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LOCATION Little Cataraqui Creek Culvert

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

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GEOCRES # 31C-159

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN LITTLE CATARAQUI CREEK CULVERT HIGHWAY 401 FROM 2.7 KM WEST OF HIGHWAY 38 EASTERLY TO HIGHWAY 15 DISTRICT 41, KINGSTON, EASTERN REGION WP: 77-99-01 AGREEMENT NO. 4005-A-000069

Submitted to:

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EASTERLY TO HIGHWAY 15
DISTRICT 41, KINGSTON, EASTERN REGION
WP: 77-99-01
AGREEMENT NO. 4005-A-000069

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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation at the existing Little Cataraqui Creek Culvert at Highway 401 (Site 7-154) near Kingston, Ontario. This investigation forms part of the overall project which involves the widening of Highway 401 from 2.7 km west of Highway 38 easterly to Highway 15. The foundation design component includes the placement of a recreational culvert within the CP Rail overhead opening, a new bridge structure at the Montreal Street underpass, and the extension of the existing culvert at Little Cataraqui Creek. This report addresses the proposed extension to both ends of the existing culvert at Little Cataraqui Creek.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed culvert extension by drilling boreholes, and carrying out in-situ tests and laboratory tests on selected samples, where appropriate. Based on our interpretation of the data obtained, recommendations on the foundation aspects of design of the proposed works are provided. Comments are also provided on anticipated construction problems where they may affect the design of the proposed culvert structure.

A plan view of the existing and proposed Highway 401 alignment at 1:1000 scale and a profile view of the existing Little Cataraqui Creek Culvert at 1:200 scale were provided to us by the MTO.

The terms of reference for the scope of work are outlined in our original proposal letter P01-1008, dated January 27, 2000 and our revised proposal letter P01-1008, dated March 21, 2000.

2.0 SITE DESCRIPTION

The site is located some 700 m to the west of the Sir John A. MacDonald Underpass at Highway 401, near Kingston, Ontario.

The topography of the site area is generally level with a regional trend sloping down to the south towards Lake Ontario. The existing Highway 401 is four-lane divided and runs east-west within the project limits. Based on available information, the approximate existing grade of Highway 401 at the culvert (Station 21+560) is about Elevation 85.5 m. The existing rigid frame open concrete culvert is 1.83 m high and spans 6.2 m, and is about 55 m long. The inlet area on the north side of Highway 401 is protected with a gabian wall about 3 m high which surrounds the culvert structure.

The area on the north side of Highway 401 consists of a mixture of bushes and mature trees. The area to the south of Highway 401 consists of a low wet area with vegetation cover consisting of grass and bushes. The lands in the vicinity of site are mainly agricultural to the north and residential to the south.

It is understood that no details of the drawings indicating the founding conditions of the culvert could be located in MTO records / files. The only documentation obtained was a previous structural inspection of the culvert carried out in August 1994 by MTO staff (copy of the inspection record provided in Appendix A). This record indicates that at the time of inspection cracks, spalls and severe scaling were observed at the south end of the structure and that the east side has settled. The report also indicates that the foundations component received a rating of 9, although there is no details of the foundations.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on April 18, 2000 and between April 20 and 24, 2000. At this time two boreholes were put down at the site. Boreholes 2-1 and 2-4 were put down just beyond the north-east and south-east limits of the existing culvert, respectively. Boreholes 2-1 and 2-4 were extended to depths of 21.9 m and 35.8 m below the existing ground surface, respectively.

The investigation was carried out using a track-mounted CME 55 drill rig supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario. In the boreholes, samples of the overburden were generally obtained at regular intervals of depth of 0.75 m and 1.5 m using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedures. In-situ vane testing was carried out throughout the upper cohesive deposit encountered in Borehole 2-4 where two Shelby tube samples of the deposit were also obtained. An MTO 'N' size vane was used and after determining the undrained shear strength the vane was rotated some 10 times, after a pause of 60 seconds to measure the remoulded strength. Bedrock was cored in NQ size in Borehole 2-4. The open boreholes were backfilled with bentonite mixed with some auger cuttings to provide an adequate seal in accordance with MOE requirements. Groundwater conditions in the open boreholes were observed throughout the drilling operation and upon completion of drilling. Piezometers were installed in Boreholes 2-1 and 2-4 to permit monitoring of the groundwater levels at these locations. The piezometers consisted of a 200 mm long slotted tip threaded into 12 mm diameter PVC rigid tubing.

The field work was supervised on a full-time basis by a member of our engineering staff who located the boreholes in the field, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples and bedrock core were identified in the field, placed in labeled containers and boxes, respectively, and transported to our laboratory in Mississauga for further examination. Water content, Atterberg limits, and grain size analyses were carried out on selected samples of the recovered soil. In addition, consolidation testing was carried out on two selected sections of the Shelby tube samples. Point load testing was also carried out on selected samples of the recovered rock core.

The borehole locations were surveyed and staked in the field by Transenco Ltd., who have professional land surveyors on staff. Based on the information provided, the northing and easting co-ordinates of the borehole locations are given in UTM, and the borehole elevations are referenced to Geodetic Datum. The co-ordinates of the boreholes are indicated on the Record of Borehole sheets and the locations of the boreholes are shown on Drawing 1.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

The site is located in the physiographic region of Southern Ontario known as the Napanee Plain (The Physiography of Southern Ontario, Chapman and Putnam, 3rd Edition, 1984). The overburden is typically shallow. The Napanee Plain, which is generally flat to undulating, has been stripped of most of its overburden during the late Wisconsonian glaciation period some 11,000 years ago.

Geologic mapping (Map 2544, Ministry of Northern Development and Mines, 1991) indicates the bedrock at the site consists of Phanerozoic rock of the middle Ordovocian age. The predominant bedrock type in the area is limestone of the Gull River Formation. The local bedrock is generally located at or near the ground surface. Bedrock outcrops were noted to the north and south of the existing culvert.

4.2 Site Stratigraphy

The detailed subsurface soil, bedrock, and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil and rock samples, are given on the attached Record of Borehole sheets following the text of this report. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsurface conditions will vary between and beyond the borehole locations.

In summary, the subsoils at the site generally consist of a surficial layer of topsoil or rockfill underlain by silty clay to clayey silt, which in turn is underlain by an upper sandy silt deposit and a lower sand deposit with varying amounts of silt. Bedrock was encountered at about Elevation 45.8 m on the south side of Highway 401. A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided in the following sections.

4.2.1 Topsoil

An 80 mm thick surficial layer of topsoil was encountered in Borehole 2-1.

4.2.2 Sand and Gravel Fill

About 610 mm of sand and gravel fill with rock fragments was encountered surficially in Borehole 2-4. The sand and gravel fill may be from construction of the existing Highway 401 embankment.

4.2.3 Silty Clay to Clayey Silt

A deposit of silty clay to clayey silt, trace sand, was encountered below the topsoil and sand and gravel fill in Boreholes 2-1 and 2-4 where the deposit is about 8.6 m and 8.1 thick, respectively. Occasional organics were noted in the upper 2.2 m and 4.6 m of the deposit in Boreholes 2-1 and 2-4, respectively. The organic contents of two selected samples of the upper portion of the silt / clay were measured at 2.5 percent and 4.5 percent. Grain size distribution curves for two selected samples of the silt / clay are shown on Figure 1. Atterberg limits testing carried out on selected samples of the deposit gave an average liquid limit and plasticity index of about 17.5 percent and 8 percent, respectively. The results are plotted on a plasticity chart on Figure 2. Based on these results, the upper portion of the deposit is classified as intermediate plasticity (Borehole 2-4, Sample 3) while the lower portion of the deposit is classified as low plasticity. The water contents measured for selected samples of the silty clay to clayey silt range from about 21 percent to 41 percent, with an average of about 30 percent; the water contents were generally near or above the liquid limit.

The results of the in-situ vane testing carried out within the deposit in Borehole 2-4 are summarized in the following table.

	Maximum Undrained Shear Strength (kPa)	*Minimum Undrained Shear Strength (kPa)*	*Average Undrained Shear Strength (kPa)*
Undisturbed Strength	Greater than 100	51	71
Remolded Strength	42	20	38

Standard Penetration testing (SPT) carried out within the silt / clay measured 'N' values of between 0 blows (i.e. the weight of the hammer) to 11 blows per 0.3 m of penetration. The higher 'N' values and higher in-situ shear strengths were generally measured in the upper portion of the deposit which indicates that there is a weathered crust present. The 'N' value and vane test results at about Elevation 73 m in both boreholes indicate the presence of a stronger layer within the deposit at this elevation. Based on the 'N' values and in-situ vane testing, the silty clay to clayey silt has a firm to stiff consistency.

Two consolidation tests were carried out on selected sections of the Shelby tube samples obtained from the silty clay to clayey silt deposit in Borehole 2-4. The void ratio versus log pressure and the total work versus pressure curves for each sample are shown on Figures 3, 3a, 4 and 4a. It is estimated that the difference between the existing vertical stress, P_O, and the apparent pre-consolidation pressure, P_C, is about 40 kPa for the sample at 3 m depth and about 250 kPa for the sample at 6 m depth. It should be noted that the sample at 3 m is silty clay with a high water content and at 6 m depth is clayey silt with a lower water content. The P_c values were estimated using the total work vs. pressure plots, Figures 3a and 4a. It should be noted that the upper sample (Borehole 2-4, Sample 3) has a higher organic content.

4.2.4 Sandy Silt

A deposit of sandy silt, trace clay, was encountered below the silt / clay and is about 6.1 m thick in Borehole 2-1 and 4.6 m thick in Borehole 2-4. Standard Penetration testing (SPT) carried out within the sandy silt measured 'N' values of between 0 blows (i.e. the weight of the hammer) to 22 blows per 0.3 m of penetration, which indicates a very loose to compact relative density. It should be noted that the low 'N' values measured may be a result of 'blowing' conditions which was encountered during the sampling of the boreholes. The natural water contents for selected samples of the sandy silt ranged from about 18 percent to 27 percent, with an average of about 22 percent.

4.2.5 Lower Sand

Below the sandy silt exists a lower granular deposit which ranges from silty sand to sand some silt. Grain size distribution curves for two selected samples of the sand are shown on Figure 5. Standard Penetration Testing carried out within the deposit measured 'N' values of between 1 blow to greater than 50 blows per 0.3 m of penetration, which indicates a very loose to very dense state of packing. In general, the sand is compact to very dense. The lower 'N' values may be a result of 'blowing' conditions which disturbed the deposit during drilling. The natural water contents measured on selected samples of the sand ranged from about 17 percent to 28 percent, with an average of about 21 percent.

Borehole 2-1 was terminated about 7.2 m within the sand and Borehole 2-4 fully penetrated the sand where the deposit was found to be about 19.1 m thick

A 1.8 m thick interlayer of sand and gravel, trace silt was encountered at about 29.9 m depth (Elevation 48.3 m) in Borehole 2-4. The sand and gravel is dense with an SPT 'N' value of 32 blows per 0.3 m of penetrated measured within the deposit. The water content for a selected sample of the sand and gravel was measured at about 16 percent.

4.2.6 Bedrock

Bedrock was encountered at about Elevation 45.8 m (about 32.4 m depth) in Borehole 2-4 and the borehole was advanced about 3.5 m into the bedrock by coring in NQ size. The rock core samples obtained consist of grey and black, fresh, fine to medium grained, diabasic gneiss which is part of the Pre-Cambrian basement. The measured fracture indices are less than 5 fractures per 0.3 m for the cored length. The Rock Quality Designation (RQD) measured on the core samples ranged from about 55 percent to 83 percent, indicating the rock mass is fair to good quality. The RQD values increase with depth. The rock is classified as strong to extremely strong; Grade 4 to 6, according to the Canadian Engineering Foundation Manual (CFEM, 3rd Edition, 1992). Strength testing carried out on five sections of the recovered core gave diametral point load indices of between 2.4 MPa to greater than 11 MPa.

