Geocres 41 P - 23

CONSULTING ENGINEERS

FOUNDATION INVESTIGATION REPORT
FOR
PROPOSED REPLACEMENT OF MONTREAL RIVER BRIDGE
HIGHWAY 65, ELK LAKE, ONTARIO

W.P. 127-88-01; SITE NO. 47-36
DISTRICT 53, NEW LISKEARD

Prepared For:
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ENCLOSURES

1.0 INTRODUCTION

DST Consulting Engineers Inc. (DST) have been retained by the Ministry of Transportation of Ontario to conduct a foundation investigation for a proposed replacement bridge on Highway 65 over the Montreal River in Elk Lake, Ontario. The new 5 span bridge will be located 5 to 10 m south of the existing bridge. New approach embankments on both sides of the bridge will extend slightly into the river which is up to approximately 6 m deep.

The purpose of this investigation has been to determine the subsurface conditions at the site for construction of the proposed bridge substructure and approach embankments. Details of the Method of Investigation are provided in Appendix 'B', and are briefly discussed in the following paragraphs.

The fieldwork was carried out during the period of January 8 to April 17, 1996 in three phases. Fieldwork had to be halted from January 27 to February 7, 1996 due to thin ice conditions (in spite of ice bridge construction), and during the period of February 16 to April 10, 1996. After analysis of the subsurface stratigraphy from the field program of Phases 1 and 2, it was decided that additional boreholes were required to confirm the subsurface soil conditions from elevation 230 to 214 m in order to prove the end bearing stratum.

The initial field program, consisting of drilling Boreholes 1, 2, 7 and 8 and static cone penetration tests with pore pressure measurements (CPTU's) 2A and 7A, was conducted utilizing a CME 750 drill rig mounted on an all terrain carrier. The second phase of the program was conducted utilizing a CME 45 trailer mounted drill rig. This lighter rig was mobilized to the site when it was determined that sufficient ice thickness to support the heavier drill rig would not be achieved due to weather and river current effects. Boreholes 3 to 6 were completed during this phase of the fieldwork. The final phase of the program, consisting of drilling Boreholes 9 and 10, was conducted utilizing a CME 750 drill rig mounted on an all terrain carrier. Boreholes 9 and 10 were advanced to confirm the subsurface conditions from elevation 230 to 214 m on the east and west bank of the river.

General borehole locations were discussed with MTO personnel prior to mobilization to the site. The boreholes were then located in the field by DST personnel as close as possible to the general layout. On completion of the Phase II of fieldwork, MTO crew surveyed the borehole locations and obtained surface elevations except for boreholes 9 and 10 which were subsequently surveyed by DST field staff.

Drawing No. 1278801-A provides the location of the boreholes as well as topographic detail provided by MTO. Details of the subsurface conditions are given on the Record of Borehole Logs and results of laboratory testing are shown thereon, where appropriate. Gradation analyses, moisture and organic content tests, Atterberg limit tests, consolidation tests and direct shear tests were also completed on representative samples obtained from the field program. The data from this laboratory program is shown on the Borehole Logs where applicable and on Enclosures 1 to 11. A discussion of these results together with our interpretation and recommendations are presented in this report.

2.0 SITE PHYSIOGRAPHY AND GEOLOGY

The site is located in the Cobalt plain of the James Region of the Canadian Shield physiographic region of Canada (Douglas, 1970). The Cobalt plain is formed of flat lying sedimentary rocks with ridges and hills formed by diabase sills or inliers of Achean crystalline rocks. The region has been deeply scoured by repeated glaciation. The Winsconsin Glaciation, which retreated approximately 9,500 - 10,000 years ago was the last period of glaciation.

The rocks in the area are Precambrian in age. The oldest rocks are volcanic, sedimentary and granitic rocks of the Abitibi greenstone belt, located to the north of the site. The rocks underlying Elk Lake are flat lying, relatively weakly metamorphosed sedimentary rocks of the Cobalt Group. These rocks are conglomerates, wackes, silt stones and mudstones (Thurston et. al. 1991).

Surficial deposits of glacial and nonglacial origin occur in the area. Glacial deposits consist of sandy bouldery till in ground moraine and hummocky moraine, glaciofluvial sand and gravel in eskers, kames, outwash plains, and deltas, and glaciolacustrine (varved) clay. Non-glacial deposits include eolian sand, alluvial sand and silt, colluvium, and organic deposits of peat. Clay deposits are frequently varved as a result of seasonal deposition effects.

Northern Ontario was glaciated by continental ice sheets at least four times during the Pleistocene. However, only the deposits of the last glaciation, the Laurentide of Wisconsinan age, are preserved in the Elk Lake area. (Read, 1979).

The site is located along the edge of a major lacustrine plain, known for varved clay. Immediately north of Elk Lake extends extensive deposits of sandy outwash plains, as indicated by aggregate pits mapped north of the site.

The topography at the site consists of gently rolling hills. The river has cut into the sediments confirming a transitional or near shore environment during glacial retreat.

The Montreal River at the site is controlled upstream by a dam, and due to a natural constriction at Elk Lake, has a noticeable current.

3.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at 10 boreholes and at two electronic cone penetrometer (piezocone) locations. The locations of the boreholes and piezocone or CPTU tests are shown on the Plan and Profile, Drawing No. 1278801-A. Details of the stratigraphy encountered in the boreholes are given on the individual Record of Borehole sheets (logs), Appendix 'C'. At some standard penetration test locations below the water tables there appeared a possibility that soil disturbance may have occurred within the zone of the test. The split spoon was therefore driven an additional depth to obtain a better indication of the soil condition past the effects of soil disturbance near the auger head. The blow count for the last 0.3 m are noted on the Boreholes Logs as N' in the remarks column, which are considered more representative of in situ conditions than the upper blow count (see footnote). The N or N' values versus depth are plotted on Enclosure 13. Dynamic cone penetration tests were also carried out at Boreholes 2, 3, 5, 6, 8, 9 and 10 and are plotted on the Borehole Logs. The CPTU data is presented in Appendix 'C'. The subsurface conditions can be summarized as follows.

At the west bank of the site (Boreholes 1, 2 and 10) the generalized stratigraphy consists of fill overlying an organic deposit which in turn is underlain by alternating layers of sand, silt and clay to approximate elevations 226 to 246 m. Beneath these strata sands and gravels exist. At the east bank (Boreholes 7, 8 and 9) the stratigraphy is similar but the organic deposit is absent. The maximum depths of penetration from existing grade were at Borehole 10 (63.9 m) and Borehole 9 (68.9 m).

In the river bed the generalized stratigraphy consists of sand with the occasional silt layers (Boreholes 4 to 6). Organic pockets and seams occur throughout the upper 10 m of the deposit. Due to the limited drill

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N = Standard Penetration Test Blow Count (blows for last 0.3 m for a penetration of 0.45 m).

