

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 30M 5-194

DIST. 4 REGION

W.P. No. 410 -85-00

CONT. No.

W. O. No.

STR. SITE No.

HWY. No. 403

LOCATION HWY 403 - STRUCTURES &  
APPROACHES  
WALKERS LINE TO HWY 25  
6 BRIDGES

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**REPORT  
TO  
MINISTRY OF TRANSPORTATION**

**GEOTECHNICAL INVESTIGATION  
DESIGN AND CONSTRUCTION OF  
DROP SHAFT AND TUNNEL  
HIGHWAY 403 AND BRONTE CREEK  
OAKVILLE, ONTARIO**

WP 410 - 85-00      DIST 4

Submitted to:

Ministry of Transportation Ontario  
Foundation Design Section  
1201 Wilson Avenue  
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September 1993

931-1346

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## 1.0 INTRODUCTION

Golder Associates Ltd. has been retained by the Ministry of Transportation Ontario (MTO) to carry out a geotechnical investigation to assist the design of a proposed drop shaft and tunnel which serves as an outlet to a storm detention basin. These works are located where the proposed extension of Highway 403 crosses Bronte Creek in Oakville, Ontario. This report presents the results of the subsurface investigation and associated laboratory testing for the site.

The purpose of the investigation was to determine the subsurface conditions at the site and to provide geotechnical engineering recommendations for the design of the drop shaft and tunnel. A proposal for carrying out the work was provided in our letter to the Ministry of Transportation Ontario dated May 17, 1993.

## 2.0 SITE AND PROJECT DESCRIPTION

The proposed site of the drop shaft and tunnel is located east of Bronte Creek and south of Halton Side Road Number 1 (Figure 1). The drop shaft and tunnel will service a storm detention basin collecting storm water from Highway 403 and storm water from the drainage ditches paralleling the existing Canadian National (CN) railway.

According to the proposed layout of the drop shaft and tunnel provided by R.V. Anderson, and Associates Limited, the total depth of the shaft would be approximately 20 m. The diameter of the shaft would be 1.8 m, enlarged to 2.6 m in the upper and lower chambers. The tunnel leading from the base of the shaft to the proposed rip rap channel would have an unlined diameter of 2.4 m, a lined diameter of 1.8 m and a total length of approximately 80 m.

The area of the site lies within the Peel Plain physiographic region which is characterized by till plain and till moraine deposits (Chapman and Putnam, 1984). Across this plain Bronte Creek has cut a deep valley exposing the Queenston shale below the till units. The till units are characterized as being clay rich and commonly contain weathered mudstone and limestone fragments. The shale bedrock below the till is essentially flat lying paleozoic sediment. The Queenston shale is a red brown silty mudstone, interbedded with harder grey siltstone layers. At the site, Bronte Creek has cut deep into the shale, forming outcrops with sub-vertical to vertical faces.

### 3.0 INVESTIGATION PROCEDURE

The field work for this investigation was carried out between May 27 and June 3, 1993 at which time four boreholes were drilled. The locations of the boreholes are shown on Drawing WP 410-85-00, with the coordinates and surface elevations summarized in Table 1.

Coordinates of the boreholes, borehole elevations and 3 ground surface profiles along the centre line of the tunnel and at 5 m offsets were provided by MTO staff. The elevations provided are understood to be referred to geodetic datum.

The boreholes were drilled using track mounted power auger drillrigs equipped for rotary drilling. These rigs were supplied and operated by a specialist drilling contractor. All boreholes were offset between 5 and 10 m from the tunnel centre line and sampled through the overburden to bedrock. The bedrock was drilled to a depth of 3 m below the proposed tunnel invert. The holes were advanced through the overburden using 100 mm diameter continuous flight solid stem augers or 200 mm diameter hollow stem augers and through the rock using NQ sized diamond drill bits. Standard Penetration testing and sampling were conducted at 0.75 m intervals in the overburden using 35 mm inside diameter open drive split spoons samplers. Core was recovered in the bedrock using double and triple tube wireline equipment.

Samples of overburden and rock core recovered from the boreholes were taken to our Mississauga laboratory for examination and testing. Because the properties of the Queenston Shale are known to be sensitive to changes in moisture content, all core were wrapped in aluminium foil and plastic wrap upon recovery to preserve its in situ moisture content. Once at the laboratory, the core was immediately placed in our humidity room for storage.

Representative samples of overburden were selected for water content, Atterberg limit and grain size analyses. Unit weight, point load, uniaxial compression (with and without lateral deformation monitoring), slake durability, chloride content, calcite content and swell tests were conducted on selected samples of rock core. Groundwater samples from the well installation in Borehole 2 have been submitted for sulphate, chloride and pH determination.

The results of these test are presented in Section 5 of the report.

The overburden and rock stratigraphy encountered in the boreholes is shown in detail on the Record of Borehole sheets following the text of the report and is summarized on Drawing WP 410-85-00. Summaries of bedrock features and joints are provided in Tables 2 and 3. Results of the field and laboratory overburden testing are presented on the Record of Borehole sheets and on Figures A-1 to A-5 in Appendix A. Results of the laboratory rock testing are presented in Tables 4 to 6 and on Figures A-6 and A-7 in Appendix A.

Groundwater levels were observed in the open boreholes during drilling and in the installed piezometers after drilling was completed. Details of the piezometer installations are shown on the Record of Borehole sheets. Notes pertaining to the groundwater conditions observed in the boreholes are shown on the Record of Borehole sheets and on Drawing 4108500-A.



#### 4.0 SUBSURFACE CONDITIONS

##### 4.1 General Stratigraphy

The subsurface conditions encountered in the boreholes put down at the site are shown in detail on the Record of Borehole sheets and summarized on Drawing 4108500-A. The soil boundaries indicated on the Record of Borehole sheets are inferred from non-continuous sampling and resistance to drilling advance. These boundaries represent transitions from one soil type to another and are not intended to define an exact plane of geological change. Conditions will vary between and beyond the borehole locations. Data concerning the rock characteristics were also only obtained at point locations. While these data are in general considered applicable for the shaft and tunnel development, local differences can be anticipated.

The subsurface conditions encountered at the top of the slope consist of topsoil and glacial till overlying highly to moderately weathered rock and slightly weathered to fresh rock at depth. Midway down the slope the subsurface conditions change with the encountered stratigraphy consisting of topsoil overlying highly to moderately weathered rock and slightly weathered rock at depth. At the bottom of the slope further changes in the subsurface conditions take place with the encountered stratigraphy consisting of topsoil and scree material overlying highly to moderately weathered rock and slightly weathered rock at depth.

In the following discussion, the individual soil and rock units are described for the purposes of geotechnical design.

##### 4.2 Topsoil

A thin layer of top soil was encountered at the ground surface at all 4 borehole locations.

#### 4.3 Silty Clay to Clayey Silt Scree

Silty clay and sandy clayey silt scree were encountered beneath the cover of topsoil in Boreholes 93-1A, 93-1B and 93-4. These materials are characterized as a stiff to very stiff red brown silty clay to sandy clayey silt with occasional rock fragments and a slightly higher natural water content than the tills of Borehole 93-3. The SPT N values ranged from 11 to 17 blows per 0.3 metres. The natural water contents of recovered samples from this stratum ranged from 17.8 to 22.4 per cent.

The grain size distribution curve of a selected sample of the silty clay scree from Borehole 93-1 is shown on Figure A-1. The percentages of gravel, sand, silt, and clay are 2, 20, 54, and 24 per cent respectively. The liquid and plastic limits from the same sample were 30 per cent and 19 per cent respectively, giving a plasticity index,  $I_p$ , of 11 per cent (Figure A-5).

The grain size distribution curve and Atterberg Limits of a selected sample of the sandy clayey silt scree from Borehole 93-4 are shown on Figures A-4 and A-5 respectively. The percentages of gravel, sand, silt and clay are 15, 34, 39 and 12 per cent respectively while the liquid and plastic limits are 24 per cent and 15 per cent, giving a plasticity index of about 9 per cent.

The silty clay and sandy clayey silt scree strata of Boreholes 93-1A, 93-1B and 93-4 were immediately underlain by a thin layer of grey sandy silt, which was closely followed by the weathered bedrock surface.

#### 4.4 Silty Clay to Clayey Silt Till

An interlayered silty clay and clayey silt till stratum was encountered beneath the topsoil in Borehole 93-3. The till was characterized by interlayered zones of very stiff to hard brown sandy clayey silt till and hard red brown sandy silty clay till with occasional weathered rock fragments. In Borehole 93-3, the zones were separated by a thin layer of very dense sandy silt till as described below. The N values, as determined from the Standard Penetration Test, in the upper sandy clayey silt till zone ranged from 25 to 71 blows per 0.3 m penetration. The

natural water content of the recovered samples from the upper till ranged from 10.7 to 13.0 per cent. Standard Penetration Tests were also carried out in the lower till zone, but due to its hard consistency, it was not practical to advance the sampler the entire 450 mm required to establish an N value. The N values were inferred as greater than 100 blows per 0.3 m penetration. The natural water content of the recovered samples from the lower till ranged from 6.8 per cent to 8.9 per cent.

The grain size distribution curve for a sample of the lower sandy silty clay till in Borehole 93-3 is shown in Figure A-2. The percentages of gravel, sand, silt and clay are 17, 25, 41 and 17 per cent respectively. The liquid and plastic limits of the same sample were 25 per cent and 17 per cent respectively, giving a plasticity index,  $I_p$ , of 9 per cent (Figure A-5).

A second grain size analysis and Atterberg Limit determination was conducted on a sample of the sandy clayey silt till immediately above the highly weathered bedrock surface of Borehole 93-3. The grain size distribution curve and plasticity limits are plotted in Figures A-3 and A-5 respectively. The percentages of gravel, sand, silt and clay are 20, 35, 37 and 8 per cent while the liquid and plastic limits are 18 per cent and 13 per cent, giving a plasticity index of about 5 per cent.

#### **4.5     Sandy Silt Till**

A layer of sandy silt till was encountered as one of the interlayered till strata of Borehole 93-3. The sandy silt till is characterized by a very dense red brown sandy silt with trace clay and trace gravel. Only one SPT N value of 88 blows per 0.3 metres could be determined due to the limited thickness of the stratum. The natural water content of the recovered sandy silt till sample was 8.5 per cent.

#### **4.6     Queenston Shale, Highly Weathered to Fresh Bedrock**

At all four borehole locations, the overburden materials are underlain by bedrock of the Queenston Formation. Bedrock was encountered at elevation 151.9 m at Borehole 93-3, 145.5 m at Borehole 93-2, 138.1 m at Borehole 93-4 and 137.8 m at Borehole 93-1. Based on

the observed topography at the site (river terraces bounded by steeply dipping banks) and the bedded nature of the rock mass, it is expected that these changes in bedrock elevation are a consequence of a stepped rather than a sloped bedrock surface. However, because of the limited number of boreholes, the presence of these steps can only be assumed.

Bedrock consists of a thinly to medium bedded, red brown, medium grained, non-porous, silty mudstone, interbedded with grey, coarse grained, non-porous siltstone up to 0.34 m in thickness. Close to the overburden/bedrock contact the rock is highly to moderately weathered. Completely weathered rock was not encountered in any of the boreholes as the interface zone appears to have been protected by relatively harder siltstone interbeds close to the rock surface. In such cases these interbeds have provided a protective cap to the softer silty mudstone beds below and limited the extent of weathering. A transition was noted downwards in all holes from moderately weathered to slightly weathered rock near one of the distinct siltstone interbeds again providing a relatively sharp boundary. This transition, which seems to be relatively sharp, was logged at elevation 147.2 m in Borehole 93-3, at elevation 144.4 m in Borehole 93-2, at elevation 135.7 m in Borehole 93-4 and at elevation 133.6 m in Borehole 93-1. Below this boundary the rock was noted to be slightly weathered to the completion depth of Boreholes 93-1, 93-2 and 93-4. However, in Borehole 93-3 a zone of fresh rock was encountered from elevation 141.0 m to the end of the hole, at elevation 131.8 m.

The rock core recovered from the boreholes was generally fractured in the highly to moderately weathered zones, with the degree of fracturing decreasing noticeably with depth in the slightly weathered to fresh rock. In the majority of cases, the fractures were associated with horizontal bedding, appearing as smooth, planar, clean surfaces oriented at 90 degrees to the core axis. Occasionally some fractures exhibited clay infilling, typically in intervals of core where a high degree of fracturing occurred. Only a few non-horizontal joints were encountered in the core and typically these were observed to be cross-joints between bedding planes.

Total core recoveries (TCR) ranged from 50 to 100 per cent in the highly to moderately weathered zones and from 90 to 100 per cent in the slightly weathered to fresh zones. The

lower core recoveries were always associated with the first runs, where the run began in overburden or highly weathered material. The solid core recovery (SCR) ranged from 30 to 80 per cent in the highly to moderately weathered zones and from 80 to 100 per cent in the slightly weathered to fresh zones. The rock quality designation (RQD) ranged from 10 to 70 per cent in the highly weathered to moderately weathered zones, from 50 to 100 per cent in the slightly weathered zones and from 95 to 100 per cent in the fresh zones.

In addition to the core, two nearby outcrops were mapped for structural discontinuities. During the outcrop mapping special attention was paid to any vertical or sub-vertical features, as it was unlikely that they would be encountered in the core from the vertical boreholes. The outcrops are situated on the east and west banks of Bronte Creek, within 100 m of the proposed portal location (Figure 2).

The results of the outcrop mapping are summarized in Figure 3 in the form of a rose diagram and a lower hemisphere, equal area stereonet. All data are reported in terms of dip and dip direction, with bearings taken with respect to magnetic north. From the mapping three distinct sub-vertical to vertical joint sets were identified (JS1, JS2 and JS3) oriented at 85/215, 85/290 and 90/335. All three sets were most persistent in the harder siltstone interbeds, with the individual joint traces rarely continuing into the softer silty mudstone. The approximate joint spacings were 0.8 m for JS1, 0.5 m for JS2 and 0.5 m for JS3.

#### 4.7 Groundwater Conditions

Groundwater levels, as measured and documented during the drilling in open boreholes and subsequent to the field investigation in the installed piezometers are tabulated below.

Given the stratigraphy at this site, it is likely that an upper groundwater table exists with a phreatic surface at or immediately above the terraced bedrock surface, in the clayey silt till and scree strata. Three levels of piezometer were installed in Borehole 93-3 at the crest of slope in order to help delineate these different groundwater levels. It appears that there may be a consistent gradient and resulting groundwater flow towards Bronte Creek. There were, however, no seepage points or spring lines observed on the slope.

Borehole Reference Number (Elevation), m	Depth to Groundwater (Elevation) m						
	Date (Day/Month)						
	27/06	28/05	01/06	02/06	03/06	07/06	21/06
93-1A (140.2)	-	-	-	-	Dry*	-	-
93-1B 8(140.2)	-	-	-	-	Dry*	5.7 (134.5)	6.8 (133.4)
93-2 (146.8)	-	-	10.4* (136.4)	-	-	11.4 (135.4)	11.5 (135.3)
93-3 (162.6) (Cased Borehole)	8.8* (153.8)	-	9.3* (153.3)	6.9* (155.7)	-	-	-
93-3 Shallow	-	-	-	-	-	5.8 (156.8)	6.4 (156.2)
93-3 Intermediate	-	-	-	-	-	9.1 (153.5)	10.0 (152.6)
93-3 Deep	-	-	-	-	-	7.8 (154.8)	12.7 (149.9)
93-4 (141.2)	-	-	-	5.2* (136.0)	-	6.0 (135.2)	6.4 (134.8)

\* Water level questionable due to presence of drilling water or lack of recovery time.  
 - Piezometer not installed, no reading available.

It should be noted that the piezometric groundwater level within the subsoil and underlying bedrock is subject to fluctuation not only due to precipitation conditions, but also due to seasonal variations. The water levels given above may not necessarily reflect stabilized conditions and may vary from the conditions which are encountered during construction.

## 5.0 RESULTS OF LABORATORY TESTING

All soil testing was conducted according to ASTM Standards. For rock mechanics tests, the methods and procedures suggested by the International Society of Rock Mechanics (ISRM) were generally followed. Minor Modifications to the ISRM suggested procedures for the swelling tests were made as recommended by Lo et al, 1978.

### 5.1 Strength Testing

Point load strength, uniaxial compressive strength and modulus tests were conducted on representative samples of rock core.

Point load tests were conducted at 1 m intervals along the entire length of rock core from Borehole 93-3 and at 1 m intervals from rock core in Boreholes 93-1, 93-2 and 93-4 between the proposed elevations of the tunnel obvert and invert. At each sample location both axial and diametral tests were conducted to determine the ratio of strength perpendicular to versus parallel to bedding (Table 4). In the silty mudstone samples the average point load index ( $I_{50}$ ) was 1.6 MPa across bedding and 0.9 MPa parallel to bedding, yielding a ratio of axial to diametral strength of 1.7. In the siltstone interbeds the average  $I_{50}$  was 2.8 MPa across and 1.4 MPa parallel to bedding yielding, a ratio of axial to diametral strength of about 2.

Uniaxial compressive strengths were measured on 4 representative samples of silty mudstone taken from Borehole 93-2 at elevation 135.7 m and from Borehole 93-3 at elevations 145.6 m, 144.5 m and 140.0m. The measured compressive strengths ranged from 23.6 MPa to 35.4 MPa with an average strength of 29.2 MPa. This classifies the silty mudstone as weak rock to medium strong rock in terms of Deere and Miller's classification of rock strengths (Table 5). Only the argillaceous units were tested for uniaxial compressive strength, thus assuming a ratio of 1.8 between the index strengths for the silty mudstone compared to the siltstone samples based on the axial point load results, extrapolation for the siltstone would suggest an average compressive strength of about 52 MPa, which classifies it as medium strong to strong rock.

In addition to testing for compressive strength, one sample from Borehole 93-2 at elevation 135.7 m and one sample from Borehole 93-3 at elevation 140.0 m were monitored for axial and lateral deformations during the compression test. The results of the testing indicated an average elastic modulus of 4.2 GPa and an average Poisson's ratio of 0.28 for the silty mudstone.

## 5.2 Slake Durability

Slake durability tests were carried out on two representative rock samples from the site; one from Borehole 93-2 at elevation 135.5 m and the other from Borehole 93-3 at elevation 139.6 m. The second cycle slake durability index ( $I_{ds}$ ) of the samples ranged from 87.6 to 93.7 per cent. These results indicate that the rock tested is classified as having high durability. Similar rock at the site should therefore slake only marginally when exposed to cycles of wetting and drying (Franklin and Gruspier, 1983). It should be noted however that some zones of distinctly greater fracturing and degradation are evident in the rock outcrops at the creek. Thus, differences in behaviour with depth and different layers may be anticipated.

## 5.3 Calcite, Chloride and Swell Tests

Two vertical swell tests are currently being carried out on representative samples of the rock core. The first test sample was taken from Borehole 93-2 at elevation 135.6 m and the second from Borehole 93-3 at elevation 139.8 m. The first test is a free swell test, while the second is confined vertically with a 590 kPa load equivalent to the weight of overburden at the sample location. For the vertically confined test, the sample was loaded on day 7 after the sample had swelled 0.13 per cent.

Individual plots of the experimental data for the two tests are given in Figures A-6 to A-7 in Appendix A. A summary of the rates of the measured time-dependent deformation is presented in Table 6. For this report, data is presented up to September 13, 1993, after 100 days of testing. The free swell sample has indicated an average swelling potential of about 0.1 percent per logarithmic cycle (between 10 and 100 days) based on extrapolation of the data between 20 and 55 days. However, it is noted that there is negligible swelling after 60



days of testing (See Figure A-6). The vertically confined sample had shrunk 0.17 percent due to the application of the vertical load on day 7 and had shown no swelling since that time.

The calcite contents of the tested rock specimens were also determined from 4 samples adjacent to the swell test sample locations. All calcite contents were relatively high with the two samples from Borehole 93-2 having a mean calcite content of 24.3 per cent and the two samples from Borehole 93-3 having a mean calcite content of 18.8 per cent (Table 6).

The results of both the free and confined swell tests as described above indicate that the tested samples exhibit little tendency to swell, which is consistent with the swelling characteristics of the Queenston Shale Formation rocks which consist of high calcite contents.

#### **5.4 Groundwater Chloride, Sulphate and pH Results**

Having submitted a groundwater sample from the developed Borehole 2 to an independent, accredited laboratory for chemical testing, it has been found that the chloride and sulphate contents and the pH are all within acceptable limits for normal construction materials and methods. The chloride and sulphate contents of the groundwater were found to be 44.0 mg/L and 119 mg/L respectively, while the pH was found to be near neutral, at 7.33. These results, along with their associated quality control results, are included in Table A.1 in the appendix of this report.

## **6.0 INTERPRETATION OF SUBSURFACE CONDITIONS AND RECOMMENDATIONS**

This part of the report represents our interpretation of the factual borehole results, together with engineering recommendations for the geotechnical design of the proposed tunnel and shaft. Comments on specific construction features which may influence the design are also provided. This information is provided for the guidance of the design engineers only. Contractors bidding on or undertaking the work should review the factual results of the information for construction, and make their own interpretation of the factual borehole and laboratory test data as it affects their proposed construction techniques, schedule and equipment capabilities.

### **6.1 Tunnel Excavation**

According to preliminary sketches provided by R.V. Anderson and Associates Limited, the collar of the proposed drop shaft would be located at the top of the slope at elevation 155 m. The proposed tunnel extends from the base of the drop shaft in a south-southwest direction towards Bronte Creek with a 45 degree bend towards the south in the last 10 m of the tunnel.

The preliminary general arrangement of the tunnel provided by R.V. Anderson indicates that the invert elevation of the tunnel would be at about 135 m at the outlet structure. Based on an internal diameter of 1.8 m, the excavated width of the tunnel would be about 2.4 m. Figure 4 shows the proposed excavation lines for the drop shaft, tunnel, and outlet structure superimposed on the results of the field investigation. In addition to a profile of the RQDs measured from the core, profiles of the fracture frequency, fracture locations and fracture dip relative to the core axis have been included at each borehole location.

As discussed previously, the overburden bedrock interface at the borehole locations tends to suggest that the bedrock profile is in the form of a series of terraces reflecting the competence of some of the more silty bands within the rock mass. When these elevations are superimposed on the proposed tunnel alignment, there is little or no rock cover to the tunnel at the locations of Boreholes 93-1 and 93-4. Upstream of these boreholes, the bedrock surface

is anticipated to remain relatively level and then to rise steeply to the elevation of the bedrock surface encountered in Borehole 93-2, in a similar form to the steep rock face exposed along the west bank of Bronte Creek to the south of the site. It is understood that the invert elevation of the tunnel near the outlet is tied to the elevation of the Bronte Creek channel; therefore little flexibility exists in lowering the tunnel invert in order to increase the rock cover for the section near the portal.

## 6.2 Tunnel Portal

On the basis of the accessibility for removal of excavated material, the preferred excavation procedure is to develop the tunnel from the downstream end towards the proposed shaft location. In view of the anticipated bedrock surface profile described above, consideration has to be given to the following factors in determining the location of the tunnel portal:

- i) tunnelling condition;
- ii) tunnel support requirements;
- iii) environmental impact of the surface excavation for developing the tunnel portal.

Two alternative locations for the tunnel portal are discussed below:

### *Alternative 1: Develop Portal in Area of Shallow Rock Cover*

This alternative consists of locating the tunnel portal between Boreholes 93-1 and 93-4, approximately 4 m upstream of Borehole 93-1 where the overburden is about 3 m thick. The length of open cut for this alternative would be about 15 m, measuring from the current bank of the Creek shown in the longitudinal section. The maximum depth of the open cut would be about 6 m, 3 m in overburden and 3 m in highly to moderately weathered rock.

The main objective of this alternative is to minimize the environmental impact of the excavation to the site by limiting the extent of the surface excavation required to develop the tunnel portal. However, due to the low bedrock level and the relatively level bedrock surface at the tunnel outlet, the initial section of the tunnel up to about Borehole 93-2 (approximately

24 m in length) would have little to no rock cover depending on the actual rock profile between Boreholes 93-2 and 93-4. Within this section, the tunnel crown would approximately be located at the overburden/bedrock interface while the walls of the tunnel would be in the highly to moderately weathered rocky. Tunnelling rates through this zone would be slow with support requirements for this mixed face condition relatively high. Beyond this section, rock cover would increase substantially, thus tunnelling conditions should improve, allowing reduction of support requirements.

