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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
CULVERTS
LIMOGES ROAD (COUNTY ROAD 5) INTERCHANGE
HIGHWAY 417
W.P. 258-98-00
LIMOGES, ONTARIO**

Submitted to:

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April 2005

04-1120-013-1 (5000)



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PART A

**FOUNDATION INVESTIGATION
CULVERTS**

**LIMOGES ROAD (COUNTY ROAD 5) INTERCHANGE
HIGHWAY 417
W.P. 258-98-00
LIMOGES, ONTARIO**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Ltd. (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's proposal P31-2107, dated August 2003, that forms part of the Consultant's Agreement (Number P.O.4005-A-000316) for this project. This report addresses three culverts at the Limoges Road (County Road 5) interchange. Two of the culverts are to be provided at new ramps where they will cross South Indian Creek, while the third culvert is to replace the existing twin SPPA culverts which carry South Indian Creek under Highway 417. The work was carried out in accordance with the Quality Control Plan for this project dated February 2004.

2.0 SITE DESCRIPTION

The Limoges Road interchange on Highway 417 is located about 25 km east of the City of Ottawa, about 12 km west of the Town of Casselman, and about 1.5 km south of the Village of Limoges. Limoges Road is carried over Highway 417 on a five span, post-tensioned concrete, two lane bridge. The terrain in this area is flat and relatively poorly drained. The existing ground in the vicinity of the S-E and N-W ramps is at about Elevation 67 to 67.5 m. Ramps currently exist in the southwest quadrant (W-N/S ramp and N/S-E ramps) and northeast quadrant (E-N/S and N/S-W ramps) of the interchange. South Indian Creek flows from northwest to southeast through the interchange, crossing under Highway 417 and the existing N/S-E ramp in approximately 100 m long twin 4270 x 3050 mm SPPA culverts, located about 100 m to the west of the underpass structure. The creek then flows under the bridge structure, through the second most southerly span. In the area of the interchange, the creek channel is about 5 m wide and about 2 m deep relative to the natural ground level, with fairly steep side slopes.

The existing twin SPPA culverts are understood to be corroded and in need of replacement.

There is no treatment at the existing culvert inlets and outlets.

At the proposed S-E and N-W ramp culverts, the ramp profile grades are presently proposed for about Elevation 69.6 m, indicating an embankment height of about 2 to 2.6 m above the general grade. For the culvert replacement under Highway 417, the profile grade of the highway will be maintained at about elevation 69.3 m. The invert levels of the N-W, Highway 417, and S-E ramp culverts are to be about 65.55 m, 65.35 m, and 65.20 m, respectively. Those culverts will also be about 38.5 m, 100 m, and 40 m long, respectively.

The foundation investigation for the design of the existing bridge at this interchange was carried out in 1969 and the results of that investigations are summarized in the Ministry of Transportation, Ontario's GEOCREC No. 31G-042, *Foundation Investigation Report, For Proposed Crossing at Hwy. 417, Eastbound and Westbound Lanes, And Country Road 5 Relocation, Twps. Of Cambridge and Russell, Country of Russell, District No. 9 (Ottawa), W.J. 69-F-84 -- W.P.35-66-09*, dated December 1969.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The field work for this investigation was carried out between April 29 and May 5, 2004 at which time eight boreholes (numbered 04-1 and 04-8) were advanced in the culvert foundation areas, as follows:

- Boreholes 04-1 and 04-2: proposed new S-E Ramp culvert
- Boreholes 04-3 to 04-6: proposed Highway 417 replacement culvert
- Boreholes 04-7 and 04-8: proposed new N-W Ramp culvert

The boreholes were advanced to depths ranging from 10.4 to 11.9 m using hollow stem augers and a bombardier-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario.

The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers. 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.

In addition, five relatively undisturbed, 75 millimetre diameter thin walled Shelby tube samples of the cohesive soil deposits were obtained using a fixed piston sampler.

Piezometers were installed in Boreholes 04-2, 04-3, 04-6 and 04-7 to permit monitoring of the groundwater level at these locations. The piezometers consist of a 20 mm outside diameter rigid PVC tubing with a 0.6 m long slotted tip that is sealed at a selected depth interval within the boreholes. The holes were backfilled with bentonite mixed with soil cuttings; bentonite gravel was placed as well to provide seals between the granular and cohesive deposits. The site conditions were restored following completion of the field work.

The field work was supervised throughout by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and rock 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. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. The laboratory testing included water content, Atterberg limit, and grain size distribution testing on selected soil samples, as well as three laboratory oedometer consolidation tests.

The groundwater levels were measured in the standpipes in Boreholes 04-1 and 04-3 on May 20, 2004.

The borehole locations were initially selected by Golder Associates and then surveyed by MH.

The borehole locations, including MTM NAD83 northing and easting coordinates, and ground surface elevations referenced to geodetic datum are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
04-1	S-E Ramp culvert	5019895.70	403590.09	67.0
04-2	S-E Ramp culvert	5019886.92	403580.28	67.4
04-3	Hwy 417 replacement culvert	5019982.82	403395.44	68.3
04-4	Hwy 417 replacement culvert	5020004.34	403345.98	69.0
04-5	Hwy 417 replacement culvert	5020025.12	403353.51	68.7
04-6	Hwy 417 replacement culvert	5020039.95	403309.76	68.6
04-7	N-W Ramp culvert	5020085.60	403262.04	67.3
04-8	N-W Ramp culvert	5020087.60	403242.87	67.5

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site lies within the minor physiographic region known as the Ottawa Valley Clay Plain, which is part of the Ottawa-St. Lawrence Lowlands. The Ottawa Valley Clay Plain region 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. Most of this minor physiographic region is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain by igneous and metamorphic bedrock of the Precambrian Shield.

The terrain in the vicinity of this site is relatively flat.

Published geologic mapping as well as the existing GEOCRESS information indicate this site to be underlain by a surficial deposit of silt and sand (of deltaic or estuarine origin) underlain by a thick deposit of Champlain Sea clay. That deposit is indicated to extend to a depth in the order of 45 to 55 metres, and overlies a very thin layer of glacial till which overlies shale bedrock. Published geologic mapping indicates the bedrock in this area to consist of shale and siltstone of the Queenstone formation.

4.2 Site Stratigraphy

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets and on Figures 1 to 9. 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 locations.

In general, the subsoils at the site, are as follows:

- Culverts beneath proposed S-E and N-W ramps: approximately 2.2 to 3.9 m of sandy organic alluvium, locally overlying up to about 1 m of sand and silt, overlying soft silty clay
- Highway 417 Replacement Culvert: approximately 1.6 to 2.4 m of fill material overlying approximately 2.8 to 3.7 m of sand and silt, overlying soft silty clay.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections, and stratigraphic profiles and sections of this site are shown on Drawings 1 and 2.

4.2.1 Crushed Stone, Rockfill, and Sand Fill Material

Fill materials associated with the existing lanes of Highway 417 exist at ground surface at Boreholes 04-3 to 04-6. The fill material consists of approximately 0.5 to 0.9 m of crushed stone and/or rockfill, followed by 0.9 to 1.9 m of fine sand fill. Standard penetration test 'N' values for the sand fill ranging from 3 to 20 blows per 0.3 m of penetration indicate the sand to be very loose to compact. The results of grain size distribution testing carried out on one sample of the sand fill are provided on Figure 1.

In Boreholes 04-7 and 04-8, at the proposed N-W Ramp culvert, approximately 0.3 to 0.4 m of sand fill exist at ground surface, covered by a thin veneer of topsoil.

4.2.2 Topsoil and Silty Sand Alluvium

Topsoil was encountered discontinuously at ground surface at the borehole locations. Borehole 04-1 (S-E Ramp culvert site) encountered 200 mm of topsoil at ground surface. Borehole 04-6 (right shoulder of WBL of Hwy 417) encountered 100 mm of topsoil fill overlying the rockfill at that location. Boreholes 04-7 and 04-8 (N-W Ramp culvert site) encountered less than 50 mm of topsoil overlying the aforementioned sand fill.

In the four boreholes put down adjacent to the proposed new ramp culverts (04-1, 04-2, 04-7 and 04-8) a layer of organic alluvium exists beneath the thin surficial topsoil and/or fill layers, where present, or at ground surface. The results of grain size distribution testing carried out on two samples of the alluvium are provided on Figure 2 and indicate the material to generally consist of silty sand. The alluvium at those locations ranges in thickness from about 2.2 to 3.9 m. Approximately 0.5 m of alluvium were also encountered beneath the roadway fill materials in Borehole 04-3, put down near the south end of the Highway 417 replacement culvert.

Standard penetration test 'N' values for the alluvium material range from 1 to 4 blows per 0.3 m of penetration, indicating a very loose relative density. The measured water content of the alluvium ranges from approximately 46 to 140 percent, with the higher values reflecting the organic content.

4.2.3 Silt and Sand

In Boreholes 04-1 and 04-2 at the S-E Ramp culvert site, the alluvium is underlain by 0.6 and 0.9 m of silty fine sand and silt, some sand, respectively. In Boreholes 04-3 to 04-6, the fill materials are underlain by 2.8 to 3.7 m of a deposit ranging in composition from sandy silt to silty sand. The results of grain size distribution testing carried out on five samples of this material are provided on Figure 3.

Standard penetration tests carried out within the silty sand/sandy silt give 'N' values ranging from 1 to 13 blows per 0.3 m of penetration, indicating the material to be very loose to compact.

4.2.4 Silty Clay

A thick deposit of silty clay underlies the silty sand / sandy silt or alluvium at all of the borehole locations. The deposit extends to at least the depth investigated (10.4 to 11.9 m) and, based on the information available in the report for the existing Limoges Road underpass, the deposit is expected to extend to considerably greater depth (in the order of at least 45 m). The deposit contains occasional silty fine sand or sandy silt seams, and is generally grey in colour with red brown streaking. The results of grain size distribution testing carried out on one selected sample of the silty clay are shown on Figure 4.

The results of Atterberg Limit testing indicate liquid limits ranging from about 27 to 64 percent and plasticity index values ranging from about 11 percent to 37 percent. These results are summarized on the plasticity chart on Figures 5 and indicate the deposit to consist of clayey silt of low plasticity to clay of high plasticity. The measured natural water content of the silty clay ranges from about 40 percent to 80 percent, and is generally greater than the measured liquid limit.

Standard Penetration Testing (SPT) measured 'N' values ranging from 'manual pressure' to 1 blow per 0.3 m of penetration. The results of the in situ vane tests are summarized on Figure 9. The measured undrained shear strength of the silty clay deposit generally varies from about 19 kPa to 47 kPa, generally increasing with depth, and indicating a soft to stiff consistency. Occasional anomalous higher undrained shear strength measurements of 57 to 73 kPa may reflect the presence of sand seams within the deposit.

The sensitivity of the deposit, as estimated from the field vane tests, ranges from about 2 to 15, indicating a highly sensitive soil.

Consolidation tests (oedometer) were performed on three samples from this stratum. The results are summarized below.

Borehole (Sample)	Elevation (Depth) (m)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	$\sigma_p' - \sigma_{vo}'$ (kPa)	e_o	C_r	C_c
04-4 (7)	62.6 (6.4)	68	120	1.76	52	0.870	0.019	0.42
04-7 (4)	63.32 (4.0)	39	80	2.0	41	1.575	0.022	1.18
04-2 (6)	62.4 (5.0)	50	115	2.3	65	1.21	0.011	0.88

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}'

$\sigma_p' - \sigma_{vo}'$ is the magnitude of 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 6 to 8, respectively. As noted in the above table, the magnitude of overconsolidation (the difference between the measured preconsolidation pressure and calculated existing effective stress level, $\sigma_p' - \sigma_{vo}'$) is indicated to range from about 41 to 65 kPa.

Figure 9 provides a summary of the results of the oedometer consolidation testing carried out as part of this investigation as well as the results of four oedometer consolidation tests previously carried out by MTO as part of the design of the existing underpass structures. A summary of the water content, Atterberg limit, and in situ vane testing carried out as part of this investigation is also provided.

4.2.5 Groundwater

The water levels in the piezometers in Boreholes 04-2, 04-3, 04-6 and 04-7, were measured on May 20, 2004. The piezometers were sealed into the silty clay deposit, with the exception of Borehole 04-3, which was installed in the silty sand/ sandy silt deposit.