N' = Number of blows for last 0.3 m when sampler is driven beyond depth for standard penetration test.

rig size (because of ice thickness), the maximum penetration depth below river bed which could be achieved was 19 m. The depth of water at the time of investigation varied between 4.6 and 6.7 m.

The individual strata are briefly described in the following paragraphs.

3.1 Fill

The fill consists of a mixture of sand, silt and gravel with organic layers and pockets throughout. The thickness varies between 2 and 3 m. The condition of the fill is very loose to compact as indicated by the N values from the standard penetration tests varying from 1 to 21 blows/0.3 m.

The boreholes on the west bank were located near the existing shorelines. It should be noted that thicker layers of fill exist up slope within the approach embankments.

3.2 Organics/Peat

Beneath the fill on the west bank of the site, an organic and/or peat layer mixed with sand and silt exists. The organic layer varies in thickness from 0.5 to 4.1 m. Furthermore, deeper soil deposits also have various layers and pockets with significant organic content. Organic content tests conducted on samples from Boreholes 1 and 2 between depths of 2 and 10 m indicate an organic content varying between 0.3 and 44%. The organic content for the individual samples are shown on Enclosure 6.

A distinct organic layer is absent in Boreholes 4, 5 and 6 in the riverbed but thin organic pockets and layers were noted in the upper 10 m of the sand stratum.

To characterize the compressibility of the organic soils, consolidation tests were conducted on samples from Boreholes 2 at depths of 4.6 and 5.3 m. The test results are shown on Enclosures 7, 8 and 9. The moisture contents obtained from the organic samples are shown on the Borehole Logs, and vary between 11 and 229%. For this test the samples were oven dried to a constant weight at a temperature of 85°C. Reference to Enclosure 6 indicates that the moisture content varies directly with the organic content.

Both the compressibility and strength characteristics of the organic material is of importance to the project. Undrained material strength is normally applicable to approach embankment construction over peat. The piezocone results indicate thin zones with undrained behaviour and an interpreted undrained shear strength of 30 to 40 kPa.

As a indication of the undrained strength to be expected following construction of a new embankment, a single quick direct shear test was carried out on a 'disturbed' sample of 18% organic soil with no fibre content and a minimal sand content (mainly silt) and consolidated at 88 kPa. Results are given on Enclosure 11 and indicate a shear strength of at least 35 kPa at 3% strain.

The drained strength parameters of the peat have not been measured for this project, however published data indicates that at large displacements (which remove the reinforcing effect of fibres) an effective friction angle of 28° to 32° is applicable (Landva and La Rochelle, 1983).

The compressibility of organic soils is very high, and laboratory consolidation tests were carried out on 70 mm diameter shelby tube samples for defining both primary (short term) and secondary (long term) consolidation parameters. Multi-stage tests were carried out, whereby the primary consolidation (which occurred very rapidly) was estimated from a settlement/log-time plot. An additional single increment test was also carried out to a 73 kPa stress to provide better definition of the primary and secondary consolidation as recommended by MacFarlane (1969) and Landva and La Rochelle (1983). Consolidation testing results have been plotted on Enclosures 7, 8 and 9.

The compressibility characteristics of the peat have, for this report, been interpreted in the form of a constrained tangent modulus. A tangent modulus gives deformation or strain in terms of the applied load directly without the need for calculating void ratio (Canadian Geotechnical Society, 1985). This method is recommended for organic soil by Rowe (1984) and MacFarlane (1969). Furthermore, an alternate method which uses void ratio and a compression ratio, C_c, can introduce inaccuracies in establishing the initial very low effective stresses in organic soils and their large effect on the calculated settlement (Landva and La Rochelle, 1983). The tangent modulus (slope of the stress-strain curve) determined from oedometer or consolidation tests is a constrained modulus which is particularly applicable to settlement of embankments over peat. The rate of primary consolidation in all cases was rapid, being complete within a few minutes of laboratory loading.

Secondary consolidation was also considered, where a load increment at 73 kPa was maintained for several days. These results are given on Enclosure 9 and give a rate of secondary consolidation $C_{\alpha\epsilon}$ (change in strain per log cycle of time) ranging from 1 to 3%. This method of characterizing long term settlement of peat is recommended by MacFarlane (1969) and Mesri (1973).

3.3 Sand

The sand deposits have a varying silt content varying from 5 to 35%, as noted from the gradation analysis conducted on representative samples. The gradation analyses are indicated on the Borehole Logs where applicable and graphically on Enclosures 2 to 5, which indicates some of the sand is potentially liquefiable under earthquake loadings (See Section 4.5). The grainsize data suggests hydraulic conductivities of the sand are between 10^{-2} and 10^{-4} cm/sec.

The condition of the sand deposits varies from loose to very dense as indicated from the N values from the standard penetration tests and DCPT data. The sand layer varies in thickness from 3 to greater than

15 m.

To provide an estimate of the lower bound of the shear strength of the sand, drained direct shear tests were conducted on a normally consolidated sample from Borehole 3 at a depth of 21.3 m indicating an internal angle of friction (ϕ) of 37 degrees.

The CPTU tests inferred N values and ϕ angles (above 20 m) are plotted on the CPTU data sheets Appendix 'C'. These indicate highly variable zones with interpreted ϕ values of between 31° and 41° with N values interpreted between from less than 10 to 40 blows/0.3 m.

3.4 Silt

Two distinct silt deposits were noted in the deep boreholes. The upper layer occurs between a depth of 10 and 15 m below grade and varies in thickness from 4.6 to 7.1 m. This layer has a varying sand content and is in a loose to compact condition according to the N values of 3 to 23 blows per 0.3 m (as well as CPTU interpreted N values).

The second silt deposit is present 22 to 33.5 m below grade and varies in thickness from 9.1 to 22.0 m. Sufficient amounts of clay are present in some layers to impact plasticity to silt (Atterberg Limit Test, Enclosure 2). The silt is frequently interbedded with clay lenses of various thicknesses, ranging from less than 5 mm to greater than 100 mm. Drained direct shear tests were conducted on the samples of the silt from Borehole 2 at a depth of 36.6 m, Borehole 4 at 18.3 m and Borehole 7 and 34.8 m, which indicated internal angles of friction of 30, 36 and 29 degrees respectively. Gradation analysis conducted on samples from this deposit are shown on the Borehole Logs where applicable, and Enclosures 2 to 5, indicating that many of these soils may be categorized as potentially liquefiable under earthquake loads (see Section 4.5).

3.5 Clay Lenses

Clay lenses (varves) are interbedded within the silt deposits. These lenses are typically from 5 to more than 100 mm in thickness.

Atterberg limits conducted on the clay indicate medium to high plasticity with a liquid limit ranging between 34 and 79 and a plasticity index ranging between 15 and 43 (Enclosure 1A). A consolidation test conducted on a clay sample from Borehole 2 at a depth of 39.6 m is presented on Enclosure 10.