***Alternative 2: Develop Portal in Area of Deep Rock Cover***

The second alternative is to locate the portal close to the location of Borehole 93-2 where a steep rising rock face is anticipated. This alternative would entail substantial surface excavation to expose the rock face from which the tunnel would start. The length of the open cut could be in excess of 30 m and the depth of the cut would be about 11 to 12 m at the upstream end. Overburden excavation could be up to 7 m and rock excavation would generally be about 3 m but would increase near the portal. Due to the depth of the overburden excavation, the areal extent of the excavation would be fairly substantial if side slopes were used for the overburden excavation. If sheet pile walls or soldier pile and lagging walls were used to reduce the overburden excavation, a substantial length of wall would be required.

From a geotechnical point of view, the merit of this alternative is to provide relatively better tunnelling condition by eliminating the mixed face tunnelling section described in alternative 1. The main concern about this alternative is its environmental impact on the slope since a potentially large open excavation area would be required to develop the portal.

The construction aspects of these two alternatives are presented in subsequent sections of this report. Since the two alternatives will involve very different cost, schedule and environmental implications geotechnical factors will be key in evaluating the final design.

### 6.3 Tunnel Excavation Method and Support

#### *Alternative 1: Develop Portal in Area of Shallow Rock Cover*

The required open excavation for this alternative would be of limited length. The open cut in the overburden could be made with a temporary slope of 1.5:1 (Horizontal:Vertical) whereas the rock portion could be cut with a vertical to subvertical face. Due to the fractured nature of the rock close to the bedrock surface and the susceptibility of the rock to degradation due to exposure to potential wetting and drying cycles, surficial unravelling of the exposed rock face should be anticipated. Undercutting of the siltstone layer could occur due to weathering of the underlying softer mudstones. Shotcreting the exposed rock face would help to seal the rock from degradation. The overburden slope should also be covered with heavy tarps to reduce the potential of surficial washing of the overburden slopes along the approach to the tunnel portal.

Since there is little or no rock cover for the initial section of the tunnel, the excavation should be portalled by setting up a series of steel sets in front of the portal from which tunnelling can be developed. The exposed rock face at the portal should be shotcreted to stabilize the rock prior to tunnelling. To advance the tunnel, a forepoling technique consisting of the use of timber forepoling or steel poling plates would be required to temporarily support the overburden or fractured rock ahead of the face prior to tunnel advance. Manual or mechanical equipment (such as hydraulically operated rock breakers) would likely be required to advance this section of the tunnel.

Tunnel support would be in the form of steel set support, tightly blocked and fully lagged to support the overburden and side pressures. For design of the steel set support, the following design pressures are recommended:

$$P_v = \gamma_{ob} D B_t \text{ [kN / m length of tunnel]}$$

If vertical rock cut is stable:

$$P_{hmin} = \text{minimal}$$

If vertical rock cut exhibits squeezing into the opening:

$$P_{hmax} = K_o \gamma_{ob} D H_t + 0.5 K_o \gamma_{rock} H_t^2 \text{ [kN / m length of tunnel]}$$

where:  $\gamma_{ob}$  = unit weight of the overburden (22.9 kN/m<sup>3</sup>)

$\gamma_{rock}$  = unit weight of the rock (26.4 kN/m<sup>3</sup>)

$K_o$  = coefficient of earth pressure at rest (0.5)

$D$  = depth of overburden cover above crown (m)

$H_t$  = height of the tunnel (m)

$B_t$  = width of the tunnel (m)

It is assumed that little arching could be developed in the overburden material with the water table approximately at the tunnel invert (based on the piezometric levels recorded up to June, 15 days after completion of drilling). As the rock is relatively soft the legs of the steel sets should be strapped one to each other and each set should be dowelled into the rock to provide stability to the sets. The use of steel set support would likely continue until the tunnel has reached the position of Borehole 93-2. The steel set support and lagging should be kept no more than 1 m back from the face of the tunnel at all times to avoid loosening of the rock and overburden.

Based on the drill core data, beyond Borehole 93-2 the rock quality should improve and rock cover over the tunnel would increase. In this zone the frequency of fractures/jointing in the rock is also expected to decrease as indicated by the borehole records. Consequently, the mechanical method of excavation may become less efficient particularly where layers of relatively competent siltstones are present with the mudstone. Use of a drill-and-blast procedures will probably be required once the more competent sections of rock have been reached. Carefully controlled blasting techniques however are critical to minimize overbreak and reduce blast induced damage. The contractor should therefore employ a qualified blaster and the services of a qualified blasting engineer for the design of the blasts. Blast vibration monitoring should also be carried out to ensure that specified vibration limits are not exceeded. No matter what the rock conditions, blasting should be avoided until there is at least 3 m of rock cover over the tunnel crown to avoid destabilization of the overburden slope.

While carefully controlled blasting is considered acceptable for excavating some sections of the tunnel, the environmental constraints on the use of blasting for construction at this site should be reviewed with the Ministry of the Environment (MOE) to ensure that all the environmental concerns related to blasting are properly addressed.

For the rock section of the tunnel, the design considerations for tunnel support design are based on the performance data from similar tunnels in the same rock formation or other rock formations of similar strength and character. Beyond the position of Borehole 93-2, it is anticipated that the rock loading would generally result from surficial spalling due to oversteering and delamination along bedding planes and contacts with reduction zones and siltstone layers within the rock unit. The use of rock bolts, mesh and silica fume shotcrete would provide adequate tunnel support. Welded wire mesh would be required for the roof arch of the tunnel. Shotcrete would be required for the roof arch and sidewalls of the tunnel. In general rock bolts of 1.8 m length and of 25 mm diameter, fully resin grouted would be required to be installed at 1.2 m spacing across the tunnel and 1.5 m centres along the tunnel. Bolting of the sidewall of the tunnel would be optional depending on the encountered conditions. During tunnelling, especially in the more weathered rock zones bolts and mesh support would need to be installed within 2 m of the tunnel face, with shotcrete no more than 6 m applied at some distance behind the face.

Records of the piezometers indicate that the groundwater levels are at the invert of the tunnel at Boreholes 93-1, 93-2 and 93-4. The groundwater level at the shaft location is slightly above the overburden/bedrock interface at Borehole 93-3. Consequently, seepage into the initial section of the tunnel would likely be due to infiltration from surface. As the tunnel is advanced further into the slope, seepage would be controlled by the presence of open fractures in the rock and the hydraulic conductivity of the intact rock. Normally seepage would not be significant and can be controlled by pumping during construction. To facilitate tunnel construction, the tunnel invert should be graded with a slight rise towards the shaft to avoid accumulation of construction water or seepage at the tunnel face.

***Alternative 2: Develop Portal in Area of Deep Rock Cover***

For this alternative, the portal would be developed at an exposed face of the rock after removal of the overburden material. The overburden slopes could be excavated to a temporary slopes of 1.5:1 (Horizontal:Vertical) and the rock sidewalls could be excavated to vertical faces. Alternatively, the overburden excavation could be supported by sheet pile walls or soldier pile and lagging walls. If these methods were to be used, they should be designed for the earth pressure distributions shown in Figure 5. The exposed rock face adjacent to and on either side of the tunnel portal should be supported with patterned fully resin grouted rock bolts on a 1.5 m dice 5 pattern with shotcrete and mesh as required. This support would be required to be fully installed prior to tunnelling. The initial section of the tunnel would then be advanced using by steel set support until the tunnel has reached Borehole 93-2 where rock bolts, wire mesh and shotcrete could again be used for tunnel support.

The recommendations for tunnel support, the discussion on the tunnelling methods and the assessment of the consequences of seepage and the method for seepage control for this alternative would be the same as those for the section of tunnel beyond Borehole 93-2 in the first alternative.

**6.4    Shaft Excavation**

Based on the general arrangement of the shaft structure, the shaft collar would be at elevation 155 m. The internal diameter of the shaft would be 1.2 m except over the upper and lower sections where the diameter would be increased to 1.8 m. The nominal excavated diameter of the shaft would be about 1.8 m with the lower chamber enlarged to about 2.4 m diameter. The shaft would penetrate about 3.2 m of overburden and about 17 m of rock. We understand that the grade will be lowered in the area of the shaft. Since the shaft collar is located at the 155 m elevation bench, it is anticipated that excavation for the cut slope would be completed to elevation 155 m prior to the shaft excavation.



At the proposed shaft location, the shaft would penetrate about 3.2 m of sandy silty clay to sandy clayey silt tills. Below the overburden, the shaft would penetrate about 3.8 m of highly to moderately weathered interbedded silty mudstone with RQDs in the range of 15 to 60 %. Below this weathered zone, the shaft would be excavated through relatively competent interbedded silty mudstone with RQDs in excess of 80 %. Occasional fracture zones would also be present (Record of Borehole sheet 93-3). Excavation through the overburden and highly fractured mudstone at the shaft could be carried out in an open cut with side slopes of 1.5:1 (Horizontal:Vertical). The groundwater level is at about elevation 153 m, which is approximately 2m below the proposed collar elevation.

Alternatively, if the site condition is such that an open cut in the overburden for the shaft excavation is not considered practical, the overburden excavation can be carried out within a braced soldier pile wall installed to bedrock or a bolted liner plated wall or steel sheet pile wall driven to the bedrock. However, the hard overburden material above the bedrock may be difficult for penetration of the sheet piles. The sheeting supporting the overburden should be designed to resist full earth and groundwater pressures as well as the surcharge due to construction loadings as shown in Figure 5. In many cases involving vertical shaft construction the contact zone between the overburden and bedrock exhibits the least desirable characteristics of both materials. For this reason, the earth pressure diagram extends through the overburden and into the highly to moderately weathered rock portion of the shaft.

The shaft construction method to be adopted should be such that it will avoid loss of ground into the shaft during construction, whether the excavation is carried out in open cut or within a sheeted or braced wall. It is a normal shaft sinking technique to use drill-and-blast method or mechanical equipment in the weathered zone. The limitation of mechanical equipment in terms of progress rate and breaking through the siltstone interbeds has been discussed previously in regard to possible tunnelling methods.

If blasting for the shaft excavation becomes necessary due to presence of hard siltstone units, it will be necessary to use carefully controlled techniques to avoid overbreak and possible loosening or undermining of any overburden support system. In addition, where structures may be cast or formed the peak particle velocity resulting from blasting should not exceed the

specified values for the type of structure. The need to review the environmental constraints with MOE on the use of a drill-and-blast method for construction at this site has also been discussed previously in regard to possible tunnelling methods.

In addition to the vertical shaft construction, there is a side tunnel of 1.2 m internal diameter branching into the shaft 7 m below the collar. From the construction point of view, it would be preferable to excavate the side tunnel from the shaft when the shaft is sunk to that level. This tunnel could be excavated to about 6 m from the shaft, where it would join the open cut for the remaining part of this side tunnel excavation. This excavation procedure keeps the shaft excavation a sufficient distance away from the open trench excavation for the side tunnel.

Based on the borehole data, it is not anticipated that stability problems in the rock portion of the shaft will be of significant concern. However surficial spalling of the sidewall may occur due to oversteering around the periphery of the shaft, if concentration of the ground stress around the shaft exceeds the rock mass strength. Surficial unravelling at localized fracture zone and due to drying may also occur during construction. However it is envisaged that conventional rock support using spot bolting will suffice. Typically 1.8 m long, 25 mm diameter, fully resin grouted rock bolts and welded wire mesh placed strategically over the shaft walls will be adequate for safety requirements. Additional bolting would be required at the intersection of the shaft with the lower and upper tunnels. Here rock support at the side tunnel may need to be patterned bolted with 1.2 m long rock bolts with welded wire mesh support. Shotcreting would again be required to protect the rock along the shaft and the side tunnel from spalling due to drying.

#### **6.5 Other Construction Considerations**

It should be noted that the rock mass contains a high percentage of chloride and the groundwater may also have a high chloride content. Chloride contents measured on two samples taken from Borehole 93-3 at an elevation of 140.1 m, indicated relatively low chloride contents of 0.24 mg/g and 0.26 mg/g. However, other excavations in Queenston Shale Formation rocks have encountered high chloride concentrations in the rock, which could

potentially provide a source for brine contamination of the groundwater. It is therefore imperative that the excavated materials from the tunnel and shaft be disposed of in a designated area where the leaching of salts from the stockpile of excavated material would not affect any water course. In addition, disposal of seepage and construction water from the tunnel and shaft should be in accordance with MOE requirements. Based on the test results of the groundwater sample from Borehole 2, (see Table A-1) the chloride and sulphate contents are below the levels for which special provision for sulphate and chloride resistance is required for the concrete structures in contact with the groundwater. However, it should be noted that water from external sources such as the water to be contained in the shaft and tunnel may have higher sulphate and chloride concentrations that require special provisions.

#### **6.6 Design Considerations for Tunnel and Shaft Linings**

The design considerations for the tunnel and shaft linings should focus on the following factors:

- i) The pressure on the linings due to rock loading and hydrostatic pressure; and
- ii) The pressure on the linings due to time-dependent deformation of the rock after the linings are in place.

##### ***Pressure on Lining due to Rock Load and Hydrostatic Pressure***

For the section of the tunnel where the rock cover is minimal, the vertical and side pressures from the overburden and the upper part of bedrock can be compiled using the formulae in Section 6.3. As the primary tunnel support (steel sets) should be designed to resist the range of vertical and side pressures anticipated; the pressure on the final lining should be minimal.

For the tunnel section supported by rock bolts, mesh and shotcrete, the loading on the lining is also considered to be small as the tunnel will be stabilized by the primary tunnel support system prior to placement of the lining. Similarly for the shaft, the shaft walls will be stabilized by the rock bolts, mesh and shotcrete installed prior to the installation of the lining.

For the design of tunnel and shaft lining for hydrostatic pressures, reference should be made to the groundwater levels recorded in the piezometers installed along the tunnel alignment. The water head for the lining design should relate to the piezometric head measured.

***Pressure on Lining due to Time Dependent Rock Deformation***

The time dependent (swelling) deformation of the Queenston Formation has been a subject of extensive research (Lee and Lo, 1989). The swelling mechanism of the mudstone is that the mudstone swells as a result of the dilution of pore water salt concentration resulting in the expansion of spacing between clay particles. Both processes of osmosis (liquid flow from low to high concentration regions) and diffusion (ion transfer from high to low concentration regions) are responsible for the dilution of pore water salt concentration. The necessary and sufficient conditions for swelling of the Queenston rocks to take place are:

- i) relief of initial stresses, this serve as the initiating mechanism;
- ii) accessibility to fresh water; and
- iii) an outward salt concentration gradient from the pore fluid within the rock to the ambient fluid in the tunnel.

If all these conditions are met, swelling of the rock occurs, while no swelling is expected if none of these conditions are satisfied. If only one or two conditions are met, the rock may or may not swell. Swelling is also affected by the calcite content of the rock. The higher the calcite content, the lower will be the swelling potential (defined as the swelling strain per logarithm cycle of time from 10 to 100 days). For Queenston rocks where appreciable swelling potential has been recorded, the calcite contents of the samples generally range from 3 to 6 per cent. It has also to be noted that under sufficient confinement swelling will be suppressed.

For design purposes, it is necessary to know the stress distribution around the tunnel or shaft, the site specific swelling characteristics of the rock, the diffusion rate of the ions, such that

the magnitude of the swelling and the extent of the swelling zone around the tunnel or shaft can be determined. Since the scope of investigation for the site specific characteristics of the rock is limited for this project, the effect of rock swelling on design was assessed on the basis of a set of assumptions for the relevant factors that may influence the swelling of the rock.

Results of laboratory swelling tests on the samples recovered from Boreholes 93-2 and 93-3 are summarized in Table 6. Currently, the results of the chloride testing of the rock suggest that the chloride content is approximately 0.25 mg/g. This, coupled with the high calcite content (19 to 24%) suggests a low swelling potential. Results from the swelling tests are consistent with this assessment.

For the purpose of an assessment for the design of the tunnel and shaft lining for swelling effects, an estimate of the long term deformation of the rock due to swelling was made based on the following assumptions:

- i) the rock would have minimal swelling potential under the in situ groundwater conditions. It would swell when in contact with fresh water which may seep through the tunnel and shaft lining;
- ii) the free swelling potentials of the rock in the vertical and horizontal directions are approximately 0.1% and 0.06% per log cycle of time respectively and swelling would be suppressed when the external pressure exceeds 0.6 MPa in the vertical direction.
- iii) the zone of swelling would be limited to about 3 m from the boundary of the excavation due to the slow rate of diffusion of the ions; and
- iv) the horizontal in situ stress is assumed to be approximately 6 MPa normal to the tunnel and approximately 4 MPa parallel to the tunnel with the vertical stress assumed to be represented by the overburden pressure.

On the basis of these assumptions, the estimated maximum long term deformations of the rock at the springline, haunches and crown of the tunnel are 1.5, 2.5 and 1.5 mm respectively.

Similarly the estimated maximum long term deformations of the intact rock mass around the shaft would be about 0.5 mm. The estimated magnitudes of rock deformation are the unrestrained deformations and do not take into account interaction with the concrete lining. Based on the laboratory test results, the swelling in the vertical direction would be suppressed by about 0.6 MPa. No confined swelling tests were carried out on horizontal samples. Based on the ratio of the vertical and horizontal swelling potential of Queenston Shale at other sites, it is inferred that the suppression pressure for horizontal swelling at this site could be about 0.4 MPa.

Available field data from similar sized tunnels excavated elsewhere in this Queenston Formation rocks indicate that there is no appreciable time dependent deformation if the tunnel is relatively dry or if it is not subjected to any fresh water influx. Rock deformation due to stress relief would generally stabilize within 10 to 30 days after completion of excavation. Thus, if the lining is installed after about 30 days, there would be minimal pressure on the lining due to stress relief effects. This should however be verified in the field by monitoring the rock deformation during construction of the tunnel and shaft. The monitoring scheme would comprise the installation of convergence points on the walls of the tunnel and shaft at selected sections. Movements across these convergence points would be measured by means of a tape extensometer.

Rock swelling however can take place when the structure is in operation, during which time there would be fresh water within the structure such that there would be a potential for the water to seep out of the lining into the surrounding rock. This would initiate the swelling of the rock, which would then impose pressure on the lining. The above estimates of the long term deformations of the rock and the estimate of suppression pressure will provide a basis for the lining design.

## 6.7 Geotechnical Review

We recommend that when the design of the tunnel and shaft excavation is underway, the geotechnical aspects of the designs and drawings be reviewed to ensure that the recommendations presented in the report are applicable to the designs as finalized, particularly

with regard to the section of tunnel where the rock cover is very shallow. We are also available to provide assistance in the preparation of the technical specifications related to the geotechnical aspects of the project and in the evaluation of the tenders.

During construction, we would recommend that regular visits to the site be made by an experienced geotechnical engineer to inspect and interpret the exposed subsurface condition, to review the stability of the tunnel and shaft excavation and to interpret the results of the displacement monitoring program. This type of on-site review would ensure that the geotechnical aspects are adequately covered during the construction phase of the project particularly when tunnelling through the section where rock cover to the tunnel is very shallow.

It is essential that the Contractor ensures that the construction procedures, methods and equipment which are intended to be adopted are appropriate for the range of geotechnical conditions likely to be encountered on the project.

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**TABLE 1****SUMMARY OF BOREHOLE LOCATIONS AND DEPTHS**

Borehole Reference Number	Locations of Borehole (1)		Distance from Proposed Drop Shaft (m)	Surface Elevation (2) (m)
	Northing	Easting		
93-1A	4 808 678.5	279 283.5	69	140.2
93-1B	4 808 678.8	279 283.0	69	140.2
93-2	4 808 702.9	279 300.3	40	146.9
93-3	4 808 735.9	279 322.0	0	162.6
93-4	4 808 684.7	279 297.2	56	141.2

Borehole Reference Number	Depth of Hole (m)	Bedrock Thickness (3) (m)	Overburden Thickness (m)	Depth of Rock Cover Above Crown of Proposed Tunnel (m)
93-1	9.30	6.80	2.50	0.0
93-2	15.67	15.06	0.61	7.7
93-3	20.72	19.99	10.73	14.1
93-4	9.30	6.22	3.08	0.3

## Notes:

1. Borehole locations provided by MTO Surveyors.
2. All elevations are referred to Canadian Geodetic Datum.
3. Thickness of bedrock intersected in borehole.

**TABLE 2****SUMMARY OF BEDROCK FEATURES**

Borehole Reference Number (Elevation), m	Depth (m)	Elevation (m)	Thickness (m)	Feature
93-1A (140.19)	2.44	137.75	-	Bedrock Surface
	2.50	137.69	0.12	Siltstone Layer
	3.05	137.14	-	End of Drillhole
93-1B (140.20)	2.50	137.70	-	Bedrock Surface
	2.61	137.59	0.21	Siltstone Layer
	3.38	136.82	0.06	Siltstone Layer
	3.84	136.36	0.06	Siltstone Layer
	4.08	136.12	0.06	Siltstone Layer
	4.23	135.97	0.03	Siltstone Layer
	4.53	135.67	0.03	Siltstone Layer
	5.43	134.77	0.07	Soft Rock Layer
	5.99	134.21	0.03	Siltstone Layer
	6.18	134.02	0.09	Siltstone Layer
	6.79	133.41	0.15	Siltstone Layer
	7.27	132.93	0.03	Siltstone Layer
	7.70	132.50	0.15	Siltstone Layer
	7.79	132.41	0.03	Siltstone Layer
	8.19	132.01	0.09	Siltstone Layer
	8.55	131.65	0.03	Siltstone Layer
	9.01	131.19	0.03	Siltstone Layer
	9.30	130.90	-	End of Drillhole
93-2 (146.84)	0.61	146.83	-	Bedrock Surface
	1.83	145.01	0.30	Siltstone Layer
	3.10	143.74	0.02	Siltstone Layer
	4.29	142.55	0.05	Siltstone Layer
	4.82	142.02	0.05	Siltstone Layer
	5.08	141.76	0.10	Siltstone Layer
	5.66	141.18	0.05	Siltstone Layer
	4.85	141.99	-	Occasional Vugs
	6.47	140.37	0.07	Siltstone Layer
	6.65	140.19	-	Occasional vugs
	7.95	138.89	0.12	Siltstone Layer
	8.53	138.31	0.46	Siltstone Layer
	9.42	137.42	0.07	Siltstone Layer
	10.03	136.81	0.02	Siltstone Layer
	01.08	136.76	0.02	Siltstone Layer
	10.21	136.63	0.05	Siltstone Layer
	10.92	135.92	0.12	Siltstone Layer
	11.28	135.56	0.10	Siltstone Layer
	11.92	134.92	-	Occasional vugs
	12.42	134.42	0.28	Siltstone Layer
	13.06	133.78	0.23	Siltstone Layer
	13.11	133.73	-	Occasional vugs
	13.67	133.17	0.05	Siltstone Layer
	14.50	132.34	0.03	Siltstone Layer
	14.76	132.08	0.05	Siltstone Layer
	15.52	131.32	0.03	Siltstone Layer
	15.67	131.17	-	End of Drillhole

**TABLE 2 (continued)****SUMMARY OF BEDROCK FEATURES**

Borehole Reference Number (Elevation), m	Depth (m)	Elevation (m)	Thickness (m)	Feature
93-3 (162.5)	10.73	151.77	-	Borehole surface
	11.52	150.98	0.06	Siltstone Layer
	12.47	150.03	0.06	Siltstone Layer
	12.89	149.61	0.06	Siltstone Layer
	14.34	148.16	0.34	Siltstone Layer
	14.69	147.81	0.24	Siltstone Layer
	15.99	146.51	0.03	Siltstone Layer
	16.79	145.71	0.06	Siltstone Layer
	17.25	145.25	0.06	Siltstone Layer
	17.44	145.06	0.07	Siltstone Layer
	17.55	144.95	0.03	Siltstone Layer
	17.64	144.86	0.03	Siltstone Layer
	17.98	144.52	0.18	Siltstone Layer
	18.98	143.52	0.03	Siltstone Layer
	19.45	143.05	0.06	Siltstone Layer
	19.77	142.73	0.03	Siltstone Layer
	19.97	142.53	0.19	Siltstone Layer
	20.35	142.15	0.03	Siltstone Layer
	20.56	141.94	0.09	Siltstone Layer
	21.18	141.32	0.06	Siltstone Layer
	21.95	140.55	0.13	Siltstone Layer
	22.16	140.34	0.05	Siltstone Layer
	22.51	139.99	0.03	Siltstone Layer
	22.56	139.94	-	Gypsum Vein
	22.77	139.73	0.13	Siltstone Layer
	23.06	139.44	0.03	Siltstone Layer
	23.70	138.80	0.03	Siltstone Layer
	23.88	138.62	0.16	Siltstone Layer
	24.16	138.34	0.15	Siltstone Layer
	24.57	137.93	0.12	Siltstone Layer
	24.69	137.81	0.12	Siltstone Layer
	25.35	137.15	0.09	Siltstone Layer
	27.74	134.76	0.12	Siltstone Layer
	27.97	134.53	0.15	Siltstone Layer
	28.40	134.1	0.09	Siltstone Layer
	28.53	133.97	-	Gypsum Vein
	28.67	133.83	0.03	Siltstone Layer
	29.17	133.33	-	Gypsum Vein
	29.59	132.91	0.03	Siltstone Layer
	29.75	132.75	0.06	Siltstone Layer
	30.32	132.18	0.03	Siltstone Layer
	30.66	131.84	-	Gypsum Vein
	30.72	131.78	-	End of Drillhole
93-4 (141.17)	3.10	138.07	-	Bedrock Surface
	3.91	137.26	0.08	Siltstone Layer
	5.00	136.17	0.03	Siltstone Layer
	5.28	135.89	-	Occasional Vugs
	5.49	135.68	-	Occasional Vugs
	5.49	135.68	0.10	Siltstone Layer
	6.07	135.10	0.05	Siltstone Layer
	6.68	134.49	0.05	Siltstone Layer
	7.37	133.80	0.23	Siltstone Layer
	7.98	133.19	0.03	Siltstone Layer
	8.81	132.36	0.03	Siltstone Layer
	9.14	132.03	0.03	Siltstone Layer
	9.33	131.84	-	End of Borehole