The water levels in the piezometers are summarized in the table below:

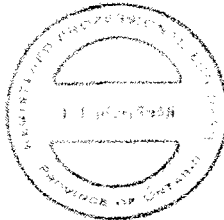
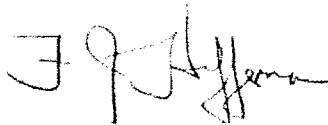
Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
04-2	67.4	0.7	66.7	May 20, 2004
04-3	68.3	1.8	66.5	May 20, 2004
04-6	68.6	1.7	66.9	May 20, 2004
04-7	67.3	1.3	66.0	May 20, 2004

The levels reflect a groundwater table which is probably typically at about Elevation 66.5 to 67 m.

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

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PART B

**FOUNDATION INVESTIGATION AND DESIGN
CULVERTS
LIMOGES ROAD (COUNTY ROAD 5) INTERCHANGE
HIGHWAY 417
W.P. 258-98-00
LIMOGES, ONTARIO**

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the two proposed new culverts to be constructed at crossings of South Indian Creek on the new S-E and N-W ramps at the Limoges Road interchange on Highway 417, as well the replacement culvert for that which presently carries South Indian Creek under Highway 417. 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

New S-E and N-W ramps are to be provided in the southeast and northwest quadrants of the interchange, respectively, and new culverts will be constructed to carry those ramps over South Indian Creek. It is presently proposed for both culverts to be 5.0 x 2.4 m concrete box or rigid-frame open box culverts. In addition, it is understood that the existing twin SPPA culverts under Highway 417 have not performed well and have previously been proposed to be replaced with a new 5.0 x 2.4 m concrete box culvert. It should however be noted that, at the time of the writing of this report, an alternative option of re-lining the existing culverts has been selected as the preferred option and this culvert is therefore not presently planned to be replaced. However, for completeness, foundation engineering guidelines for that work are still provided.

It is also understood that construction of the S-E and N-W ramps, and the associated culverts, has been deleted from the current contract.

In general, the subsoils at the sites of the proposed culverts beneath the new S-E and N-W ramps consist of approximately 2.2 to 3.9 m of sandy organic alluvium, locally overlying up to about 1 m of sand and silt, overlying soft silty clay. The subsurface conditions at the site of the proposed Highway 417 replacement culvert consist of approximately 1.6 to 2.4 m of fill material overlying approximately 2.8 to 3.7 m of sand and silt, overlying soft silty clay.

Based on the subsurface information available at the culvert locations, it is considered generally feasible to found the proposed culverts on the native soils providing the specified subgrade preparation is carried out and the estimated settlements can be tolerated. The culverts should be designed to withstand the appropriate weight of fill and traffic loadings.

5.2 Culvert Foundations

It is presently proposed for all three culverts to be 5.0 x 2.4 m concrete box or rigid-frame open box culverts. The new S-E and N-W ramps will be constructed with embankments that will be about 2 to 2.5 m above the general grade on either side of the creek and about 4 to 4.5 metres above the creek bed level. At the Highway 417 replacement culvert, the profile grade of the highway will be maintained at about elevation 69.3 m, which is about 4 metres above the proposed invert level for the replacement culvert. The N-W Ramp, Highway 417, and S-E Ramp culverts are to be about 38.5 m, 100 m, and 40 m long, respectively, with invert levels at about Elevations 65.55 m, 65.35 m and 65.2 m, respectively.

As discussed below, it is considered feasible to support all three proposed culvert on the inorganic native subgrade soils, although the settlements of the culverts for the S-E and N-W ramps will be significant. For the Highway 417 replacement culvert, the existing roadway grades will be maintained, so the structure's settlements should not be excessive provided the structure can be designed in accordance with the SLS resistance specified herein. For all three structures, closed box culverts are therefore preferred, from a foundation design perspective.

Despite the large anticipated settlements for the culverts on the proposed N-W and S-E ramps, and the low SLS resistance available for the Highway 417 replacement culvert, supporting these structures on deep foundations is not considered to be a practical (or necessary) option since the available information indicates the bedrock surface to be at about 45 to 55 m depth, resulting in long and expensive piles. In addition, for the N-W and S-E ramp culverts, the resulting differential settlement between the ramp embankments and a culvert structure founded on end-bearing piles would result in severe distortion (i.e., bump) of the roadway surface.

Two other options that could be considered for reducing the post-construction settlements for the N-W and S-E ramp culverts would be to use light weight fills to construct the embankments or alternatively to pre-load the area, in conjunction with wick drains and an overall surcharge.

Recommendations for closed box culvert design are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the foundation options is presented in Tables 1A and 1B following the text of this report.

5.2.1 Axial Geotechnical Resistance

5.2.1.1 S-E and N-W Ramp Culvert

In general, the subsoils at the sites of the proposed culverts beneath the new S-E and N-W ramps consist of approximately 2.2 to 3.9 m of sandy organic alluvium, locally overlying up to about 1 m of sand and silt, overlying soft silty clay.

As discussed below, the significant stratum from a foundation design perspective for these two culverts is the soft clay deposit present at shallow depth. This stratum is quite compressible, such that settlements may be significant and it will be important for the culvert to be tolerant of those settlements. It will also be helpful to keep the foundations as shallow as possible to limit the stress increase on the silty clay deposit at depth. A closed box culvert is therefore preferred, from a foundation design perspective.

In addition, the surficial organic alluvium is a weak and compressible subgrade and should be removed from beneath both culverts wherever the material extends below the planned founding level and be replaced with compacted engineered fill; Granular A would generally be suitable for this purpose, though Granular B Type II may need to be substituted for the first lift where the subgrade is particularly wet, as should be expected for the wet subgrade conditions anticipated within the excavations. The replacement should be carried out wherever the alluvium lies within the zone of influence of the foundations, which is considered to extend out and down from the edge of the culvert foundations at an inclination of 1 horizontal to 1 vertical. Given the expected excavation depths of up to 2 m below founding level, the engineered fill pad will extend, locally, up to about 4 metres (2 metres each side) wider than the width of the culvert. However there will be other locations where the underside of the alluvium will be very close to the founding level of the culverts. Based on the borehole data, the subexcavation elevation for the S-E Ramp varies from Elevation 62.9 to 64.0 m. For the N-W Ramp, the subexcavation elevation would be 64.7 m. However it should be confirmed at the time of construction that the underside of the alluvium, between borehole locations, does not extend below these levels.

The factored geotechnical resistance at Ultimate Limit States (ULS) for the culverts will be controlled by both the combined shear strength of the engineered fill pad and the soft grey silty clay, based on potential 'punching' failure through the engineered fill into the soft clay. The factored resistance is therefore a function of the thickness of engineered fill which will underlie the foundations. Based on the minimum expected thickness of engineered fill beneath the culverts of about 0.5 m, and assuming a bearing width of about 5 to 6 m, the ULS factored resistance may be taken as 125 kPa

Based on the results of the laboratory oedometer consolidation testing and in situ vane testing, it is considered that the available overconsolidation (i.e., the difference between the preconsolidation pressure and the existing effective stress level) within the upper, softer portions of the silty clay deposit is about 40 kPa. Stress increases greater than about 80 percent of that value would be expected to result in excessive settlements, due to both primary consolidation and secondary compression of the deposit. However the available data also indicates the deposit at greater depth to be nearly normally consolidated.

Based on the above, and in the absence of the loading from the adjacent embankments, the Serviceability Limit States resistance for these culvert foundations, based on 25 mm of settlement, would be 60 kilopascals (gross).

However, as discussed subsequently in Sections 5.2.2 and 5.6 of this report, if the adjacent embankments are constructed to their full design height using conventional earth borrow materials, the subgrade settlements will exceed 25 mm, essentially regardless of the foundation loads from the culverts. That is, the settlement of the culverts for the S-E and N-W Ramps will largely be controlled by the stress increase from the weight of the ramp embankments. It is not therefore feasible to specify a geotechnical resistance at Serviceability Limit States for the design of these culverts based on 25 mm of settlement.

Options for reducing the post-construction settlement of the embankments, and therefore of the culverts, are discussed in Section 5.6 of this report.

5.2.1.2 Highway 417 Replacement Culvert

The subsurface conditions at the site of the proposed Highway 417 replacement culvert consist of approximately 1.6 to 2.4 m of fill material overlying approximately 2.8 to 3.7 m of sand and silt, overlying soft silty clay.

A closed box culvert is also preferred for this structure in that it will result in more distributed stress on the subgrade and lesser stress increase on the soft silty clay deposit that exists at depth.

The factored geotechnical resistance at Ultimate Limit States (ULS) for the culvert will be controlled by both the combined shear strength of the limited thicknesses of the very loose to compact sand and silt that will underlie the culvert directly, as well as the shear strength of the soft grey silty clay at depth. The critical failure mode is one based on potential 'punching' failure through the sand and silt into the soft clay. Based on the minimum expected thickness of sand and silt beneath the culvert of about 1.0 m, and assuming a bearing width of about 5 to 6 m, the ULS factored resistance may be taken as 110 kPa.

The geotechnical resistance at Serviceability Limits States (SLS) is controlled by the compressibility of the soft clay deposit present at shallow depth below founding level. In considering the height and weight of the existing fills within the eastbound and westbound lanes, the available information indicates that the stress level within the silty clay deposit is at its preconsolidation pressure. That is, the deposit is essentially normally consolidated. Therefore any increase in stress above the existing value can be expected to result in settlements that would be significant. The resulting overlying roadway settlements would also be entirely differential in relation to the existing roadway on either side of the culvert. It is therefore proposed that the resistance at Serviceability Limits States (SLS) be taken as 75 kPa (gross), which is slightly below the calculated existing effective stress level at the expected founding level of the culvert.

5.2.2 Estimated Settlement for N-W and S-E Ramp Culverts

Given the soft, sensitive, and compressible silty clay deposit that underlies this site, the settlement of the culverts for the S-E and N-W Ramps will be controlled by the stress increase on that deposit from the weight of the ramp embankments. Accordingly, as described above, it is not possible to provide an SLS bearing resistance for 25 mm of settlement that is independent of the load from the embankment; the two are inter-related.

Up to approximately 2.5 m high embankments are to be constructed for the ramps at the culvert locations, and, if conventional embankment borrow materials are used, having a unit weight in the range of 20 to 22 kN/m³, the stress imposed on the embankment subgrade will be about 50 to 55 kPa. In addition, significant thicknesses of organic alluvium will have to be excavated from beneath the culvert areas and replaced with heavier granular fill. The difference in unit weight between the engineered fill and the alluvium it replaces will result in additional stress increase on the soft silty clay deposit.

Considering the oedometer consolidation test data from this investigation as well as the results of four tests previously completed by MTO for the design of the existing bridge and approach embankments, it is considered that the preconsolidation pressure of the deposit, at shallow depth, is about 80 kPa, and increases somewhat with depth. The existing effective stress level within the upper portions of the deposit is considered to be in the order of 40 kPa. It is therefore estimated that the weight of the embankment fill and additional stress from the alluvium replacement will raise the effective stress level within the upper portion of the silty clay deposit in excess of its preconsolidation pressure. At greater depth, the available information indicates that the deposit may in fact be normally consolidated and the resulting stress will therefore be significantly in excess of the material's preconsolidation pressure. Significant settlements can therefore be expected due to consolidation of the silty clay deposit.

It is estimated that the resulting culvert settlements could be in the order of 500 to 700 mm. However, based on the coefficients of consolidation indicated from the laboratory testing and considering the significant thickness of the deposit, which results in a long drainage path for displaced pore water, it is estimated that this settlement would take in the order of 30 years to occur. About 3 years would be required for 30 percent of the settlement to occur; 60 percent of the settlement would occur in about 9 years.

It should be noted that the settlement magnitude given above is based on the (likely conservative) assumption that the contact stress under the culvert is equal to the weight of the adjacent embankment. It should also be noted that the settlement estimates given above exceed the settlement estimates for the adjacent embankments (as described in Section 5.6 of this report) due to the additional net stress increase that will occur under the culverts as a result of replacement of the alluvium beneath the culvert areas with heavier engineered fill.

The above settlement estimate can be refined once further details are available on the ramp and culvert design.

It should be noted that the settlements will be non-uniform along the length of the culverts, due to the variable fill height across the embankment width. The settlement at the centre of the embankment, and culvert, will be greater than that beneath the toe of the embankment (ends of the culverts). In addition to potentially impacting the culvert's hydraulic performance, these settlements would result in stresses within the culvert and irregular subsidence of the roadway surface. The distortion of the overlying roadway may be quite noticeable and require periodic maintenance.