3.6 Sands and Gravels

Sands and gravels with occasional layers of cobbles are found below the silt and clay stratum in Boreholes 7, 9 and 10. This stratum extends to the depth of penetration of the boreholes.

The condition of the sands and gravels varies from compact to very dense, as indicated from the N values from standard penetration tests, which varied from 13 to 102 blows for 0.3 m. Dynamic cone test blow counts ranged from about 30 to more than 300 blows/ft. Grainsize analysis from a sample of this stratum from Borehole 9 at a depth of 54.9 m is shown on Enclosure 5.

3.7 Bedrock

The depth to bedrock is unknown at this site. The bedrock type is expected to be greywacke based on published geological information.

3.8 Groundwater

Groundwater levels in the open boreholes were observed during the drilling and at completion of each hole. A Casagrande type piezometer, an open standpipe piezometer and a pneumatic piezometer where installed in Boreholes 7, 8 and 1 respectively.

The river level was at Elevation 279.4 m at the time of the investigation. MTO data indicated the river level was at Elevation 279.204 m on September 17, 1992. It should be noted that the river level is controlled due to the hydro dam upstream.

The following groundwater levels were measured in piezometers and standpipes at the on shore boreholes.

TABLE 1

BOREHOLE	DATE	DEPTH TO WATER (m)	ELEVATION (m)	COMMENTS
1	Feb 16	0	279.6	Frozen
7	Feb 16	2.3	280.5	Frozen
8	Feb 16	3.5	279.4	

Based on the above data as well as CPTU dissipation tests, the porewater pressure distribution at depth is essentially hydrostatic (ie. no artesian pressure).

4.0 DISCUSSION AND RECOMMENDATIONS

The proposed bridge is to consist of a 5 span, 2 lane structure located about 5 to 10 m south of the existing bridge.

The bridge deck will be about 2 m higher than the existing bridge and the approach embankments will extend into the river. The conceptual design of the bridge is not defined at present. Details of proposed configuration, that is pier type, size, spacing, loading are yet unknown. The existing bridge is supported on timber piles, and is understood to be structurally in poor condition with large areas of concrete spalling visible.

4.1 Foundation System

The upper soils at the site consist of loose organic and alluvial soils with low strengths and high compressibilities, therefore a shallow foundation system consisting of footings is not suitable for this site. The recommended foundation system for the bridge abutments and piers utilizes driven piles. Pile capacities are dependent on the length and type of pile used. The different pile types considered suitable to carry the proposed structural loading of about 8000 kN per abutment are discussed below. Maximum capacities at ULS of 1300 kN are recommended, subject to confirmation of structural capacity.

Lighter loads may be supported on shallow piles to elevation 255, although capacities are low.

Shorter steel and timber piles have been reviewed. However, because of the loose compressible nature of the upper soil strata, capacities will be low and such piles are not recommended.

The soil stratigraphy across the river varies considerably, particularly in the upper 30 m. Design recommendations are currently based on the more critical stratigraphy of the west side of the river.

Improved capacities for shallow (tip elevation 255) piles are be available on the east side of the river.

Due to the variability of the site soil conditions along the bridge centreline, the unusually long.pile lengths anticipated (60 m), and the importance of the minimizing piling costs (by avoiding unnecessary driving to 'dead refusal' and by maximizing capacity without costly static load tests) confirmation of the pile capacities should be undertaken during the initial stages of construction utilizing dynamic pile testing (OHBDC Section 6-9.7.5).

4.1.1 Abutment Foundations

The abutments may be supported on deep driven steel piles consisting of H-piles or closed end steel pipe piles, as summarized in Table 1. While concrete piles are also feasible, they have not been further considered due to the complex nature of the stratigraphy and the difficulties associated with field adjustments to pile lengths.

H-piles may be driven to support light to high loadings. A minimum pile section of HP310 x 79 is recommended for light loadings. Capacities are primarily dependent on the length of pile utilized. For moderate loads, it is recommended that piles be driven a minimum nominal 45 m below grade (tip elevation 235 m). Should higher capacities be required, the piles may be driven into the sand and gravel zone in the order of 60 m (nominal) below grade (tip elevation 220 m). A large capacity hammer with a minimum rated driving energy of 55 kJ is recommended for advancing the piles. Heavier pile sections will be required at moderate to high loadings to sustain the anticipated driving stresses.

Closed end steel pipe piles may also be considered for the foundation system. The pipe piles should be driven to a nominal depth of 45 m (Elevation 235 m). These will act as displacement piles and will densify the cohesionless soils as they are installed. The minimum driving energy of the hammer should be as noted above for the H-piles. Shallower, low capacity piles may also be used, although a large number of piles will be required.

The structural design of the pile capacity will have to include loads resulting from the negative skin friction or drag induced by consolidation of the existing fill/organic materials with the addition of the proposed embankment fill. In the structural design of the pile, the following separate loading cases must be considered: 1) permanent dead load plus drag load, but no live load; and 2) permanent dead load plus live load, but no drag load. For this site the drag load can be calculated using negative skin friction values of 20 kPa for the embedded length of pile above Elevation 272 m. This value applies to the exterior perimeter of the pile.

The following compressive axial capacity values are recommended for the design of the piles for the abutments. (Note that short displacement piles have a higher capacity on the west side). The following capacities are contingent on dynamic testing a minimum of one piles at each abutment location. Without testing the pile capacity should be reduced 20%.

TABLE 2

Pile Type	Approximate Tip Elevation m (Approx. Pile Lengths)	Factored Axial Capacity at ULS kN	Axial Capacity at SLS kN
HP310 x 79	255 (25)	290	220
HP310 x 110	235 (45)	850	640
HP310 x 110	220 (60)	1300	975
Pipe Pile (324 mm dia)	235 (45)	850	640
Pipe Pile (324 mm dia) East	255 (25)	225	170
Pipe Pile (324 mm dia) West	255 (25)	450	340

Notes: Pile lengths estimated from elevation 280 m. Actual lengths will vary depending on pile cap elevations.

Pile caps at the abutments should have a minimum of 2.2 m earth cover as frost protection. Bottoms of abutment footings supported on piles should be placed at least 1 m below scoured bed elevation, otherwise sheet piles should be used in accordance with Ontario Highway Bridge Design Code requirements.

Pile spacing, clearance and embedment should be in accordance with Ontario Highway Bridge Design Code (OHBDC) 6-11.

4.1.2 Pier Foundations

The design of the pier foundation system should also consist of driven piles as discussed above. The following compressive axial capacity values are recommended for the design of the pile, and applies for the case where capacities are confirmed by dynamic testing of a minimum 1 pile per pier.