**TABLE 3****SUMMARY OF JOINT DEPTH, ELEVATION AND DESCRIPTION**

Borehole Reference Number (Elevation)	Depth (m)	Elevation (m)	Dip with Respect to Core Axis (degrees)	Joint Description
93-1B (140.20)	5.49	134.71	45	Stepped
	7.7	132.49	45	Rough
93-2 (146.84)	2.83	144.01	70	Smooth, Stained
	2.90	143.94	70	Smooth, Stained
	3.32	143.52	70	Smooth
	3.51	143.33	30	Uneven, Rough, Soft Infill
	5.58	141.26	70	Rough
	8.08	138.76	70	Smooth
	11.49	135.35	20	Rough
93-3 (162.50)	12.37	150.13	45	Rough
	14.05	148.45	10	Rough
	14.60	147.90	0	Rough, Soft Infill
	14.84	147.66	0	Rough
	15.27	147.23	60	Smooth
	15.33	147.17	70	Smooth
	25.45	137.05	75	Rough
93-4 (141.17)	4.15	137.02	5	Rough, Stained
	5.03	136.14	5	Rough
	5.43	135.44	75	Smooth, Stained
	5.64	135.53	5	Smooth, Stained
	6.77	134.40	5	Rough, Stained
	8.32	132.85	60	Smooth, Stained

**TABLE 4****RESULTS OF AXIAL AND DIAMETRAL POINT LOAD TESTS**

Borehole Number	Sample Depth (m)	Sample Elev. (m)	Lithology	Axial Index Is 50 (MPa)	Diametral Index Is 50 (MPa)	Ratio of Axial to Diametral Strength
93-1B	2.0	138.3	Mudstone	SS	0.169	1.943
	3.0	137.3	Mudstone	SS	0.15	
	4.0	136.3	Mudstone	SS	0.527	
	5.0	135.3	Siltstone	5.480	2.820	
93-2	8.0	138.1	Muddy Siltstone	1.836	0.753	2.438
	9.0	137.1	Mudstone	1.347	1.129	1.193
	10.0	136.1	Muddy Siltstone	1.595	1.129	1.413
	11.0	135.1	Mudstone	1.102	1.317	0.837
93-3	12.0	150.6	Mudstone	0.067	0.151	0.444
	13.0	149.6	Mudstone	0.383	0.226	1.695
	14.0	148.6	Mudstone	4.303	0.263	16.361
	15.0	147.6	Mudstone	SS	0.263	
	15.7	146.9	Siltstone	1.700	1.411	1.205
	16.0	146.6	Mudstone	1.525	0.903	1.689
	17.0	145.6	Mudstone	1.548	0.941	1.645
	18.0	144.6	Siltstone	2.171	1.694	1.282
	19.0	143.6	Mudstone	2.258	0.828	2.727
	19.6	143.0	Siltstone	2.290	1.506	1.521
	20.0	142.6	Mudstone	1.881	0.565	3.329
	21.0	141.6	Mudstone	1.578	2.446	0.645
	21.8	140.8	Siltstone	SS	0.753	
	22.0	140.6	Siltstone	SS	1.035	
	23.0	139.6	Mudstone	2.350	1.882	1.249
	24.0	138.6	Muddy Siltstone	SS	1.506	
	24.4	138.3	Siltstone	3.662	3.011	1.216
	25.0	137.6	Mudstone	3.011	1.317	2.286
	26.0	136.6	Mudstone	2.387	2.258	1.057
	27.0	135.6	Mudstone	1.855	1.035	1.792
	28.0	134.6	Muddy Siltstone	2.530	2.446	1.034
	29.0	133.6	Mudstone	1.773	1.694	1.047
	30.0	132.6	Mudstone	1.694	1.129	1.500
	30.3	132.3	Siltstone	1.827	2.108	0.867
93-4	4.0	137.2	Muddy Siltstone	4.048	0.132	30.667
	5.0	136.2	Mudstone	0.361	0.753	0.479
	6.0	135.2	Mudstone	1.290	1.035	1.246
	6.2	135.0	Siltstone	3.880	1.317	2.946
	7.0	134.2	Mudstone	1.349	0.941	1.434
Average Is 50 (Mudstone)				1.56	0.93	1.67
Average Is 50 (Siltstone)				2.82	1.36	2.06

Notes: SS: Sample sheared rather than fractured vertically during test

**TABLE 5****DEERE AND MILLER'S  
INTACT ROCK STRENGTH CLASSIFICATION**

<b>Grade</b>	<b>Description</b>	<b>Field Identification</b>	<b>Approx. Range of Uniaxial Compressive Strength (MPa)</b>
R0	Extremely weak rock	Indented by thumbnail	1 - 5
R1	Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	5 - 20
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	20 - 25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25 - 50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50 - 100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100 - 250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	> 250

**TABLE 6****RESULTS OF CALCITE CONTENT,  
CHLORIDE CONTENT AND SWELL TESTS**

Borehole	Sample Elevation (m)	Rate of Time Dependent Deformation (% Strain/log cycle of time) <sup>(1)</sup>	Calcite Content (%)	Chloride Content (mg/g)	Test Type
93-2	135.6	0.1%*			Free Swell
	136.3		23.3		
	136.3		25.2		
91-3	139.6	0			Vert. Confined <sup>(2)</sup>
	139.6		18.9		
	139.6		18.7		
	140.0			0.24	
	140.0			0.26	

<sup>(1)</sup> Results after 100 days of testing.

<sup>(2)</sup> Vertical Pressure = 0.59 MPa

\* Extrapolated from data between 30 and 55 days. After 60 days of testing no further swelling was noted.

- Note:
1. For test procedures for swelling tests refer to Lo et. al., 1978
  2. In free swelling test sample was allowed to deform without constraint.
  3. In vertically confined test the sample was confined in the vertical direction by the applied load specified.
  4. See Figures A6 to A7 for results of swell tests up to September 13, 1993

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**DYNAMIC CONE PENETRATION TEST:** CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS:

$c_u$ (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$kPa^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	$m^2/s$	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_f$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	$kg/m^3$	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	$kN/m^3$	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	$kg/m^3$	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	$kN/m^3$	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	$kg/m^3$	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	$kN/m^3$	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	$kg/m^3$	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	$m^3/s$	RATE OF DISCHARGE
$\gamma_d$	$kN/m^3$	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	$kg/m^3$	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	$kN/m^3$	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	$kg/m^3$	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	$kN/m^2$	SEEPAGE FORCE
$\gamma'$	$kN/m^3$	UNIT WEIGHT OF SUBMERGED SOIL						



## LIST OF ABBREVIATIONS

The abbreviation commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

### I. SAMPLE TYPES

*AS* auger sample  
*CS* chunk sample  
*DO* drive open  
*DS* Denison type sample  
*FS* foil sample  
*RC* rock core  
*ST* slotted tube  
*TO* thin-walled, open  
*TP* thin-walled, piston  
*WS* wash sample

### II. PENETRATION RESISTANCES

#### Dynamic Penetration Resistance:

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 0.3 m (12 in.).

#### Standard Penetration Resistance, *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 0.3 m (12 in.).

*WH* sampler advanced by static weight—weight, hammer

*PH* sampler advanced by pressure—pressure, hydraulic

*PM* sampler advanced by pressure—pressure, manual

### III. SOIL DESCRIPTION

(a) <i>Cohesionless Soils</i>	
	' <i>N</i> ' Blows/0.30m or Blows/ft.
Relative Density	
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	kPa	' <i>Cu</i> ' psf.
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	over 200	over 4000

### IV. SOIL TESTS

*C* consolidation test  
*H* hydrometer analysis  
*M* sieve analysis  
*MH* combined analysis, sieve and hydrometer<sup>1</sup>  
*Q* undrained triaxial<sup>2</sup>  
*R* consolidated undrained triaxial<sup>2</sup>  
*S* drained triaxial  
*U* unconfined compression  
*V* field vane test

#### NOTES:

<sup>1</sup>Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

<sup>2</sup>Undrained triaxial tests in which pore pressures are measured are shown as  $\bar{Q}$  or  $\bar{R}$ .

## LIST OF SYMBOLS

### I. GENERAL

$\pi$	= 3.1416
$e$	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of $a$
$\log_{10} a$ or $\log a$	logarithm of $a$ to base 10
$t$	time
$g$	acceleration due to gravity
$V$	volume
$W$	weight
$M$	moment
$F$	factor of safety

### II. STRESS AND STRAIN

$u$	pore pressure
$\sigma$	normal stress
$\sigma'$	normal effective stress ( $\bar{\sigma}$ is also used)
$\tau$	shear stress
$\epsilon$	linear strain
$\epsilon_{xy}$	shear strain
$\nu$	Poisson's ratio ( $\mu$ is also used)
$E$	modulus of linear deformation (Young's modulus)
$G$	modulus of shear deformation
$K$	modulus of compressibility
$\eta$	coefficient of viscosity

### III. SOIL PROPERTIES

#### (a) Unit weight

$\gamma$	unit weight of soil (bulk density)
$\gamma_s$	unit weight of solid particles
$\gamma_w$	unit weight of water
$\gamma_d$	unit dry weight of soil (dry density)
$\gamma'$	unit weight of submerged soil
$G_s$	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
$e$	void ratio
$n$	porosity
$w$	water content
$S_r$	degree of saturation

#### (b) Consistency

$w_L$	liquid limit
$w_P$	plastic limit
$I_P$	plasticity index
$w_s$	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
$D_r$	relative density = $(e_{max} - e) / (e_{max} - e_{min})$

#### (c) Permeability

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

#### (d) Consolidation (one-dimensional)

$m_v$	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
$C_c$	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
$c_s$	coefficient of consolidation
$T_v$	time factor = $c_s t / d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

#### (e) Shear strength

$\tau_f$	shear strength
$c'$	effective cohesion
$\phi'$	effective angle of shearing resistance, or friction
$c_u$	apparent cohesion*
$\phi_u$	apparent angle of shearing resistance, or friction
$\mu$	coefficient of friction
$S_i$	sensitivity

}	in terms of effective stress
$\tau_f = c' + \sigma' \tan \phi'$	

}	in terms of total stress
$\tau_f = c_u + \sigma \tan \phi_u$	

\*For the case of a saturated cohesive soil,  $\phi_u = 0$  and the strength  $\tau_f = c_u$  is taken as half the undrained compressive strength.

## METRIC

Sta. 9+930.89: 0/S 2.50 m Lt.

W P 410-85-00

LOCATION Co-ords. 4, 808, 678.47 N; 279, 283E

ORIGINATED BY JMTW

DIST 4 HWY 403

BOREHOLE TYPE Hollow Stem Augers, NQ-3 Rock Core

COMPILED BY JMTW

DATUM Geodetic

DATE June 03, 1993

CHECKED BY F.J.H.

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

**+<sup>3</sup>, x<sup>5</sup> :** Numbers refer to Sensitivity

20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 93-1B

METRIC

W P 410-85-00 LOCATION Co-ords. 4, 808, 678.83N; 279,282.97E ORIGINATED BY JMTW  
 DIST 4 HWY 403 BOREHOLE TYPE Hollow Stem Augers, NQ-3 Rock Core COMPILED BY JMTW  
 DATUM Geodetic DATE June 03, 1993 CHECKED BY F.J.H.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH m	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
140.2	Ground Surface																GR SA SI CL
0.0	Stiff to very stiff, dry to moist red brown SILTY CLAY, some sand, trace gravel and cobble fragments, occ. grey sandy silt layers. (SCREE)						140										
137.7			1	NQ RC	TCR=48% SCR=27% RQD=8%		138										
2.5	Moderately weathered, thinly to medium bedded red brown, medium grained, non-porous, silty MUDSTONE, inter-bedded with grey, coarse grained, non-porous SILTSTONE (QUEENSTON FORMATION).		2	NQ RC	TCR=100% SCR=64% RQD=36%		136										S
			3	NQ RC	TCR=85% SCR=52% RQD=36%		134										S
133.5			4	NQ RC	TCR=95% SCR=70% RQD=62%		132										S
6.7	Slightly weathered, thinly to medium bedded red brown, medium grained, non-porous, silty MUDSTONE, inter-bedded with grey, coarse grained, non-porous SILTSTONE (QUEENSTON FORMATION)		5	NQ RC	TCR=103% SCR=100% RQD=92%		130										S
130.9																	S
9.3	End of Borehole						130										S
	* Borehole grouted to surface upon completion * TCR: Total Core Recovery SCR: Solid Core Recovery RQD: Rock Quality Designation * S: Siltstone Layer G: Gypsum Vein V: Vugs Present																

W.L. in piezometer at Elevation 133.5 m on June 21, 1993

# RECORD OF BOREHOLE No 93-2

METRIC

W P 410-85-00

LOCATION Co-ords. 4, 808,702.87N; 279, 300.26E

ORIGINATED BY SVB

DIST 4 HWY 403

BOREHOLE TYPE Solid Stem Augers, NQ Rock Core

COMPILED BY JMTW

DATUM Geodetic

DATE May 27, 1993

CHECKED BY F.J.H.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH m	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
146.8	Ground Surface													
0.0	Topsoil		1	SS	6									
146.2														
0.6	Highly weathered, medium grained, silty MUDSTONE. (QUEENSTON FORMATION)		2	SS	34		146							
145.0														
1.8	Moderately weathered, silty MUDSTONE, inter- bedded with grey,		3	NQ RC	TCR= 74% SCR= 66% RQD= 39%									S
144.4														
2.4	coarse grained, non- porous SILTSTONE (QUEENSTON FORMATION)						144							S
			4	NQ RC	TCR= 91% SCR= 79% RQD= 65%									S
	Slightly weathered, thinly to medium bedded red brown, medium grained, non-porous, silty MUDSTONE, inter- bedded with grey,													
	coarse grained, non- porous SILTSTONE (QUEENSTON FORMATION)		5	NQ RC	TCR= 99% SCR= 87% RQD= 76%		142							S
140.9														
5.9	Fresh, thinly to medium bedded, red brown, medium grained, non- porous, silty MUDSTONE, interbedded with grey, coarse grained, non- porous SILTSTONE (QUEENSTON FORMATION)		6	NQ RC	TCR= 93% SCR= 93% RQD= 84%		140							S
			7	NQ RC	TCR= 100% SCR= 98% RQD= 97%									S
							138							S
			8	NQ RC	TCR= 92% SCR= 74% RQD= 70%									S
			9	NQ RC	TCR= 100% SCR= 92% RQD= 88%		136							S
			10	NQ RC	TCR= 100% SCR= 100% RQD= 57%									S
							134							S
			11	NQ RC	TCR= 100% SCR= 100% RQD= 81%									S
			12	NQ RC	TCR= 91% SCR= 81%		132							S

+3, x5: Numbers refer to  
Sensitivity

20  
15-5 (%) STRAIN AT FAILURE  
10

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 93-2 (cont.) METRIC

W P 410-85-00 LOCATION Sta. 9+960.50; O/S 3.20m Lt. ORIGINATED BY SVB  
 DIST 4 HWY 403 BOREHOLE TYPE Solid Stem Augers. NO Rock Core COMPILED BY JMTW  
 DATUM Geodetic DATE May 27, 1993 CHECKED BY F.J.H.

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 (%) GR SA SI CL
ELEV DEPTH m	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
131.2			12	NQ RC	RQD=53%												
15.6	End of Borehole						131										
	* Borehole grouted to surface upon completion * TCR: Total Core Recovery SCR: Solid Core Recovery RQD: Rock Quality Designation * S: Siltstone Layer G: Gypsum Vein V: Vugs Present																

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 93-3

METRIC

W P 410-85-00 LOCATION Co-ords. 4, 808, 735.93 N; 279, 321.95 E ORIGINATED BY JMTW  
 DIST 4 HWY 403 BOREHOLE TYPE Solid Stem Augers, NQ-3 Rock Core, NW Casing COMPILED BY JMTW  
 DATUM Geodetic DATE May 27, 1993 CHECKED BY F.J.H.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH m	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
162.6	Ground Surface																
162.4	Topsoil																
0.2																	
	Very stiff to hard, dry mottled brown sandy CLAYEY SILT, some gravel, occ. mudstone fragments and oxidation partings. (TILL)		1	SS	25		162							o			
			2	SS	33									o			
			3	SS	46		160							o			
			4	SS	72									o			
			5	SS	71									o			
157.7			6	A B	SS 80/175mm		158							o			
4.9	Very dense, dry to moist, sandy SILT, trace clay, trace gravel. (TILL)		7	SS	77/75mm									o			17 25 41 17
157.4			8	SS	105/100mm									o			
5.2	Hard, dry, grey brown to red brown sandy SILTY CLAY, some gravel, trace weathered mudstone fragments. (TILL)		9	SS	110/150mm		156							o			
			10	SS	88									o			
154.8			11	SS	118/250mm		154							o			
			12	SS	77/200mm									o			
			13	SS	111/75mm									o			20 35 37 8
151.8			14	SS	116/300mm		152							o			
10.7	Highly weathered, thinly to medium bedded red brown, medium grained, non-porous, silty MUDSTONE. (QUEENSTON FORMATION)		15	SS	110/225mm									o			S
			16	NQ RC	TCR=95% SCR=65% RDP=59%									o			S
150.1			17	NQ RC	TCR=95% SCR=68% RDP=52%		150							o			S
12.4	Moderately weathered, thinly to medium bedded red brown, medium grained, non-porous, silty MUDSTONE, inter-bedded with grey, coarse grained, non-porous SILTSTONE (QUEENSTON FORMATION)		18	NQ RC	TCR=97% SCR=62%		148							o			S

+3, x5: Numbers refer to 20  
Sensitivity 15-5 (%) STRAIN AT FAILURE  
10

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 93-3 (cont.) METRIC

Sta. 10+000.00; O/S 5.00 m Lt.

W P 410-85-00

LOCATION Co-ords. 4, 808, 735.93N; 279, 321.95E

ORIGINATED BY JMTW

DIST 4 HWY 403

BOREHOLE TYPE Solid Stem Augers, NQ-3 Rock Core, NW Casing

COMPILED BY JMTW

DATUM Geodetic

DATE May 27, 1993

CHECKED BY F.J.H.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH m	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
								SHEAR STRENGTH kPa							PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
							WATER CONTENT (%)			10	20	30					
147.1			18		RQD=28%												
15.5	Slightly weathered, thinly to medium bedded red brown, medium grained, non-porous, silty MUDSTONE, interbedded with grey, coarse grained, non-porous SILTSTONE (QUEENSTON FORMATION)		19	NQ RC	TCR=97% SCR=97% RQD=97%		146								S		
			20	NQ RC	TCR=95% SCR=90% RQD=90%		144								S		
			21	NQ RC	TCR=100% SCR=100% RQD=100%										S		
			22	NQ RC	TCR=100% SCR=93% RQD=75%		142								S		
			23	NQ RC	TCR=98% SCR=96% RQD=95%		140								S		
141.0	Fresh, thinly to medium bedded, red brown, medium grained, non-porous, silty MUDSTONE, interbedded with grey, coarse grained, non-porous SILTSTONE (QUEENSTON FORMATION)		24	NQ RC	TCR=100% SCR=99% RQD=99%		138								S		
21.6			25	NQ RC	TCR=100% SCR=99% RQD=100%										S		
			26	NQ RC	TCR=100% SCR=100% RQD=100%		136								S		
			27	NQ RC	TCR=97% SCR=97% RQD=97%		134								S		
			28	NQ RC	TCR=100%										S		

OFFICE REPORT ON SOIL EXPLORATION



# RECORD OF BOREHOLE No 93-3(cont.) METRIC

Sta. 10+000.00; O/S 5.00 m Lt.