4.2.7 Groundwater Conditions

The water level in the open boreholes was observed during and upon completion of the drilling operation. A piezometer was installed in both Boreholes 2-1 and 2-4 to permit monitoring of the groundwater level at these locations. Details of the piezometer installations and water level measurements are shown on the attached Record of Borehole sheets.

A summary of the water level monitoring results for the subject site is provided in the following table.

Borehole	*In Piezometer at Completion of Installation*		*In Piezometer May 21, 2000*	
	Depth (m)	*Elevation (m)*	*Depth (m)*	*Elevation (m)*
2-1	6.6	71.9	At ground surface	78.5
2-4	2.7*	80.9	1.8*	80.0

* above ground surface

The piezometer tip in Borehole 2-1 is within the sandy silt deposit at about Elevation 67 m. The piezometer tip in Borehole 2-4 is at Elevation 67 m and is within the sand and gravel interlayer which overlies the bedrock. The water level measurements indicate that there is artesian pressure conditions within the deep coarser granular deposits close to the bedrock surface. The main confining layer is the upper silty clay to clayey silt deposit; however, there appears to be some dissipation of pressure occurring within the sandy silt deposit. This conclusion is based on the "lower" water level as measured in Borehole 2-1 with the piezometer tip at a higher elevation in comparison to Borehole 2-4.

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

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PART B – FOUNDATION DESIGN
MONTREAL STREET UNDERPASS AT HIGHWAY 401
HIGHWAY 401 FROM 2.7 KM WEST OF HIGHWAY 38
EASTERLY TO HIGHWAY 15
DISTRICT 41, KINGSTON, EASTERN REGION
WP: 78-99-01
AGREEMENT NO. 4005-A-000069

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the geotechnical aspects of design of the proposed culvert extension at Little Cataraqui Creek, based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. 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 method and scheduling.

It is understood that Highway 401 will be widened from 2.7 km west of Highway 38 easterly to Highway 15. This widening will include the placement of a recreational culvert within the existing CP Rail overhead opening, a new bridge structure at the Montreal Street underpass, and an extension of the existing rigid frame culvert structure at the Little Cataraqui Creek crossing. The works described in this report are associated with the proposed culvert extension at Little Cataraqui Creek.

It is understood that the existing culvert at Little Cataraqui Creek will be extended at each end to accommodate the proposed highway widening of about 5 m north and south of the existing road limits. Based on the information provided, the final grade of Highway 401 at the culvert structure will remain unchanged at about Elevation 85.5 m. The existing culvert obvert is at about Elevation 79.5 m on the north side and Elevation 79.4 m on the south side.

5.2 Background Information

As discussed previously, it is understood that there are no records in MTO files with respect to the founding conditions or the type of the culvert. Based on the subsurface conditions as established at the site, it is unlikely that a 6.1 m x 1.83 m concrete open footing culvert is supported on spread footings. This statement is made based on the relatively low 'N' values and vane strengths at shallow depth together with the organic nature of the upper portion of the silty clay / clayey silt deposit which extends to about 8.7 m depth.

The terms of reference for this project indicates that the existing concrete culvert is an open footing structure without any details of the foundation support. However special efforts were made in July 2000 to ascertain the foundation support of the existing culvert. At this stage we developed three shallow foundation options for the proposed culvert extension with the assumption that the existing culvert is a rigid frame open footing culvert. In addition, a fourth option was also developed supporting the culvert on end bearing steel H-piles. In August 2000, the MTO, Eastern Region Structural Section informed us, based on their recent site inspection, that the existing culvert is not an open footing culvert as indicated on their previous plans and inspection reports. Their recent site visit concluded that the existing culvert is a rigid frame box culvert without any signs of distress nor any instability of slopes of the existing rockfill embankment.

The geotechnical investigation carried out for the pavement design included the following three holes:

Station 12+500 (near culvert)	-	480 mm	asphalt (total)		
	-	120 mm	"A"		
	-	400 mm	sand cushion over rockfill (embankment)		
Station 21+562 (eastbound passing lane)	-	360 mm	asphalt (total)	comprised of:	
	-			1 lift HL 1	} }extra asphalt
				1 lift HL 4	}
	-			1 lift HL 1	
	-			2 lifts HL 4	
Station 21+628	-	390 mm	asphalt (total)		

The pavement structure in the area of the culvert has apparently been performing well over the last ten years which may indicate that any embankment settlement which occurred probably took place over the first twenty-years following construction. The above pavement structure thicknesses encountered in the geotechnical investigation are indicative of settlement of the highway embankment where constructed over the silty clay / clayey silt deposit. Considering a design asphalt thickness of 270 mm for this major highway, the extra asphalt thickness suggests that there may have been some 200 mm of settlement near the culvert location. A widened embankment to accommodate the culvert extension will induce additional settlements.

5.3 Culvert Foundation Options

The shallow subsoils at this site consist of a firm to stiff clay to clayey silt deposit underlain at 8.6 m depth by loose to compact sandy sit. The upper 2 m to 4.5 m of the silty clay / clayey silt deposit contains up to 5 percent organics. Given these conditions in combination with the known founding conditions of the existing culvert, it is recommended that the widened portion of the culvert be supported on shallow foundations. The allowable bearing pressure for shallow foundations would be governed by the settlement of the cohesive deposit under the widened embankment loading and there would be differential settlement between the existing culvert and the widened portion. The magnitude of total and differential settlement would be dependant on the type of embankment material over the culvert extension.

The final choice is dependent upon the magnitude of settlements beneath the extension and the existing culvert. The options to be considered for the shallow and deep foundation support for the culvert extension are as follows:

Shallow Foundations

Option I: Embankment widening with rockfill
Option II: Embankment widening with light-weight slag (Litex 149)
Option III: Embankment widening with polystyrene blocks

Deep Foundations

Option IV: Embankment widening with rockfill

The above mentioned embankment options with appropriate foundation support for the extension were previously discussed in our facsimile transmissions dated July 27, 2000 July 31, 2000, August 31, 2000 and November 08, 2000.

Based on the preceding discussion, the options for the widening of the culvert are summarized in details as follows:

Option I

The rockfill in embankment widening construction sequence should be carried out as shown on Figure 6. In the upper 2 m to 4.5 m of silty clay to clayey silt deposit there was some minor organic inclusion. In order to provide adequate support for the culvert foundation, the foundation level in this deposit should be inspected. If organic sections are encountered they should be sub-excavated and replaced with compacted granular material. The anticipated settlements for the rockfill option are shown on Figure 6. It should be noted settlements do occur underneath the existing culvert due to the superimposed load of the widened rockfill embankment. A construction joint should be provided in order to accommodate these settlements.

For foundation design purposes a factored resistance at ULS of 250 kPa may be used. The geotechnical resistance at SLS for an equivalent loading (rockfill and culvert) of 90 kPa and the associated deformations are given on Figure 11 for design purposes.

Option II

In this option the light-weight water cooled blast furnace slag (Litex 143) Type I material will be used in place of rockfill for the widening of the embankment. The recommendations for Option I are equally applicable with regard to sub-excavation and design values at ULS. The geotechnical resistance at SLS for an equivalent loading (light-weight fill and culvert) of 75 kPa and the associated deformations are given on Figure 11.

A granular separator to minimize loss of fines into the rockfill is required as shown on Figures 7 and 8. Special compaction measures are required for the placement of slag material for the embankment widening. Particle breakage or crushing of particles will occur during compaction if slag materials are over-compacted. Therefore, careful construction control is required to achieve adequate compaction without crushing. The slag material should be placed in loose lifts of 300 mm and compacted by eight passes of a manually guided tamper such as Bomag BPR 30/38 or equivalent in accordance with OPSS 206.07. The estimated costs for the supply and placement of slag will be in the order of $70 cu/m to $80 cu/m.

Option III To minimize settlements at the base of the culvert, the widening of the embankment can be constructed using polystyrene blocks as shown on Figures 9 and 10. For this option sub-excavation / backfilling requirements will be similar to Option I. The anticipated settlements will be negligible consequently a construction joint is not necessary. The factored ULS values given for Option I can be used. The estimated cost for the supply and installation of the polystyrene blocks would be in the order of \$100 to \$110 per cu.m. A 1.0 m thick earth cover will be necessary on the slope to protect styrofoam blocks. The subgrade and pavement structure of at least 1.0 m thickness will provide adequate thickness to prevent icing on the travelled portion of the highway. The existing rockfill slope should be trimmed in order to install the polystyrene blocks as shown on Figure 9.

Deep Foundations

Option IV For this option end bearing steel 'H' piles would be driven to practical refusal within the very dense granular deposit or to the bedrock surface. The pile capacities for a HP 310 x 100 for the north and south extension are given on Figure 12. Since the existing culvert is a rigid frame box, not an open footing culvert, this option is not a viable option.

The three shallow foundation options are equally suitable from the geotechnical point of view. The final selection of the option, however, depends upon the structural design considerations that can accommodate the anticipated settlements beneath the existing and the extension of the culvert. The settlements for the polystyrene blocks (Option III) will be negligible but the material costs may be high. With regard to costs, the least expensive option is rockfill (Option I) for the widening of the embankment, however, the anticipated settlements will be somewhat larger compared to the light-weight fill option (Option II). The duration for the settlements for rockfill (Option I) and light-weight fill (Option II) will be in the order of 18 to 24 months.

It is understood wing walls may be required at the entrance and exist of the box culvert. These wing walls will have to be supported either on shallow or deep foundations depending upon the loading requirements. These wing walls have to be designed to resist the backfill and sloping surcharge pressures behind the walls. Further, some differential settlements can be anticipated between the wing wall and the culvert depending upon the choice of shallow or deep foundation support for the wings walls. In addition, dewatering and shoring requirements will be necessary for the construction of the wing walls. In view of this, it is more practical and economical to have a longer culvert extension to satisfy the design requirements. A reduced length of the extension is possible provided such length satisfies scour and hydrological requirements.

A suitable construction joint will be necessary to accommodate the anticipated settlements. A non-rigid connection between the extension and the existing culvert will allow the design to accommodate the anticipated settlements. However, a rigid connection that can tolerate the anticipated settlements is equally suitable. It should be noted that depending upon the option selected, the existing culvert will undergo further settlements due to the superimposed loads of widened embankment.

The bedding and levelling pad for the culvert extensions should be as per current Ministry's standards OPSD 803.010. The materials discussed for the three options are suitable to satisfy backfill requirements behind the culvert. A clay seal of minimum thickness of 0.3 m will be required for the embankment constructed with granular material at the upstream or inlet side of culvert. This seal should be keyed into the natural subsoil and extend to a minimum horizontal distance of 2.0 m on either side of the culvert and extend to high water level. The material for clay seal shall be as per Ministry's standard specification OPSS 1205.

Adequate scour protection should be provided at both ends of the culvert, consistent with stream flow volume and velocity. Care should be taken to prevent creek water entering the foundation excavation of the culvert extensions. Some form of creek diversion to divert the flow around the work area may be necessary to facilitate construction.

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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
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

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

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

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

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils

Consistency	c_u, s_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

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

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

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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 (con't.)