TABLE 3

Pile Type	Approximate Tip Elevation m (Approx. Length)	Factored Axial Capacity at ULS kN	Axial Capacity at SLS kN
HP310 x 79	255 (25)	230	173
HP310 x 110	235 (45)	850	640
HP310 x 110	220 (60)	1300	975
Pipe Pile (324 mm dia)	235 (45)	850	640
Pipe Pile (324 mm dia)	255 (25)	180	135

Pile lengths estimated from elevation 280 m. Actual lengths will vary depending on pile cap elevations. Pile spacing, clearance and embedment should be in accordance with OHBDC 6-11. We have assumed scour protection will be provided at the piers with sheet piles or protective aprons (OHBDC 1-9.6.3).

4.1.3 Pile Installation

For H-piles driven to elevation 220 m into the sand and gravel layer, the pile should be reinforced with a steel plate per MTO standard DD-3301 to prevent damage to the pile tip. Pile splice and driving shoe details should be in accordance with Ontario Provincial Standard Drawings OPSD-3301.00 for H-piles and OPSD 3302.00 for steel pipe piles.

Because of the variable soil conditions and unusually long piles, the predictability of pile capacity is less precise than for conventional shorter piles in uniform soils and driven to refusal on sound bedrock. The capacity of the piles should be confirmed by a pile load test program (dynamic testing of one pile at each pier and abutment location).

All aspects of the pile materials and installation should conform to the requirements of the Ontario Highway Bridge Design Code.

Driving records should be kept for each pile. Information to be recorded should include (but not be limited to) pile dimensions, hammer type, rated energy, ram weight, cap block type and weight, anvil weight, number of blows for each foot of penetration and final set. All pile driving equipment must be in good working order. The suitability of the proposed driving equipment should be verified by the geotechnical engineer prior to construction.

It is recommended that pile driving be monitored and the "setting" of all piles be controlled by reference to the Hiley Dynamic Pile Driving Formula, in accordance with MTO Standards SS103-10 or SS103-11, assuming:

TABLE 4

Pile Type	Approximate Tip Elevation m (Approx. Length)	Ultimate Capacity kN
HP310 x 79	255 (25)	520
HP310 x 110	235 (45)	1920
HP310 x 110	220 (60)	2925
Pipe Pile (324 mm dia)	235 (45)	1920
Pipe Pile (324 mm dia)	255 (25)	405

The elevation of the tops of driven piles should be measured immediately after driving. If uplift occurs in any piles during the driving of adjacent piles, the displaced piles should be re-driven to at least their previous final elevation and final set.

It should be noted that during test drilling, the "B" size casing and augers experienced significant soil "freeze". Hard pile driving should therefore be anticipated following stoppages in the driving sequence.

Where piles are driven in groups, they should be driven from the centre outwards. In general, all piles in a group should be driven to a similar tip elevation.

4.2 Lateral Pile Capacity

Lateral loads will be induced onto the foundation by loads such as traffic, soil backfill, ice loads, current loads and possibly temperature effects. These must be transferred to the underlying soils. Given the poor soil conditions in the upper strata of the site, the lateral capacity of vertical piles at the abutments should be taken as zero. Lateral loads should be resisted with battered piles.

Lateral loadings at the pier locations may be resisted using either battered piles or the lateral capacity of the piles. The capacity of piles under lateral load varies with the deformation for piles. The following lateral capacities (SLS) are recommended, assuming that some protection to the existing riverbed is provided that the load is applied at the pile cap and no unsupported pile length is used.

TABLE 5

Deformation	Lateral Capacity
25 mm 10 mm	50 kN 30 kN

4.3 Lateral Earth Pressures

Earth pressures should be computed as per Section 6.7.4.5 of the OHBDC and the coefficient of earth pressure at rest should only be used for rigid and unyielding walls. The Granular 'A' or 'B', Type 1 backfill should be in accordance with Ontario Provincial Standard Specifications (OPSS 1010). The following parameters are recommended for the granular backfill when calculating earth pressures on walls.

TABLE 6

	Granular 'A'	Granular 'B', Type 1	Clear Stone
Angle of Internal Friction	$\phi = 35°$	$\phi = 30°$	$\phi = 30°$
Unit Weight (kN/m^3)	$\gamma = 23$	$\gamma = 21$	$\gamma = 22$

4.4 Approach Embankments

The proposed finished grade is expected to be in the order of 2 m higher than the existing grade. The stability and settlement of the existing soils are discussed below in Section 4.4.1 and 4.4.2. The fill should consist of a clear stone or rockfill to above the water level at time of construction and the remaining portion may consist of well compacted Granular 'B', Type 1 fill. On the west embankment a 2 m wide bench should be placed near midslope of the embankment (approximately elevation 280 m). Widening of the existing side slopes should be carried out in accordance with OPSD 208.01 (benches).

The approach fill should be protected by a rip rap revetment to at least 1.0 m above the high water level. Toe protection should be provided to prevent undermining of the revetment. Granular 'A' material (or non-woven geotextile) should be provided as separation between the rip rap and soil to prevent loss of fines.

4.4.1 Approach Embankment Stability

Slope stability analysis has been completed for both the east and west approach fill areas. These were carried out using the Bishop simplified method of analysis using the software SLOPE/W by Geo-Slope International. Three cases of critical conditions were analyzed, one with a traffic loading of 40 kN/m, one with a seismic force (seismic zonal velocity of 0.04 m/s) and one with a 2 m drawdown of the river. Results are given on Table 5. An example of the computer analysis section is provided as Enclosure 12.

TABLE 7

FACTORS OF SAFETY AGAINST SLOPE FAILURE
FOR THE EAST AND WEST EMBANKMENTS

EMBANKMENTS	TRAFFIC LOADING ONLY	EARTHQUAKE LOADING Zv = 0.04	DRAWDOWN OF RIVER TO ELEVATION 277.0 m
East embankment 2h:1v slope	1.36	1.21	1.34
West embankment with bench 2h:1v slope	-	1.25	-
West embankment with bench 2.5h:1v slope	1.67	1.41	1.53

4.4.1.1 East Embankment

The configuration for the east embankment may be constructed with side slopes of a 2 horizontal to 1 vertical without a bench, providing the fill materials used below water level consists of clear stone or riprap. From our analysis a minimum factor of safety of 1.3 was obtained for this configuration in all cases except earthquake loading. For the earthquake loading the inclusion of a geogrid type reinforcement system will be required (Enclosure 15) to increase the calculated factor of safety to above 1.3. Should a safety factor of 1.2 under earthquake loading conditions be suitable, then geogrid may be deleted.

4.4.1.2 West Embankment

The west embankment, which is more critical than the east embankment due to the underlying organic layer, requires a slope configuration of 2.5 horizontal:1 vertical slopes and a 2 m wide bench placed near mid-slope in order to achieve a safety factor of 1.3.