W P 410-85-00 LOCATION Co-ords. 4, 808,735.93N; 279,321.95E ORIGINATED BY JMTW  
 DIST 4 HWY 403 BOREHOLE TYPE Solid Stem Augers, NQ-3 Rock Core, NW Casing COMPILED BY JMTW  
 DATUM Geodetic DATE May 27, 1993 CHECKED BY F.J.H.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH m	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100					
131.8			28	NQ RC	SCR = 100% ROD = 97%	132										
30.7	End of Borehole															
	* Borehole grouted to surface upon completion * TCR: Total Core Recovery SRC: Solid Core Recovery ROD: Rock Quality Designation * S: Siltstone Layer G: Gypsum Vein V: Vugs Present					130										

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 93-4

METRIC

W P 410-85-00

LOCATION

Sta. 9+944.00; 0/S 5.00 m Rt.  
Co-ords. 4, 808, 684.69N; 279, 297.23E

ORIGINATED BY SVB

DIST 4 HWY 403

BOREHOLE TYPE Solid Stem Augers, NO Rock Core

COMPILED BY JMTW

DATUM Geodetic

DATE June 01, 1993

CHECKED BY F.J.H.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH m	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100					
141.2	Ground Surface															
0.0	Topsoil		1	SS	11											
140.6																
0.6	Stiff to very stiff, moist, red brown, sandy CLAYEY SILT, some sand, trace gravel, occ. cobble fragments. (SCREE)		2	SS	17											
			3	SS	17											
			4	SS	17											
138.1																
3.1	Moderately weathered, thinly to medium bedded red brown, medium grained, non-porous, silty MUDSTONE, inter- bedded with grey, coarse grained, non- porous SILTSTONE (QUEENSTON FORMATION)		5	SS	60											
			6	NQ RC	TCR= 97% SCR= 87% RQD= 35%											
135.7																
5.5	Slightly weathered, thinly to medium bedded red brown, medium grained, non-porous, silty MUDSTONE, inter- bedded with grey, coarse grained, non- porous SILTSTONE (QUEENSTON FORMATION)		7	NQ RC	TCR= 100% SCR= 74% RQD= 57%											
			8	NQ RC	TCR= 100% SCR= 82% RQD= 68%											
			9	NQ RC	TCR= 100% SCR= 83% RQD= 82%											
131.8																
9.4	End of Borehole															
	* Borehole grouted to surface upon completion.															
	* TCR: Total Core Recovery SCR: Solid Core Recovery RQD: Rock Quality Designation															
	* S: Siltstone Layer G: Gypsum Vein V: Vugs Present															

+3, x5: Numbers refer to  
Sensitivity

20  
15  $\div$  5 (%) STRAIN AT FAILURE  
10

OFFICE REPORT ON SOIL EXPLORATION

# BARRINGER LABORATORIES

5735 McAdam Road  
Mississauga, Ontario  
L4Z 1N9  
Tel: (416) 890-8588  
Fax: (416) 890-8575  
Wats: 1-800-263-9040

15-Jul-93

GOLDER ASSOCIATES  
2180 Meadowvale Boulevard  
Mississauga, ON  
L5N 5S3

Page: 1  
Copy: 1 of 1  
Set: 1

Attn: Mr. John Wilkinson  
Project: 931-1346

PO #:

Received: 22-Jun-93 09:02

Job: 935933

Status: Final

## Water Samples

Sample Id	pH pH Elec. pH Units	SO4= IC mg/L	Cl- IC mg/L
W1 2	7.33	119.	44.0
Blank	5.42	<0.5	<0.1
QC Standard (actual)	4.46	599.	252.
QC Standard (expected)	4.45	600.	250.
Repeat	7.33	120.	38.8

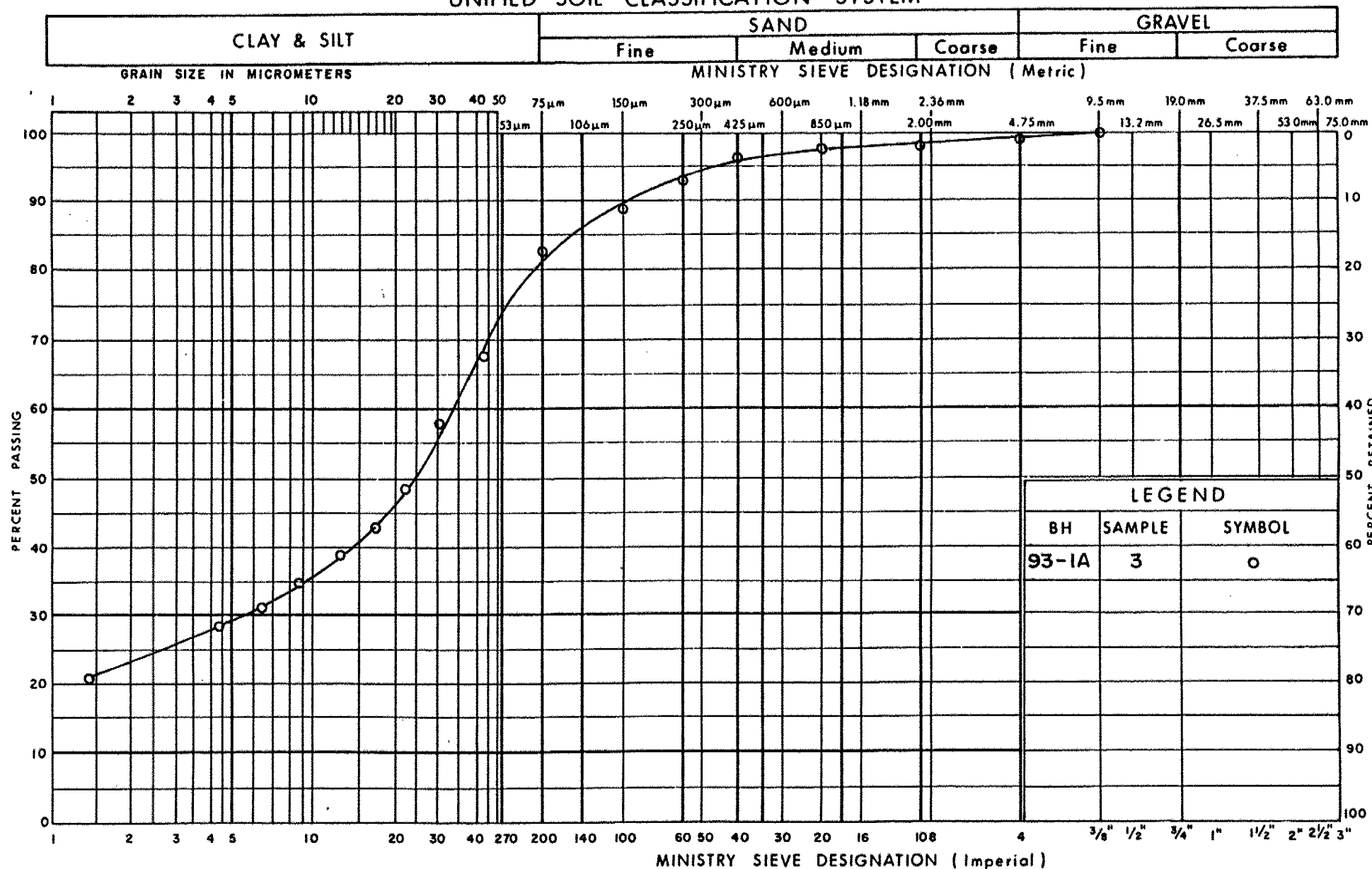
TABLE A-1

Job approved by:

Signed: 

.....  
Mike Muneswar  
Manager, Environmental Inorganic Services

## UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

Ministry of  
Transportation

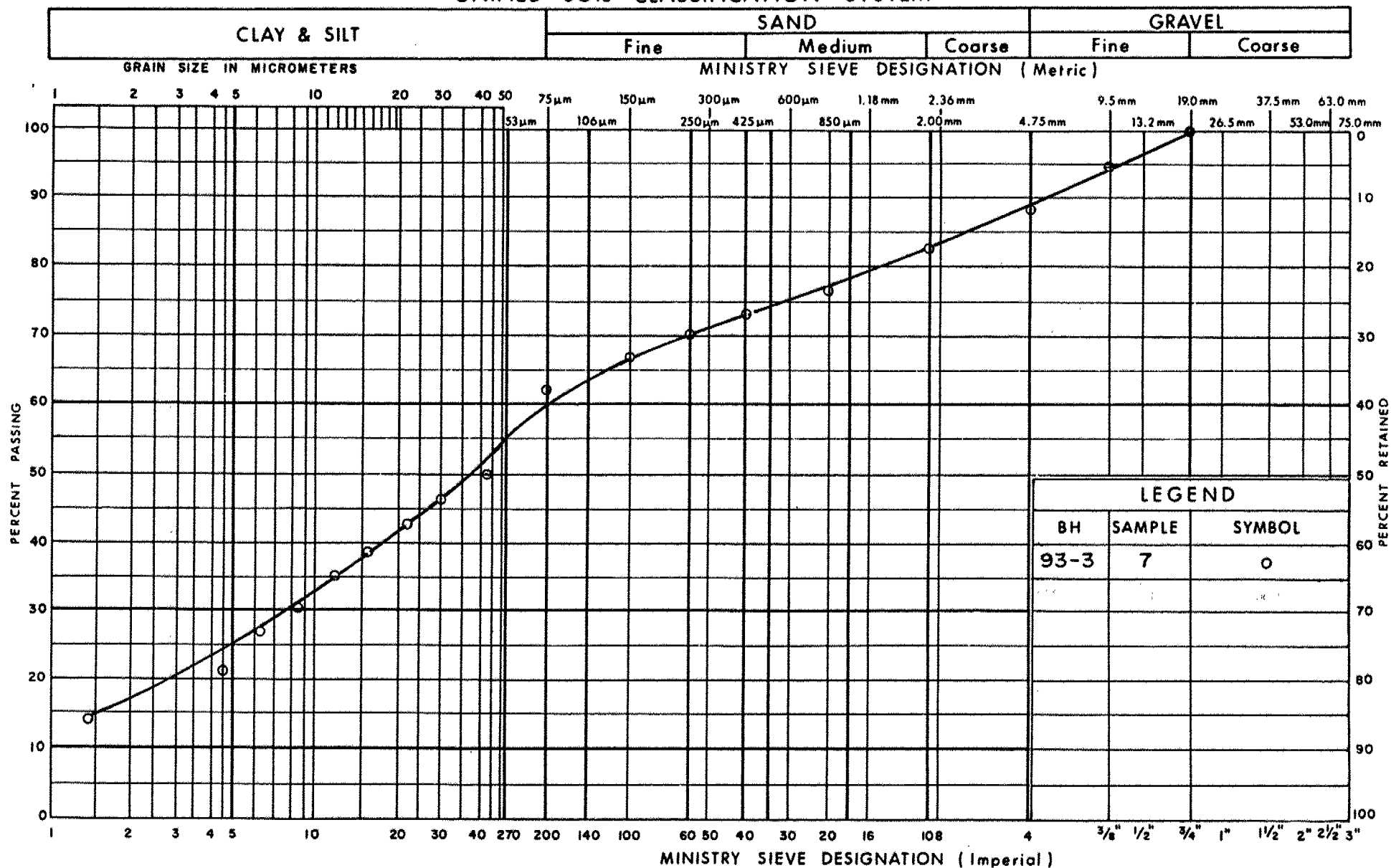
## GRAIN SIZE DISTRIBUTION

### SILTY CLAY (SCREE)

FIG No    A-1

W P    410-85-00

## UNIFIED SOIL CLASSIFICATION SYSTEM

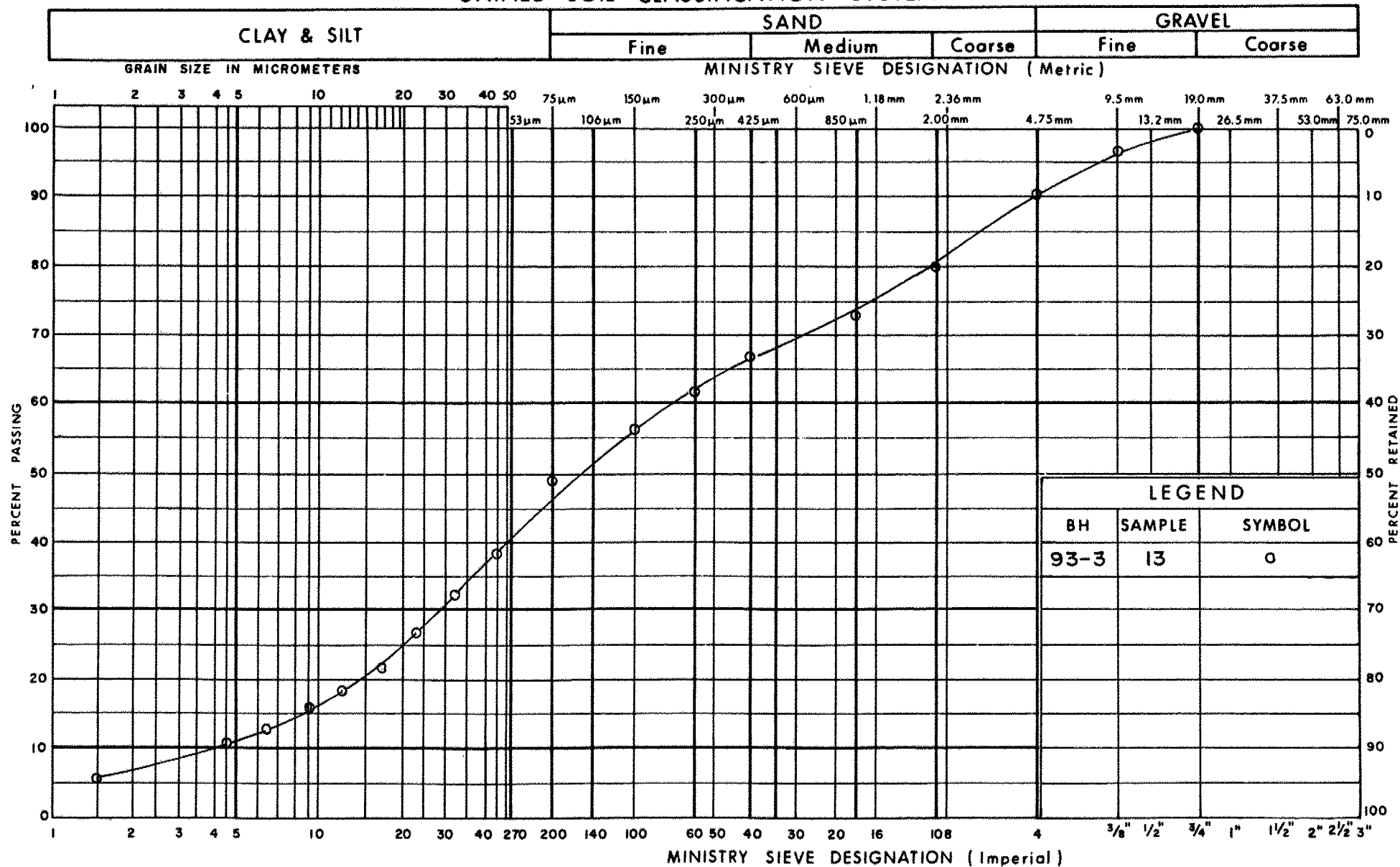


Ministry of  
Transportation

**GRAIN SIZE DISTRIBUTION**  
SANDY SILTY CLAY ( TILL )

FIG No A-2  
W P 410-85-00

## UNIFIED SOIL CLASSIFICATION SYSTEM

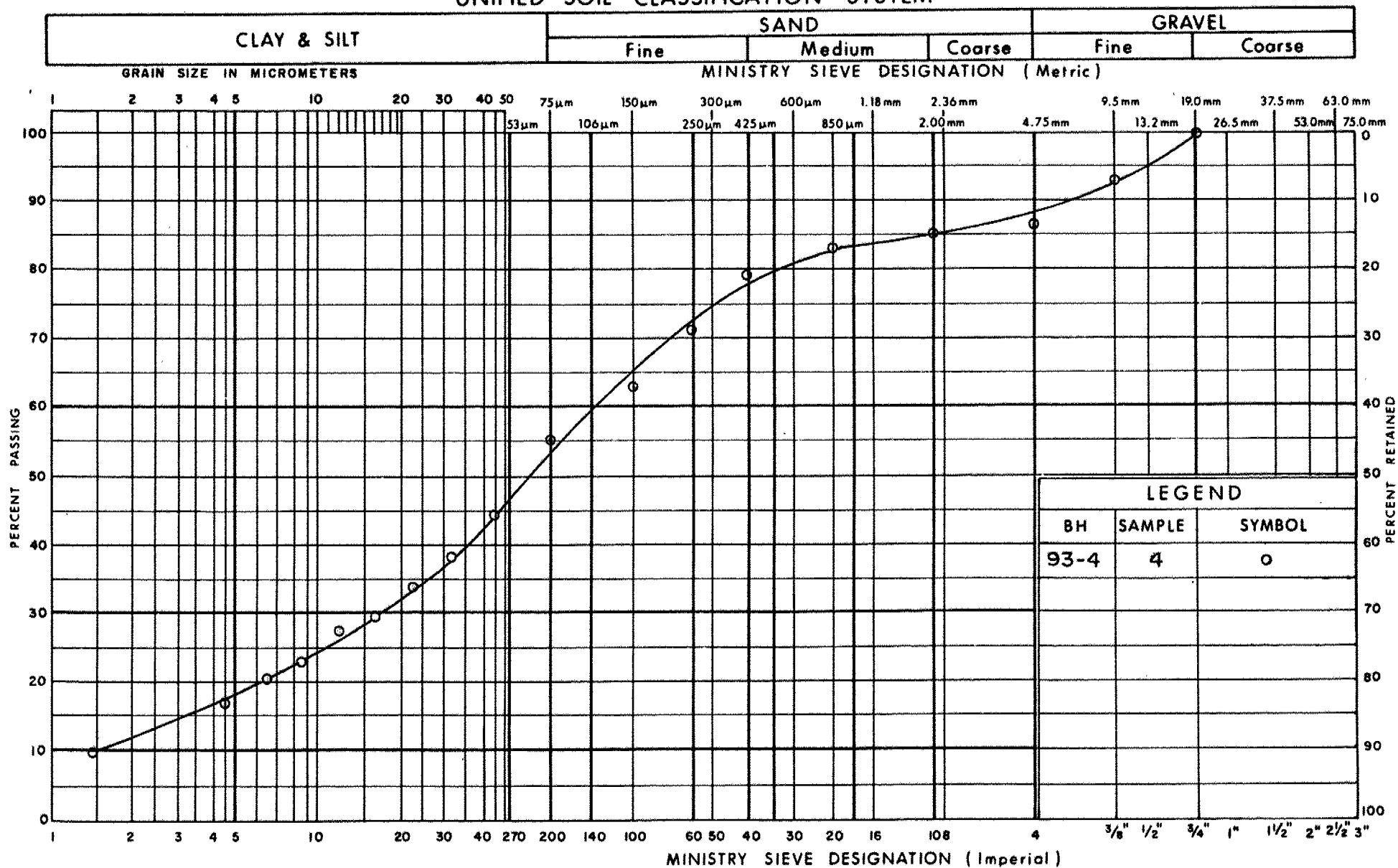


Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
SANDY CLAYEY SILT (TILL)

FIG No A-3  
WP 410-85-00

## UNIFIED SOIL CLASSIFICATION SYSTEM



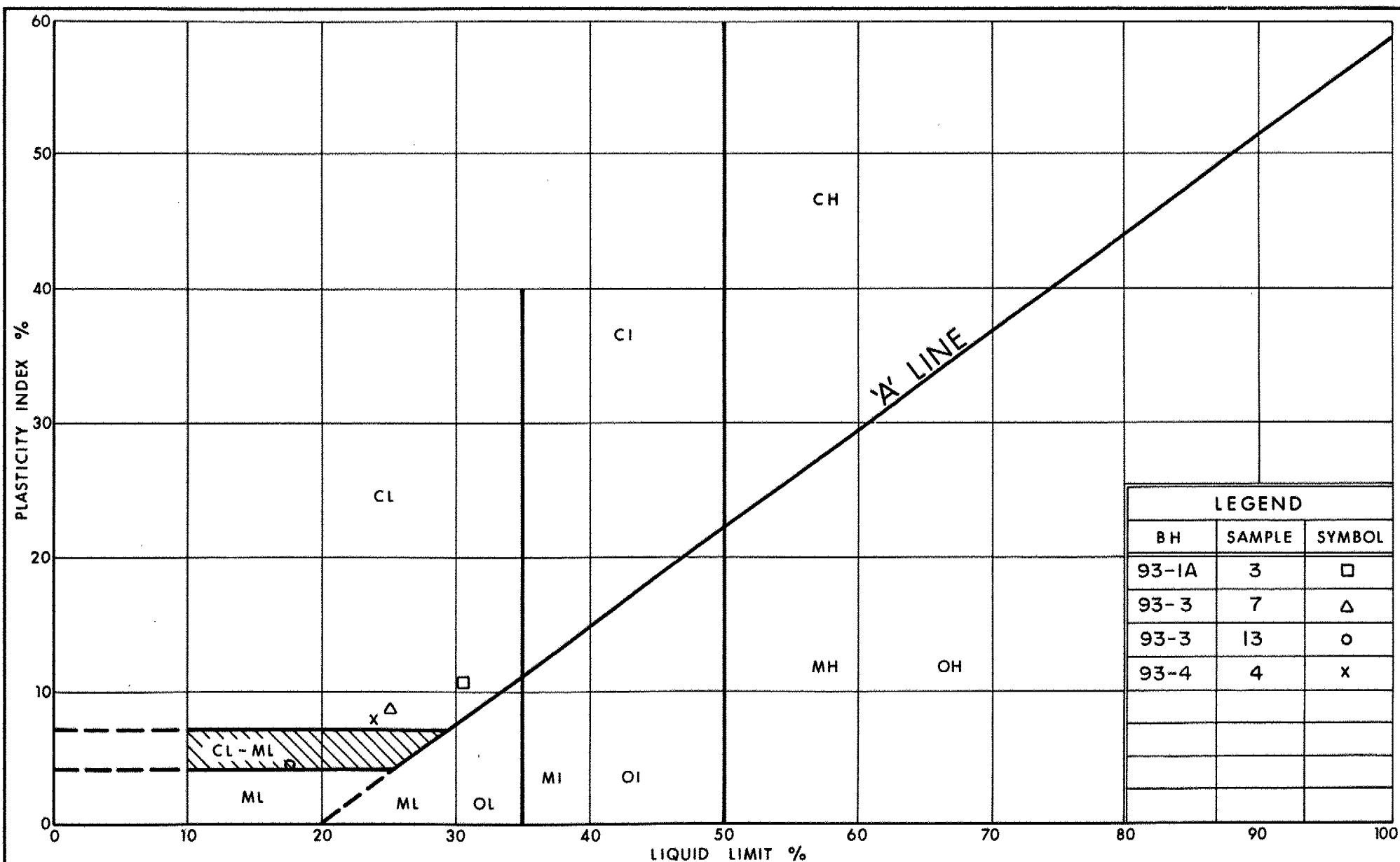
Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

### SANDY CLAYEY SILT (SCREE)

FIG No A-4

W P 410-85-00



Ministry of  
Transportation  
Ontario

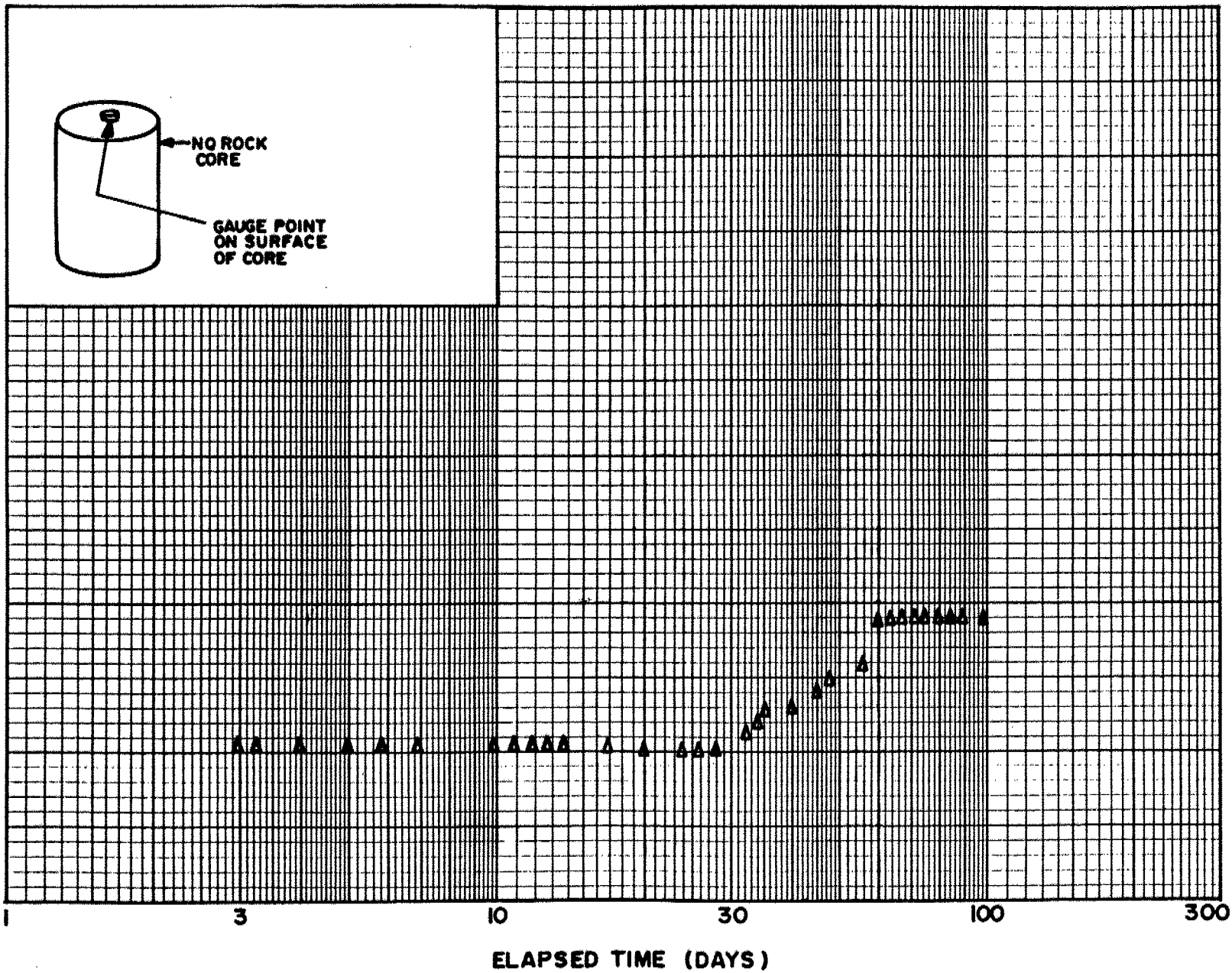
## PLASTICITY CHART

FIG No A-5  
W P 410-85-00



FREE SWELLING TEST  
SAMPLE LOCATION: BOREHOLE 93-2 (ELEV. 135.6m)