Given that it is not likely feasible to reduce the ramp heights significantly, consideration may therefore be given to the following three options:

1. Accepting the potential impacts of the expected settlements on the performance of the overlying roadway and culvert, but designing the culvert to be relatively tolerant of those settlements,
2. Using light weight fill (such as EPS) for the embankment construction on either side of the culvert and thereby reduce the stress increase on the compressible clay deposit to a level such that the foundation settlements will be within acceptable tolerances.
3. Pre-loading the site and allowing the settlements to occur in advance of culvert construction.

For Option 1, if these settlements are to be accepted, a closed box culvert made with pre-cast concrete sections would be preferred. A rigid frame, open-box type culvert would not be expected to perform well and should not be used. By not connecting the sections together, the closed box culvert would be able to accommodate some settlement and flexure without damaging the culvert. However the exterior of the joints, over the top and sides of the culvert, should be covered with geotextile to avoid having the backfill materials enter the joints when they open. Cast-in-place closed box culverts could also be considered, provided the culverts were constructed in several, short independent sections, and would therefore behave similarly to the pre-cast culvert option. Consideration should be given to cambering the culvert such that it would have the proper grade after some proportion of the settlement is complete.

Preliminary guidelines on the use of EPS fill (Option 2, above) are provided in Section 5.6 of this report. However, if essentially the full height of the embankment (other than the pavement materials) was constructed using EPS, such that the embankment weight would no longer control the settlement of the culvert, then, as described previously, the SLS resistance that could be used for the design of the culvert, based on 25 mm of settlement, would be 60 kilopascals (gross).

Preliminary guidelines for pre-loading (Option 3) are also provided in Section 5.6, however it should be noted that this option would likely involve the placement of a surcharge and the installation of wick drains to accelerate the settlement. The creek would also need to be temporarily diverted, for the duration of the pre-loading.

A summary comparison of the advantages, disadvantages, relative costs, and risks associated with these options is presented in Table 1A following the text of this report. A recommendation for a preferred option is provided in Section 5.6 of this report, inasmuch as the design of the culvert foundations and the approach embankments are inter-related.

5.2.3 Resistance to Lateral Loads

The resistance to lateral forces/sliding should be calculated in accordance with Section 6.7.5 of the CHBDC, using the following parameters:

Interface and Loading Condition	Parameter
Concrete – Granular A pad: short or long term loading	Effective friction angle = 33 degrees
Granular A pad – clay subgrade: short term loading	Undrained cohesion = 12 kPa
Granular A pad - clay subgrade: long term loading	Effective friction angle of 25 degrees

These are unfactored values; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

5.2.4 Frost Protection

If the South Indian Creek is expected to neither freeze completely nor be dry during the winter months, then no frost protection other than the bedding material need to be provided for the culvert bearing surface.

5.3 Bedding and Backfill

The bedding, backfill and levelling pad requirements for the culverts should be in accordance with OPSD 803.010. The culverts should be designed for the full overburden pressure and live load, assuming an embankment fill unit weight of 21 kN/m³.

The box culverts should be provided with a minimum 400 mm of OPSS Granular 'A' bedding. Additional Granular 'A' shall be placed according to the depth of subexcavation required. The bedding should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Where the subgrade is particularly wet, Granular B Type II should be substituted for the first lift, since its

compactability is generally less sensitive to the water content. Groundwater and surface water control will be required at the culvert locations to avoid “pumping” of water into the fill during compaction.

Backfill to the culvert walls and wing walls should consist of granular fill meeting the specifications for OPSS Granular ‘A’ or Granular ‘B’ Type II (but with less than 5 per cent passing the 200 sieve). The backfill should be placed in lifts not exceeding 200 mm loose thickness and compacted to 95 per cent Standard Proctor dry density. The fill depth during placement should be maintained equal on both sides of the culvert with one side not exceeding the other by more than 400 mm.

5.4 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the walls. It should be noted that where there is sloping ground behind the walls the coefficient of lateral earth pressure must be adjusted to account for the slope.

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

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

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

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

EARTH FILL	
Soil unit weight:	21 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33 (level ground) 0.73 (2H:1V slope)
At rest, K_o	0.50 (level ground) Note 1 (2H:1V slope)

Note 1: For sloping backfill and at-rests conditions, the weight of the sloping backfill located above the top of the wall should be treated as a surcharge.

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

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27 (level ground) 0.62 (2H:1V slope)	0.31 (level ground) 0.68 (2H:1V slope)
At rest, K_o	0.43 (level ground) Note 1 (2H:1V slope)	0.47 (level ground) Note 1 (2H:1V slope)

Note 1: For sloping backfill and at-rests conditions, the weight of the sloping backfill located above the top of the wall should be treated as a surcharge.

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

Seismic loading will result in increased lateral earth pressures acting on the wall stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the National Building Code of Canada, this site is located in Seismic Zone 4. The site-specific zonal acceleration ratio for Ottawa (the nearest listed location) is 0.2. Based on experience, for the subsurface conditions at this site, a 10 per cent amplification of the ground motion could occur, resulting in an increase in the ground surface acceleration to between 0.22g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.22$.

In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.33$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.11$). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v . Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3 k_h$, $k_v = 0$, and $k_v = -2/3 k_h$.

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

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.40	0.32	0.36
Non-yielding wall	0.80	0.63	0.71

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.22. This corresponds to displacements of up to 55 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d)$$

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

- Where sloping backfill is provided, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

5.5 Erosion Protection

The soils exposed at creek bank level consist of sandy alluvium (S-E and N-W Ramp culverts) and the silt and sand deposit (Highway 417 replacement culvert). The grain size distribution of those deposits, as shown on Figures 2 and 3, indicate that 100 per cent of the particles are smaller than 1 mm and the mean particle size is less than 0.08 mm. If the creek velocities warrant, provision should be made for scour and erosion protection.

In order to prevent creek water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal or cut-off wall should be provided at the upstream end of the culvert. The clay seal should have a minimum thickness of 0.3 m. It should be keyed into the native subsoil, and extended a minimum horizontal distance of 2 m on either side of the culvert inlet openings and a minimum vertical height equal to the high water level. The material for the clay seal should be in accordance with the requirements of OPSS 1205.

Erosion protection should be provided to the culvert as appropriate. Consideration could be given to the use of suitable non-woven geotextiles and rip-rap to provide erosion protection based on hydraulic requirements.

In addition, sediment control such as silt fences and / or erosion control blankets may be required during construction and diversion of the water course to mitigate migration of fine soil particles into the water courses.

5.6 Embankments for New Ramps

5.6.1 Embankment Subgrade Preparation and Construction

For the approach embankments to the culverts on the N-W and S-E ramps, any topsoil, organic matter and softened / loosened soils should be stripped from beneath the embankment footprints and all subgrade soils should be proof-rolled prior to fill placement.

Embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing

should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

5.6.2 Embankment Settlement

The settlements of the embankments adjacent to the culverts will result from both compression of the native soils and from within the embankments 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, would occur after construction.

As described previously Section 5.2.2 of this report, the stress increase on the soft, sensitive, and compressible silty clay deposit that underlies this site from the weight of the ramp embankments will result in significant settlements. Up to approximately 2.5 m high embankments are to be constructed for the ramps at the culvert locations, and, if conventional embankment borrow materials are used, having a unit weight in the range of 20 to 22 kN/m³, the stress imposed on the embankment subgrade will be about 50 to 55 kPa. Considering the oedometer consolidation test data from this investigation as well as the results of four tests previously completed by MTO for the design of the existing bridge and approach embankments, it is estimated that the embankment subgrade settlements could be in the order of 400 to 500 mm. These estimated settlement magnitudes have been established from the results of primary consolidation analyses, calibrated to the results of previous monitoring of the settlement of other bridge approach embankments in this area (i.e., Anderson Road). The estimates are therefore considered, within their order of accuracy, to include both primary consolidation and secondary compression.

It is estimated that this settlement would take in the order of 30 years to occur. About 3 years would be required for 30 percent of the settlement to occur; 60 percent of the settlement would occur in about 9 years.

The above settlement estimate can be refined once further details are available on the embankment design.

It should be noted that the settlements will be non-uniform along the width of the embankments. The settlement estimate given above corresponds to the centre of the embankment. The settlement beneath the toe of the embankment would be less. Some loss of super-elevation could be expected. The settlements of the roadway surface directly over the culverts could exceed those of the adjacent embankments, due to the additional stress increase resulting from sub-excavation of alluvium and replacement with heavier engineered fill, such that a depression could form over

the culvert. Similar depressions are visible elsewhere along Highway 417. The severity of the distortion would depend on the magnitude of the differential settlement between the culvert and embankment. The distortion may require periodic maintenance. As described previously, these settlements would also impact on the performance of the culvert.

If the settlement magnitudes described above cannot be tolerated, in relation to either the culvert or roadway performance, the following options could be considered, as also described in Table 1A:

1. Option 1: At least partially constructing the ramp embankments using light weight fill, such as EPS.
2. Option 2A: Pre-loading the embankment and culvert footprint, with a surcharge
3. Option 2B: Pre-loading the embankment and culvert footprint, with a surcharge and wick drains.

For Option1, as a preliminary guideline, in the absence of detailed subsurface information along the full ramp length, it is considered that the EPS thickness would need to be such that the combined thickness of the earth fill and pavement structure would be no more than 1.25 metres, if the ramp settlements are to be maintained at more manageable levels (i.e., less than about 150 mm total settlement with minor differential settlement). Although other light weight fill materials could be considered, such as slag, the quantity of light weight fill required at this site is relatively small, and it would therefore be likely un-economic to import those materials. Also, the EPS would be more effective in reducing the loading.

For Option 2A, the embankment and culvert footprints would be pre-loaded to the design grade plus a surcharge, if feasible, and the settlements allowed to occur prior to culvert construction and paving of the ramp. However the results of stability analyses indicate that unless the pre-load slopes are flatter than 4H:1V (typically the flattest desirable slope), the surcharge would be limited to 1 m height; stability analyses indicate that this geometry would have the minimum required factor of safety of 1.3 against instability. Therefore, considering this limited allowable surcharge as well as the significant thickness of the deposit and the associated slow rate of consolidation, it is considered that the pre-loading time required to achieve a sufficient portion of the total settlements would be several years, which is likely impractical for this project.

It is therefore considered that the pre-load/surcharge would need to be applied in conjunction with wick drains (i.e., Option 2B) to reduce the preloading time. As an example, wick drains could be installed and a pre-load with an additional 1.0 m surcharge of fill material could be left in-place for a period of about one year and then removed, such that the post-construction settlements would be reduced. Further, more detailed, analyses would need to be carried out to estimate the magnitude of the resulting post-construction settlements.

Additional foundation investigation would be required to fully evaluate/design these options.

Advantages, disadvantages, relative costs, risks and consequences for these options, which relate also to the foundation design for the culverts, are provided in Table 1A. Since it is considered that the cost for light weight fill would be significantly more than for either pre-loading options, and that pre-loading would only be practical if used in conjunction with a surcharge and wick drains, it is considered that Option 2B is preferred.

5.6.3 Embankment Stability

It is understood that the embankment side slopes on the N-W and S-E ramps are to be ultimately inclined at no steeper than 3 horizontal to 1 vertical (3H:1V). With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, these approach embankments will have a factor of safety of 1.6 against deep-seated slope instability, which is considered acceptable. Under seismic loading conditions, using a seismic coefficient of 0.1, the embankments would have a factor of safety of 1.2 against deep-seated instability, which is also acceptable. The stability analyses were carried out using the following parameters:

Soil Deposit	Bulk Unit Weight	Effective Friction Angle	Undrained Shear Strength
Embankment Fill	22 kN/m ³	32°	—
Alluvium	16.9 kN/m ³	27	-
Grey Silty Clay	18 kN/m ³	—	17.5 kPa – Note 1

NOTES:

1 - Undrained shear strength reduced from measured value based on plasticity.

It should be noted however that the factor of safety, under 'static' loading conditions for 2H:1V side slopes (which is steeper than presently proposed for this site) would be (slightly) less than 1.3 and the slopes should therefore not be that steep.

5.7 Construction Considerations

5.7.1 Groundwater and Surface Water Control

Based on the water levels measured in the piezometers, the groundwater level is expected to be essentially at the creek water level. It is assumed that coffer dams will need to be constructed to bypass the flows around the culvert areas during construction.

It is understood that, based on the proposed invert levels, culvert floor thicknesses, and the required substrate within the culverts, the excavations for all three culvert will extend about 2.0 to 2.5 metres below the water level in silt, sand, and alluvium. Relatively significant groundwater flow should be expected. It should be noted that the volume of groundwater pumped by the contractor could conceivably exceed 50,000 Litres per day and, if the contractor expects to do so, then the contractor will need to apply for a Permit-to-Take-Water from the Ministry of the Environment.