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)

(c) 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

(d) Consolidation (one-dimensional)

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

(e) Shear Strength

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

Notes: 1. $\tau = c' + \sigma' \tan \phi'$
2. Shear strength = (Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

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

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

Description and Notes

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

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
II - Parallel To	
$\perp$ - Perpendicular To	

Form...G.A.-R-3

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 2-1

METRIC

PROJECT 001-1119
W.P. 77-99-01
DIST 41 HWY 401
DATUM Geodetic
LOCATION N 4903877.06; E 302550.33
BOREHOLE TYPE 108mm I.D. Hollow Stem Augers
DATE April 18, 2000
ORIGINATED BY SB
COMPILED BY DKB
CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
78.47	GROUND SURFACE					
78.39 / 0.08	TOPSOIL					
	Silty Clay to Clayey Silt, trace sand, occ. organics to 2.2m depth Firm to stiff Brown becoming grey at 0.7m depth Moist Organic content of Sample 2 = 2.2%	1	SS	2	78	
		2	SS	5	77	
		3	SS	WH		
		4	SS	5	76	
		5	SS	6	75	0 2 77 21
		6	SS	7	74	
		7	SS	11	73	
		8	SS	3	72	
		9	SS	1	71, 70	
69.78 / 8.69	Sandy Silt, trace clay Very loose Grey Wet					
		10	SS	3	69	
		11	SS	1	68, 67	
		12	SS	WH	66	
		13	SS	WH	65, 64	
63.69 / 14.78						

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

40.6

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 2-1

METRIC

PROJECT 001-1119

W.P. 77-99-01 LOCATION N 4903877.06; E 302550.33 ORIGINATED BY SB

DIST 41 HWY 401 BOREHOLE TYPE 108mm I.D. Hollow Stem Augers COMPILED BY DKB

DATUM Geodetic DATE April 18, 2000 CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)
	— CONTINUED FROM PREVIOUS PAGE —					
	Silty Sand to Sand, some silt Dense to very dense Grey and brown Wet	14	SS	107	63	o
					62	
		15	SS	127	61	
		16	SS	34	60	o
					59	
		17	SS	78	58	
	END OF BOREHOLE				57	
56.52 21.95	END OF CONEHOLE					

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 (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

Note:
1. Water level measured in piezometer at 6.6m depth (El. 71.9m) upon completion of installation.
2. Water level measured in piezometer at ground surface (El. 78.5m) on May 21,2000.

+³, ×³: Numbers refer to Sensitivity

O ³% STRAIN AT FAILURE

ON_MOT 001-1119.GPJ ON_MOT.GDT 19/6/00

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 2-4

METRIC

PROJECT 001-1119

W.P. 77-99-01

DIST 41 HWY 401

DATUM Geodetic

LOCATION N 4903807.92; E 302523.73

BOREHOLE TYPE 108mm I.D. Hollow Stem Augers

DATE April 20-24, 2000

ORIGINATED BY SB

COMPILED BY DKB

CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
78.18	GROUND SURFACE				
0.00	Sand and Gravel with Rock fragments (Rockfill)				
77.57 / 0.61	Silty Clay to Clayey Silt, trace sand, occ. organics to 5.2m depth; Firm to stiff; Grey; Moist; Organic content of Sample 3 = 4.5%	1	SS	9	
		2	SS	9	
		3	75 TO	PH	
		4	SS	2	
		5	75 TO	PH	0 0 70 30
		6	SS	4	
69.49 / 8.69	Sandy Silt, trace clay; Loose to compact; Grey; Wet	7	SS	8	
		8	SS	4	
		9	SS	22	
64.92 / 13.26	Silty Sand to Sand, some silt; Very loose to compact; Grey and brown; Wet	10	SS	6	

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 (%): 10 20 30

UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 78 77 76 75 74 73 72 71 70 69 68 67 66 65 64

Continued Next Page

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

RECORD OF BOREHOLE No 2-4

METRIC

PROJECT 001-1119 | W.P. 77-99-01 | DIST 41 | HWY 401

LOCATION N 4903807.92; E 302523.73

BOREHOLE TYPE 108mm I.D. Hollow Stem Augers

DATUM Geodetic | DATE April 20-24, 2000

ORIGINATED BY SB | COMPILED BY DKB | CHECKED BY ASP

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty Sand to Sand, some silt Very loose to compact Grey and brown Wet	11	SS	4	63	
		12	SS	17	62	0 81 19 0
		13	SS	3	60	
		14	SS	21	59	
		15	SS	15	57	
		16	SS	29	55	
		17	SS	22	54	
		18	SS	1	52	0 76 24 0
		19	SS	4	51	
48.31					49	

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 (%) 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 2-4

METRIC

PROJECT 001-1119

W.P. 77-89-01

DIST 41 HWY 401

DATUM Geodetic

LOCATION N 4903807.92; E 302523.73

BOREHOLE TYPE 108mm I.D. Hollow Stem Augers

DATE April 20-24, 2000

ORIGINATED BY SB

COMPILED BY DKB

CHECKED BY ASP

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT w (%)
29.87	Sand and Gravel, trace silt Dense Brown Wet	20	SS	32	~17
46.48 31.70	Silty Sand Compact Grey and brown Wet	21	SS	24	
45.79 32.39	Fresh, grey and black, weakly foliated, fine-medium grained, strong DIABASIC GNEISS. Some chloritization. Pre-cambrian basement. Bedrock cored from 32.58m to 35.84m depth For bedrock coring details, refer to Record of Drillhole 2-4.				
42.34 35.84	END OF HOLE Note: 1. Water level measured in piezometer at 2.7m above ground surface (El. 80.9m) upon completion of installation. 2. Water level measured in piezometer at 1.8m above ground surface (El. 80.0m) on May 21,2000.				

Elevation scale: 48, 47, 46, 45, 44, 43

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30); 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

ON_MOT 001-1119.GPJ ON_MOT.GDT 16/6/00

PROJECT: 001-1119

RECORD OF DRILLHOLE: 2-4

LOCATION: N 4903807.92; E 302523.73

INCLINATION: -90°   AZIMUTH: —

DRILLING DATE: April 25, 2000

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DATUM: Geodetic

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
			46.62						
33	NQ Core	Fresh, grey and black, weakly foliated, fine-medium grained, strong DIABASIC GNEISS. Some chloritization. Pre-cambrian basement.	32.56	1	0.5	100	J,R,UE J,R,UE J,R,UE	1.1	
34				2	0.4	100	J,R,W J,R,PL J,R,PL	1.1 1.1	
35			42.34 / 35.84	3	0.5	100			
36		END OF HOLE							

Column headings: RECOVERY (TOTAL CORE %, SOLID CORE %), R.Q.D. %, FRACT. INDEX PER 0.3, DIP w.r.t. CORE AXIS, HYDRAULIC CONDUCTIVITY k, cm/sec, SYMBOLIC LOG