FIGURE A-6



Date JUNE 18, 1993  
Project 93-1346

Golden Associates

Drawn MHW  
Ck'd CH

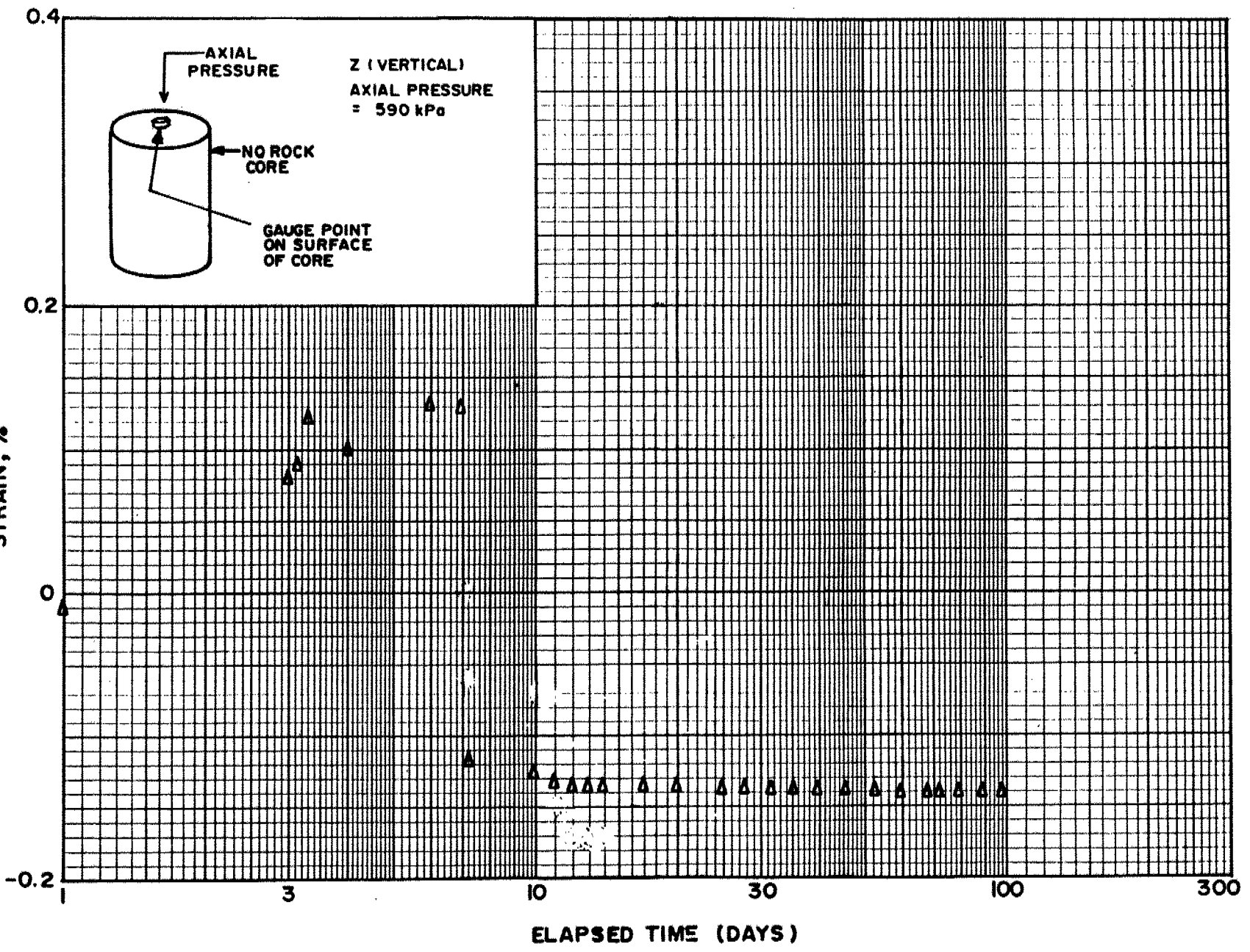
SEMI - CONFINED SWELLING TEST  
SAMPLE LOCATION : BOREHOLE 93-3 (ELEV.139.8m)

FIGURE A-7

Date JUNE 18, 1993  
Project 93-1-1346

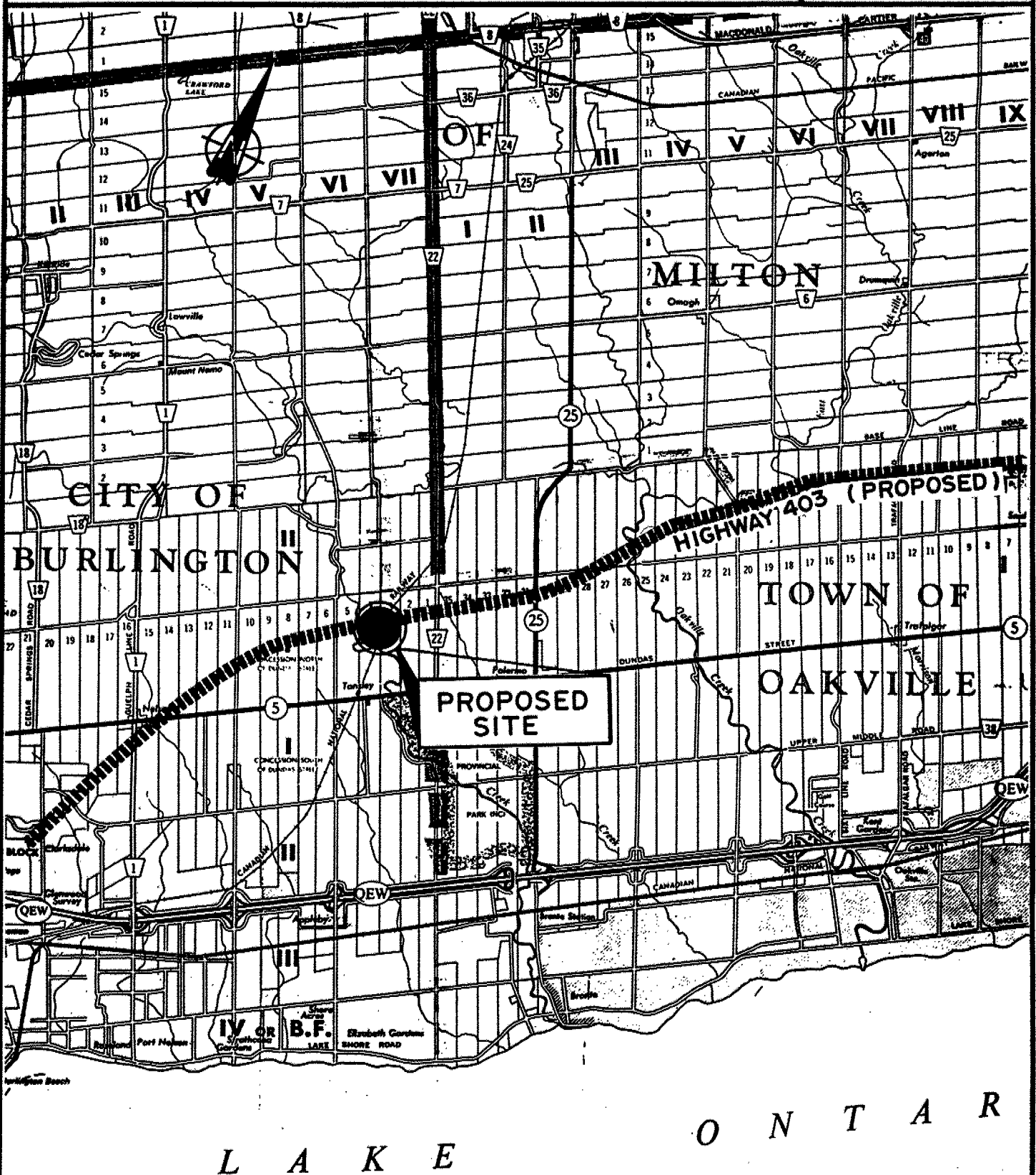
**Goldier Associates**

Drawn MHW  
Ckcd [Signature]



# SITE LOCATION PLAN

FIGURE I



Date JUNE 17, 1993

Project 931-1346

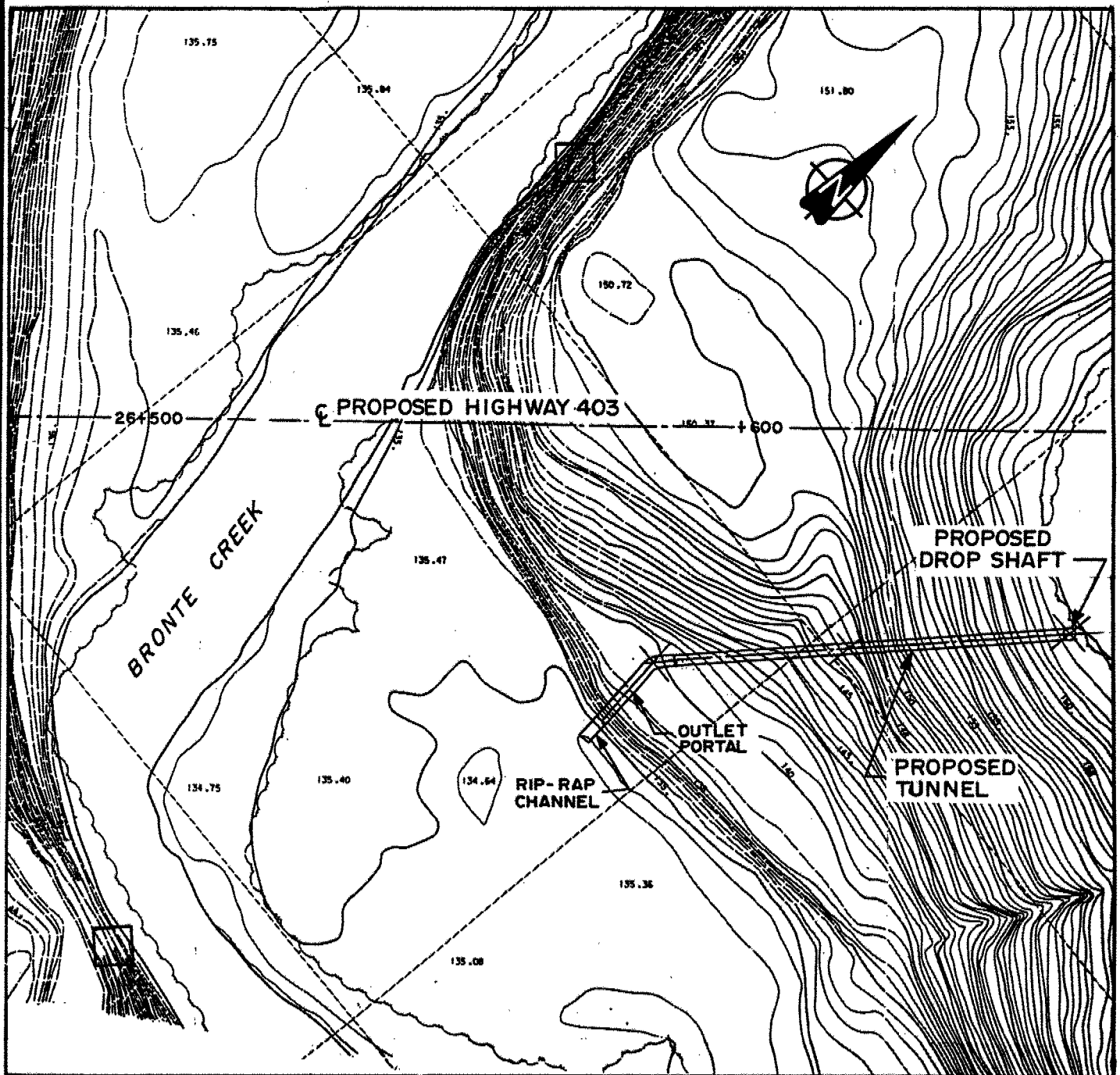
Golder Associates

Drawn MHW

Chkd. *[Signature]*

# MAPPED OUTCROP LOCATIONS ALONG BRONTE CR.

FIGURE 2



□ OUTCROPS USED TO MAP  
VERTICAL TO SUBVERTICAL  
JOINTING

SCALE 1 : 1000

NOTES : 1) CONTOUR ELEVATIONS IN METRES  
ABOVE GEODETIC DATUM.

Date JUNE 17, 1993

Project 931-1346

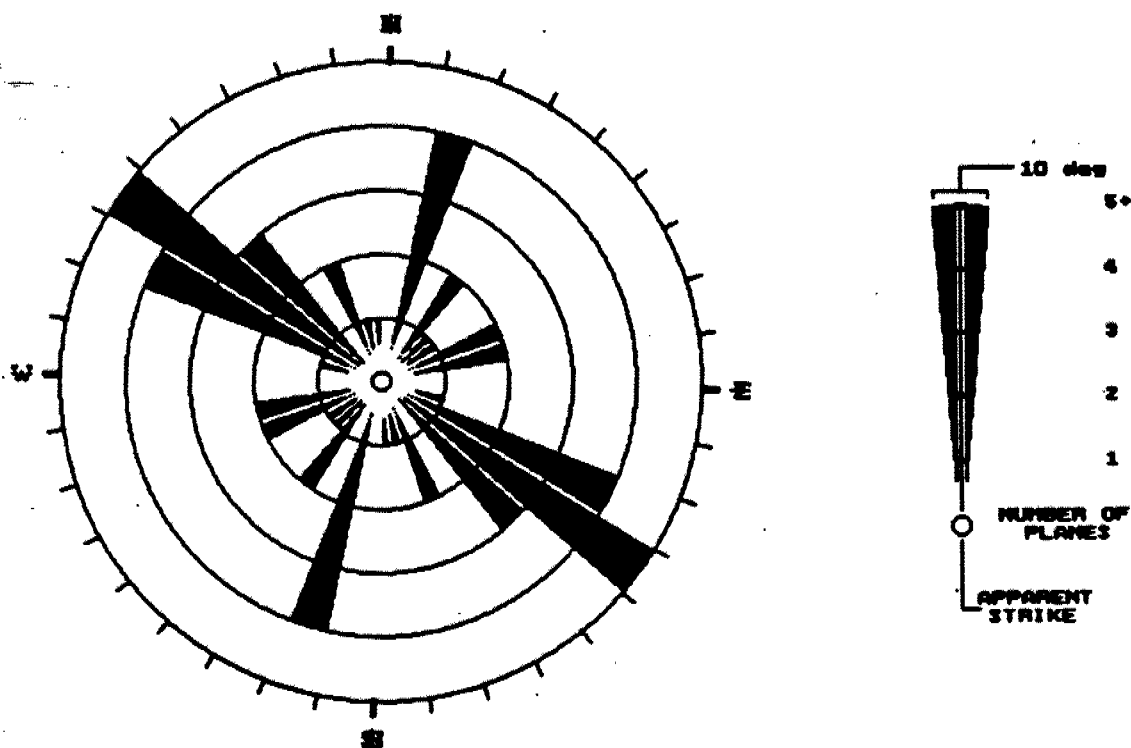
**Golder Associates**

Drawn MHW

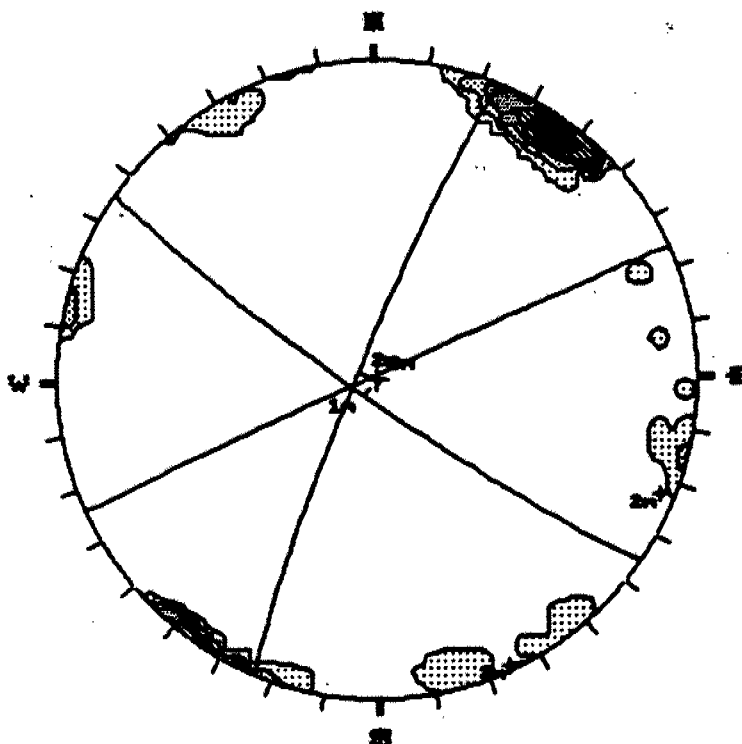
Chkd. [Signature]

# RESULTS OF STRUCTURAL MAPPING AT OUTCROPS ALONG BRONTE CREEK

FIGURE 3



ROSE DIAGRAM



LOWER HEMISPHERE, EQUAL AREA STEREO NET

- NOTES: 1) FOR OUTCROP LOCATIONS, SEE FIGURE 2.  
2) ALL ORIENTATIONS MEASURED WITH RESPECT TO TRUE NORTH.

Date JUNE 17, 1993  
Project 931-1346

**Golder Associates**

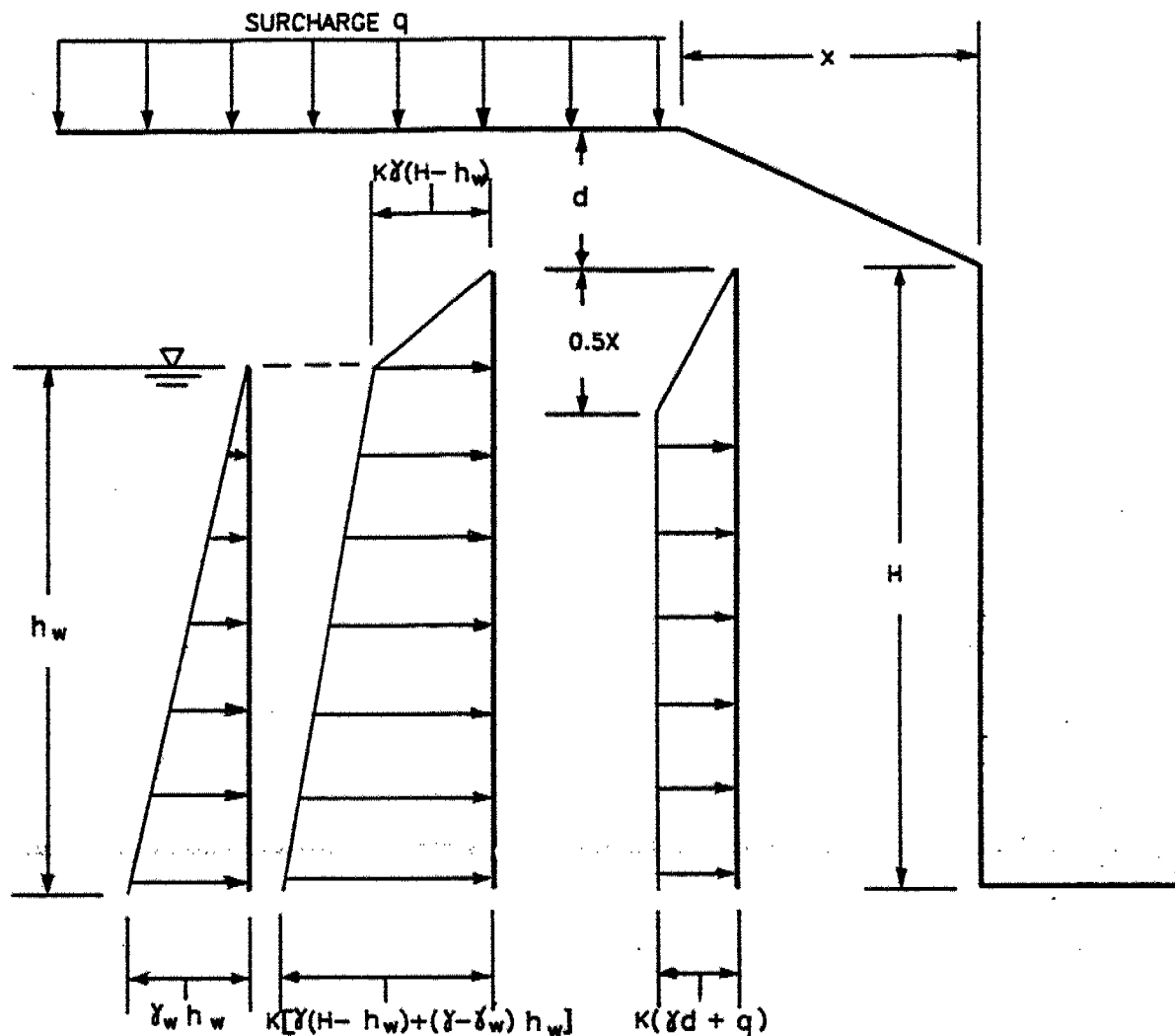
Drawn MHW  
Chkd. cy

FORM 17000000 JUL 1989

FORM 17000000 JUL 1989

# DESIGN LATERAL EARTH PRESSURES TEMPORARY SUPPORT SYSTEM

FIGURE 5



$\gamma$  = UNIT WEIGHT OF SOIL

$\gamma_w$  = UNIT WEIGHT OF WATER

$K$  = EARTH PRESSURE COEFFICIENT

(REFER TO TEXT OF REPORT FOR DESIGN VALUES)