The inflow of groundwater could also loosen and disturb the subgrade.

It is therefore considered that pre-pumping from shallow wells in the silt and sand would probably be required, to lower the groundwater level below the excavation level in advance of construction.

Consideration should be given to including an NSSP in the contract documents that alerts the contractor to potential groundwater inflow and the related potential disturbance of the alluvium and sand/silt subgrade soils.

For all three sites, the rate of groundwater inflow and the potential for subgrade disturbance could be reduced by constructing a cofferdam around the excavations (on all sides), composed of interlocking steel sheet piling that fully penetrates the surficial alluvium and silt/sand deposits and extends into the underlying, less permeable silty clay.

5.7.2 Excavations

The contractor should be aware that trafficking over the soft silty clay subgrade and the wet silt and sand material may not be possible and an Operational Constraint or a non-standard Special Provision should be included in the contract in this regard, which directs the contractor to not travel on the subgrade surface with equipment.

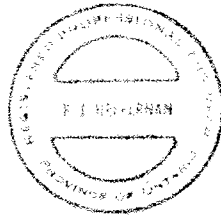
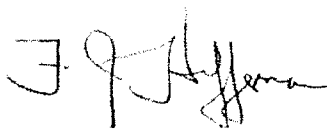
Temporary excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The soft silty clay soils are classified as Type 4 soils, according to the OHSA, and excavations and extend to, or into, that stratum should be made with side slopes no steeper than 3 horizontal to 1 vertical. Above the groundwater level, the silt, sand, and sandy alluvium are classified as Type 3 soils, according to the OHSA, and excavations should be made with side slopes no steeper than 1 horizontal to 1 vertical. Below the groundwater level, those soils would be classified as Type 4 soils, according to the OHSA, and excavations should be made with side slopes no steeper than 3 horizontal to 1 vertical.

Excavation support for roadway protection may be required at the Highway 417 replacement culvert. Where required, the temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 539S01. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 539S01. However the design of that shoring must consider the soft silty clay deposit at depth and the potential for basal instability of the excavation. This possibility should be considered even where the soft clay deposit will not be exposed by the excavation but will exist within a depth below the excavation bottom that is less than 70 percent of the excavation width. However preliminary calculations indicate that it may not in fact be feasible to construct a shored excavation with vertical walls that will have an adequate factor of safety against basal instability.

For all three culvert sites, excavated soil should not be stockpiled adjacent to the crest of excavation side slopes (or above shoring) due to the potential to reduce the factor of safety against side slope or basal instability.

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TABLE 1A
COMPARISON OF FOUNDATION ALTERNATIVES
CULVERTS
LIMOGES ROAD (COUNTY ROAD 5) INTERCHANGE
N-W and S-E RAMP CULVERTS

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Concrete box culvert	<ul style="list-style-type: none"> Feasible, if structure can tolerate differential settlements 	<ul style="list-style-type: none"> Simpler design and construction 	<ul style="list-style-type: none"> Culvert will settle along its length. Design will need to prevent cracking, either through segmental design or appropriate reinforcing. Need to provide additional culvert height and/or camber so that hydraulic performance is maintained. 	<ul style="list-style-type: none"> Lower cost than other options 	<ul style="list-style-type: none"> Culvert hydraulic performance may be impacted Could have seepage through bedding and piping of material Roadway surface may become distorted, both longitudinally and transversely.
Concrete box culvert with EPS backfill / embankments	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Less settlement than with conventional backfill / embankments 	<ul style="list-style-type: none"> Additional investigation would be required to fully define required extent of EPS treatment 	<ul style="list-style-type: none"> Significantly more expensive backfill / embankments 	<ul style="list-style-type: none"> No significant risks/consequences
Concrete box culvert with surcharge pre-load.	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Significantly reduce the settlements of culvert and roadway 	<ul style="list-style-type: none"> Magnitude (height) of surcharge limited by stability of embankment Due to limited surcharge and lengthy consolidation time, preloading will delay ramp construction for several years Need to monitor settlements during pre-loading Need to divert creek. Can't pre-load diversion area. 	<ul style="list-style-type: none"> Less than EPS fill 	<ul style="list-style-type: none"> Construction could be delayed beyond current schedule

TABLE 1A (continued)
COMPARISON OF FOUNDATION ALTERNATIVES
CULVERTS
 LIMOGES ROAD (COUNTY ROAD 5) INTERCHANGE
 N-W and S-E RAMP CULVERTS

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Concrete box culvert with surcharge pre-loading and wick-drains	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Significantly reduce the settlements of culvert and roadway Less time required for pre-load. 	<ul style="list-style-type: none"> May delay ramp construction. Less certainty regarding completion date. Need to monitor settlements during pre-loading. Need to divert creek. Can't pre-load diversion area. 	<ul style="list-style-type: none"> Higher cost 	<ul style="list-style-type: none"> Construction could be delayed beyond current schedule
Rigid frame open-box culverts	<ul style="list-style-type: none"> Not feasible due to higher settlement sensitivity 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A
Piled Foundations	<ul style="list-style-type: none"> Feasible, 	<ul style="list-style-type: none"> Limited structure settlements 	<ul style="list-style-type: none"> Distortion of roadway surface at interface with piled structure 	<ul style="list-style-type: none"> Very high cost 	<ul style="list-style-type: none"> Possibility of long, flexible piles being deflected during driving.

TABLE 1B
COMPARISON OF FOUNDATION ALTERNATIVES
CULVERTS
 LIMOGES ROAD (COUNTY ROAD 5) INTERCHANGE
 Hwy 417 REPLACEMENT CULVERT

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Concrete box culvert	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Simpler design and construction 	<ul style="list-style-type: none"> Some differential settlement could occur relative to roadway surface. 	<ul style="list-style-type: none"> Lower cost than other options 	<ul style="list-style-type: none"> Possible need for subexcavation if subgrade becomes disturbed during construction. Greater care required during subgrade preparation.
Rigid frame open-box culverts	<ul style="list-style-type: none"> Not feasible due to low SLS resistance to foundation loads Technically feasible 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A
Piled Foundations	<ul style="list-style-type: none"> Technically feasible 	<ul style="list-style-type: none"> High bearing resistance Negligible settlements 	<ul style="list-style-type: none"> Culvert will be a rigid object relative to roadway surface, which may be continuing to settle. A bump would form, in the long term. 	<ul style="list-style-type: none"> Very high cost 	<ul style="list-style-type: none"> Possibility of long flexible piles being deflected during driving.

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		III. SOIL DESCRIPTION	
AS	Auger sample	(a)	Cohesionless Soils
BS	Block sample		
CS	Chunk sample	Density Index	N
DO	Drive open	(Relative Density)	Blows/300 mm
DS	Denison type sample		Or Blows/ft.
FS	Foil sample	Very loose	0 to 4
RC	Rock core	Loose	4 to 10
SC	Soil core	Compact	10 to 30
ST	Slotted tube	Dense	30 to 50
TO	Thin-walled, open	Very dense	over 50
TP	Thin-walled, piston		
WS	Wash sample	(b)	Cohesive Soils
II. PENETRATION RESISTANCE		Consistency	$C_{u2}S_u$
Standard Penetration Resistance (SPT), N:			<u>Kpa</u>
The number of blows by a 63.5 kg. (140 lb.)		Very soft	0 to 12
hammer dropped 760 mm (30 in.) required		Soft	12 to 25
to drive a 50 mm (2 in.) drive open		Firm	25 to 50
Sampler for a distance of 300 mm (12 in.)		Stiff	50 to 100
		Very stiff	100 to 200
		Hard	Over 200
			<u>Psf</u>
			0 to 250
			250 to 500
			500 to 1,000
			1,000 to 2,000
			2,000 to 4,000
			Over 4,000
Dynamic Penetration Resistance; N_d:		IV. SOIL TESTS	
The number of blows by a 63.5 kg (140 lb.)		w	water content
hammer dropped 760 mm (30 in.) to drive		w_p	plastic limited
Uncased a 50 mm (2 in.) diameter, 60° cone		w_l	liquid limit
attached to "A" size drill rods for a distance		C	consolidation (oedometer) test
of 300 mm (12 in.).		CHEM	chemical analysis (refer to text)
PH:	Sampler advanced by hydraulic pressure	CID	consolidated isotropically drained triaxial test ¹
PM:	Sampler advanced by manual pressure	CIU	consolidated isotropically undrained triaxial test
WH:	Sampler advanced by static weight of hammer		with porewater pressure measurement ¹
WR:	Sampler advanced by weight of sampler and rod	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 ($D_R = \rho_s/\rho_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density 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

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

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

2. Shear strength = (Compressive strength)/2

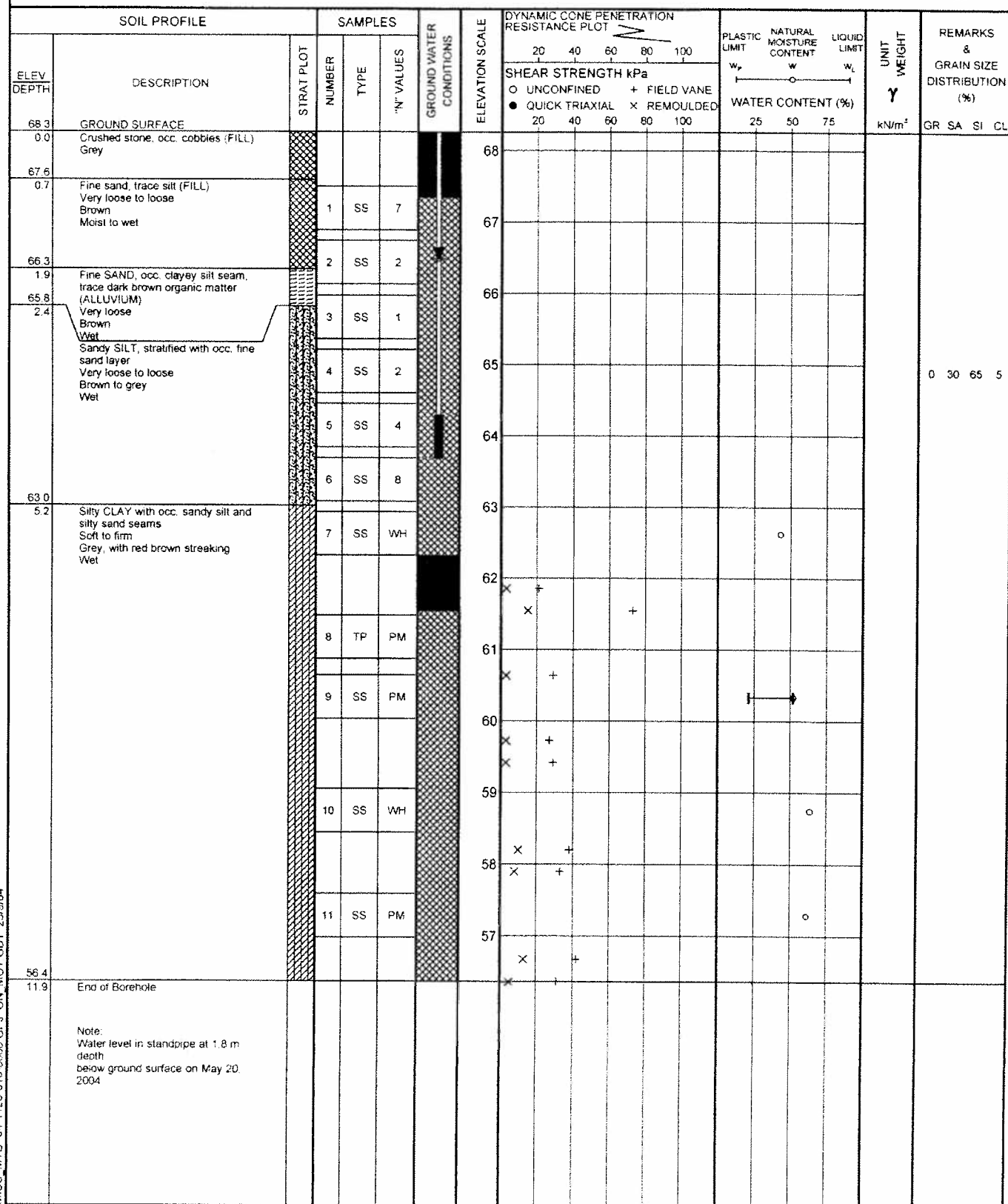
PROJECT 04-1120-013 Phase 5000			RECORD OF BOREHOLE No 04-1			1 OF 1			METRIC					
W.P. 258-98-00			LOCATION N 5019895.70, E 403590.09			ORIGINATED BY J.S.								
DIST HWY 417			BOREHOLE TYPE Power Auger 108 mm I.D.			COMPILED BY M.I.C.								
DATUM Geodetic			DATE May 5, 2004			CHECKED BY M.I.C.								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
67.0	GROUND SURFACE													
0.0	TOPSOIL													
0.2	Silty fine SAND with roots and organic matter (ALLUVIUM) Loose to very loose Grey to brown Moist to wet		1	SS	4									
			2	SS	1									0 55 43 2
			3	SS	1									
			4	SS	2									
62.9			5	SS	5									
4.1	Silty fine SAND Loose Grey Wet		6	SS	1									
62.3														
4.7	Silty CLAY, occ. silty fine sand seam Soft to firm Grey, with red brown streaking Wet		7	SS	PM									
			8	SS	PM									
			9	SS	PM									
56.7														
10.4	End of Borehole													