Depth scale: 33, 34, 35, 36, 37, 38, 39, 40, 41, 42

DRILLHOLE 1119ROCK.GPJ GLDR_CAN.GDT 16/6/00 PS

DEPTH SCALE

1 : 50

Golder Associates

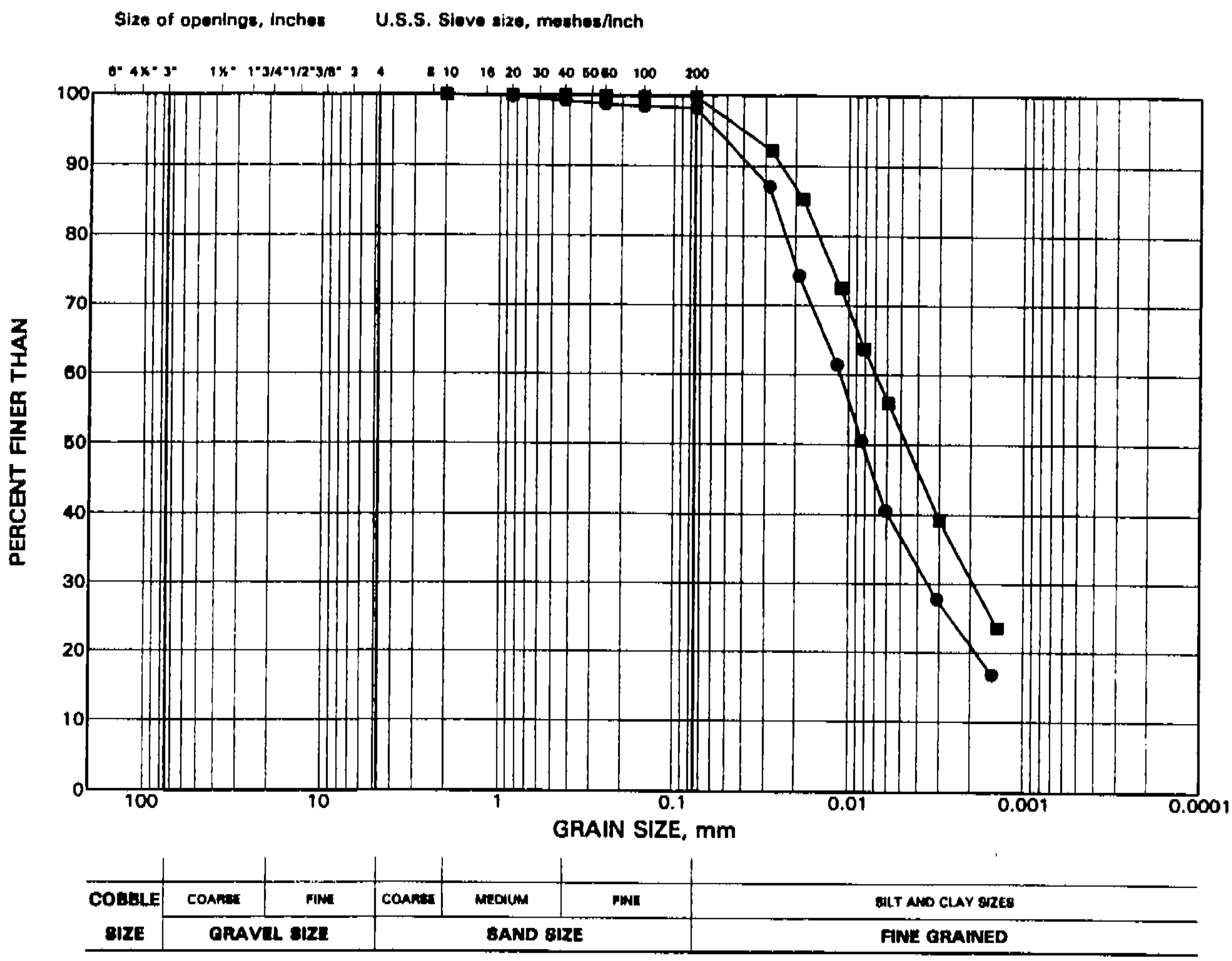
LOGGED: SB

CHECKED: MR

OVERSIZE DRAWING

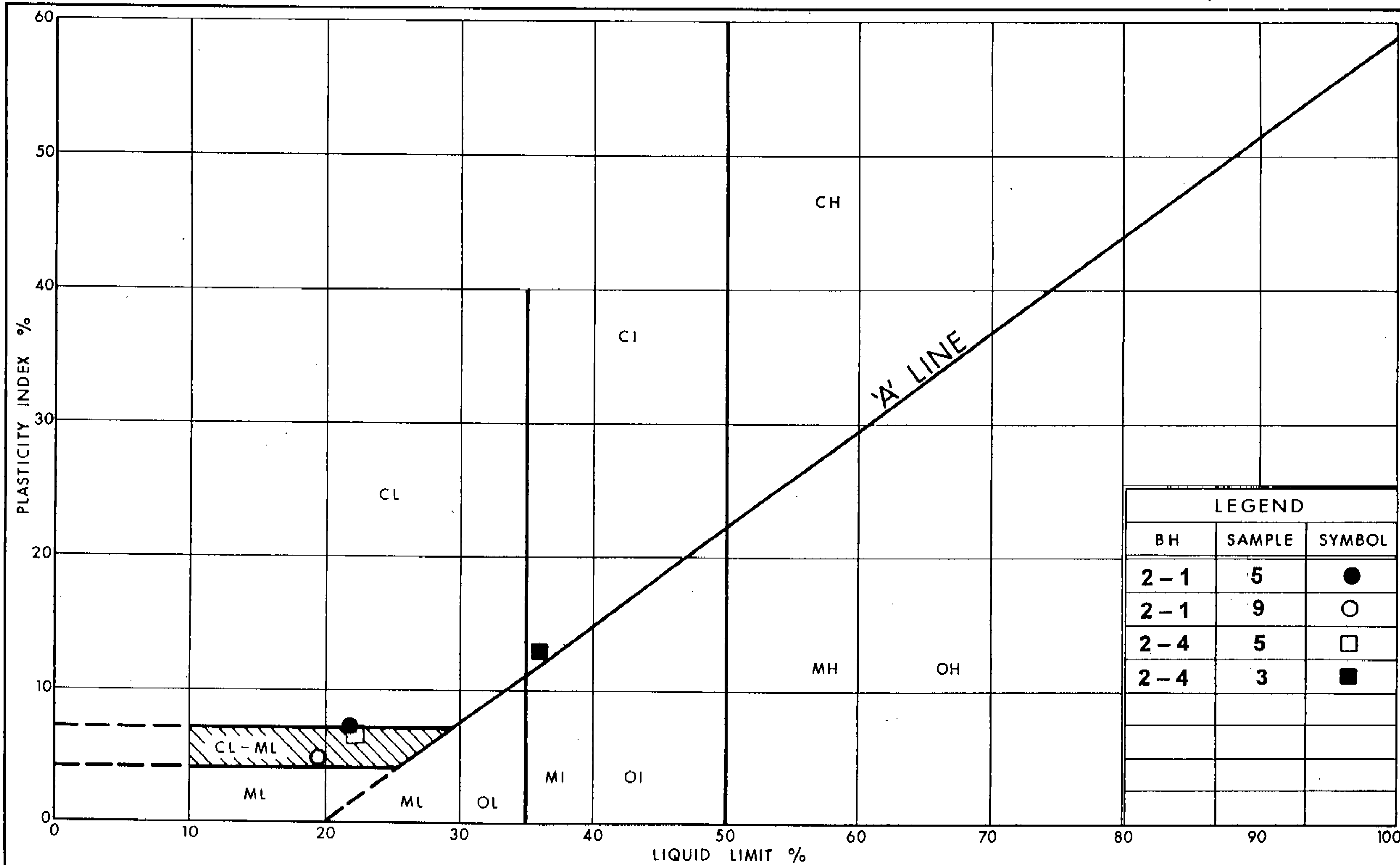
GRAIN SIZE DISTRIBUTION
Clayey Silt to Silty Clay, trace sand

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	2-1	5	75.1
■	2-4	5	71.8



Ministry of Transportation
Ontario

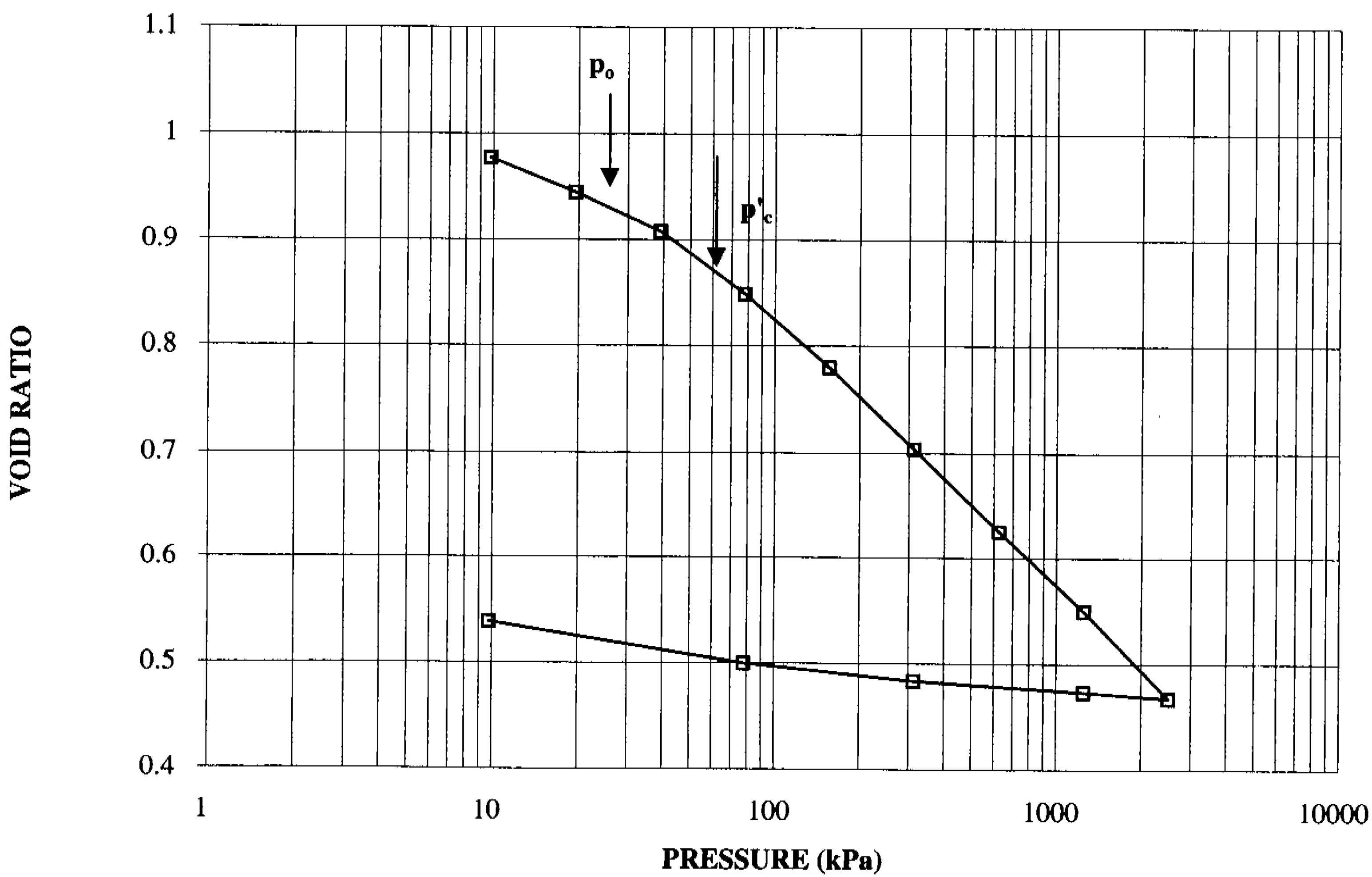
PLASTICITY CHART

CLAYEY SILT AND SILTY CLAY

FIG No 2

W P 77 – 99 – 01

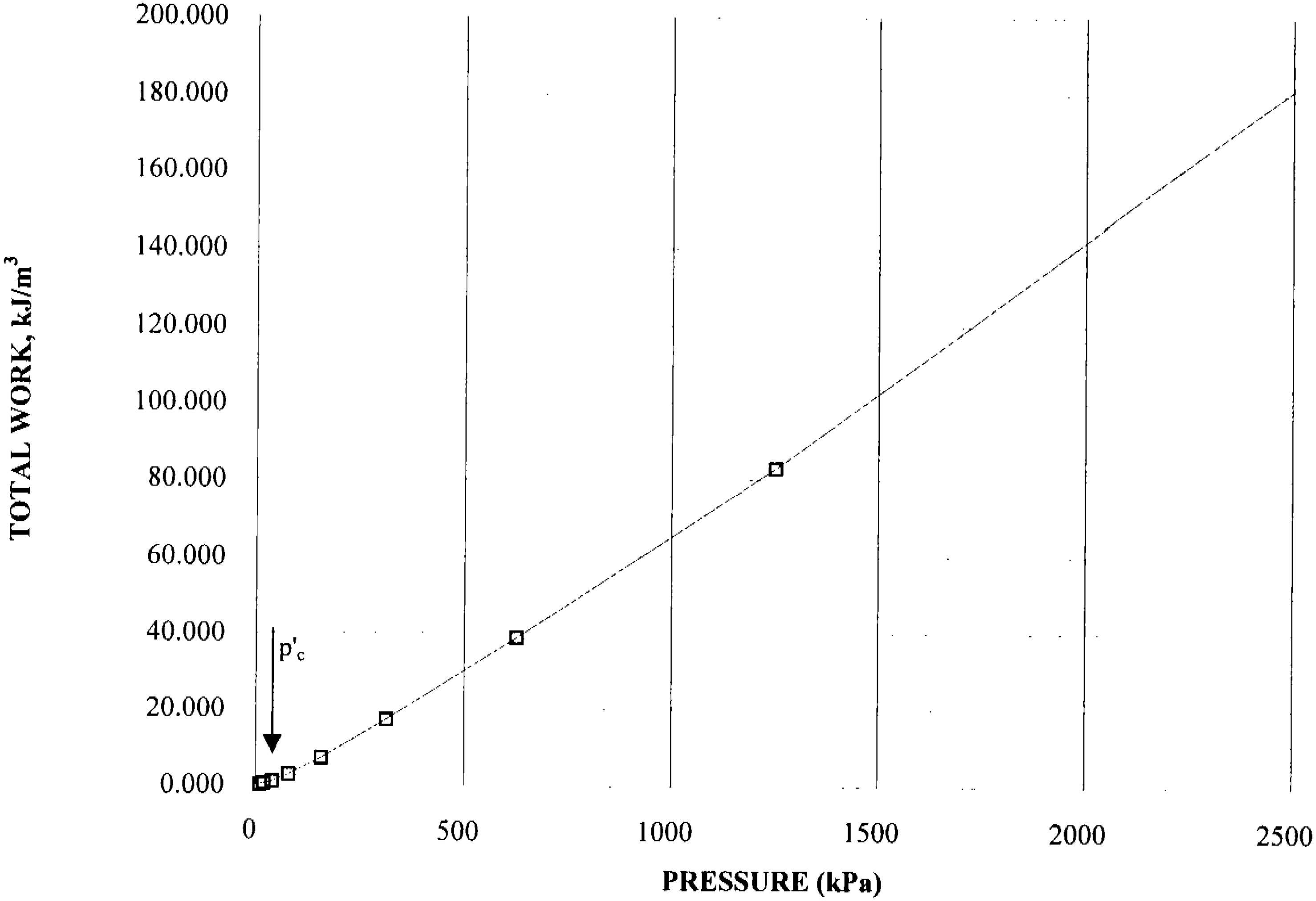
CONSOLIDATION TEST
VOID RATIO vs LOG. PRESSURE
BH 2-4 SA 3