N.B. WHERE THERE IS NO SLOPING GROUND BEHIND WALL,  $d=0$

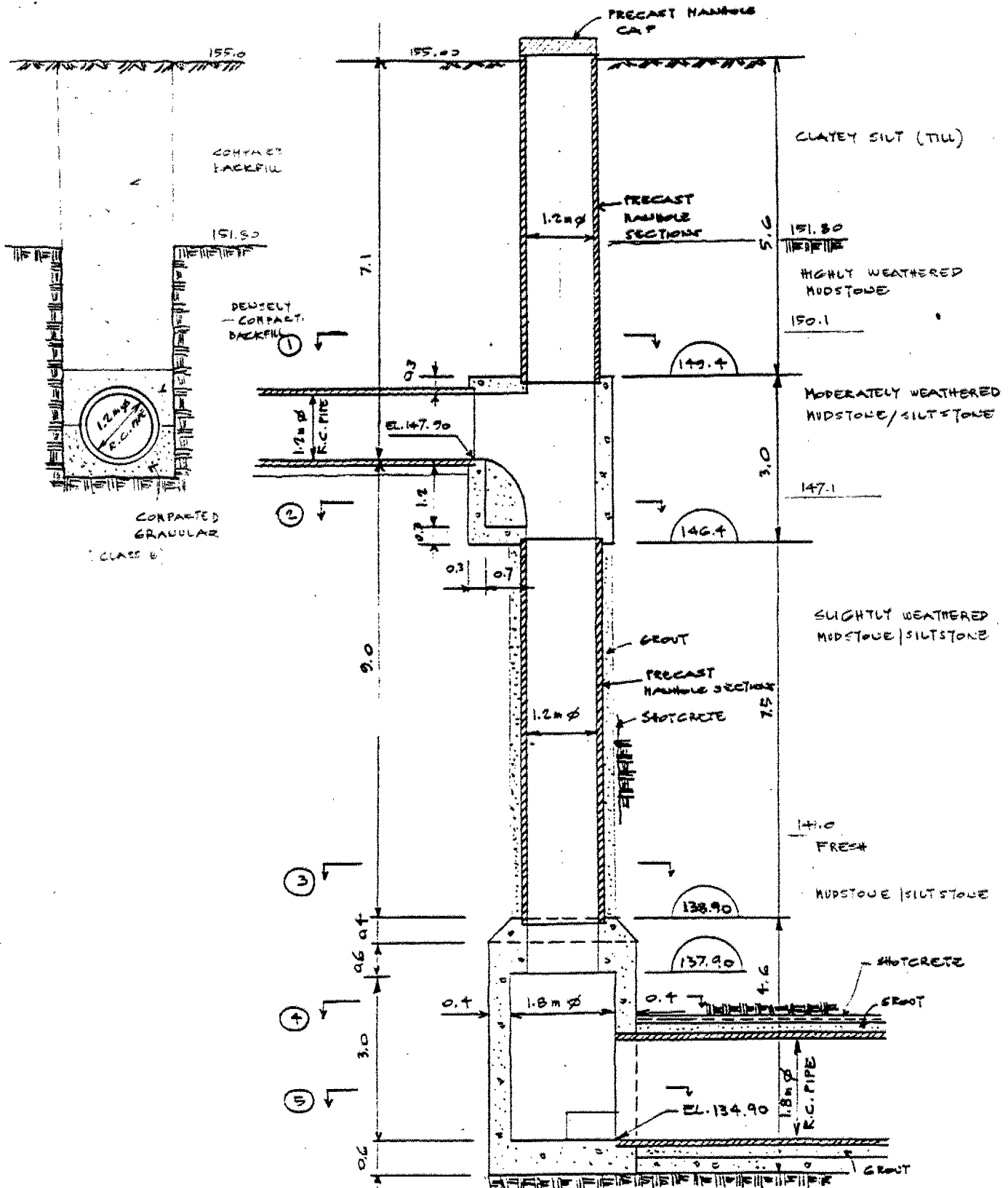
Date JUNE 17, 1993

Project 931-1346

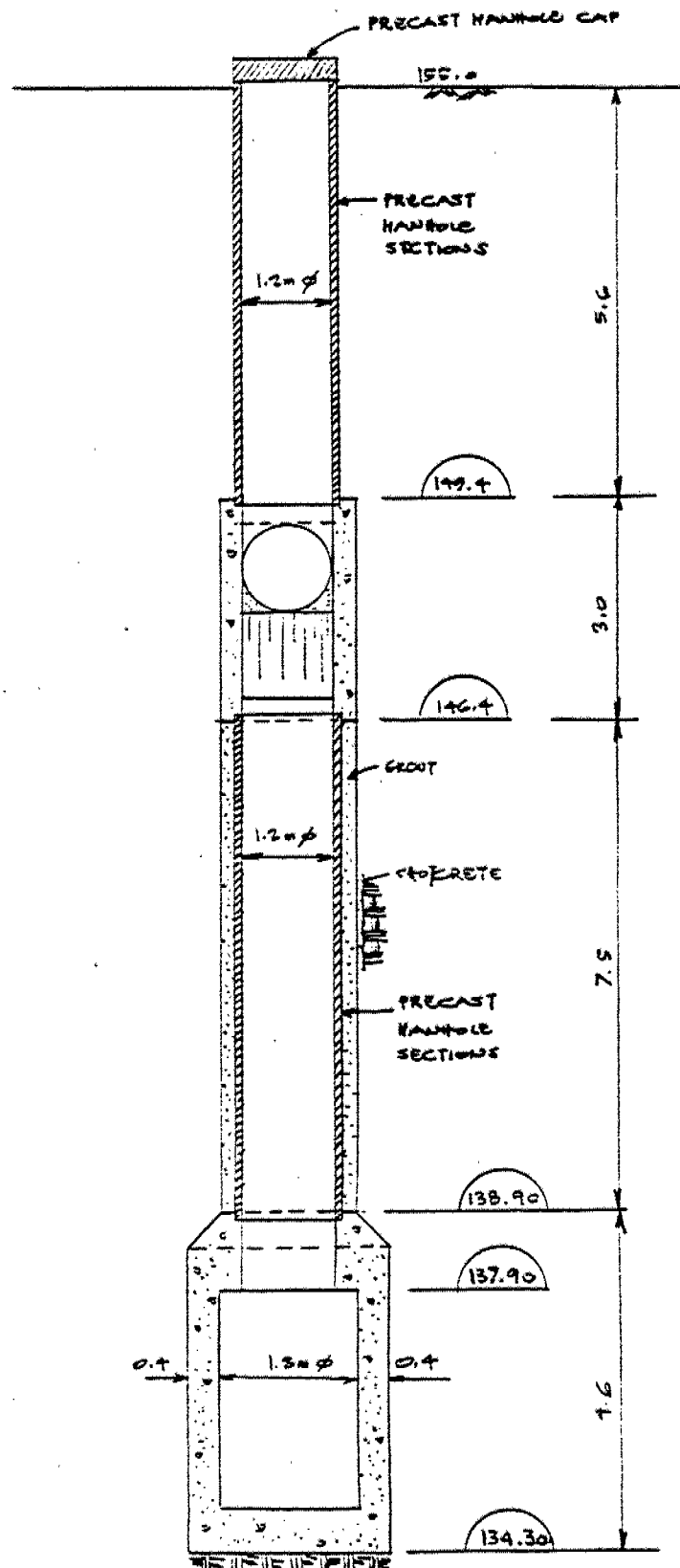
**Golder Associates**

Drawn MHW

Chkd cy



SECTION G-G



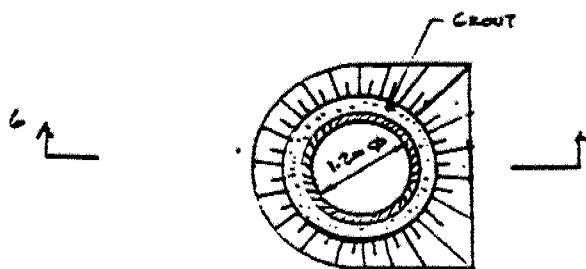
SECTION B-B

JULY 6, 93

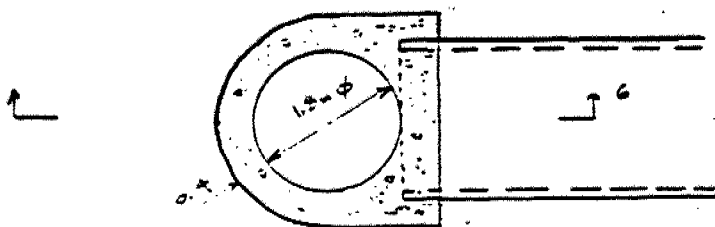


The diagram shows a mechanical assembly. A central circular component, possibly a valve or a pump head, is depicted with a yin-yang symbol. This central part is enclosed within a rectangular frame. To the left of the frame, there are several lines and labels: a dashed line labeled 'G' with an upward arrow, a solid line labeled 'T' with a vertical line segment, and another solid line labeled 'G' with a small vertical line segment. To the right of the frame, there is a single line labeled 'G' with a small vertical line segment. The entire assembly is shown in a perspective view, with lines converging towards the right.

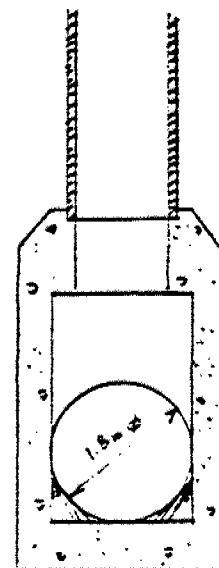
SECTION 2-2



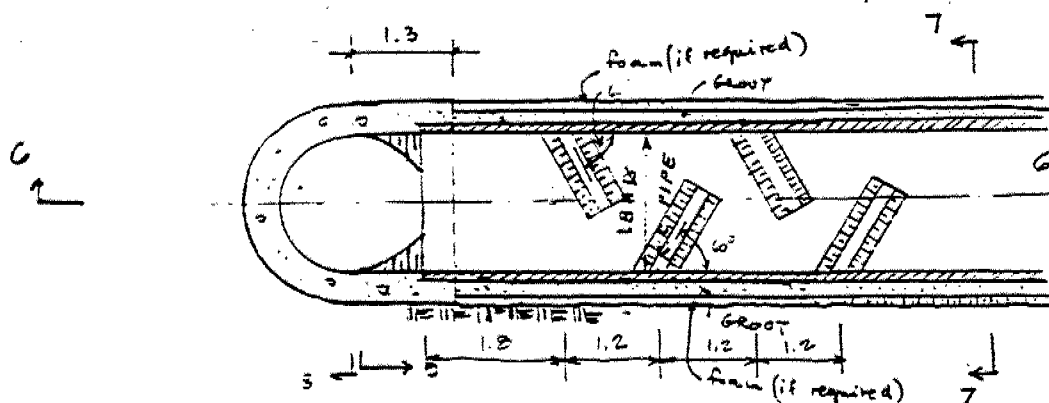
### SECTION 3-3



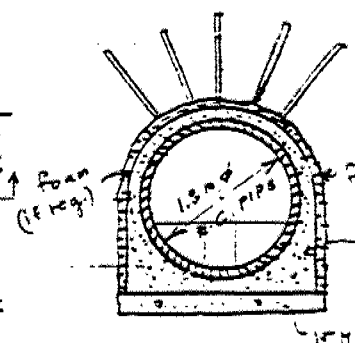
SECTION 4-4



SECTION 9-7



## Section 5-5

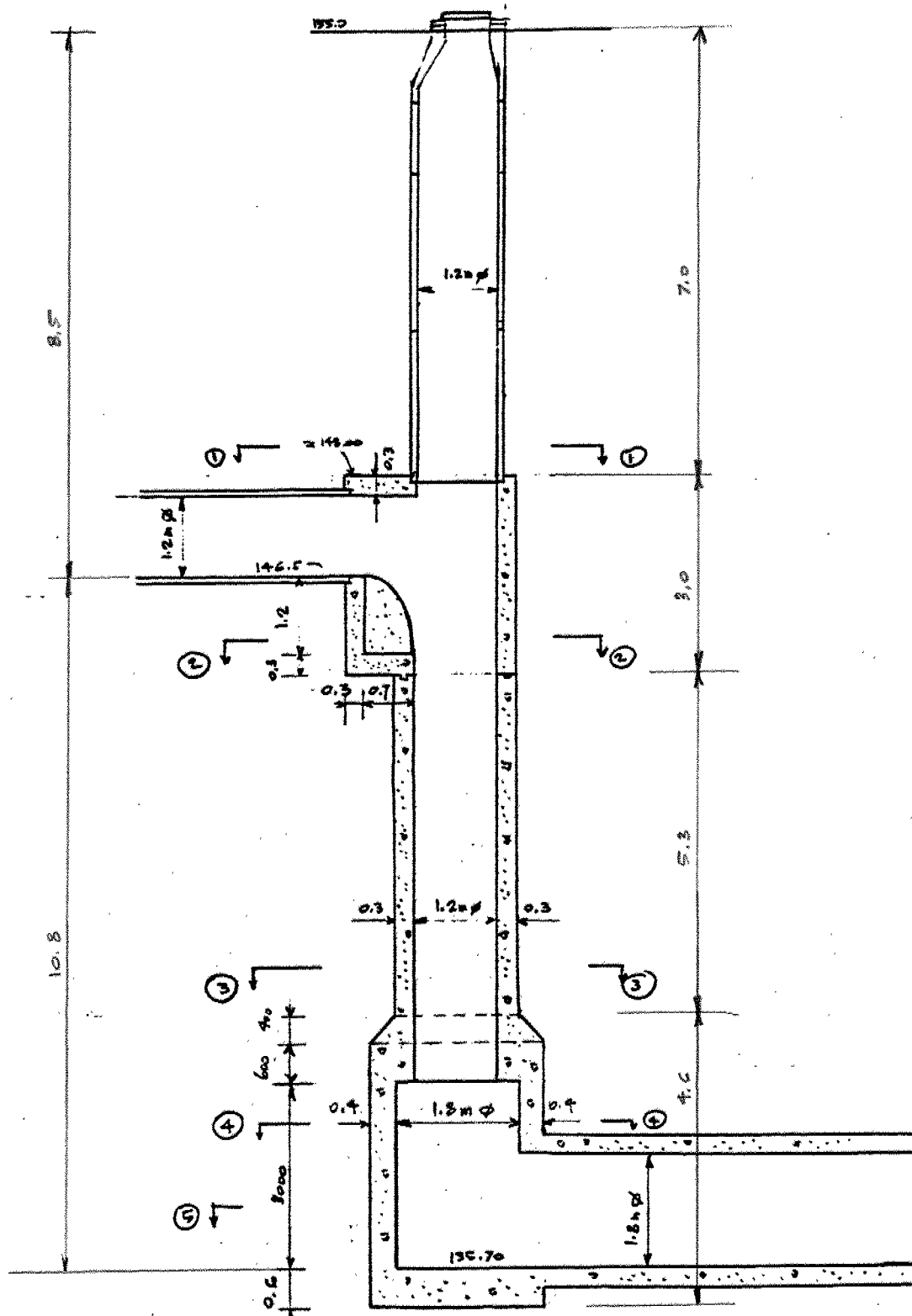


SECTION 7-7

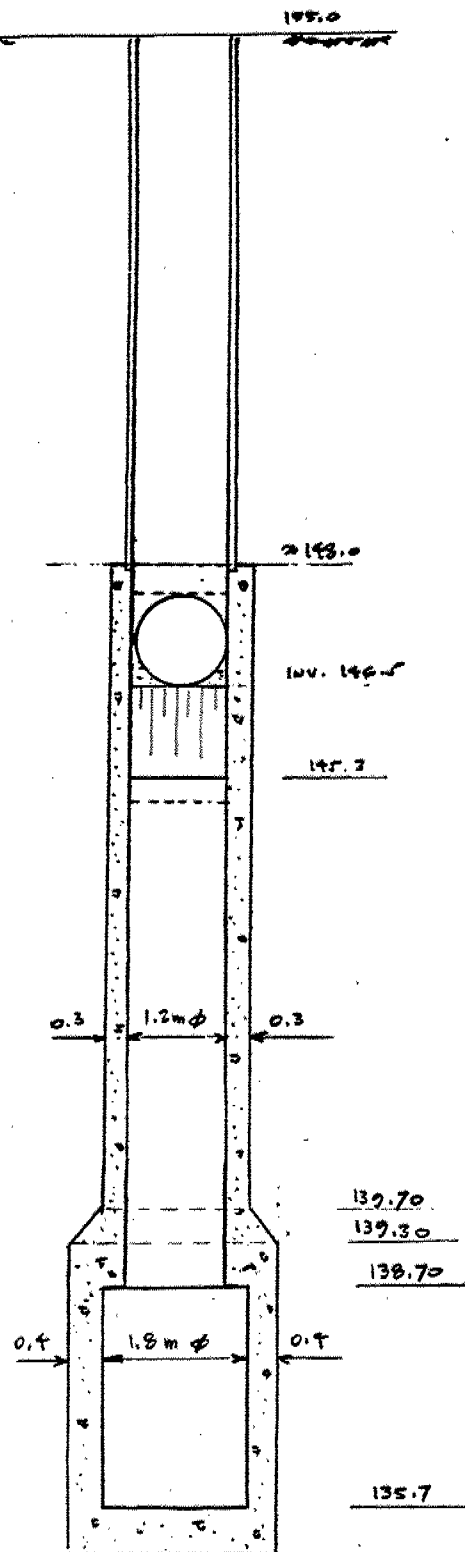
JULY 6, 93

3512

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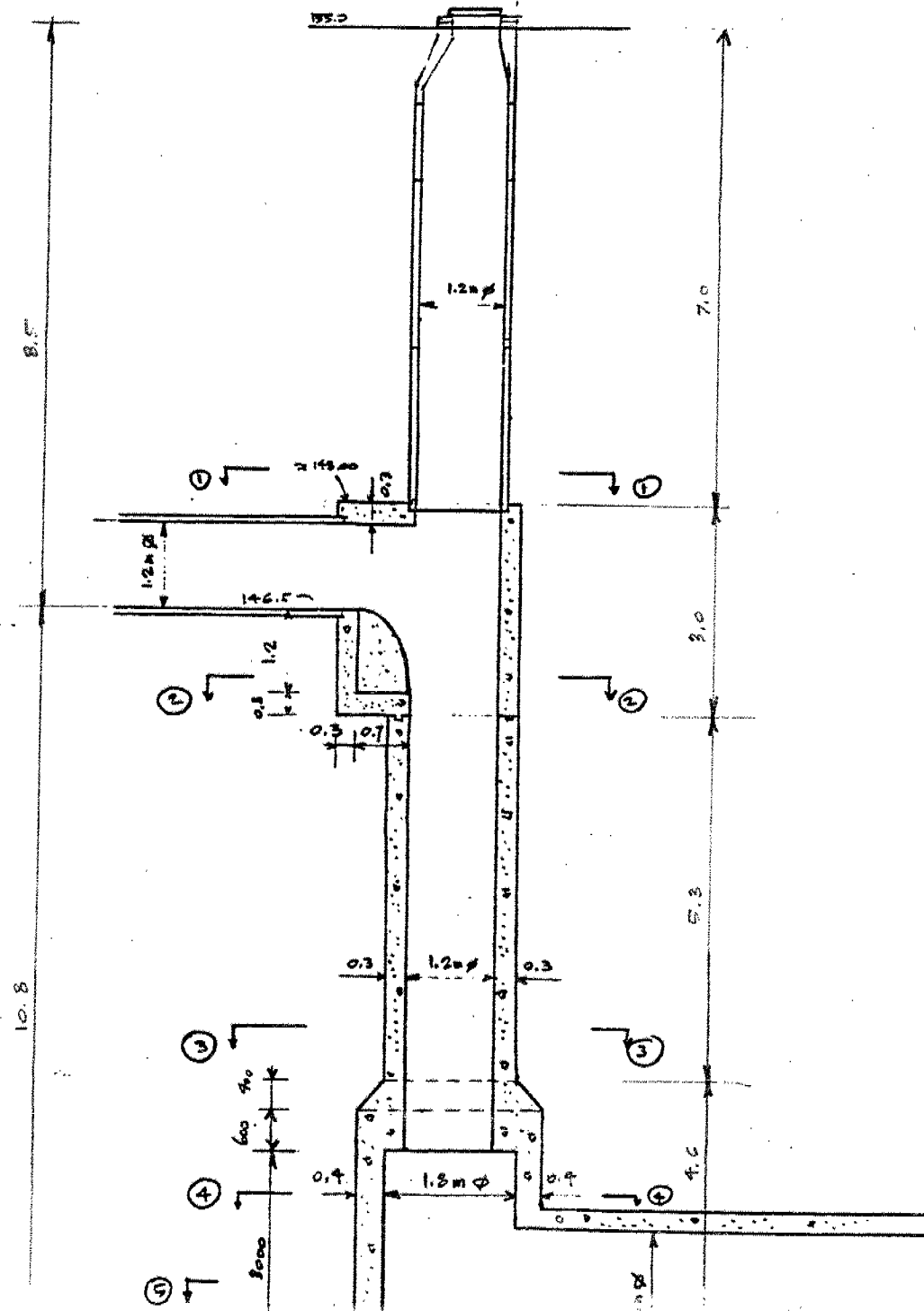


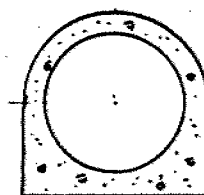
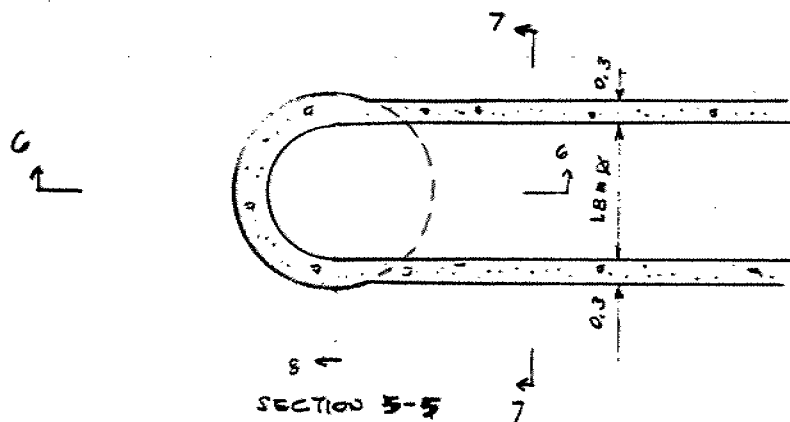
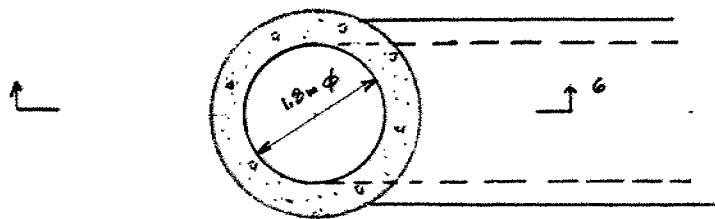
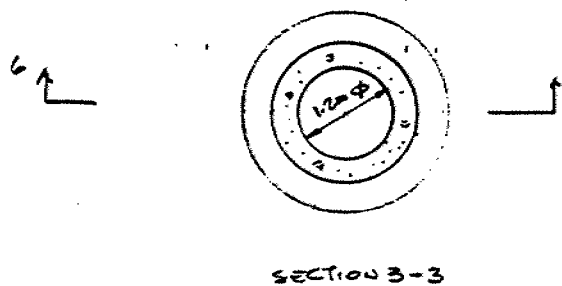
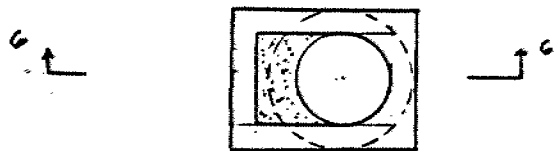
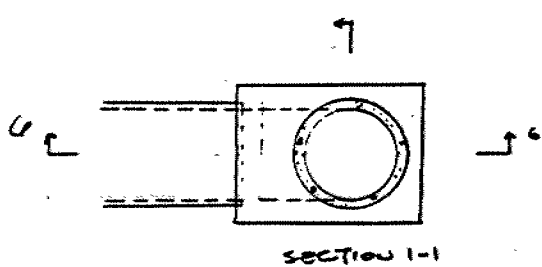
SECTION 6-6



SECTION 3-3

③





**DROP SHAFT and TUNNEL**  
**WP 410-85-00**  
**BOREHOLE LOCATIONS**

<b>ORIGINAL BOREHOLES ALONG CENTRE-LINE</b>			
BH	Co-ordinates		Elevation (m)
	Northing	Easting	
1	4 808 677	279 285.5	140.3
2	4 808 703	279 304.5	147.2
3	4 808 733	279 326.0	162.5

<b>BOREHOLES OFFSET (for practical access reasons and to avoid drilling over proposed tunnel)</b>			
BH	Co-ordinates		Elevation (m)
	Northing	Easting	
1	4 808 677.4	279 283.3	140.3
2	4 808 703.3	279 301.3	146.9
3	4 808 733.6	279 323.0	162.6
4	4 808 682.9	279 296.8	140.1

Note: Both original and offset boreholes have been staked out.

# memorandum



To: E. Salva  
Senior Project Manager  
Planning & Design, Central Region  
5th Floor, Atrium Tower

Date: 93 04 06

From: Foundation Design Section  
Room 315, Central Building

Re: Detention Pond at Hwy 403  
Between Bronte Creek and CNR Subway  
WP 410 85-00

Further to our meeting at R.V. Anderson Associates Ltd. conducted on March 24, 1993 pertaining to the detention pond design methodology, this office has undertaken an additional examination of all geotechnical considerations and their influence on the overall design. The review has also resulted in reevaluating specific geotechnical aspects for adjacent projects.

This memorandum summarizes the findings of these reviews and provides recommendations for the design and construction of relevant geotechnical aspects of the projects. The following projects have been included in the review.

- 1) Detention Pond
- 2) CNR Subway & Diversion
- 3) Hwy 403 Excavation Cut

In addition, comments have been provided regarding temporary excavation cuts and geotechnical/foundation implications of the deep shaft and tunnel adjacent to the Bronte Creek/Hwy 403 Crossing structure are also addressed.

## 1) DETENTION POND

### (i) Rate of seepage of groundwater

(a) As mentioned at the meeting, the rate of groundwater seepage into the pond is expected to be very small in view of the relatively impervious nature of the surficial cohesive heterogeneous mixture of clayey silt, sand and gravel and the underlying cohesionless heterogeneous mixture of silt, sand and gravel. Although the seepage rates are expected to yield greater flows within the underlying heterogeneous mixture

of silt, sand and gravel deposit, this seepage can be effectively controlled by employing a 0.6 m granular 'A' blanket on the slope surface of this deposit and a toe trench drain that extends for the thickness of the deposit to the bedrock surface. A trench drain is also recommended at the mid-depth bench to intercept the groundwater at the higher elevation. The trench drain-granular 'A' blanket drainage scheme will allow for groundwater drainage and simultaneously prevent slope instabilities.

The slope protection scheme shall also include rip rap or rock protection to the anticipated impounded pond high water level with the granular 'A' filter blanket between the native soil and the outer rock shell as described on page 12 of the Foundation Report (WP 410-85-00A). The granular 'A' blanket shall extend to the high pond water level or upper limit of the cohesionless heterogeneous mixture of silt, sand and gravel, whichever is greater.

Details of the recommended drainage scheme are provided on Figure 1 attached. The drainage scheme is applicable to all four(4) sides of the detention pond.

(b) It is recommended that any pond water be prevented from egressing into the groundwater drainage system. This can be achieved by constructing an impervious seal as illustrated in detail 1 in Figure 1. The impervious seal shall consist of a clay seal as defined in OPSS 1205 series.

### (ii) Subexcavation

The original Foundation Report emphasized the potential for hydrostatic uplift and recommended the excavation of the heterogeneous mixture of silt, sand and gravel to avoid potential "boiling" conditions within this material. These recommendations had been based on the presumption that the pond would be a retention "wet" pond. Having been informed at the meeting that the design would be a "dry" pond, it is hereby informed that subexcavation is **NOT** required. Alternatively, hydrostatic uplift can be controlled by the "relief" toe drains that were mentioned in (i) above. The toe drains are required throughout the excavation perimeter.

Installation of the toe drains will necessitate an additional excavation trench at the toe of the slope. A dewatering scheme will be required to facilitate the excavation of the trench within the heterogeneous mixture of silt, sand and gravel deposit. This can be achieved by conducting the excavation using an oversized excavation with perimeter ditches and sump pumping to discharge the groundwater.