MISS MTO 04-1120-013-5000 GPJ ON MOT.GDT 23/9/04

PROJECT 04-1120-013 Phase 5000			RECORD OF BOREHOLE No 04-2			1 OF 1		METRIC				
W.P. 258-98-00			LOCATION N 5019886 92 E 403580 28			ORIGINATED BY J.S.						
DIST HWY 417			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY M.I.C.						
DATUM Geodetic			DATE May 4, 2004			CHECKED BY M.I.C.						
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	W _p	W			W _L
67.4	GROUND SURFACE											
0.0	Silty fine SAND and dark brown organic SILT, some roots and wood (ALLUVIUM) Very loose Brown and dark brown Wet		1	SS	1							
			2	SS	1							
			3	SS	1							
64.0			4	SS	4							
3.4	SILT, some sand Loose Grey Wet		5	SS	6							
63.1			6	TP	PM							
4.3	Silty CLAY, occ. thin sandy silt seam Soft to firm Grey, occ. red brown streaking Wet		7	SS	WH							
			8	SS	PM							
			9	SS	PM							
57.0												
10.4	End of Borehole											
<p>Note: Water level in standpipe at 0.7 m depth below ground surface on May 20, 2004</p>												

MISS-MTO 04-1120-013-5000 GPJ ON MOT GDT 23/9/04

PROJECT <u>04-1120-013 Phase 5000</u>		RECORD OF BOREHOLE No 04-3		1 OF 1	METRIC
W.P. <u>258-98-00</u>		LOCATION <u>N 5019982 82 E 403395 44</u>		ORIGINATED BY <u>J.S.</u>	
DIST <u> </u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 108 mm I.D. Hollow Stem Auger</u>		COMPILED BY <u>M.I.C.</u>	
DATUM <u>Geodetic</u>		DATE <u>May 4, 2004</u>		CHECKED BY <u>M.I.C.</u>	



MISS-MTO 04-1120-013-5000 GR-J ON MOT GDT 23/0/04

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

PROJECT		RECORD OF BOREHOLE		No 04-4		1 OF 1		METRIC					
W.P. 258-98-00		LOCATION		N 5020004 33, E 403345 98		ORIGINATED BY		J.S.					
DIST		HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY					
M.I.C.		DATE		May 3, 2004		CHECKED BY		M.I.C.					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	25 50 75		
69.0	GROUND SURFACE												
0.0	Crushed stone (FILL) Grey												
68.3													
0.7	Fine sand, trace silt (FILL) Compact Brown Moist		1	SS	15								
67.4													
1.6	Sandy SILT with occ. fine sand layer Loose to compact Brown to grey Wet		2	SS	13								
			3	SS	6								
			4	SS	4								
			5	SS	12								
			6	SS	11								
63.9													
5.2	Silty CLAY with occ. sandy silt seam Soft to firm Grey, with red brown streaking Wet		7	TP	PM								
			8	SS	PM								
			9	SS	1								
			10	SS	PM								
57.1													
11.9	End of Borehole												

MISS MTO 04-1120-013-5000.GPJ ON MOT.GDT 23/9/04

+ 3 x 3. Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

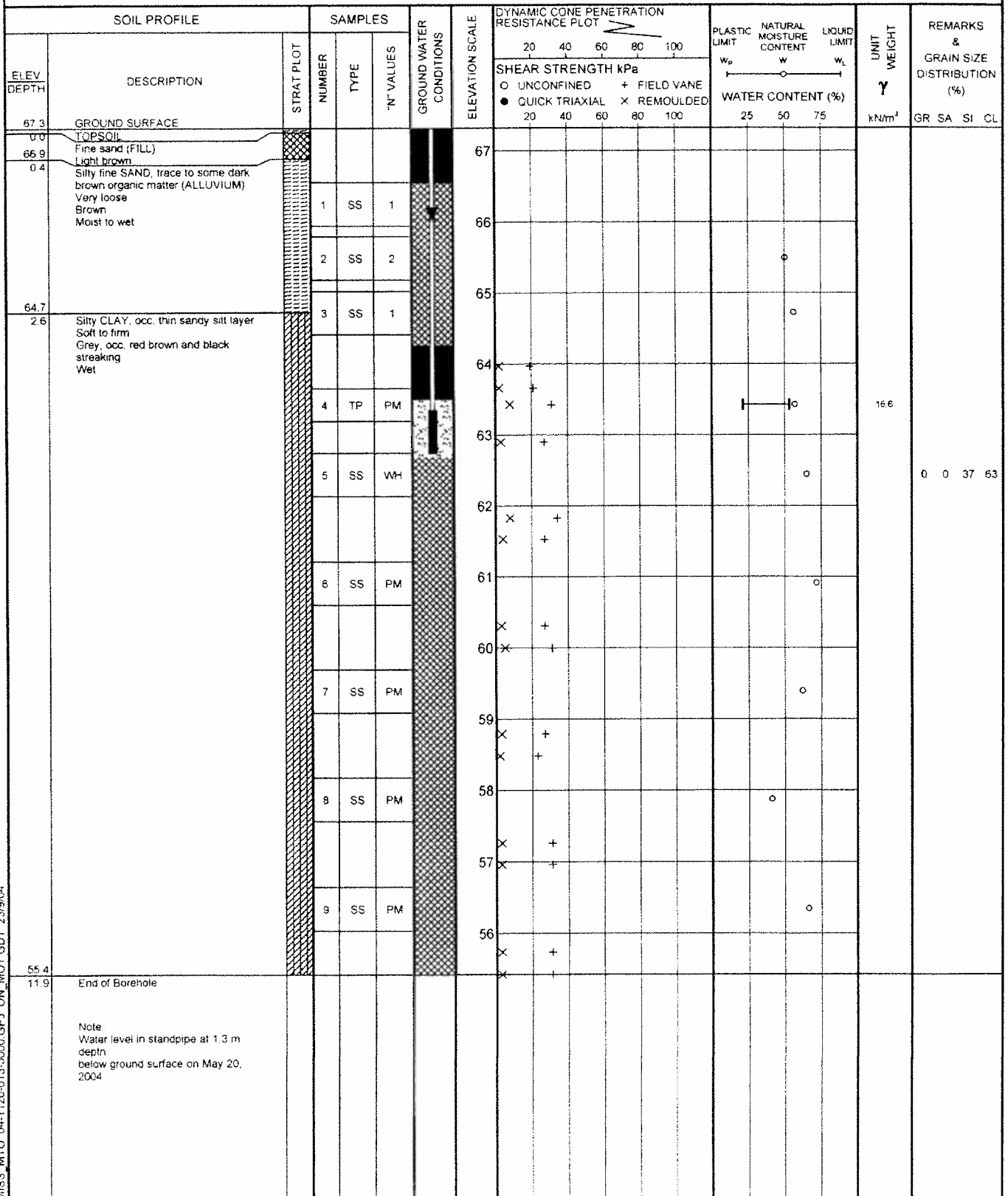
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W.P.		258-98-00		LOCATION		N 5020025 12 E 403353 51		ORIGINATED BY										
DIST		HWY 417		BOREHOLE TYPE		Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY										
DATUM		Geodetic		DATE		May 3, 2004		CHECKED BY										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20						40	60	80	100	25
68.7	GROUND SURFACE																	
0.0	Crushed stone (FILL) Grey																	
68.3																		
0.5	Fine sand, trace silt (FILL) Compact Brown Moist		1	SS	20													0 84 (16)
			2	SS	19													
66.3																		
2.4	Silty fine SAND, occ. very thin sandy silt seam Loose to compact Brown to grey Wet		3	SS	8													0 66 32 2
			4	SS	6													
			5	SS	7													
			6	SS	10													
63.2																		
5.5	Silty CLAY, occ. thin grey sandy silt seam Soft to firm Grey, with red brown streaking Wet		7	SS	1													
			8	SS	WH													
			9	SS	PM													
			10	SS	WH													
			11	SS	PM													
56.8																		
11.9	End of Borehole																	

MISS MTO 04-1120-013-5000 GPJ ON MOT GDT 23/9/04

PROJECT 04-1120-013 Phase 5000			RECORD OF BOREHOLE No 04-6			1 OF 1		METRIC					
W.P. 258-98-00			LOCATION N 5020G39 95 E 403309 76			ORIGINATED BY D.J.S.							
DIST _____ HWY 417			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY M.I.C.							
DATUM Geodetic			DATE April 30, 2004			CHECKED BY M.I.C.							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL	
68.6	GROUND SURFACE												
8.9	TOPSOIL												
67.6	Limestone ROCKFILL, with cobbles and boulders up to 0.3 m diameter												
1.0	Silty sand (FILL)		1	SS	6								
	Very loose to loose												
	Brown												
	Wet												
66.6			2	SS	3								
2.0	Sandy SILT, stratified, occ. silty sand seam												
	Very loose to loose												
	Brown to grey												
	Wet												
			3	SS	1								
			4	SS	2								
			5	SS	2								
			6	SS	5								
			7	SS	3								
62.8													
5.7	Silty CLAY, occ. sandy silt seam and red brown silty clay layer												
	Soft to firm												
	Grey												
	Wet												
			8	TP	PM								
			9	SS	PM								
			10	SS	PM								
			11	SS	PM								
56.7													
11.9	End of Borehole												
	Note Water level in standpipe at 1.7 m depth below ground surface on May 20, 2004												

MISS_MTO 04-1120-013-5000.GPJ ON MOT.GDT 23/9/04

PROJECT 04-1120-013 Phase 5000		RECORD OF BOREHOLE No 04-7		1 OF 1	METRIC
W.P. 258-96-00	LOCATION N 5020085.60, E 403262.04	ORIGINATED BY D.J.S			
DIST _____ HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY M.I.C			
DATUM Geodetic	DATE April 29, 2004	CHECKED BY M.I.C			



MISS MTO 04-1120-013-5000 GPJ ON MOT GDT 23/9/04

+ 3 x 3. Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT 04-1120-013 Phase 5000			RECORD OF BOREHOLE No 04-8			1 OF 1			METRIC					
W.P. 258-98-00			LOCATION N 5020087.60, E 403242.87			ORIGINATED BY D.J.S.								
DIST _____ HWY 417			BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger			COMPILED BY M.I.C.								
DATUM Geodetic			DATE April 29-30, 2004			CHECKED BY M.I.C.								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40					
67.5	GROUND SURFACE													
0.0	TOPSOIL													
0.3	Silty fine sand (FILL) Light brown Silty fine SAND, trace black organic matter (ALLUVIUM) Very loose Brown Moist to wet		1	SS	3									0 54 44 2
			2	SS	2									
64.7			3	SS	3									
2.7	Silty CLAY, occ. sand and silt seam Soft to firm Gray, with red brown streaking Wet		4	SS	WH									
			5	SS	WH									
			6	SS	WH									
			7	SS	PM									
			8	SS	PM									
			9	SS	PM									
55.6														
11.9	End of Borehole													