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 3

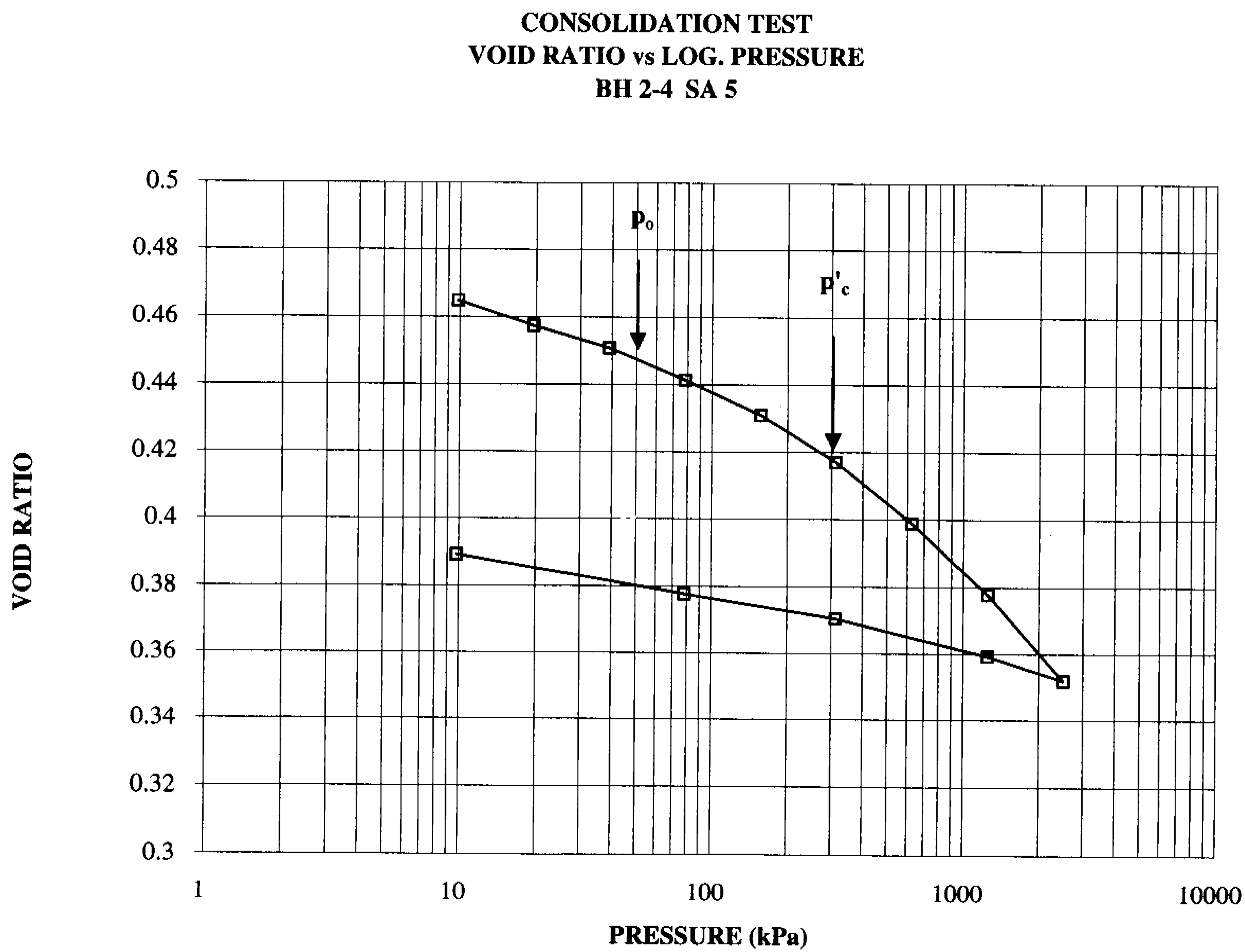
CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 2-4 SA 3



note: for consolidation test summary, refer to Appendix B

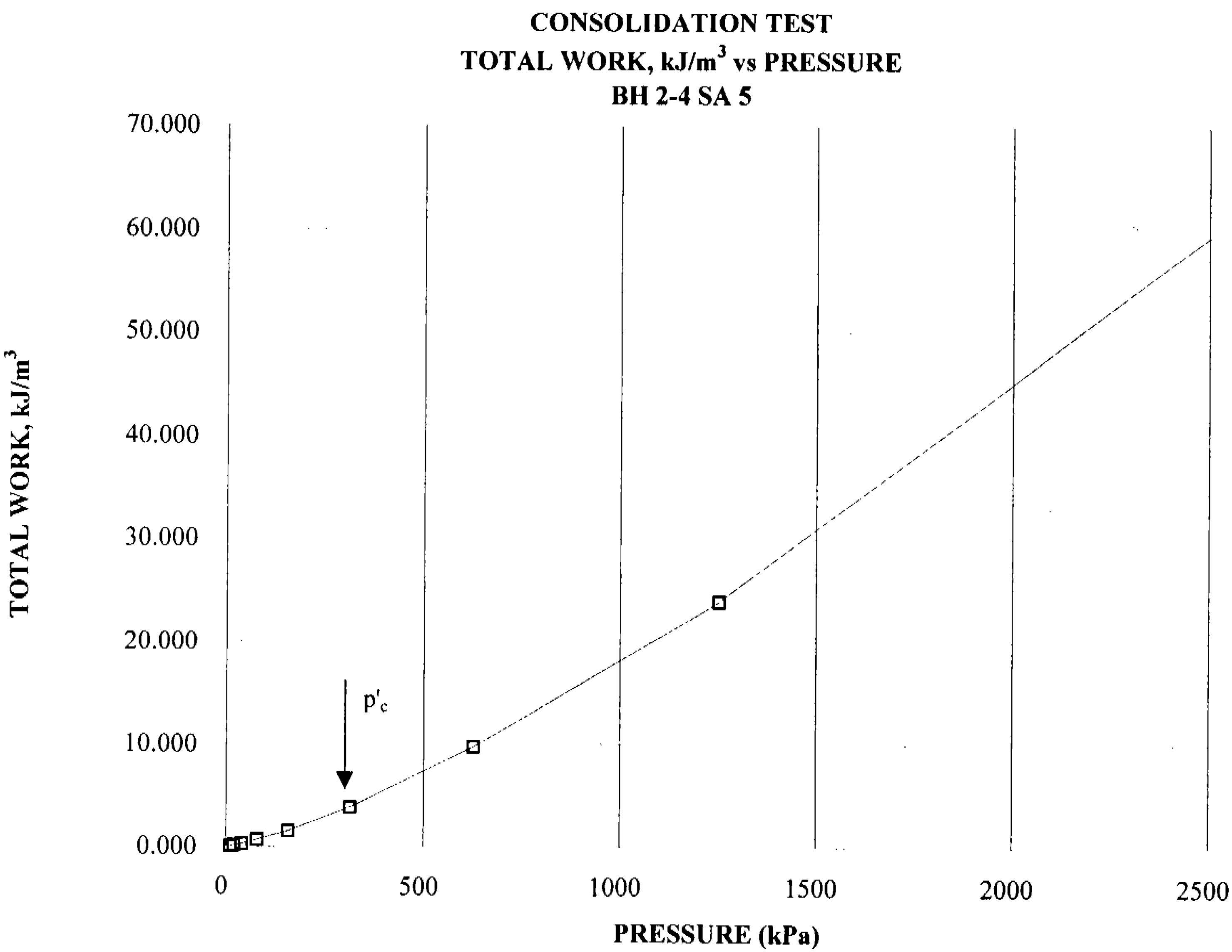
CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE

FIGURE 3A



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 4



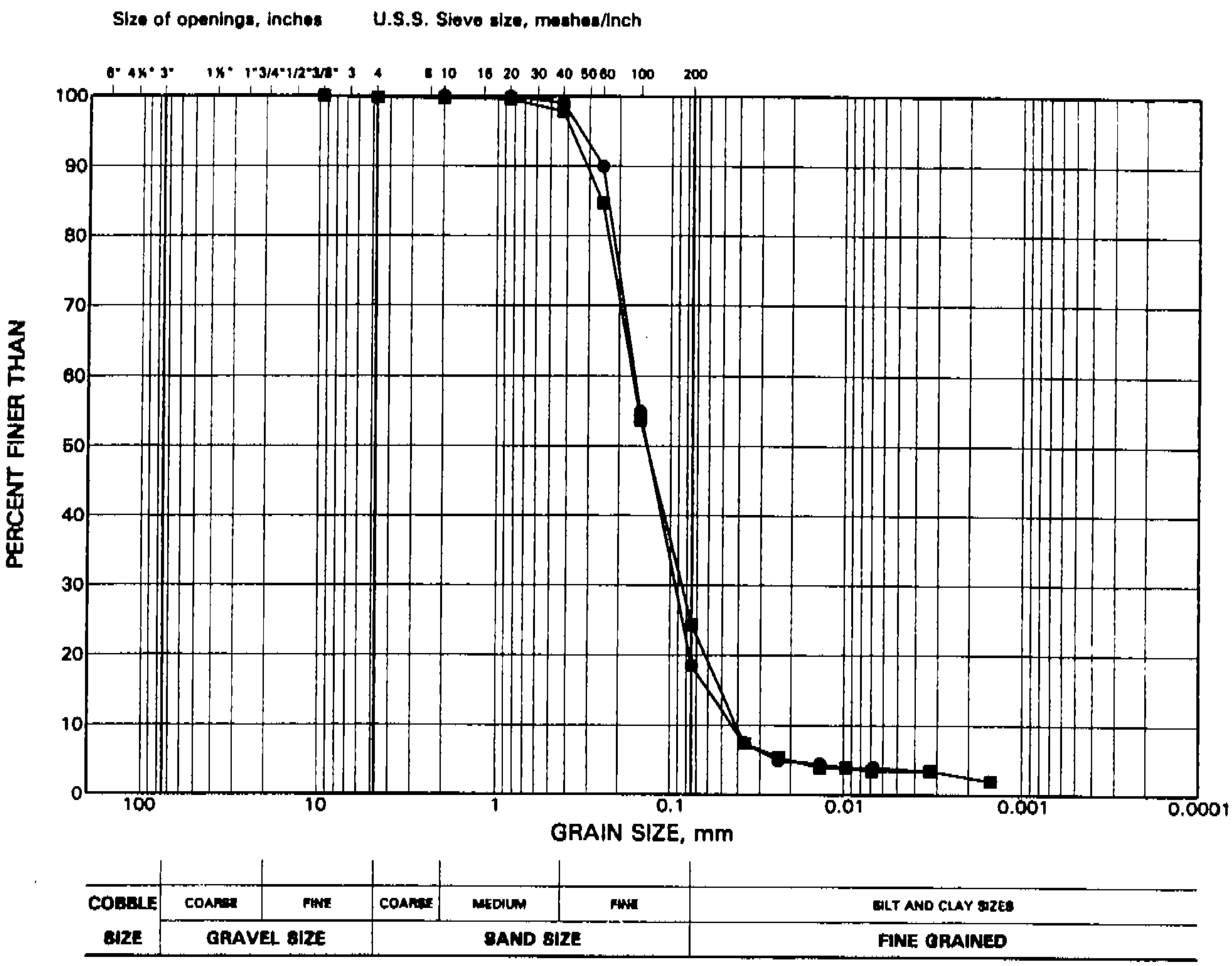
note: for consolidation test summary, refer to Appendix B

CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE

FIGURE 4A

GRAIN SIZE DISTRIBUTION
Silty Sand to Sand, some silt

FIGURE 5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	2-4	12	61.4
■	2-4	18	52.3