To reduce the possibility of slope instability caused by toe excavation, it is



recommended that the work be completed in lengths not exceeding 5 m. This should be specified in a Non-standard special provision in the contract documents. It is also suggested that the excavation take place within a trench box to ensure the safety of personnel and also to prevent toe slope instability.

The design, installation and maintenance of the temporary dewatering system should be the responsibility of the contractor. The contractor shall be notified of the potential hydrostatic uplift condition and the susceptibility of the heterogeneous mixture of silt, sand and gravel to conditions of unbalanced head.

#### (iii) Effect of Railway Loading

A further slope stability analyses was conducted to determine the minimum edge distance between the CNR detour railway loading and the crest of the adjacent excavation cut slope. An effective stress analysis was conducted incorporating the slope geometries as summarized in Table 2 of the report (WP 410-85-00A) and using an external railway loading of 120 kN/m (American Railroad Engineering Association). Based on the results, it can be concluded that the centre-line of the railroad tracks shall be a minimum of five(5) metres from the crest of the slope. The slope shall be in accordance to the geometries recommended in the Foundation Report.

A review of the CNR detour drawings attached to R.V. Anderson Associates Ltd letter dated April 2, 1993 reveals that this minimum edge distance will be satisfied.

#### (iv) Excavation Drainage System

An excavation and dewatering scheme has been proposed and is provided on page 13 of the Foundation Report (WP 410-85-00A). It is believed that this procedure will adequately facilitate the excavation.

### 2) CNR SUBWAY & DIVERSION

The CNR diversion will be located in a temporary cut approximately 3 m to 4 m deep. No dewatering or hydrostatic uplift problems are anticipated for the temporary excavation cut.

In the construction of the two span CNR Subway structure, however, excavations to

elevations of approximately 151.8 m and 150.35 m have been shown at the abutment and centre pier locations respectively (see Drawing 19 - WP 408-85-01). These excavations appear to penetrate the cohesionless heterogeneous mixture of silt, sand and gravel and hence a temporary and permanent dewatering scheme will be necessary as indicated in the Foundation Report (pg 12, WP 408-85-01) and then reinforced in preliminary design drawing reviews conducted by this office. An oversized excavation with perimeter ditches may be one method of unwatering the excavation to facilitate the foundation construction. A permanent dewatering scheme will also be required for the Hwy 403 excavation cuts at and adjacent to the abutment locations. Permanent dewatering scheme alternatives are described below.

### 3) HWY 403 EXCAVATION CUT

A review of the subsurface conditions within the proposed Hwy 403 excavation cut between Bronte Creek and the CNR was implemented to determine whether hydrostatic uplift is a concern within this area. The conditions examined reveal that there is indeed a long term potential for hydrostatic uplift of the cohesionless heterogeneous mixture of silt, sand and gravel as a result of the excavation of the surficial overlying heterogeneous mixture of clayey silt, sand and gravel between Stations 26+650 and 27+190. One method to control this condition is to modify the recommended drainage system illustrated on Figure 4 of the Bronte Creek/Hwy 403 Crossing Report (WP 410-85-01/02) such that the toe drain is extended to bedrock. This measure will intercept water flow and provide the hydrostatic relief required to prevent uplift boiling conditions below the proposed Hwy 403 grade. A revised Figure 4 reflecting this specification is attached. To ensure adequate dewatering to facilitate the trench drain installation, trench excavation can be carried out within an oversized excavation restricted to lengths of 5 m as previously discussed.

An examination of the Hwy 403 profile grade elevation in relation to the depth and thickness of the cohesionless heterogeneous mixture of silt, sand and gravel reveals that the trench drain installation within the cohesionless stratum will require excessive non pragmatic excavation cuts as the profile grade increases in the easterly direction. Therefore, a more feasible method of hydrostatic uplift relief is the installation of vertical relief wells in the cohesionless till deposit. The spacing of the relief wells depends on the efficiency of the well, the hydraulic conductivity of the cohesionless till deposit and the amount of drawdown required. It is suggested that the relief wells be installed on 30 metre centres in a staggered pattern along the north and south roadway ditches. In view of the fact that a trench drain system will be installed on the north side of the detention pond parallel to the Hwy 403 excavation cut, vertical relief wells are **NOT** required along the south roadway ditch of the Hwy 403 adjacent to the detention pond.

Relief wells should consist of a minimum 100 mm diameter, PVC plastic screen extending for the **FULL** depth through the cohesionless till deposit. A typical generic pressure relief well is shown on Figure 6 in the Appendix (taken from WP 146-74-00-3). The relief well system must be connected to a permanent discharge system.

The relief wells can be installed in augured holes using conventional drilling equipment. Temporary casing may be needed to prevent soil sloughing from the shaft of the hole and enable the PVC plastic screen installation.

#### Temporary Excavation Cuts

Any temporary excavation cut that is susceptible to hydrostatic uplift shall be safeguarded accordingly. This can be achieved by installing the permanent drainage design scheme such that it also performs as a temporary scheme or alternatively employing deep wells or well points within the underlying cohesionless till deposit. The temporary or temporary/permanent scheme shall be installed when the excavation reaches the threshold elevation that will initiate the hydrostatic uplift imbalance. These elevations are summarized in Table 1 below and the excavation should be staged accordingly. There are no major dewatering difficulties anticipated above these elevations.

**TABLE 1 - STAGE EXCAVATION ELEVATIONS**

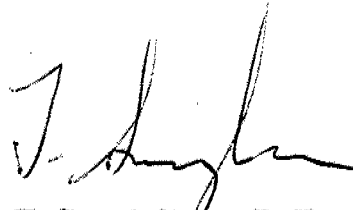
<b>Chainage</b>	<b>Elevation(m)</b>
26+650 - 27+100	156
27+100 - 27+190	159

As mentioned earlier, the design, installation and maintenance of the temporary dewatering system should be the responsibility of the contractor. The contractor shall be notified of the potential uplift condition and the susceptibility of the heterogeneous mixture of silt, sand and gravel to conditions of unbalanced head.

**PROXIMITY OF TUNNEL STRUCTURE TO EAST ABUTMENT OF EASTBOUND  
STRUCTURE**

Having reviewed the mutual influences of the proposed location of the drop shaft and the tunnel structure and the east abutment of the eastbound structure, it can be concluded that no influence exists between the structures.

If you have any questions regarding the above comments or require additional information, please do not hesitate to contact this office.

A handwritten signature in black ink, appearing to read 'T. Sangiuliano', is positioned above the printed name.

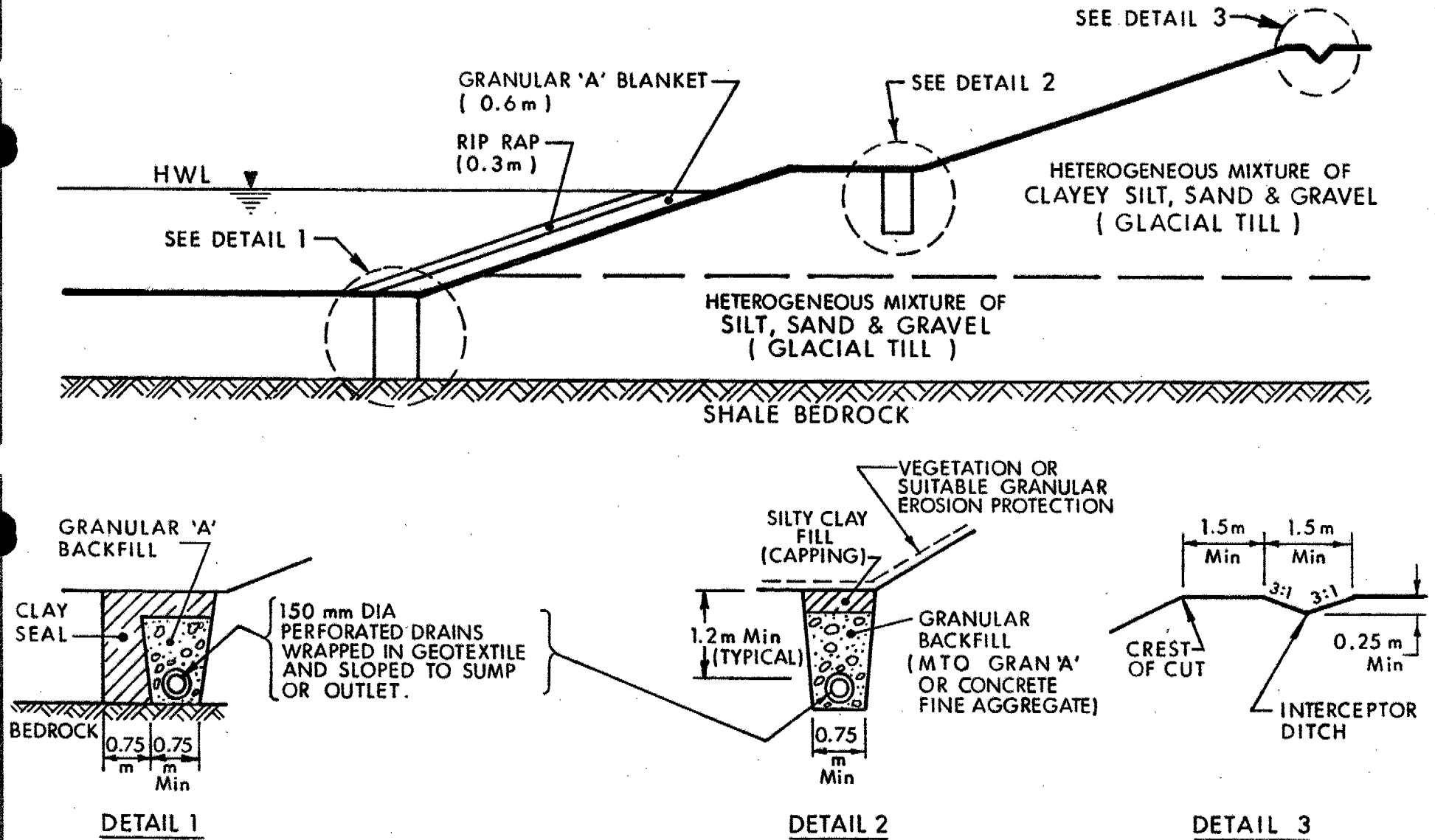
**T. Sangiuliano, P. Eng.  
Foundation Engineer**

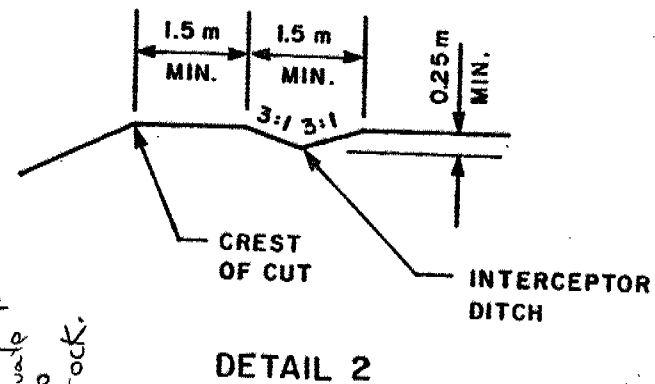
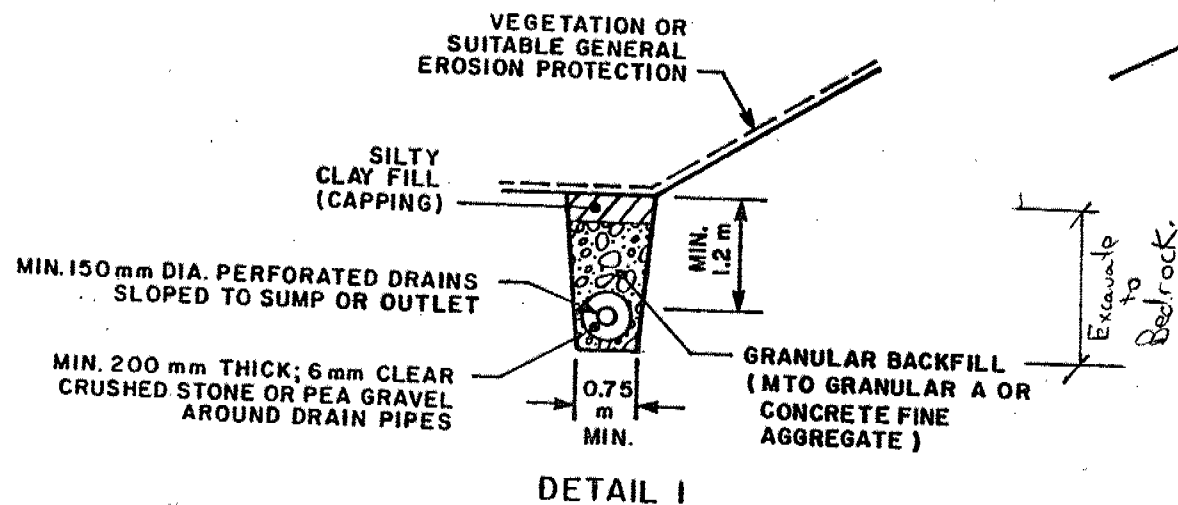
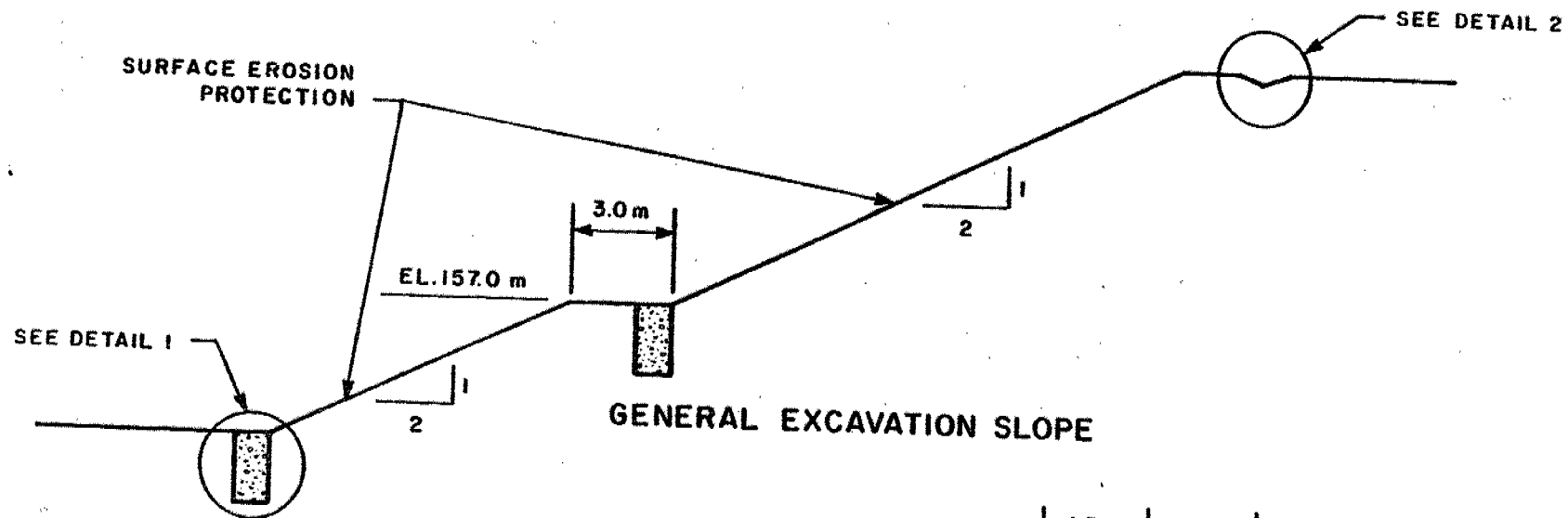
for

**P. Payer, P. Eng.  
Senior Foundation Engineer**

Figure 1 - DETENTION POND DRAINAGE/SLOPE PROTECTION SCHEME

NOT TO SCALE





Ministry of  
Transportation

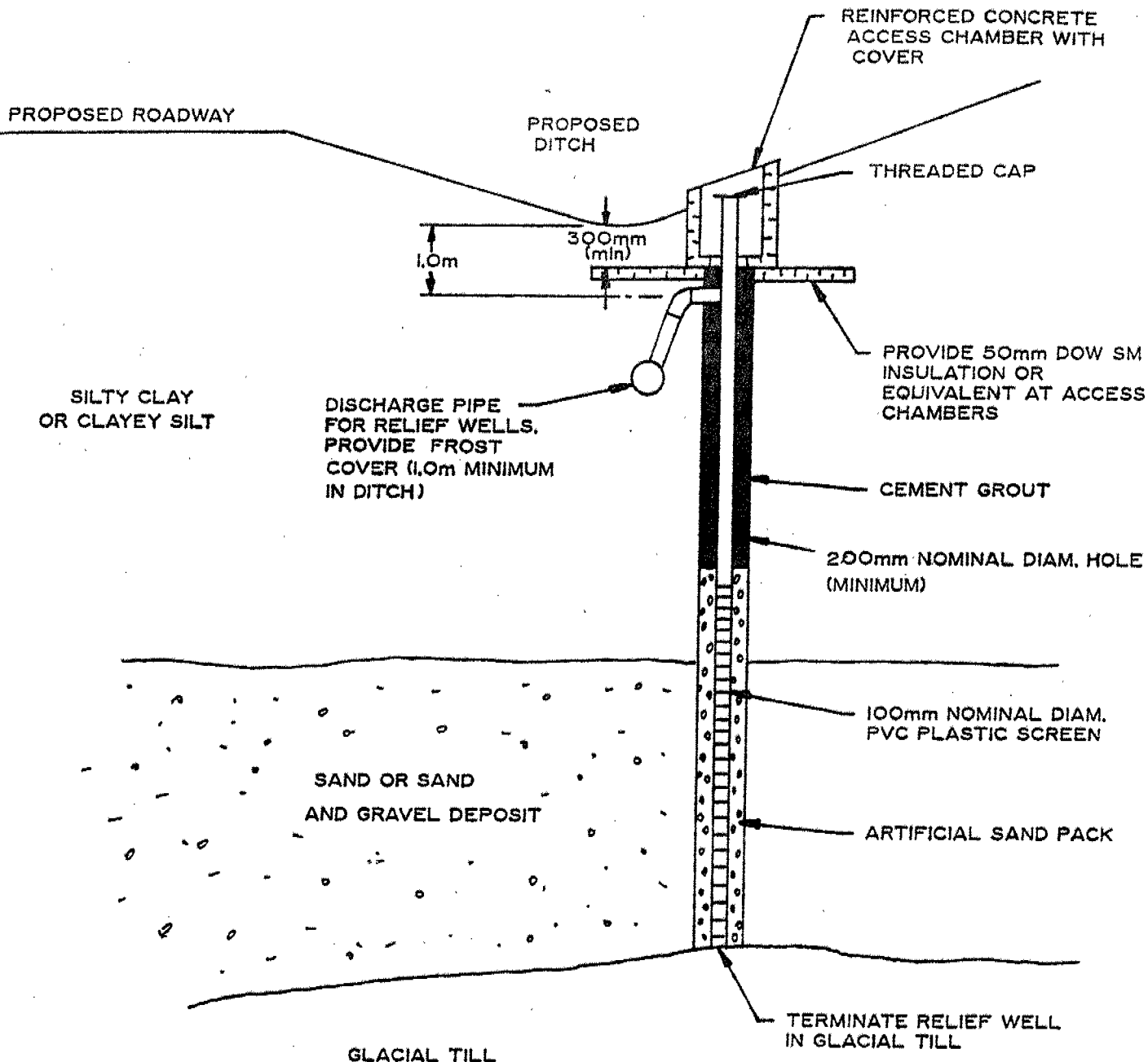
## RECOMMENDED DRAINAGE SYSTEM EAST APPROACH CUT

FIG No 4

W P 410-85-01/02

# TYPICAL PRESSURE RELIEF WELL DETAIL — SAND —

FIGURE 6  
WP 146-74-00-3



NOT TO SCALE

**SPECIAL NOTE**  
THIS DRAWING IS TO BE READ IN CONJUNCTION  
WITH ACCOMPANYING REPORT

Date OCT. 4, 1990

Project 90I-2115

Golder Associates

Drawn S.L.

Chkd. *AC*



# R.V. Anderson Associates Limited

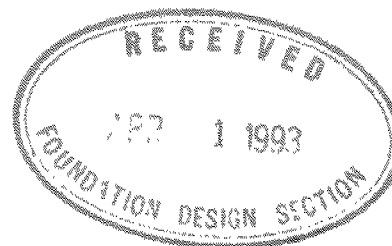
consulting engineers and architect

Suite 400, 2001 Sheppard Avenue East, Willowdale, Ontario M2J 4Z8  
Telephone (416) 497-8600 Fax (416) 497-0342

March 31, 1993

RVA 3226.10

Ministry of Transportation  
Planning & Design, Central Region  
5th Floor, Atrium Tower  
1201 Wilson Avenue  
Downsview, Ontario  
M3M 1H8



**Attention: Mr. E. Salva**  
**Senior Project Manager**

Dear Sir:

Re: Highway 403 Advance Structure Approaches  
Highway 25 to Walkers Line  
Preliminary Layout of Dry Extended Detention Pond  
& Alignment of Tunnel and Drop Shaft  
W.P. 410-85-00

Further to our meeting on March 24, 1993, we enclose herewith a preliminary layout plan of the proposed dry extended detention pond, a cross-section of the future Highway 403 at the drop shaft location and the longitudinal profile of the drop shaft and tunnel for your comments.

The stormwater runoff from Highway 403 will be intercepted by the roadside ditches on both sides of the road and enter the flow splitting structure via a 750 mm dia. stormwater pipe at the low point of the road at approximate station 26+838. The first flush will be detained in the pond for approximately 24 hours prior to its discharge into the drop shaft via the hickenbottom drain at the west end of the pond. The stormwater drainage along the CN Rail detour and both sides of the existing CN Rail tracks will be discharged into the drop shaft via the 1050 mm dia. and 1200 mm dia. sewer pipes as shown. When the water level in the pond increases beyond the first flush (ie. the discharge capacity of the hickenbottom outlet drain and the capacity of the pond), the stormwater will bypass the pond and flow into the proposed 1200 mm dia. pipe.

Vehicular access for maintenance of the drop shaft pond tunnel and the pond is proposed via a culvert on the roadside ditch of Highway 403 at station 26+775. A 4.5 m berm is provided for vehicular access to the drop shaft due to the 4 m elevation differential between the ditch and the berm.

Does this imply that it's not from?



The proposed French drain along the bottom of the berm on the south side of the pond and at the bench for the cut slope will be discharged into the proposed drainage system as shown. The exact details will be analyzed later after we received the recommendations from the Foundation Section.

The proposed alignment of the tunnel is as shown in the drawing. The longitudinal profile of the tunnel is also attached. Please note that the size of the drop shaft is anticipated to be larger than the one shown as it is expected that the tunnelling will be commencing at the drop shaft location.

We would appreciate if you would provide your comments on the above conceptual design arrangement of the dry extended detention pond and the drop shaft and tunnel alignment. The elevation and configuration of the pond will be adjusted during the detail design. By copy of this letter to Environmental and Geotechnical Sections we would appreciate if they would also provide their comments as soon as possible.

By copy of this letter to Foundation Section, we would appreciate if they could provide their comments and recommendation on the toe drain arrangement/details of the pond. We would appreciate if Foundation Section would also arrange for the site investigation required for the drop shaft and tunnel. The proposed location of the borehole with co-ordinates are shown on the drawing attached. The proposed Terms of Reference for the foundation investigation for the proposed site investigation was forwarded to you in our meeting on March 24, 1993.

Should you have any question, please do not hesitate to contact our office.