For short term conditions during and shortly after construction, there is a concern that certain zones of the organic layer may behave in an undrained manner and therefore an undrained analysis was carried out for the organic layer. The critical slip surface for this analysis travels through the organic layer found approximately between elevations 276 and 271 m. Due to a temporary low factor of safety for this situation it is recommended that fill be placed in two stages, to avoid unpredictable localized failures. The fill should first be placed to elevation 284 m and higher fill placement postponed for approximately one month to allow consolidation and strengthening of the underlying organic layers.

4.5 Liquefaction Potential Under Seismic Loading

Loose sands and silts are susceptible to liquefaction and loss in strength during an earthquake which could destabilize approach fills. To determine if the soils at this site are potentially liquefiable under cyclic loading, two analytical steps were employed. The first step involved identifying the potentially liquefiable soil layers using grainsize analysis and the use of the CPTU. The second involved applying a design earthquake and the effect of overburden confinement of these soils to estimate safety factors against liquefaction.

The grainsize distribution of a soil can be used (as a first estimate) to determine if the soil is susceptible to liquefaction. The grainsize distribution limits for cyclic liquefaction susceptibility are based on the outer most limits of several authors (R.E. Hunt, 1986).

The grainsize distribution for selected soil samples (non-cohesive sand and silt layers) have been plotted against the cyclic liquefaction susceptibility limits and are presented on Enclosures 2 to 5. Many of the samples analyzed fall within the limits indicating that, based strictly on grainsize distributions, some of the soils may be susceptible to cyclic liquefaction.

The piezocone test (CPTU) was used at this site to characterize the subsurface soils. Liquefaction potential plots consisting of cone tip bearing (q_T) plotted against the friction ratio also identified many layers of soil with a potential to be liquefiable.

A further analysis was carried out to determine the factor of safety against liquefaction of these soil layers when subjected to a design earthquake. This method also takes into account the overburden stress on the soil layer. The method used for this analysis has been developed by Seed and involves using SPT data in determining the average cyclic stress ratio developed in the field due to earthquake shaking (CSR_{Eq}) and estimating the average cyclic stress ratio required to cause liquefaction of the soil (CSR_{Qc}). The factor of safety against liquefaction can be calculated for each soil layer quantified by the CPTU (ie. every 50 mm) and is plotted on the CPTU Logs (Appendix 'C').

The design earthquake used for this analysis consisted of a magnitude (M) of 7.0 which is conservatively larger than the largest nearby earthquake recorded (near Timiskiming) of 6.2, and a velocity 0.04 m/s (OHBDC A2-1.6). For SPT data, a correlation with CPTU data was used (Robertson and Campanella, 1989).

The results of this analysis indicate that the factors of safety against liquefaction for soils subjected to the above noted design earthquakes are 1.2 or higher. This is considered acceptable for the stability of the approach embankments.

4.6 Construction Considerations

Construction of the abutments and piers may involve excavation below the groundwater and river levels. The soils at this site will require shoring and/or dewatering to control seepage and sloughing and to provide a stable base for support of concrete while curing. Construction of the piers may require the use of suitably designed coffer dams.

To eliminate problems with for pile installation through rockfill, an appropriate sleeve should be installed at pile locations prior to placement of rockfill.

All excavations should be carried out in accordance with the Occupational Health and Safety Act of Ontario.

Pile driving for the foundation piles for the new bridge structure may result in vibrations which may be unacceptable for the existing bridge structure especially since it is understood the existing bridge is in poor structural shape. Continuous monitoring of vibrations induced in the existing structure are recommended as is continuous monitoring of the existing bridge structure. Given the condition of the existing bridge, temporary structural reinforcement may be required during construction.

4.7 Scour Protection

The piers and abutment should be provided with sufficient scour protection to ensure the piles are not exposed. Scour should be calculated in accordance with Section 1-9.5 of the OHBDC for cohesionless soils.

4.8 Soil Chemical Properties - Corrosion

Three soil samples were analyzed in the laboratory for the following parameters: Resistivity, pH and redox potential and the results are indicated below:

TABLE 8

Boreholes	Sample Depth (m)	Resistivity (ohm-cm)	pH	Redox Potential (mV)
1	3	932	5.62	182
2	15	2771	7.2	58
7	10	1175	7.17	76
8	6	837	6.79	54

4.9 Cement Type

Sulphate tests carried out on soil samples gave the following results and indicate a normal Portland Cement is suitable for use at this site.

TABLE 9

Borehole Location	Depth (m)	Sulphate (mg/kg)
1	3	91.99
2	15	<0.01
7	10	93.71
8	6	31.81

5.0 LIMITATIONS OF REPORT

A description of limitations which are inherent in carrying out site investigation studies is given in Appendix 'A', and this forms an integral part of this report.

For DST CONSULTING ENGINEERS INC.

Prepared by:	Reviewed by:
Wayne Hurley, P. Eng. Manager, Thunder Bay	Mike Fabius, P. Eng. President

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APPENDIX 'A'

LIMITATIONS OF REPORT

APPENDIX 'A'

LIMITATIONS OF REPORT

The conclusions and recommendations presented in this report are based on information determined at the testhole locations. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the site investigation. It is recommended practice that the DST Consulting Engineers be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavation, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

Unless otherwise noted, the information contained herein in no way reflects on environmental aspects of either the site or the subsurface conditions.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs, e.g. the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

APPENDIX 'B'

METHOD OF INVESTIGATIONS

APPENDIX 'B'
METHOD OF INVESTIGATION

The fieldwork for this project was carried out in the period January 8 to April 17, 1996. During this period 10 boreholes and 2 electronic static cone penetration tests were put down at locations shown on Drawing No. 1278801-A.

The field program was conducted in three stages. The initial methodology was to drill the boreholes on land and construct an ice bridge over the river to provide access for and sufficient ice thickness to support the drill rig. Boreholes 1, 2, 7 and 8 at the abutments and approaches were completed from or near the shore using a CME 750 mounted on a all terrain vehicle. Holes were advanced using 80 mm id hollow stem augers and 'B' casing. Static cone penetration tests were also completed at locations 2A and 7A.

To construct the ice bridge, snow berms 0.6 m in height were constructed to aid in the flooding of the proposed ice bridge across the river. The area was frequently flooded and snow removed from the surface to thicken the ice. To support the large drill rig, an equivalent thickness of 0.6 m of blue ice is required. At the start of construction the ice thickness varied between 0.3 to 0.5 m. It was noted that as the ice became thicker and sank lower, scour at the base of the ice due to current occurred. Nevertheless, approximately 0.1 m of new ice was manufactured. At that point, unexpectedly warm weather (January 17 and 18) melted the snow berms and much of the ice thickness previously acquired. In consultation with MTO, it was decided to demobilize the drill and mobilize at a latter date, when colder temperatures returned and the ice thicknesses increased. Snow removal and monitoring of ice thickness was contracted to a local townsperson who reported back regularly to the Thunder Bay office.