MISS-MTO 04-1120-013-5000 GPJ ON MOT.CDT 23/9/04

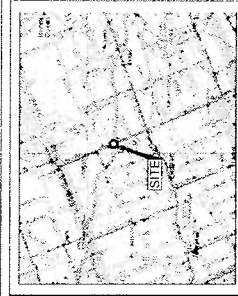
CONT No.
WP No. 258-98-00

SOIL STRATA

SHEET

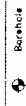


Golden Associates Ltd.
International, Ontario, Canada



KEY PLAN

LEGEND



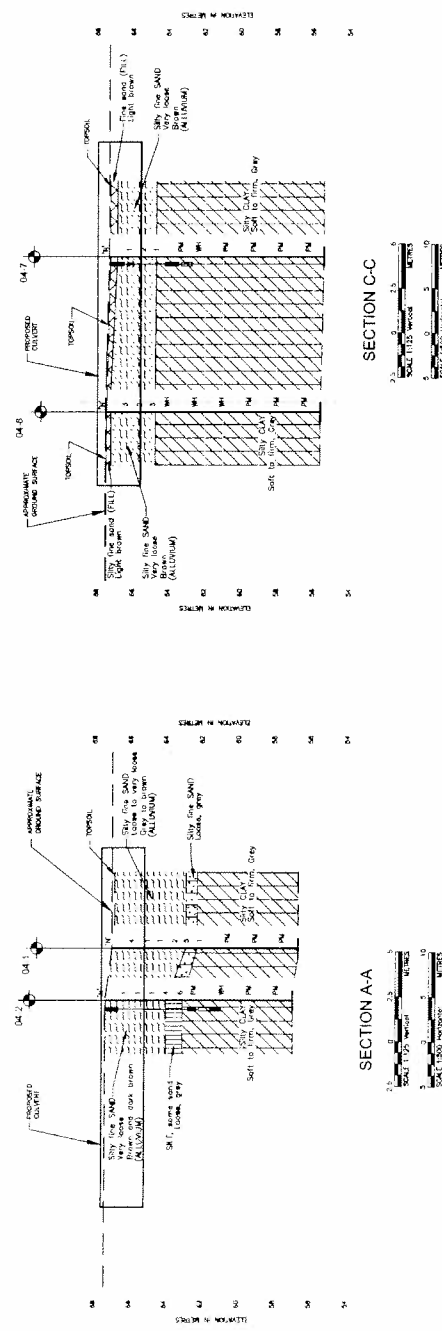
N Standard Penetration Test value
Blows/0.3m, unless otherwise stated
(Std. Pen. Test: 475 +/- blow)
100% Rock Quality Designation (RQD)
WL in piezometer (May 20, 2004)

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
04-1	67.0	501885.70	403580.09
04-2	67.4	501886.92	403580.29
04-3	68.3	501982.82	403191.44
04-4	68.0	502004.33	403345.98
04-5	68.7	502025.12	403353.51
04-6	68.6	502003.95	403359.76
04-7	67.3	502005.60	403362.04
04-8	67.5	502007.60	403362.87

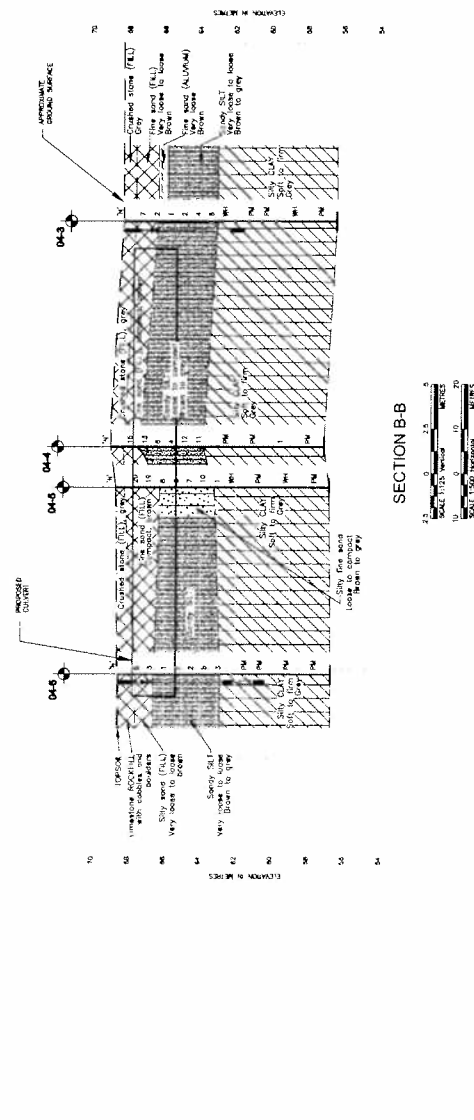
NOTES

METRIC
DIMENSIONS ARE IN METRES
UNLESS OTHERWISE SHOWN

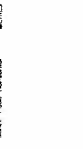
Geometric No.
PROJECT NO. 04-1, 20-013
DRAWN: J.M.C. DATE: JULY 2004
CHECKED: J.M.C. DATE: JULY 2004
REVISION