1:400

P1119006.DWG

OPTION I
LITTLE CATARAQUI CREEK
LONGITUDINAL SECTION

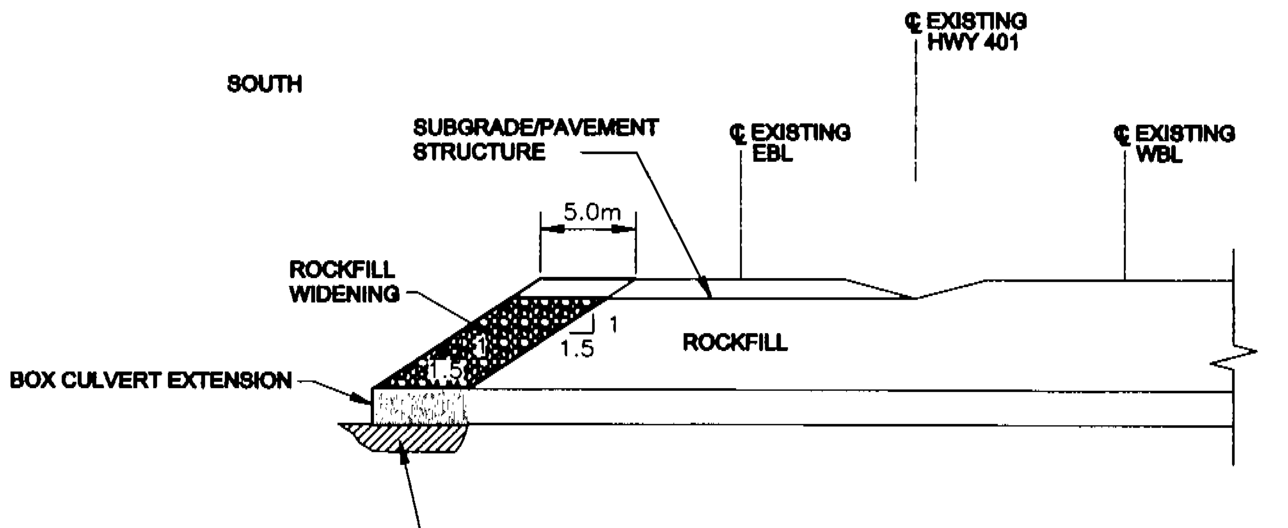
FIGURE 6

SEQUENCE OF CONSTRUCTION

1. SUB-EXCAVATE SILTY CLAY WITH ORGANICS TO ITS FULL DEPTH AND BACKFILL WITH GRANULAR 'B' TYPE 2.
2. CONSTRUCT CULVERT EXTENSIONS ON SHALLOW FOUNDATIONS. (PROVIDE CONSTRUCTION JOINT TO ACCOMMODATE DIFFERENTIAL SETTLEMENTS).
3. CONSTRUCT ROCKFILL EMBANKMENT.

NOTE :

SAME CONSTRUCTION SEQUENCE FOR NORTH SIDE WIDENING



ANTICIPATED SETTLEMENT 150 - 200mm
SHALLOW FOUNDATIONS SLS REFER TO FIGURE 11
ULS 250 kPa

Date DECEMBER, 2000

Project 001-1119

Golder Associates

Drawn. JFC

Chkd

1:400

P1119007.DWG

OPTION II
LITTLE CATARAQUI CREEK
LONGITUDINAL SECTION

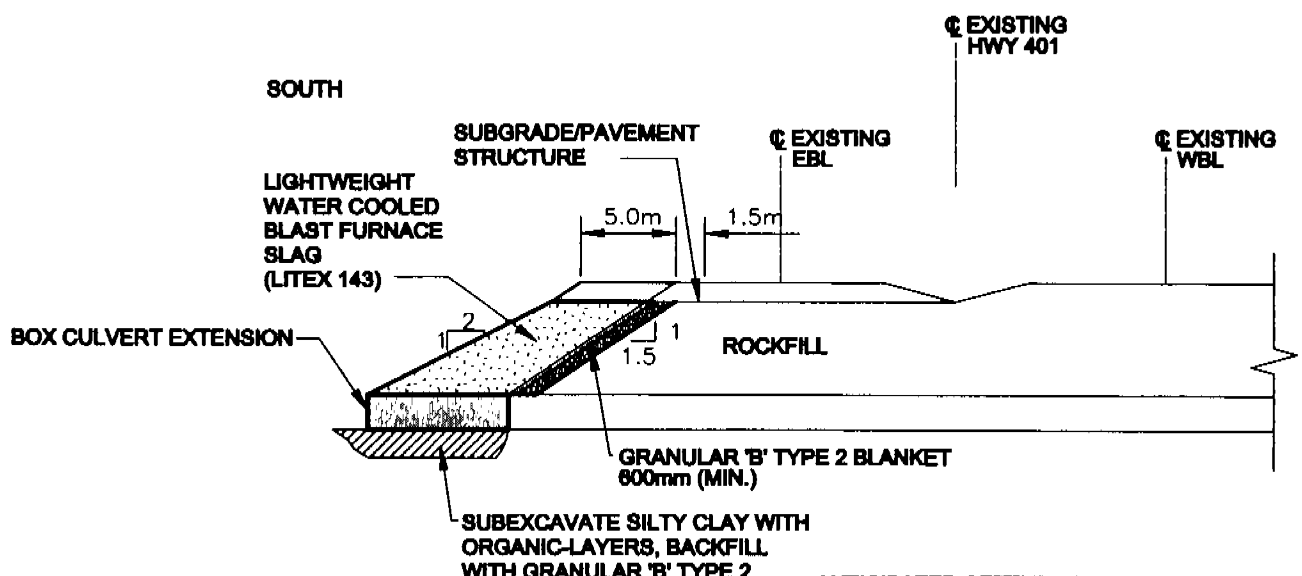
FIGURE 7

SEQUENCE OF CONSTRUCTION

1. EXCAVTE EXISTING ROCKFILL TO A DISTANCE OF 1.5m FROM THE TOP OF SLOPE.
2. SUB-EXCAVATE SILTY CLAY WITH ORGANICS LAYERS TO ITS FULL DEPTH AND BACKFILL WITH GRANULAR 'B' TYPE 2.
3. CONSTRUCT BOX CULVERT EXTENSION (PROVIDE CONSTRUCTION JOINTS TO ACCOMODATE ANTICPATED SETTLEMENTS).
4. CONSTRUCT EMBANKMENT WIDENING WITH LITEX 143 MATERIAL (PROVIDE GRANULAR BLANKET TO PREVENT LOSS OF FINES INTO ROCKFILL).

NOTE :

SAME CONSTRUCTION SEQUENCE FOR NORTH SIDE WIDENING



ANTICIPATED SETTLEMENTS 75 to 100 mm
SHALLOW FOUNDATIONS EQUIVALENT LOADING OF 75kPa
(REFER TO FIGURE 4 FOR SETTLEMENT PROFILE)
SLS REFER TO FIGURE 11
ULS 250 kPa

LITEX 143 - ESTIMATED COST $70 - $80/cu/m SUPPLY AND INSTALLATION.

Date DECEMBER 2000

Project 001-1119

Golder Associates

Drawn JFC

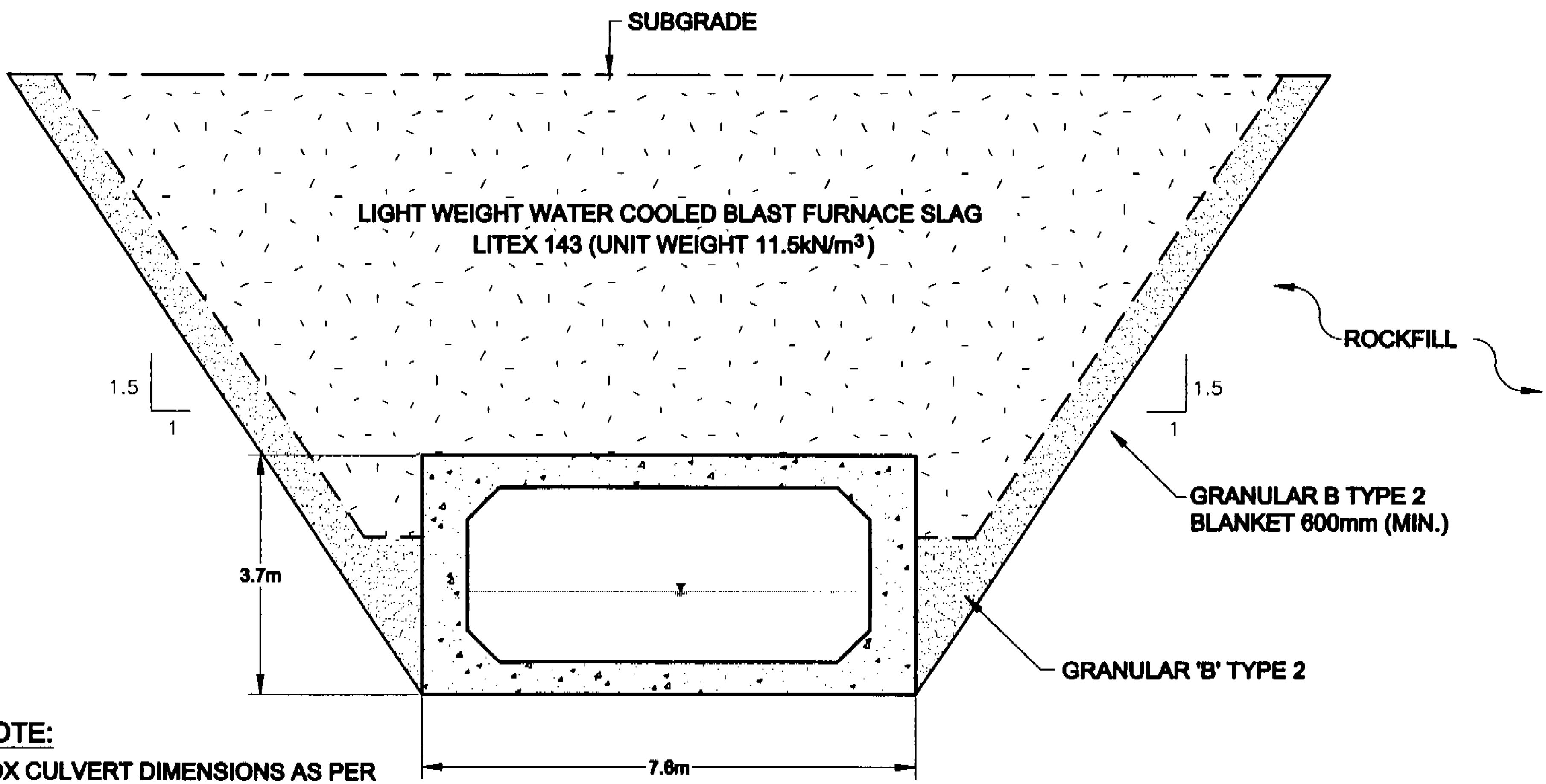
Chkd

1:100

P1119008.DWG

OPTION II
LITTLE CATARAQUI CREEK
TYPICAL CROSS-SECTION

FIGURE 8



NOTE:

BOX CULVERT DIMENSIONS AS PER
MTO INFORMATION DATED AUG. 9, 2000

Date DECEMBER, 2000

Project 001-1119

Golder Associates

Drawn. JFC

Chkd

1:400

P1119009.DWG

OPTION III
LITTLE CATARAQUI CREEK
LONGITUDINAL SECTION

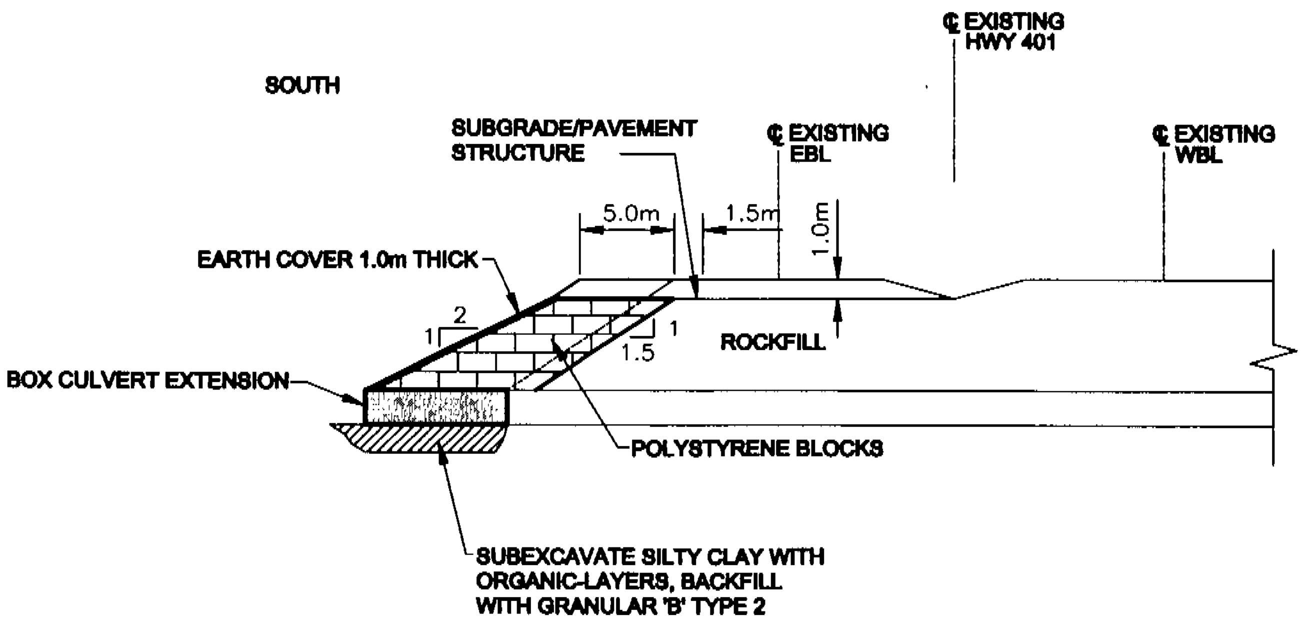
FIGURE 9

SEQUENCE OF CONSTRUCTION

1. EXCAVATE EXISTING ROCKFILL 1.5m BEHIND THE TOP OF SLOPE.
2. CARRY OUT SUB-EXCAVATION AND BACKFILLING WITH GRANULAR 'B' TYPE 2.
3. CONSTRUCT BOX CULVERT EXTENSIONS
4. CONSTRUCT EMBANKMENT WITH POLYSTYRENE BLOCKS.
5. COVER THE SLOPE WITH EARTHFILL.

NOTE :

SAME CONSTRUCTION SEQUENCE FOR NORTH SIDE WIDENING



ANTICIPATED SETTLEMENTS - NEGLIGIBLE
SHALLOW FOUNDATIONS - ULS 250 kPa
POLYSTYRENE BLOCKS - ESTIMATED COST $100-$110/cu/m SUPPLY AND INSTALLATION

Date DECEMBER 2000

Project 001-1119

Golder Associates

Drawn JFC

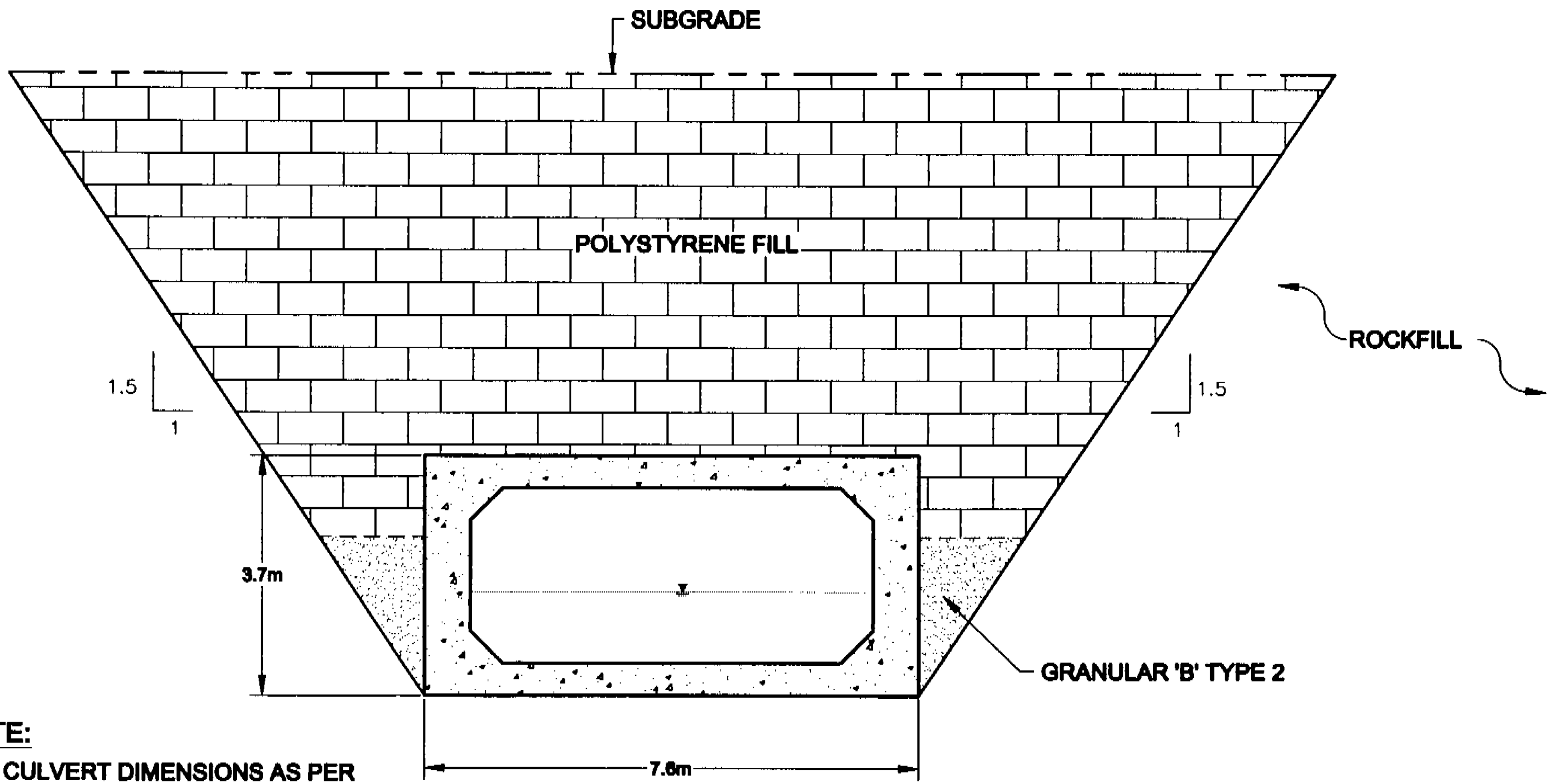
Chkd

1:100

P1119010.DWG

OPTION III
LITTLE CATARAQUI CREEK
TYPICAL CROSS-SECTION

FIGURE 10



NOTE:

BOX CULVERT DIMENSIONS AS PER
MTO INFORMATION DATED AUG. 9, 2000

Date DECEMBER, 2000

Project 001-1119

Golder Associates

Drawn. JFC

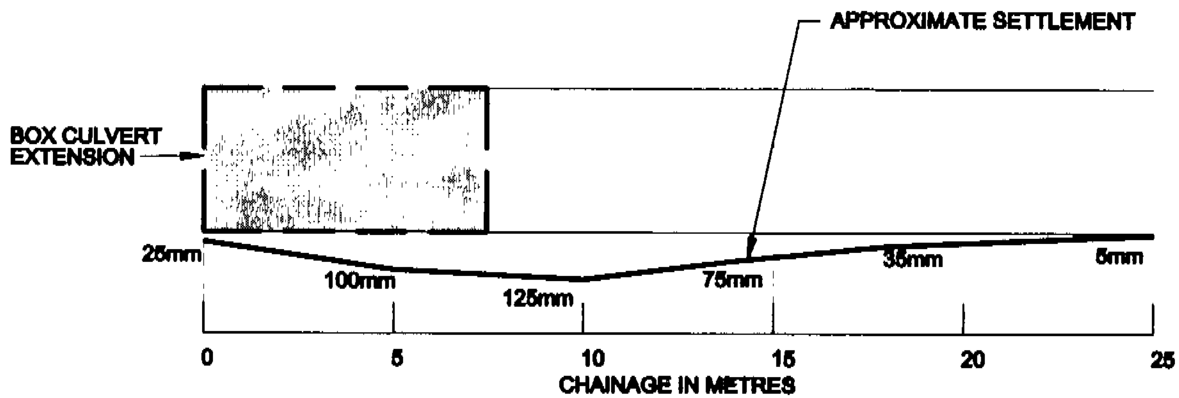
Chkd

1"=1" PS 1:200 MS

P1119011.DWG

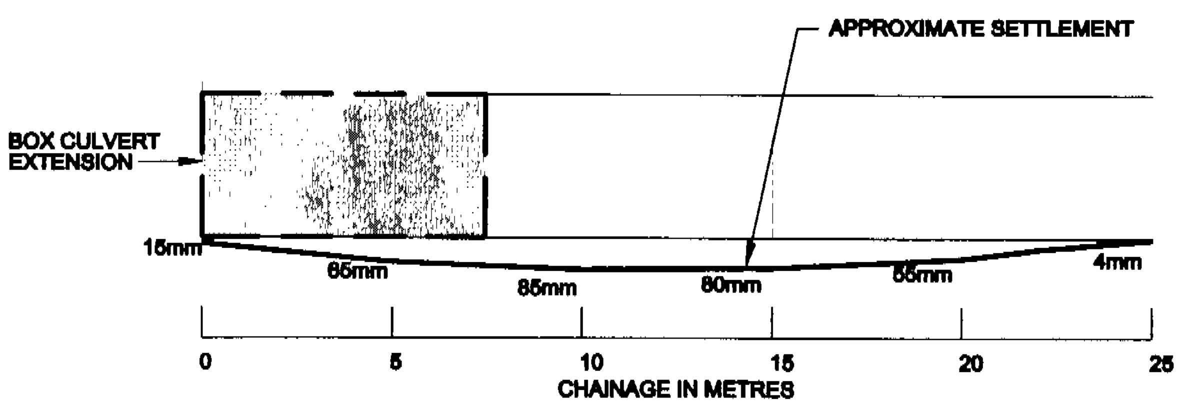
LITTLE CATARAQUI CREEK SETTLEMENT PROFILES