After a period of 3 weeks, it was apparent that the required ice thickness would not be achieved. Further discussions were held between MTO and DST personnel and it was decided to mobilize a lighter rig to

APPENDIX 'B'
METHOD OF INVESTIGATION

complete the field program. With the lighter rig, less ice thickness is required, but the rig has less torque, therefore the maximum depth of penetration is reduced.

A trailer mounted CME 45 drill rig was mobilized to the site on February 8, 1996 and completed the fieldwork February 16, 1996. The boreholes were extended to the depth indicated on the Borehole Logs utilizing hollow stem augers as well as 'B'-casing with wash boring techniques. The maximum depth of penetration during this stage was 21 m below ice surface. Difficulties were encountered at these depths with sand entering the 'B' casing by as much as 3 m, and insufficient drill torque to continue, due to "soil freeze".

After analysis of the soil stratigraphy defined in the first 2 stages of the program, it was decided that confirmation of design soil stratigraphy was required. An additional borehole was drilled on each of the east and west river banks (BH's 9 and 10) to depths below existing grade of 68.9 and 63.9 m. These boreholes were completed with a CME 750 drilled rig utilizing 'B' casing and drilling mud techniques. The work was completed during the period from April 8 to 19, 1996.

Detailed sampling consisting of standard penetration tests and 70 mm shelby tube samples was conducted in each of the boreholes. Five dynamic cone penetration tests were conducted at the site. One test location was adjacent Borehole 8 and the other tests were conducted at the base of Boreholes 2, 3, 5 and 6.

Two electronic cone penetrometer or piezocone tests (2A and 7A) with pore water pressure measurements (CPTU's) were conducted adjacent Boreholes 2 and 7 to depths of between 16.6 and 20.7 m. This test

APPENDIX 'B'
METHOD OF INVESTIGATION

consists of pushing a 10 cm^2 cone (10 tonne Hogentogler piezocone) into the soil at a constant 20 mm/s penetration rate. The resistance of the tip and the friction along the cylindrical sleeve behind the tip are measured by means of electric sensors. The pore-water pressures are measured by means of a transducer in a porous filter located immediately behind the tip. Data is collected at 50 mm depth intervals utilizing a computerized data acquisition system. The data is interpreted using University of British Columbia software CPTINT with respect to the following parameters. Methods of interpolation are outlined in "Guidelines for Geotechnical Design using CPT and CPTU" (Robertson and Campanella, 1989).

- layering and soil type (from tip resistance and pore pressure ratio)
- undrained shear strength of clay (using N_K = 15)
- drained friction angle for sands
- correlated 'N' values for the soil
- potentially liquefiable soils
- factor of safety for liquefiable soils based on a design earthquake
- in situ pore pressures, coefficient of consolidation and permeability (through dissipation testing).

Borehole Logs and plots of the CPTU data are presented in Appendix 'C'.

The boreholes were located in the field by DST personnel, and on completion of the field program the location and ground surface elevations were surveyed by MTO personnel.

The site investigation was carried out under the supervision of a geotechnical engineer from DST's

APPENDIX 'B'
METHOD OF INVESTIGATION

Thunder Bay office. Soil samples were returned to our Thunder Bay laboratory. Classification and index tests were subsequently performed in the laboratory on selected representative samples. Laboratory tests included natural moisture content, gradation (sieve and hydrometer) analyses, Atterberg limits, organic content, consolidation tests, direct shear tests and chemical analyses.

APPENDIX 'C'

RECORD OF BOREHOLE SHEETS

CPTU DATA

DST CONSULTING ENGINEERS INC.

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 1

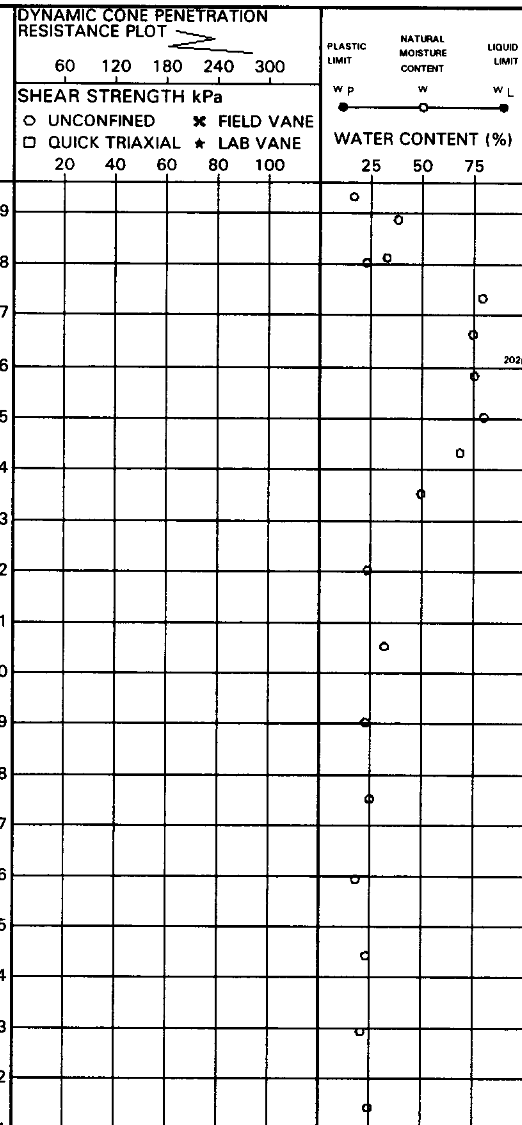
METRIC

W.P. 127-88-01 | LOCATION CO-ORDINATES 5287979 N 355011 E | ORIGINATED BY G.M.

DIST 53 HWY 65 | BOREHOLE TYPE HOLLOW STEM AUGER | COMPILED BY C.S.

DATUM Geodetic | DATE January 17th, 1996 | CHECKED BY W.H.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
279.6	**GROUND SURFACE**					
0.0	FILL - sand/silt/gravel - some organics, brown - - - - - - wood fragment	1	AS		279	
		2	SS	3		
		3	SS	1	278	
276.5		4	SS	1	277	
276.0	PEAT	5	SS	2		
3.6	SILT - trace to sandy, organics, brown, very loose - wood fragment	6	SS	1	276	
		7	TW	PH	275	
		8	SS	1	274	
273.1		9	SS	2	273	
6.5	SILT & SAND - layered, organic pockets & seams, brown/grey, very loose to loose	10	SS	3	272	N' = 9
269.1		11	SS	6	270	N' = 13
10.5	SAND - trace to silty, interbedded silt layers 0.15 to 0.3m thick, grey/brown, compact to dense	12	SS	3	268	N' = 16
		13	SS	14	267	N' = 16
		14	SS	8	265	N' = 29
		15	SS	16	264	N' = 28
		16	SS	13	262	N' = 23
260.9		17	SS	31	261	
18.7	End of Borehole at 18.7m Pneumatic Piezometer installed to 18.5m with Bentonite seal from 15.2 to 17.3m & 4.9 to 6.6m. Note: N' represents the number of blows per 0.3m for the last counts for a split spoon driven past 0.45m					