SECTION A-A

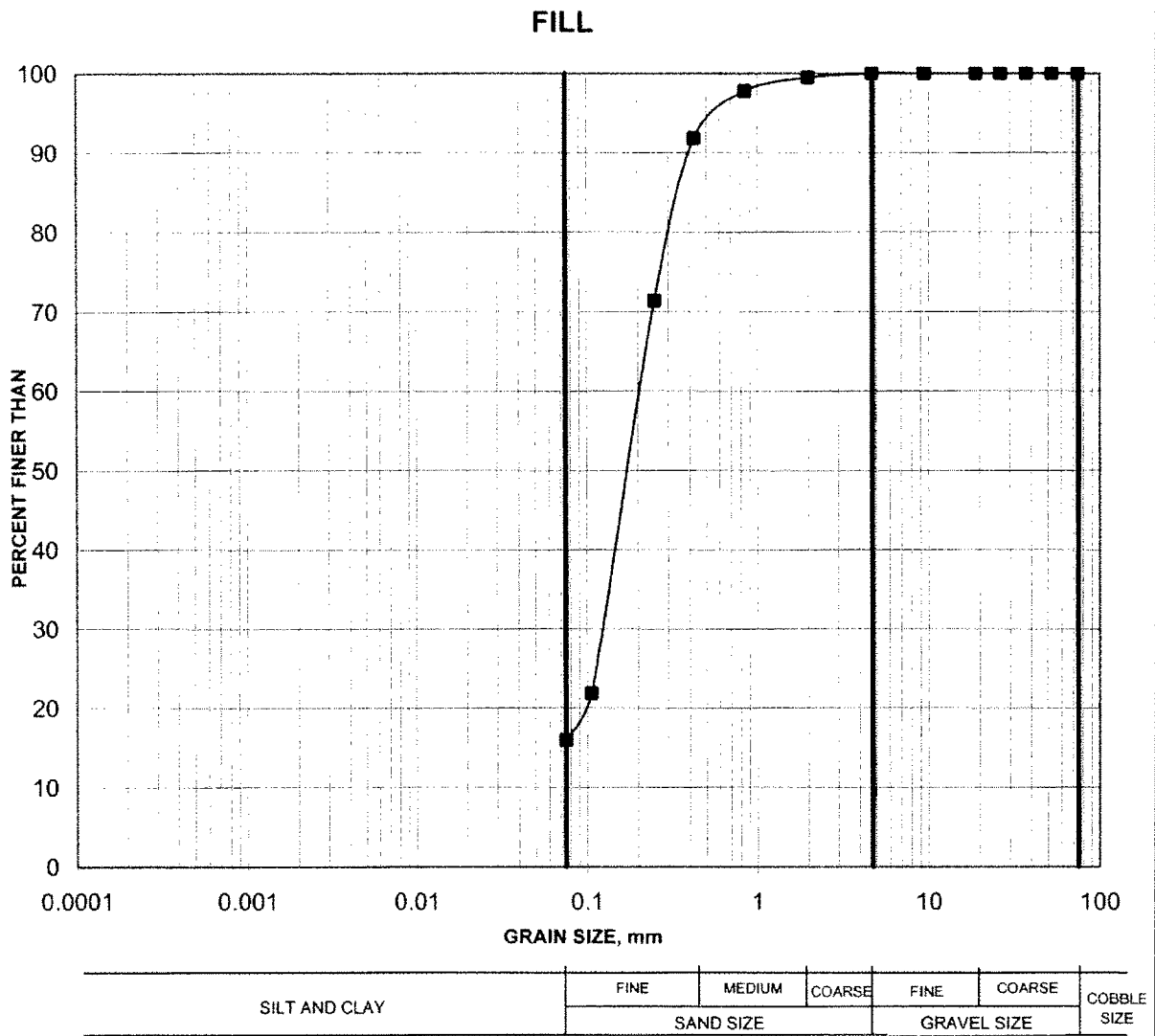


SECTION B-B



GRAIN SIZE DISTRIBUTION

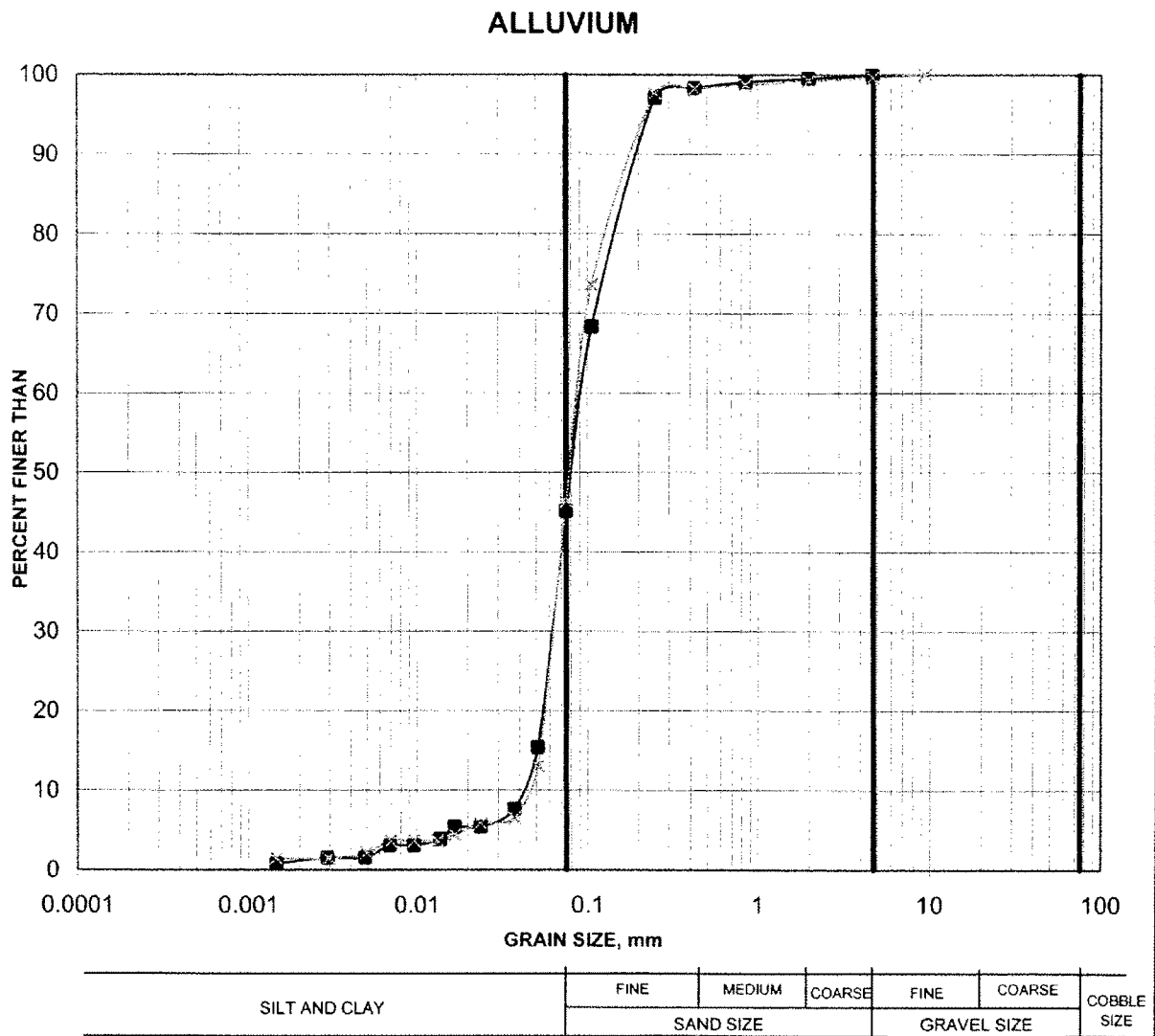
FIGURE 1



Borehole	Sample	Depth (m)
04-5	1	0.8-1.4

GRAIN SIZE DISTRIBUTION

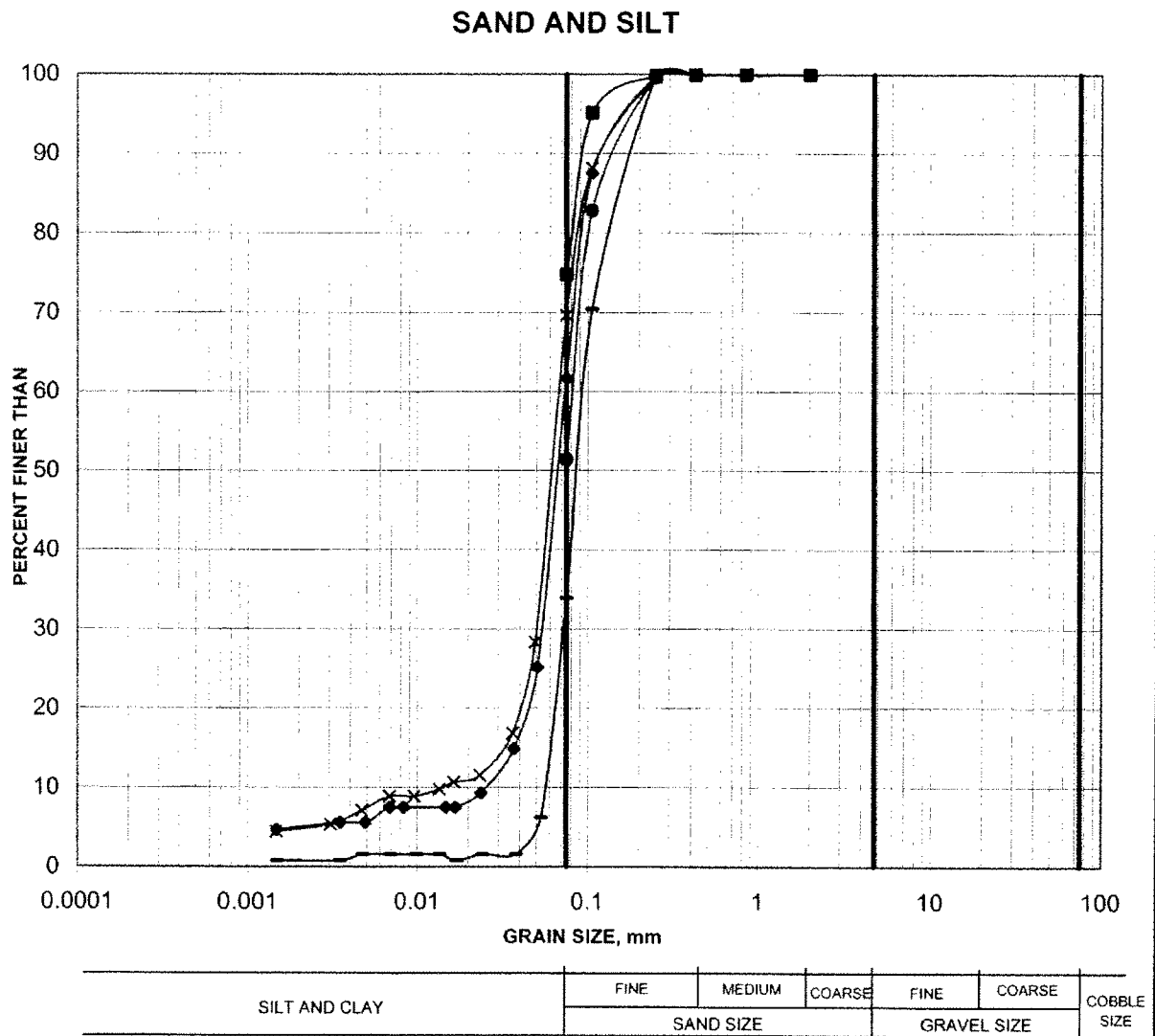
FIGURE 2



Borehole	Sample	Depth (m)
04-1	2	1.5-2.1
04-8	2	1.5-2.1

GRAIN SIZE DISTRIBUTION

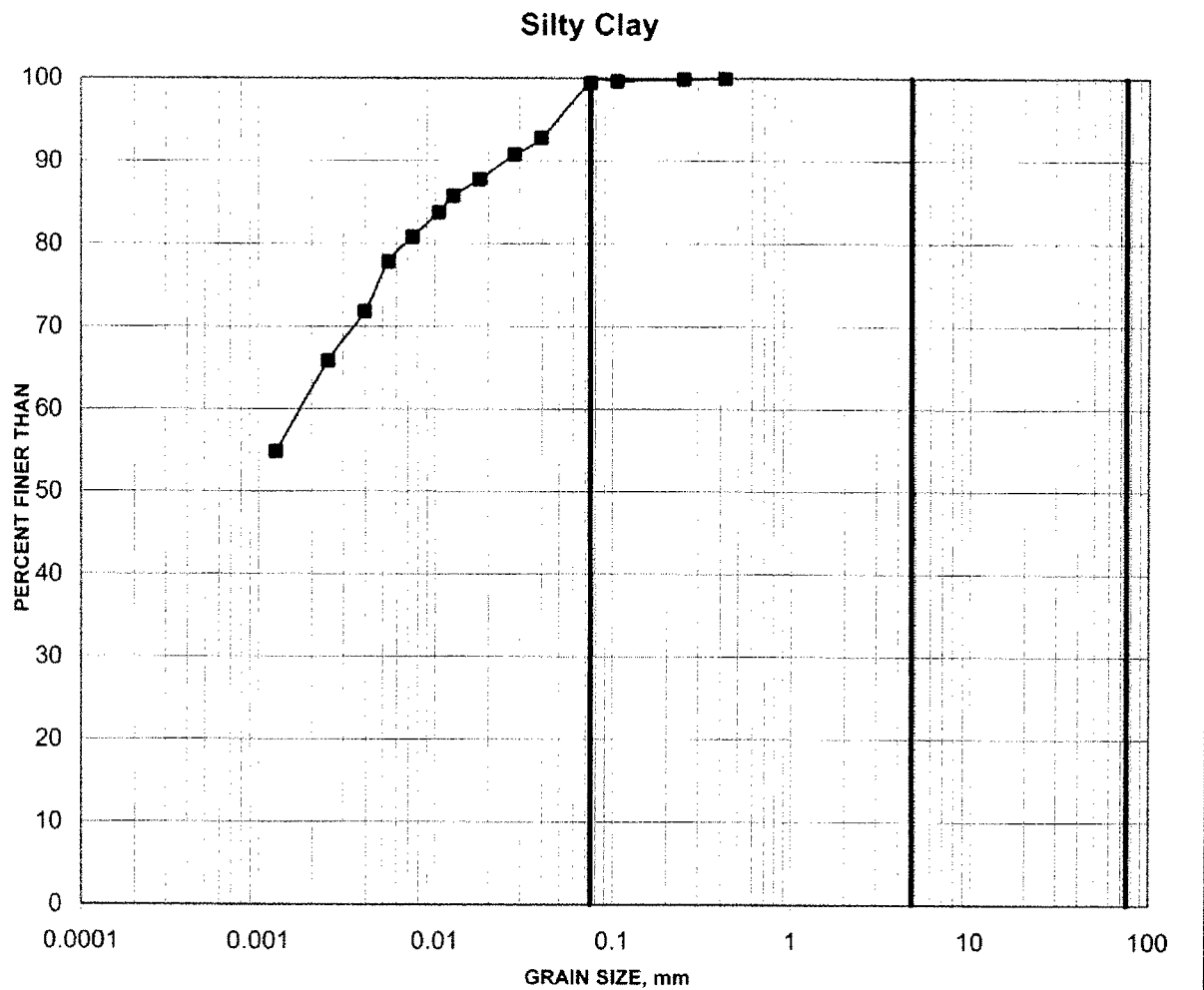
FIGURE 3



Borehole	Sample	Depth (m)
04-2	5	3.8-4.4
04-3	4	3.05-3.66
04-4	4	3.0-3.7
04-5	5	3.5-4.4
04-6	5	3.8-4.4