FIGURE 11



OPTION I

NOT TO SCALE



OPTION II

NOT TO SCALE

Date DECEMBER 2000

Project 001-1119

Golder Associates

Drawn JEC

Chkd

1:400

P1119012.DWG

OPTION IV
LITTLE CATARAQUI CREEK
LONGITUDINAL SECTION

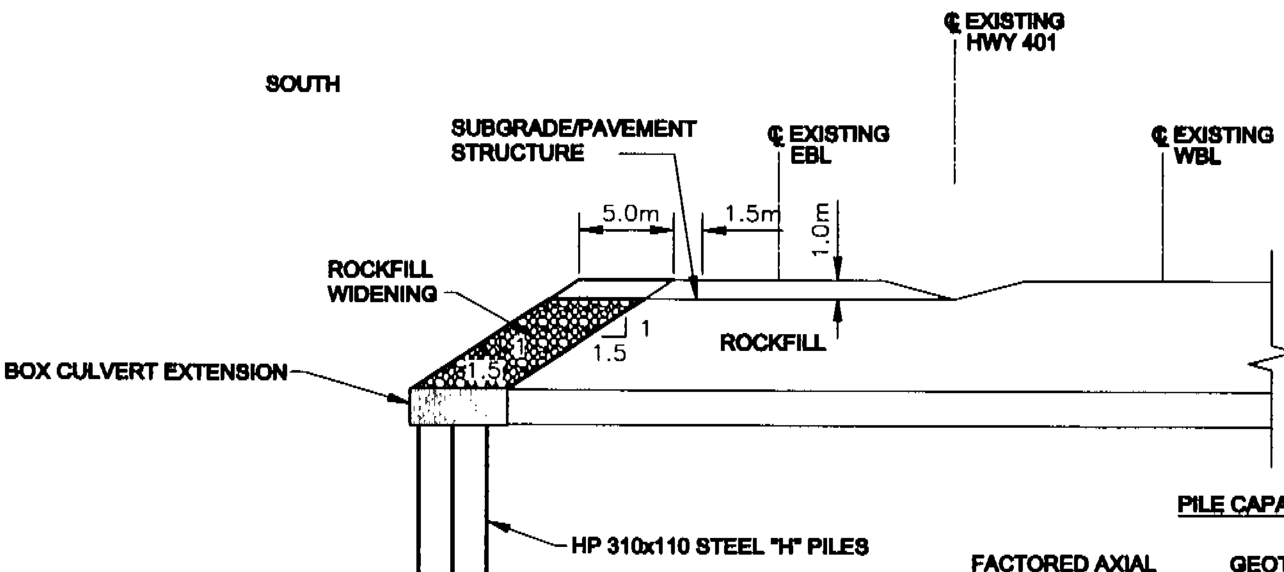
FIGURE 12

SEQUENCE OF CONSTRUCTION

1. SUB-EXCAVATE SILTY CLAY WITH ORGANICS TO ITS FULL DEPTH AND BACKFILL WITH GRANULAR 'B' TYPE 2.
2. CARRY OUT SUB-EXCAVATION AND BACKFILLING WITH (PROVIDE CONSTRUCTION JOINT TO ACCOMODATE DIFFERENTIAL SETTLEMENTS).
3. CONSTRUCT ROCKFILL EMBANKMENT

NOTE :

SAME CONSTRUCTION SEQUENCE FOR NORTH SIDE WIDENING



PILE CAPACITIES (HP 310x110)

	FACTORED AXIAL RESISTANCE AT ULS	GEOTECHNICAL AXIAL RESISTANCE AT SLS	ESTIMATED TIP ELEV.
NORTH EXTENSION	1500kN	1000kN	ELEV. 58.8±
SOUTH EXTENSION	2000kN	-	TO BEDROCK ELEV. 46.0±

UNFACTORED DOWNDRAG FORCE - 500kN (FOR HP 310x110)

Date DECEMBER, 2000

Project 001-1119

Golder Associates

Drawn JFC

Chkd

APPENDIX A

CULVERT INSPECTION REPORT

CULVERT SYNOPSIS REPORT

SITE #: 7-154

		YY/MM/DD
INSPECTION DATE	:	94/08/04
ICE INSPECTION	:	98/02/10
PICTURE DATE	:	98/02/10

INSPECTED BY: DK,PJR,BG

EQUIPMENT USED: STANDARD

LOCATION: 700 M WEST OF SIR JOHN A - CROSSES LITTLE CATARAQUI CREEK

HIGHWAY: 401 ORDER: 30
DISTRICT: 41
TOWNSHIP: KINGSTON
CONCESSION: 3
LOT: 19
STATION: 21+560

YEAR BUILT: 19 0

MATERIAL: CONCRETE
TYPE: OPEN

SPAN: 6.10 m
HEIGHT: 1.83 m

WATER DEPTH: 1.00 m
FILL DEPTH: 6.00 m

RATINGS:

SOFFIT	5
WALLS	5
INVERT	9
FOUNDATIONS	9
CHANNEL	5

DESCRIPTION:

CONCRETE CULVERT WITH GUIDERAIL ACROSS.

INSPECTION NOTES:

1. SPALLS AND CRACKS AT SOUTH END; FOUR LEACHING, WET CRACKS. EAST SIDE SETTLED.
2. MEDIAN DRAIN WITH LOTS OF DEBRIS.
3. soffit has leach stained transverse cracks at 8 m centres
4. Exposed south end of culvert has sever scaling on top and sides with one exposed rebar.
5. TREES AT SOUTH END.
6. NORTH END HAS LARGE GABIAN WALL 3.0 M HIGH.

1990 · Severe efflorescence

RECOMMENDATIONS:

MAINTENANCE BY DISTRICT:

1. REMOVE TREES AT SOUTH END.
2. MONITOR MEDIAN DRAIN.

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31C-159

DIST. 41 REGION

W.P. No. 77-99-01

CONT. No.

W. O. No.

STR. SITE No.

HWY. No. 401

LOCATION Little Cataraqui Creek Culvert

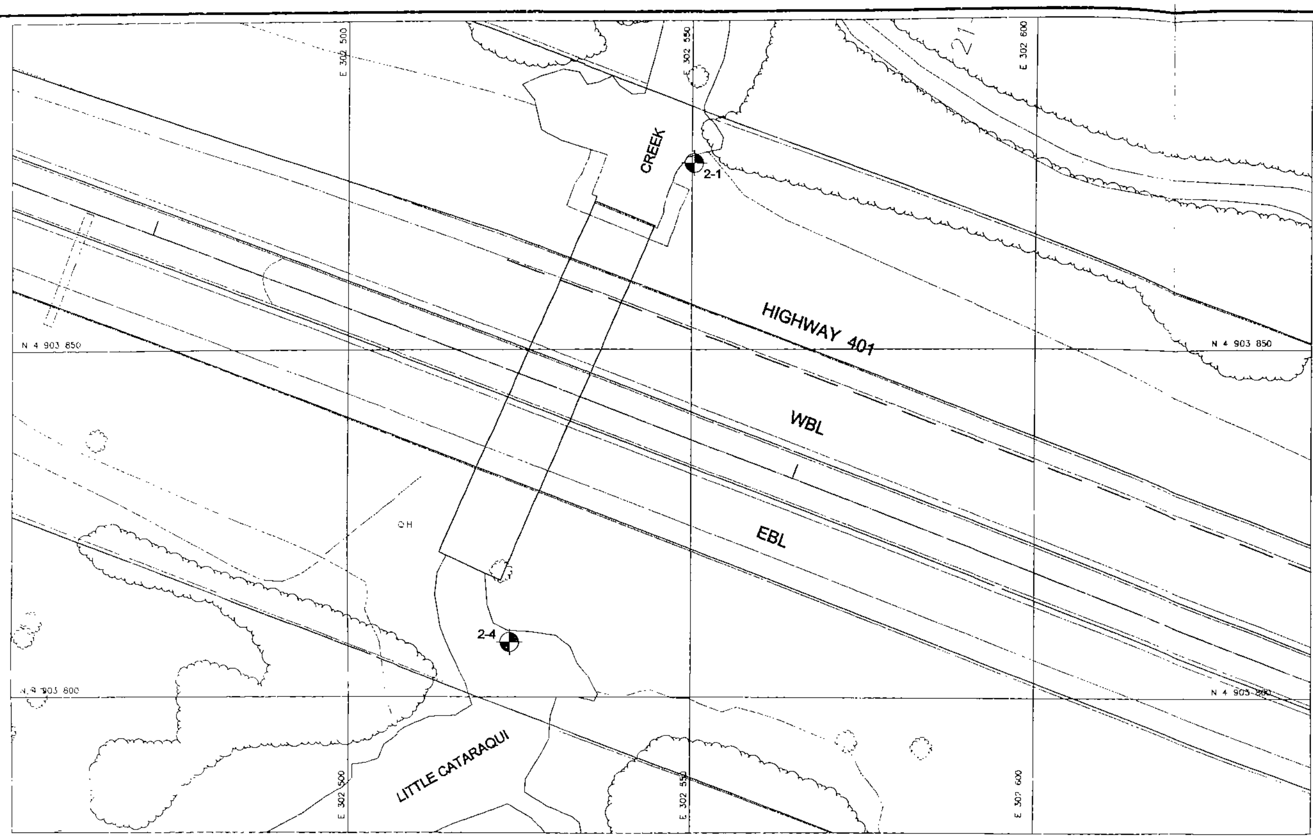
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

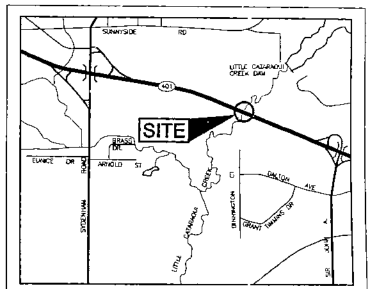
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

DIST 41 HWY 401 CONT No. WP No. 77-99-01	
HIGHWAY 401 CULVERT AT LITTLE CATARAQUI CREEK BOREHOLE LOCATIONS & SOIL STRATA	SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



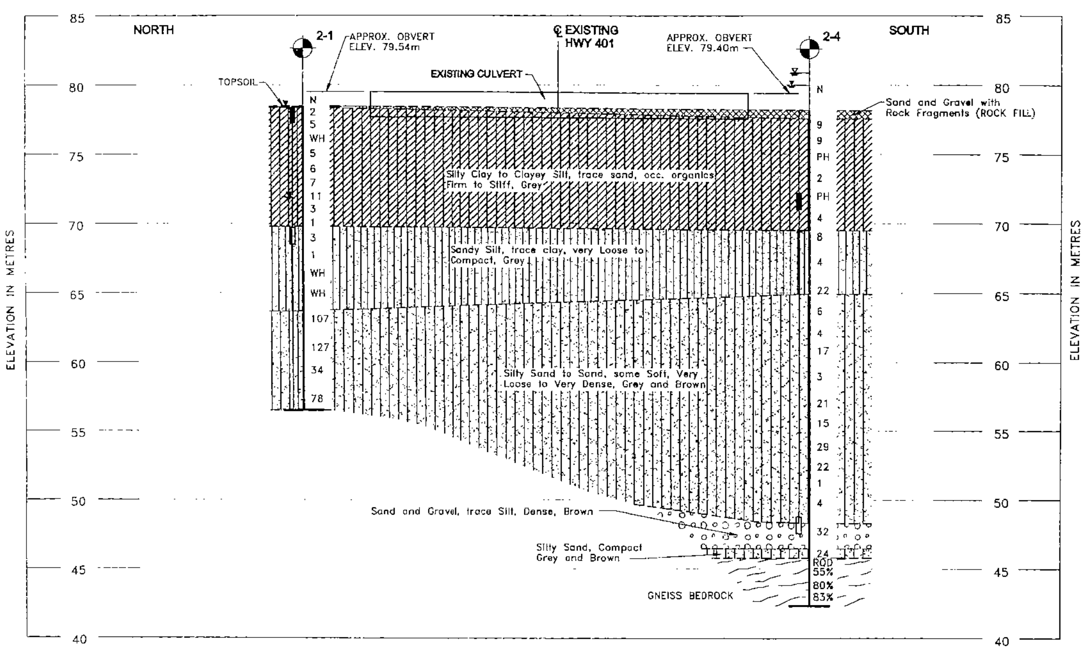
PLAN
0 10 20 30 40 METRES



LEGEND

- Borehole – Current Golder Associates Ltd. Investigation
- Seal
- Piezometer
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- WL in piezometer on May, 21/00
- WL upon completion of drilling

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
2-1	78.47	4903877.08	302550.33
2-4	78.18	4903807.92	302523.73



PROFILE ALONG LITTLE CATARAQUI CREEK

HORIZONTAL SCALE 0 10 20 30 40 METRES
VERTICAL SCALE 0 5 10 15 20 METRES

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

REFERENCE
This drawing was created from digital file "P125-800.dwg" provided by MTO, and culvert obvert elevations obtained from profile drawing "PR 21-100.DWG" provided by MTO.

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 401		PROJECT NO.: 001-1119	DIST. 41
SUBM'D. DKB	CHKD: ASP	DATE: 2000 05 08	
DRAWN: JFC	CHKD. DKB	APPD.	DWG. SITE 2

P1119002.DWG