UNIT WEIGHT γ kN/m³ | GR SA SI CL

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

15 -⊕- 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

W.P. 127-88-01	LOCATION CO-ORDINATES 5287992 N 355024 E		ORIGINATED BY G.M.
DIST 53 HWY 65	BOREHOLE TYPE HOLLOW STEM AUGER		COMPILED BY C.S.
DATUM Geodetic	DATE January 18th, 1996		CHECKED BY W.H.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
279.5	ICE SURFACE				
279.5 0.0	ICE				
0.5	FILL - sand - silty, 0.6m layer of organics	1	SS	3	
		2	SS	1	
276.6		3	SS	1	
2.9	SAND/SILT/ORGANICS/WOOD - trace clay, organic content varying from 14 to 44%, layers and pockets of wood & organics	4	SS	1	
		5	TW	PH	
		6	SS	1	0 31 61 8
		7	TW	PH	
272.5		8	SS	1	
7.0	SAND - trace to some silt, occ. organic seam (< 2mm), grey, very loose	9	SS	1	
268.7		10	SS	3	N' = 5
10.8	SAND/SILT/ORGANICS/WOOD - layered, loose	11	SS	4	N' = 8
267.4 12.1	SAND - trace to silty, trace organics, grey, loose to compact	12	SS	3	N' = 21
264.3		13	SS	5	N' = 18
15.2	SILT - interbedded with sand layers 0.05 to 0.2mm thick, grey, compact	14	SS	17	N' = 29
		15	TW	PH	
	----- - occ. 5 to 10mm clay varves	16	SS	19	N' = 22
		17	SS	23	N' = 30
257.2		18	TW	PH	
22.3	SAND - grey, compact to dense	19	SS	35	N' = 49
	----- - some silt	20	SS	2	N' = 33
	----- - trace silt	21	SS	33	0 94 6
		22	SS	27	N' = 35
249.5 30.0		23	SS	22	N' = 40

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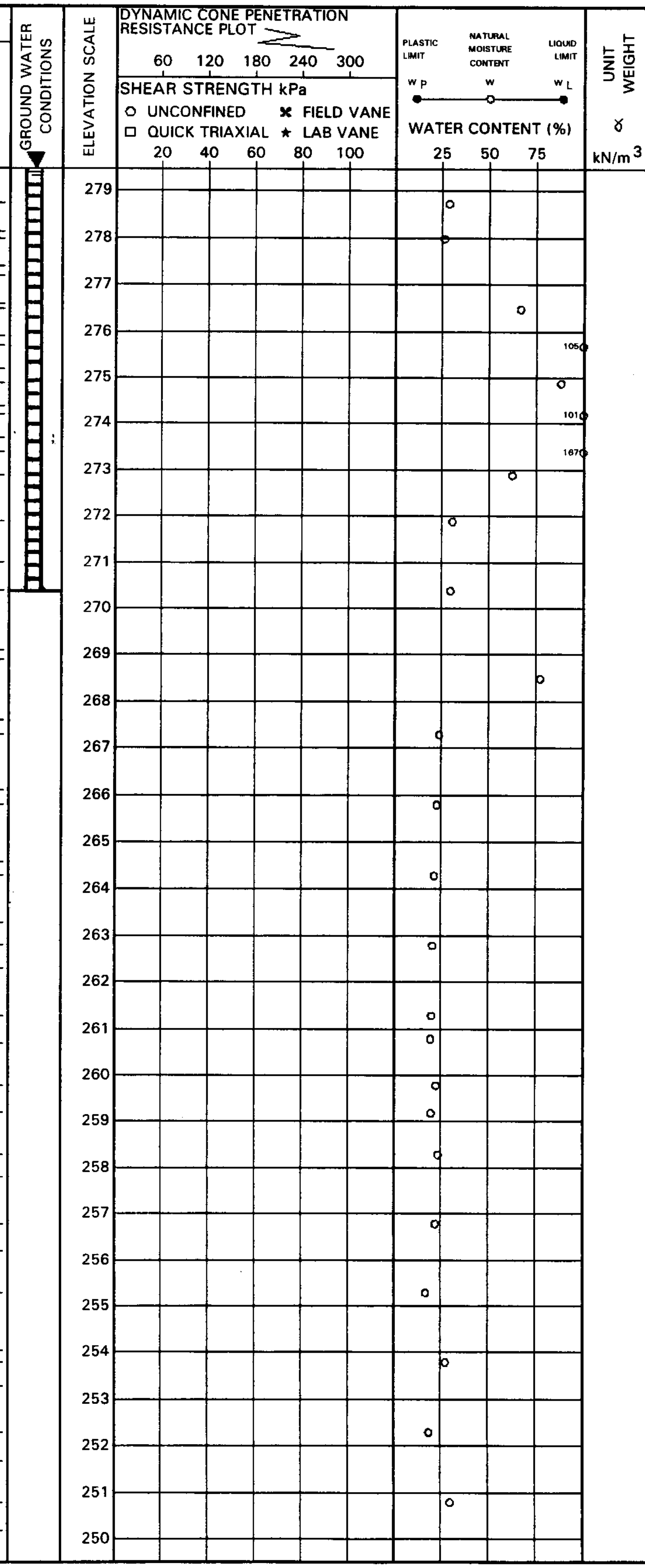


✖ 3, ★ 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

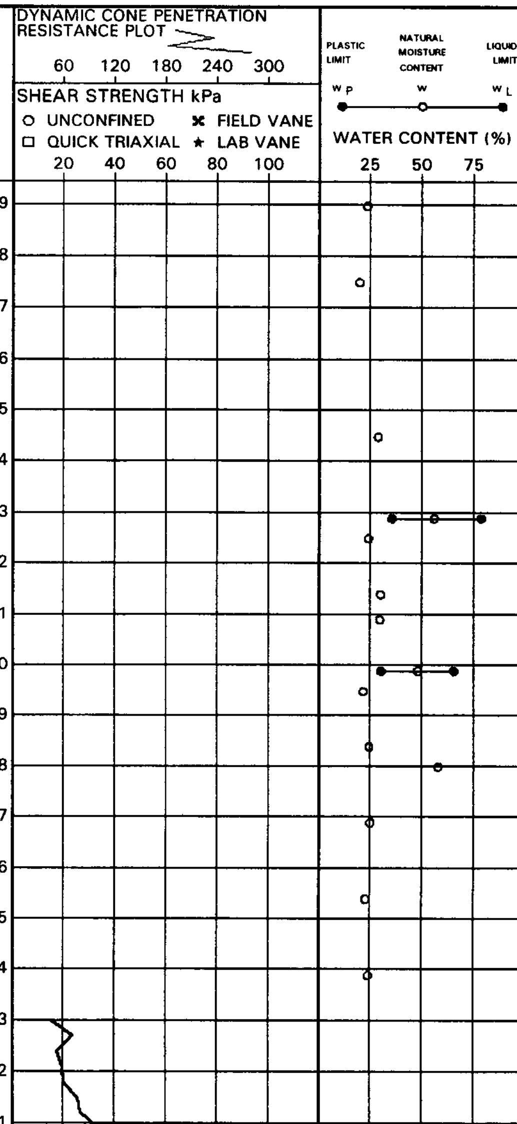
METRIC

W.P. 127-88-01
LOCATION CO-ORDINATES 5287992 N 355024 E
ORIGINATED BY G.M.