Yours very truly,

**R.V. ANDERSON ASSOCIATES LIMITED**



T.H. McColm, P.Eng.  
Project Manager

PCWL/THM/kis

cc: Mrs. C. Southey - MTO, Environmental  
Mr. L.N. Simlote - MTO, Geotechnical  
Mr. P. Payer - MTO, Foundation  
Mr. R.C. McCormick - McCormick Rankin

# RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 410-85-00 LOCATION Co-ords: N 4 809 231, E 279 294 (Sta. 9+400, E of CNR Detour) ORIGINATED BY LD  
 DIST 4 HWY 403 BOREHOLE TYPE SS Auger COMPILED BY TS  
 DATUM Geodetic DATE 92 05 19 CHECKED BY PP

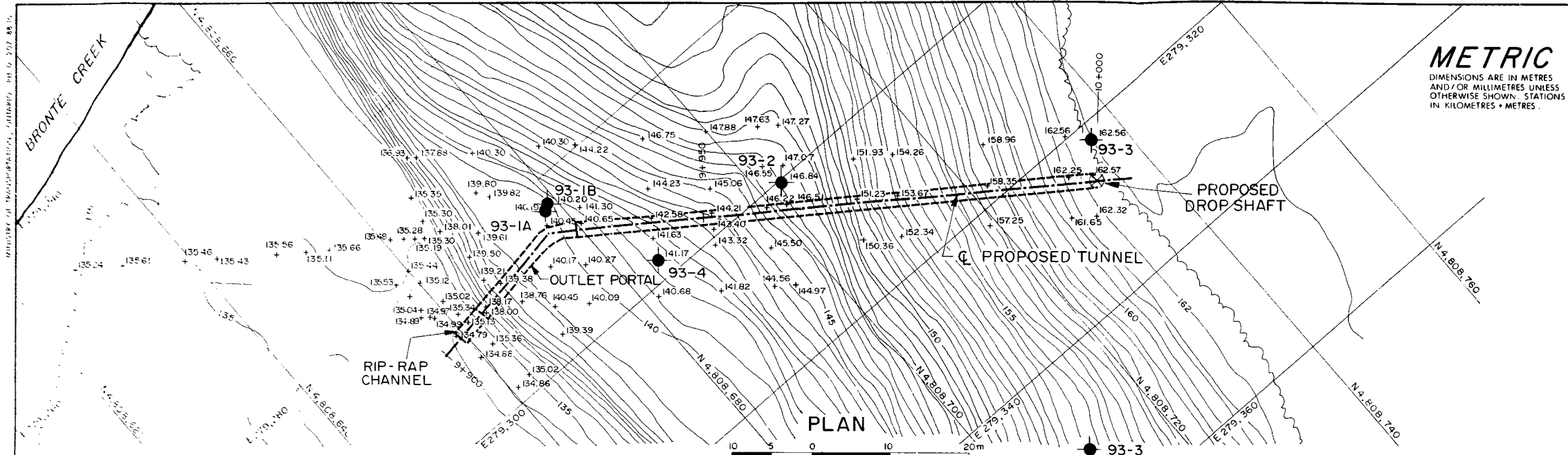
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT 7 KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
164.6	Ground Surface												
0.0													
			1	SS	35								
			2	SS	36								
			3	SS	41								
			4	SS	126								
			5	SS	69								
			6	SS	84								
160.8													
3.8			7	SS	91								
			8	SS	80								
			9	SS	**								
			10	SS	120								
156.6			12	SS	100								
8.0	End of Borehole = 92 05 21 ** Sampler Bouncing (Probable Boulder)												

RECORD OF BOREHOLE No 2

1 OF 1 METRIC

W.P. 410-85-00 LOCATION Co-ords: N 4 809 086; E 279 343 (Sta. 9+550, E of CNR Detour) ORIGINATED BY LD  
DIST 4 HWY 403 BOREHOLE TYPE SS Auger COMPILED BY TS  
DATUM Geodetic DATE 92 05 19 CHECKED BY PP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100				
163.9	Ground Surface															
0.0																
	Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till)		1	SS	34											
	Hard		2	SS	34											
			3	SS	46											
			4	SS	37											
			5	SS	39											
			6	SS	49											
	Brown		7	SS	73											
	Grey		8	SS	**											
			9	SS	81											
			10	SS	100											
156.8																
7.1	Heterogeneous Mixture of Silt, Sand and Gravel (Glacial Till)															
156.0	Grayish Red, Very Dense		11	SS	100											
7.9	End of Borehole															
	* 92 05 21															
	** Sampler Bouncing (Probable Boulder)															



**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES + METRES.

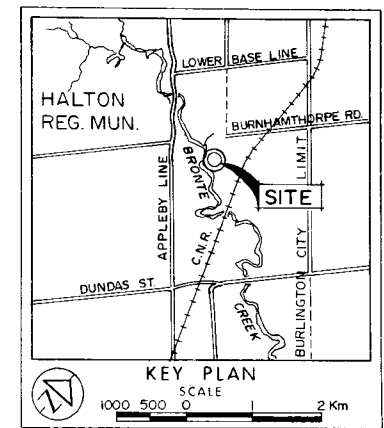
CONT No  
WP No 410-85-00

**DROP SHAFT AND  
TUNNEL ALIGNMENT**

BORE HOLE LOCATIONS & SOIL STRATA

**SHEET**

GOLDER ASSOCIATES LIMITED



- LEGEND**
- Bore Hole
  - ⊕ Dynamic Cone Penetration Test (Cone)
  - ⊕ Bore Hole & Cone
  - N Blows/0.3m (Std Pen Test, 475 J/blow)
  - CONE Blows/0.3m (60° Cone, 475 J/blow)
  - W L at time of investigation
  - Standpipe
  - Spot Elevations

No	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
93-1A	140.19	4,808,678.47	279,283.47
93-1B	140.20	4,808,678.83	279,282.97
93-2	146.84	4,808,702.87	279,300.26
93-3	162.56	4,808,735.93	279,321.95
93-4	141.17	4,808,684.69	279,297.23

**NOTE**  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

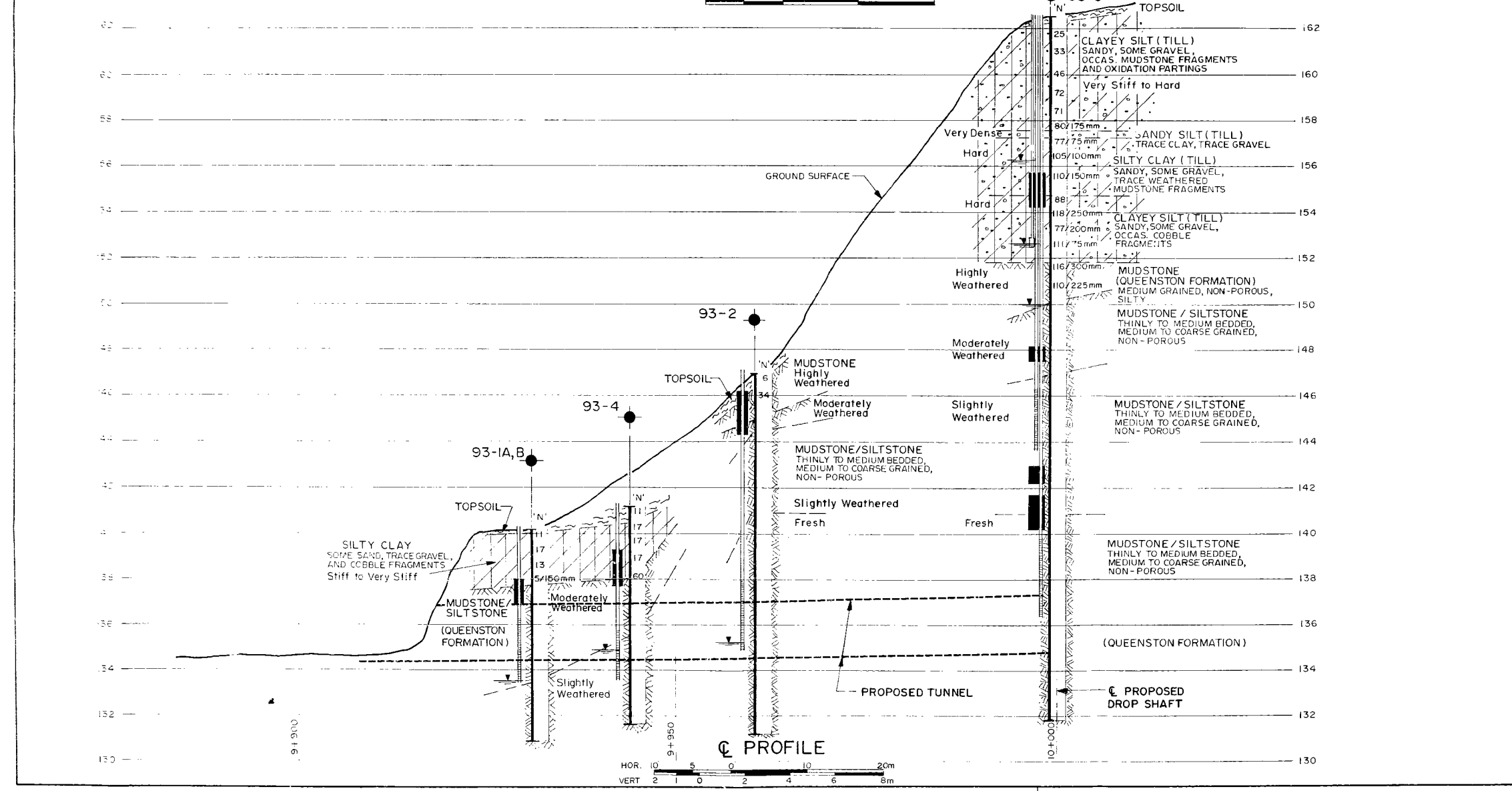
REV	DATE	BY	DESCRIPTION

Geocres No 30M5-194

HWY No. 403	DIST
SUBM'D	CHECKED
DRAWN MW	CHECKED

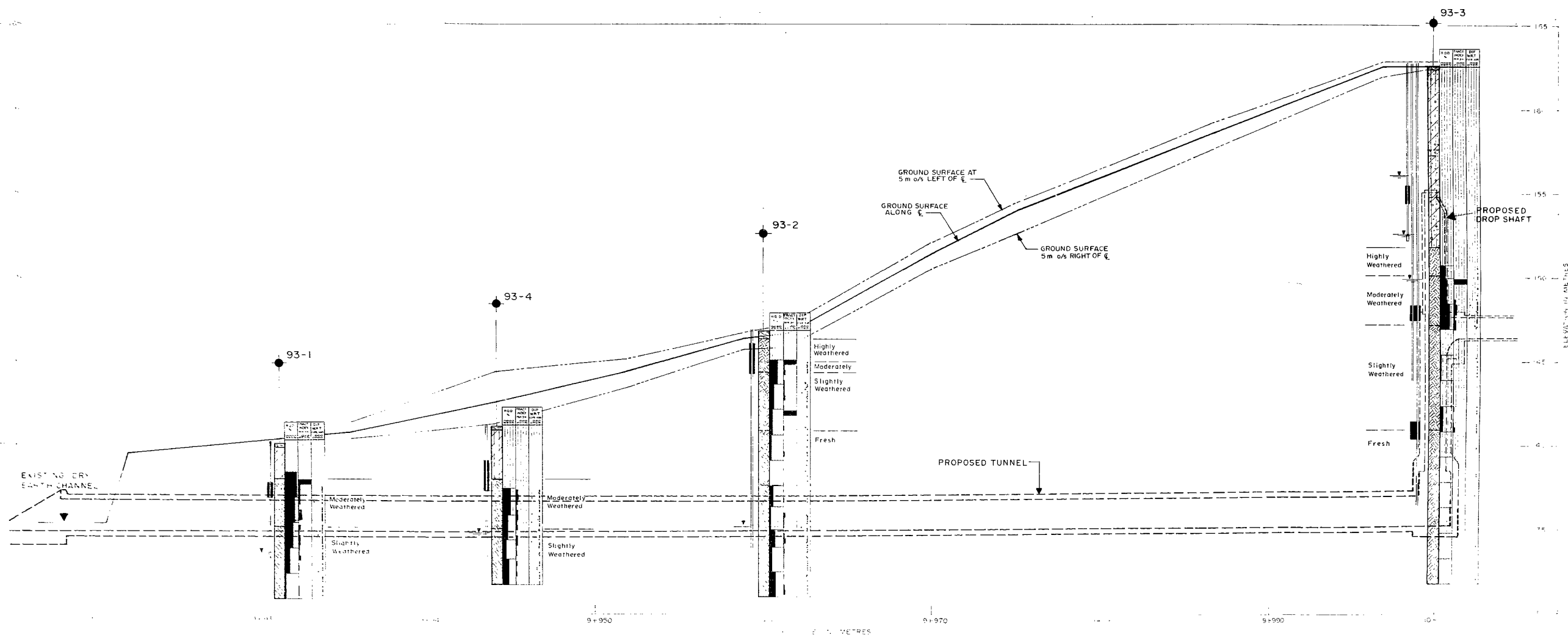
DATE	SITE
DATE	DATE

DWG 4108500-A



LONGITUDINAL SECTION THROUGH PROPOSED DROP SHAFT AND TUNNEL

FIGURE 4



**LEGEND**

93-4 - BOREHOLE IDENTIFICATION NUMBER

STRATA PLOT (SEE DESC. BELOW)

BENTONITE SEAL

WATERLEVEL IN PIEZO OR STANDPIPE

STANDPIPE

PLOTS OF ROCK QUALITY DESIGNATION, FRACTURE INDEX AND DIP with respect to core axis

**SIMPLIFIED STRATIGRAPHY**

- TILL SOIL
- SILTY CLAY SCREE
- SILTY CLAY TO CLAYEY SILT TILL
- SANDY SILT TILL
- QUEENSTON SHALE BEDROCK

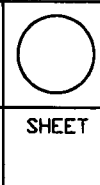
**NOTES**

1. PROPOSED DROP SHAFT, TUNNEL AND STILLING BASIN DIMENSIONS TAKEN FROM PRELIMINARY SKETCH PROVIDED BY R.V. ANDERSON LTD.
2. GROUND SURFACE PROFILE PROVIDED BY M.T.O. STAFF
3. TUNNEL ALIGNMENT BENDS 45° TO THE SOUTH AT APPROXIMATELY STATION 9+970

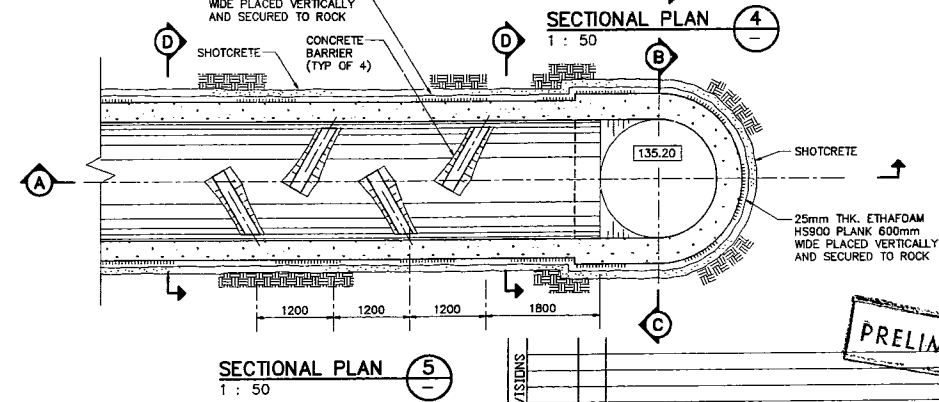
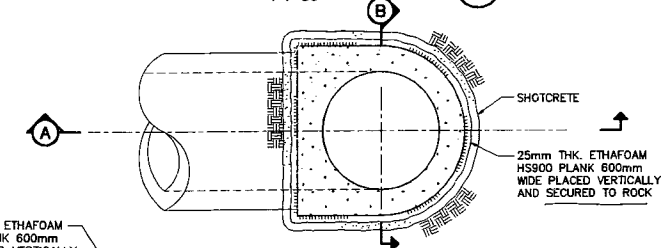
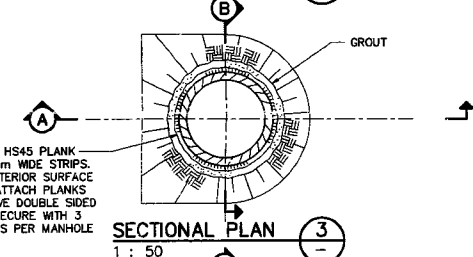
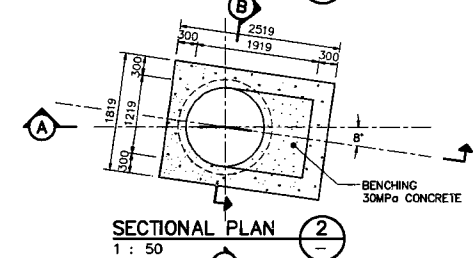
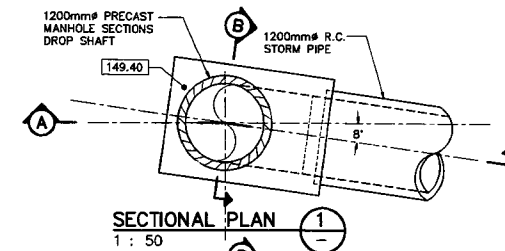
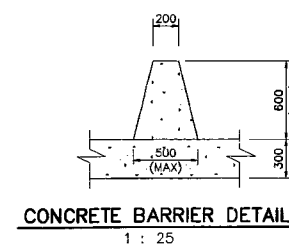
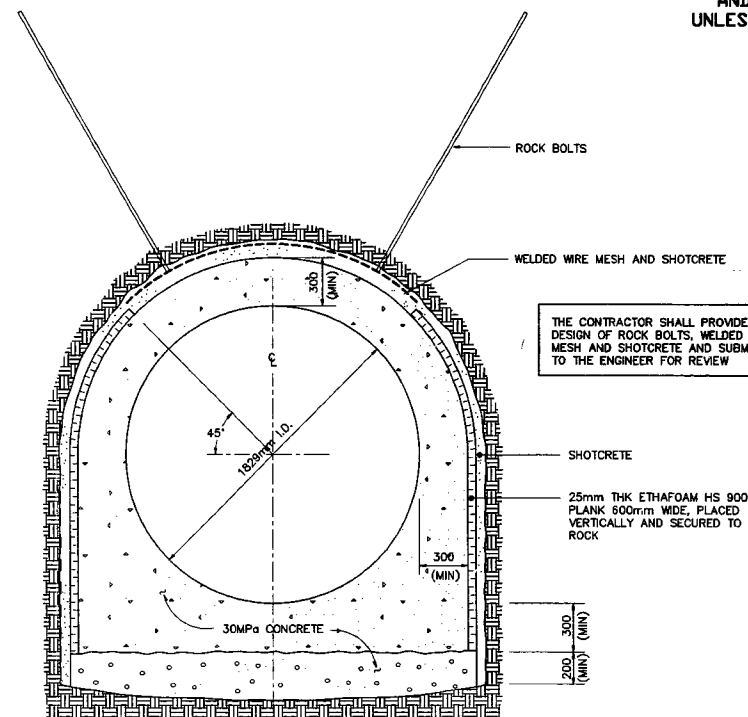
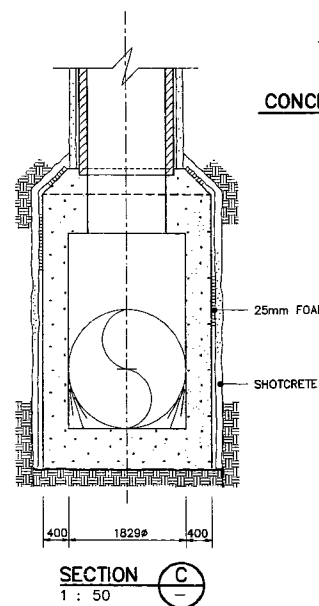
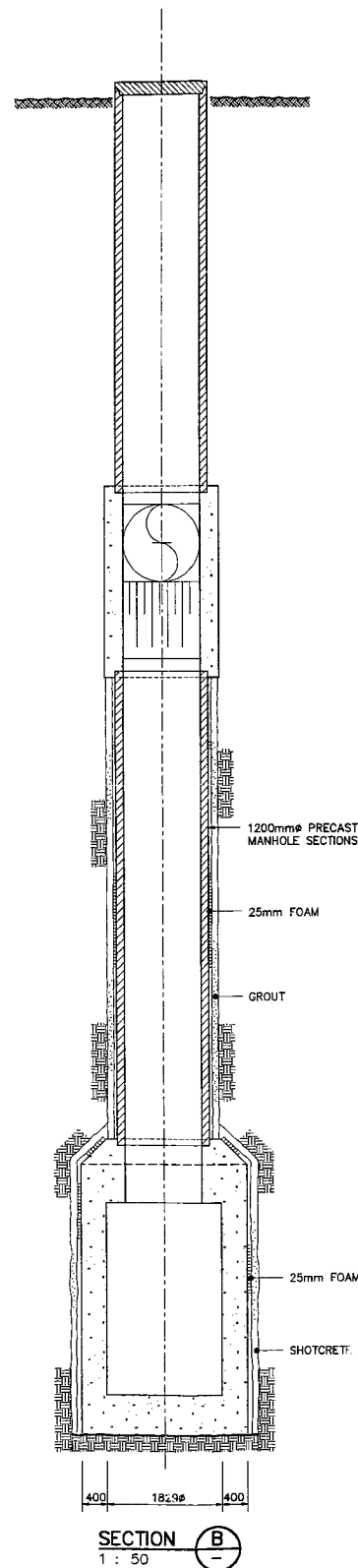
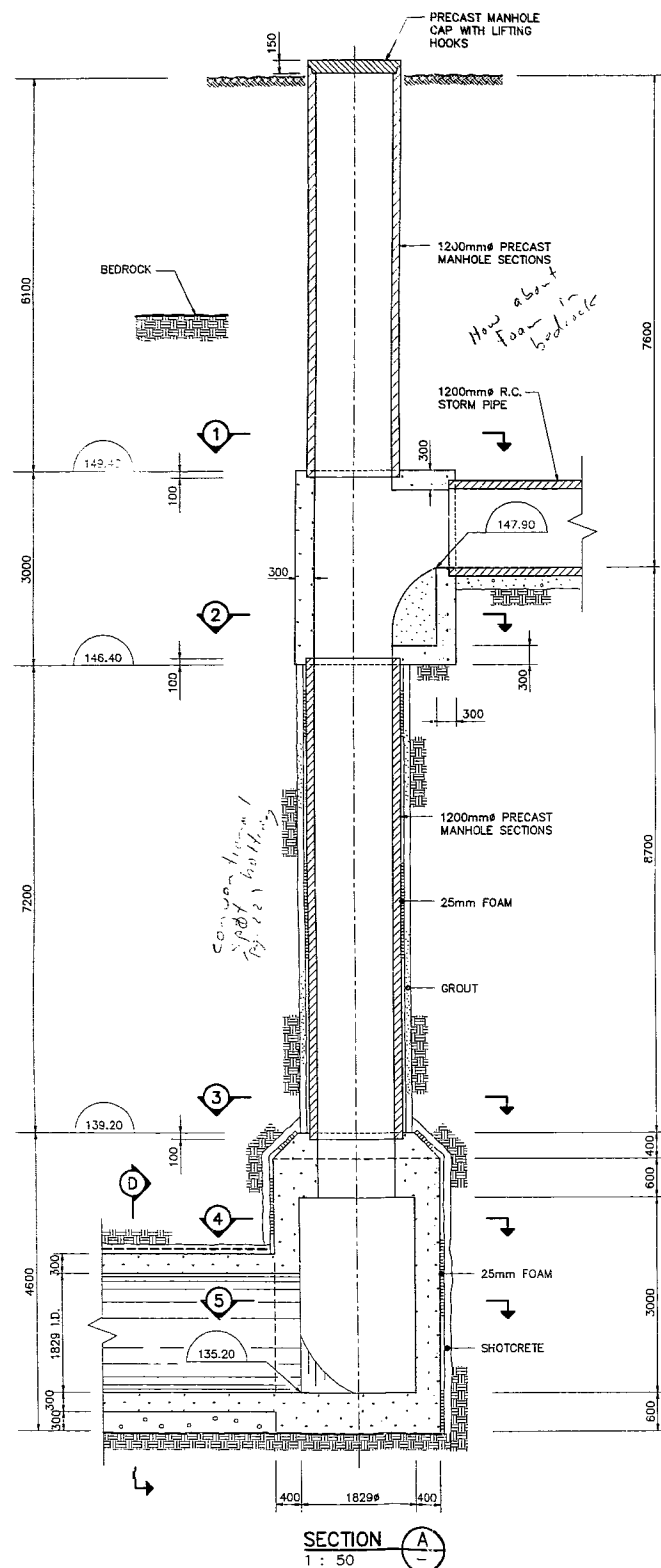
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AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT. No.  
WP. No. 410-85-00

BRONTE CREEK & HWY 403  
DROP SHAFT &  
TUNNEL OUTFALL



R.V. Anderson Associates Limited  
consulting engineers and architect



DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION	LOAD	DATE
DESIGN A.C.	CHK. J.H.R.V.	CODE			JULY 1993
DRAWN W.W.	CHK. A.C.	SITE	STRUCT.	SCHEME	DWG.

PRELIMINARY

JUL 15 1993