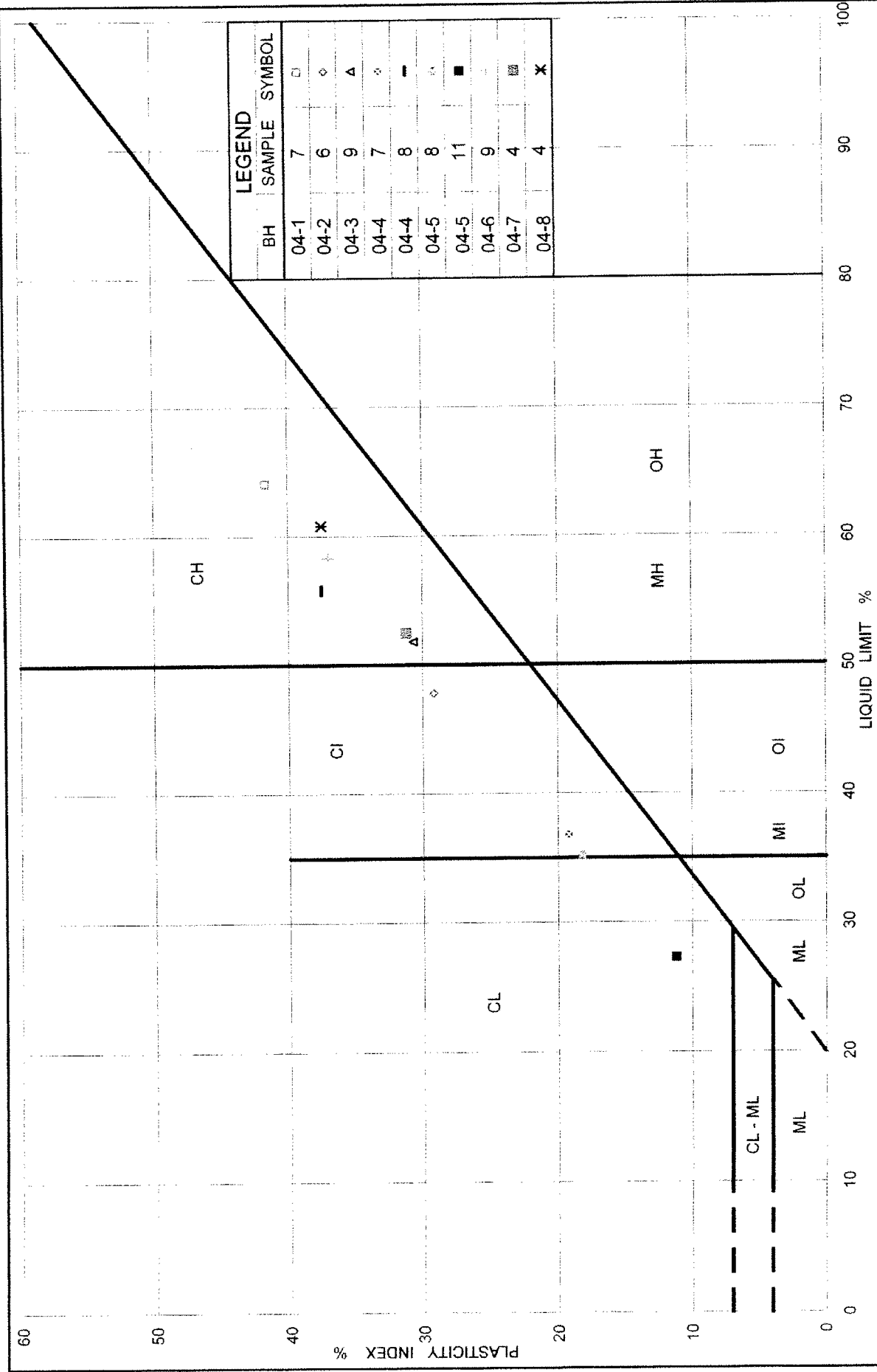
GRAIN SIZE DISTRIBUTION

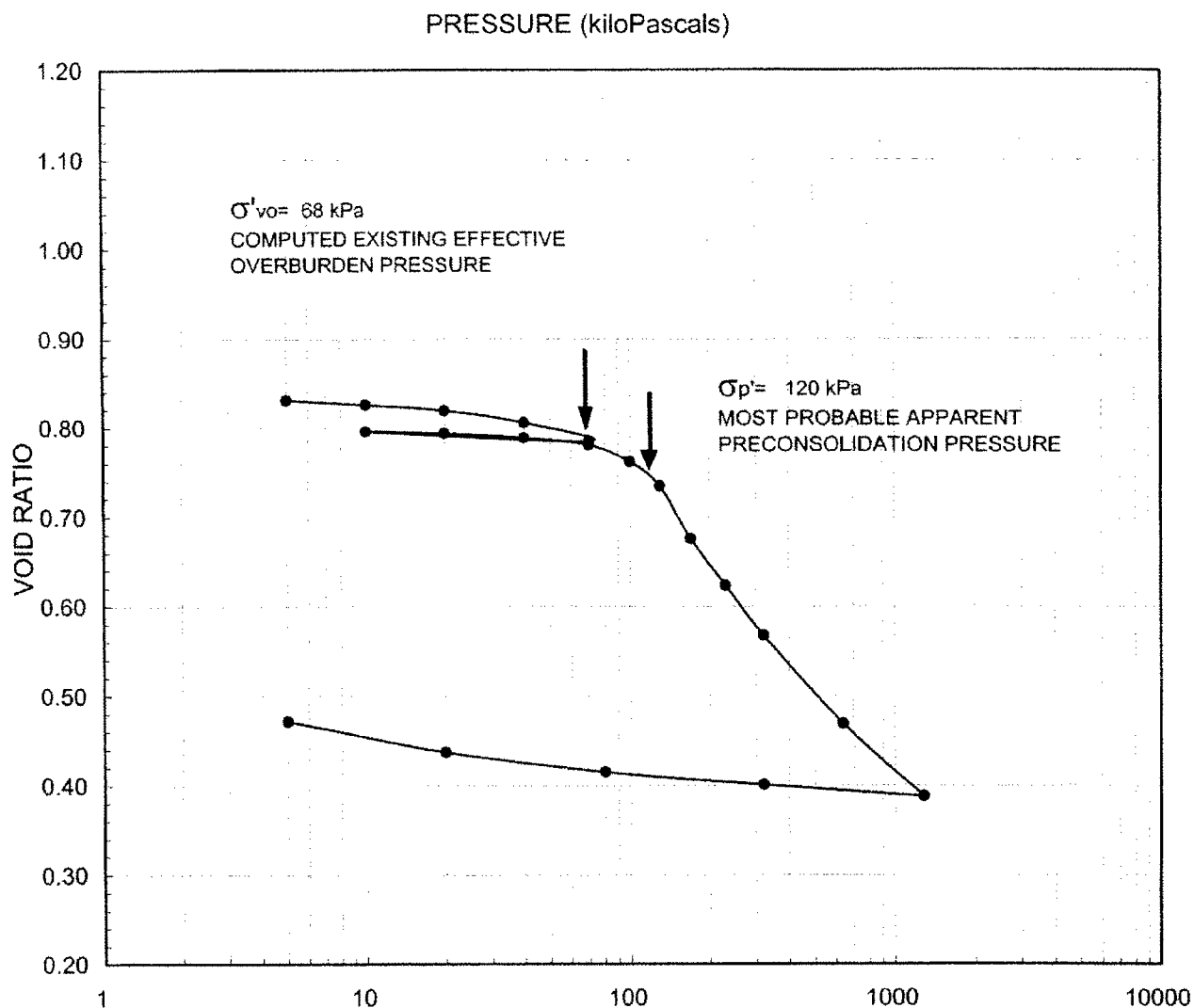
FIGURE 4



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

Borehole	Sample	Depth (m)
04-7	5	4.6-5.2





LEGEND

Borehole: 04-4	$w_i = 36.3\%$	$S_o = 110\%$
Sample: 7	$w_r = 24.3\%$	$C_c = 42.00$
Depth (m): 6.43-6.4	$w_l = 36.8\%$	$C_r = 0.019$
	$w_p = 17.6\%$	



SCALE	AS SHOWN
DATE	07/24/04
DESIGN	NA
CADD	NA
CHECK	EWK
REVIEW	JAS

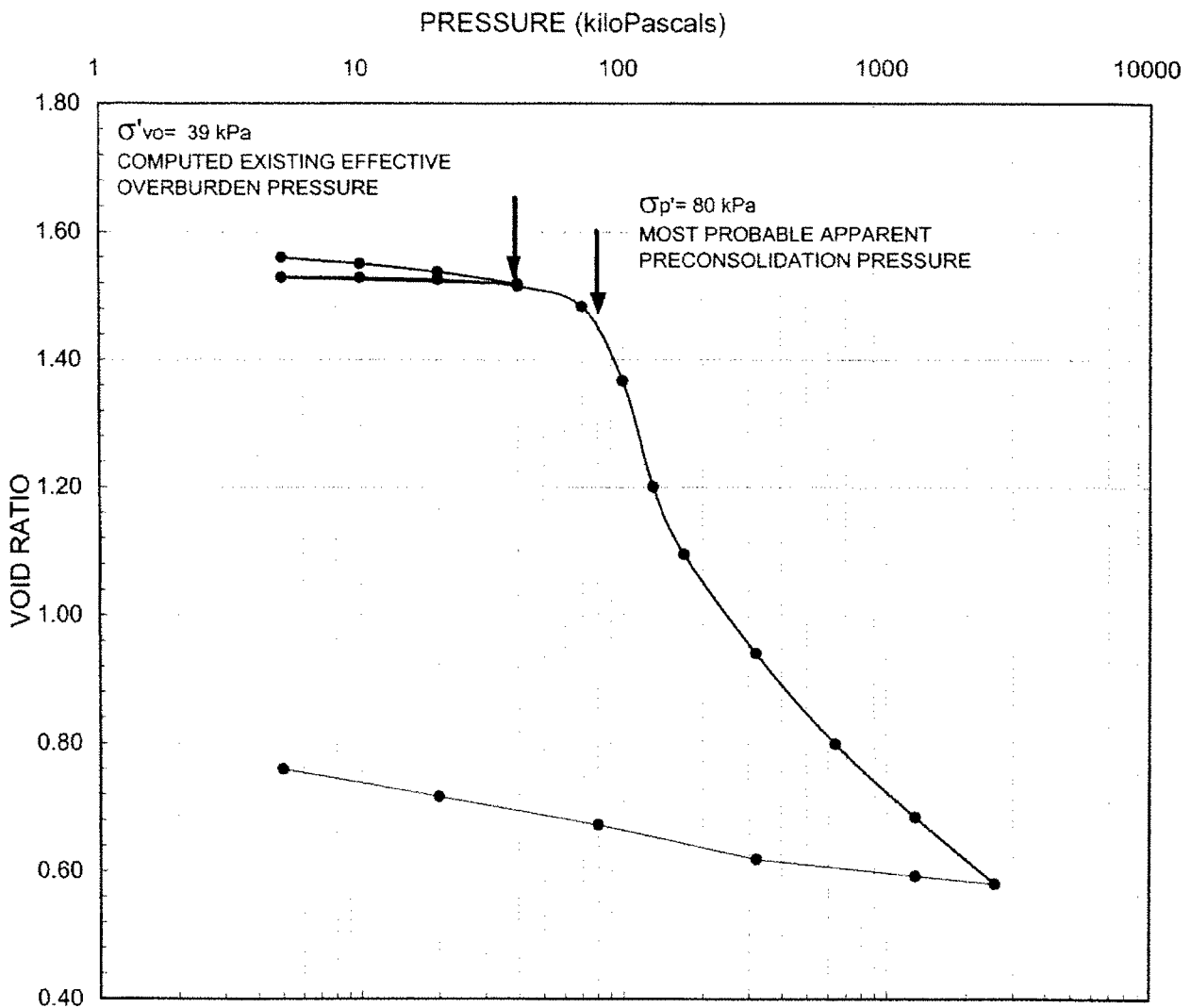
CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary

PROJECT No. 04-1120-013 REV. 0

FIGURE

6



LEGEND

Borehole: 04-7	$w_i = 56.6\%$	$S_o = 100\%$
Sample: 4	$w_f = 29.0\%$	$C_c = 1.18$
Depth (m): 4.00	$w_i = 52.5\%$	$C_r = 0.022$
Lab Vane (kPa): 31	$w_p = 21.3\%$	



SCALE	AS SHOWN
DATE	07/24/04
DESIGN	NA
CADD	NA
CHECK	EWK
REVIEW	MIC

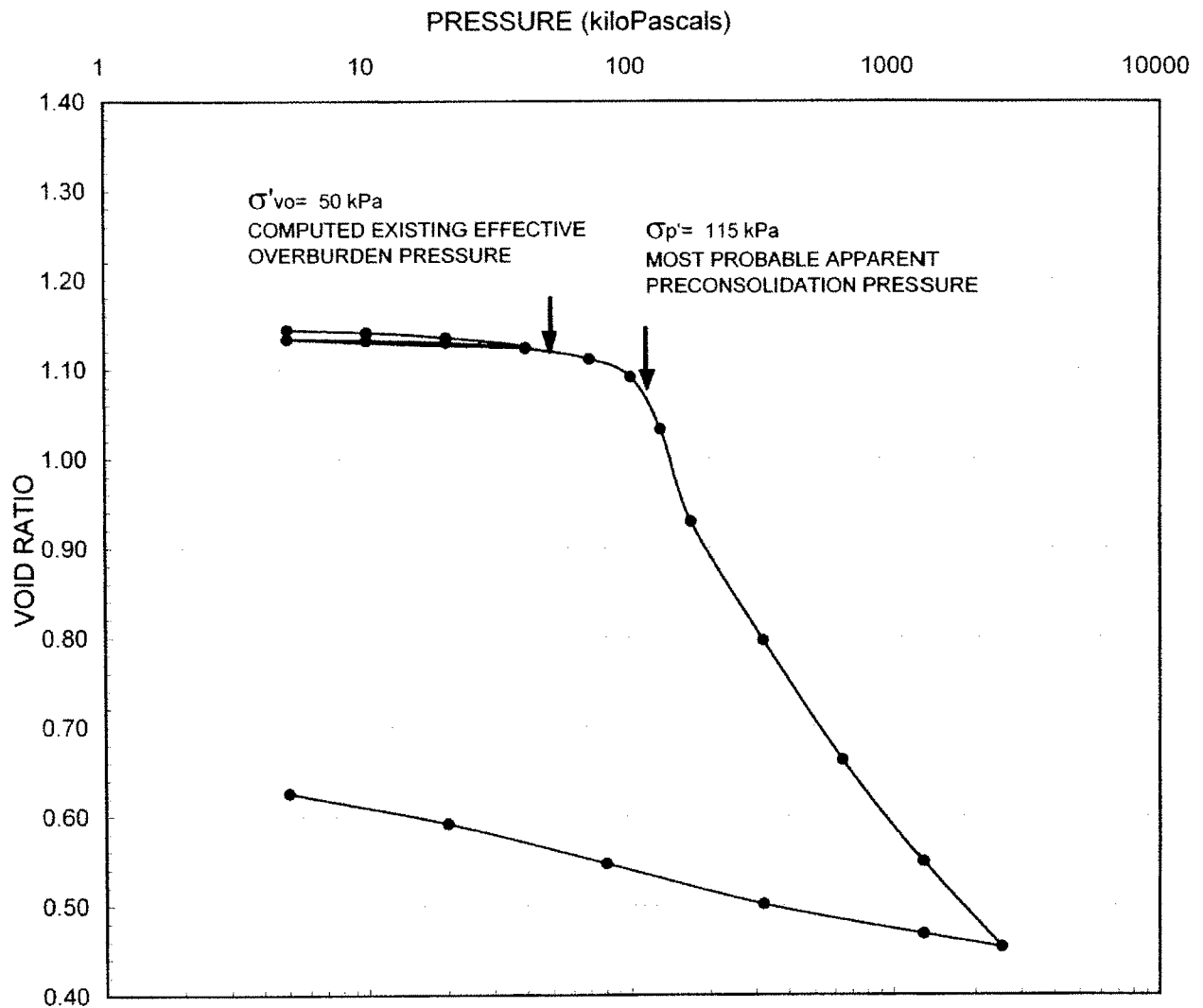
TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary
PROJECT No. 04-1120-013 REV. 0

FIGURE

7



LEGEND

Borehole: 04-2	$w_l = 44.8\%$	$S_o = 100\%$
Sample: 6	$w_r = 26.9\%$	$C_c = 0.88$
Depth (m): 5.00	$w_l = 47.8\%$	$C_r = 0.011$
	$w_p = 18.6\%$	



SCALE	AS SHOWN
DATE	09/22/04
DESIGN	NA
CADD	NA
CHECK	EWK
REVIEW	MIC

CONSOLIDATION TEST RESULTS

FILE No: Consolidation summary

PROJECT No: 04-1120-013 REV: 0

FIGURE

8