DIST 53 HWY 65
BOREHOLE TYPE HOLLOW STEM AUGER
COMPILED BY C.S.

DATUM Geodetic
DATE January 18th, 1996
CHECKED BY W.H.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
249.5	CONTINUED				
30.0	SAND & SILT - layered, grey, dense to very dense	24	SS	32	N' = 43
		25	WS		
246.0 / 33.5	SILT - occ. clay varve (5-75mm), very dense	26	SS	28	N' = 33
		27	TW	PH	
		28	SS	39	
		29	TW	PH	
		30	SS	51	
		31	TW	PH	
		32	SS	49	
233.3		33	TW	PH	
46.2	End of Borehole at 46.2m DYNAMIC CONE ONLY NO SAMPLES TAKEN				
230.8 / 48.7	End of Dynamic Cone @ 48.7m Standpipe installed to 9.1m Note: N' represents the number of blows per 0.3m for the last counts for a split spoon driven past 0.45m.				



Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; UNIT WEIGHT γ kN/m3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

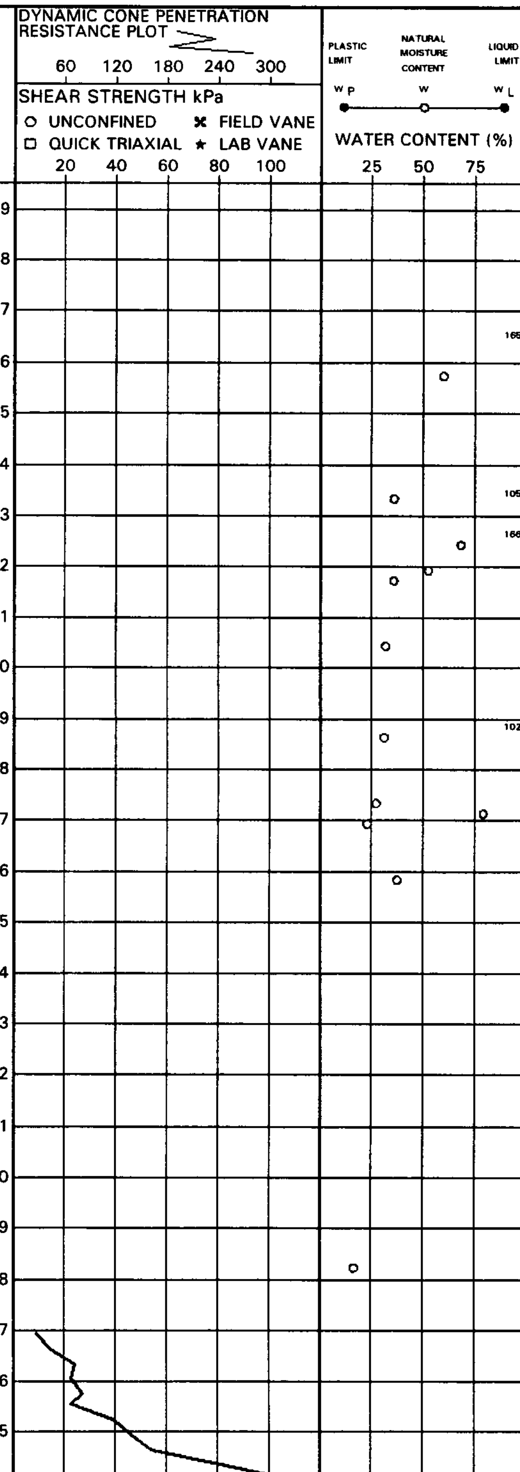
✖3, ★3: Numbers refer to Sensitivity   15-20-5-10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

METRIC

W.P. 127-88-01	LOCATION CO-ORDINATES 5288006 N 355029 E		ORIGINATED BY G.M.
DIST 53	HWY 65	BOREHOLE TYPE HOLLOW STEM AUGER	COMPILED BY C.S.
DATUM Geodetic	DATE February 13th, 1996		CHECKED BY W.H.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
279.5	ICE SURFACE				
0.0 / 278.7	ICE				
0.8 / 277.7	WATER				
1.8	SAND/SILT/ORGANICS/WOOD - grey	1	SS	1	
	----- - brown/black	2	SS	1	
	----- - SAND & PEAT - with pieces of wood, very loose	3	SS	1	
		4	SS	1	
		5	SS	1	
		6	SS	2	
271.8		7	SS	2	
7.7	SAND - trace to some silt, occ. organics seam or pocket, grey, very loose to compact	8	SS	1	
		9	SS	4	0 88 12
	- 150mm organics/wood	10	SS	3	N' = 12
	- 150mm silt layer - 150mm organics/wood	11	SS	7	N' = 6
	----- - occ. silt seam	12	SS	5	0 47 53 N' = 9
257.2		13	SS	35	0 90 11 N' = 49
22.3 / 253.9	End of Borehole at 22.3m DYNAMIC CONE ONLY NO SAMPLES TAKEN				
25.6	End of Dynamic Cone at 25.6m Note: N' represents the number of blows per 0.3m for the last counts for a split spoon driven past 0.45m				



GROUND WATER CONDITIONS | UNIT WEIGHT γ kN/m³

× ³, ★ ³: Numbers refer to Sensitivity   15 ⊕ 5 (20, 10) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

METRIC

W.P. 127-88-01	LOCATION CO-ORDINATES 5288016 N 355052 E		ORIGINATED BY G.M.
DIST 53 HWY 65	BOREHOLE TYPE HOLLOW STEM AUGER		COMPILED BY C.S.
DATUM Geodetic	DATE February 13th, 1996		CHECKED BY W.H.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
279.4	ICE SURFACE				
278.8 / 0.0	ICE				
0.6	WATER				
272.7 / 6.7	SAND & GRAVEL - trace organics, grey	1	WS		
270.4 / 9.0	SAND - trace to some gravel, grey, very loose to loose	2	SS	1	6 88 6
	-----	3	SS	2	
	- trace to some silt, trace organics, layered	4	SS	1	
	- 100mm organics & wood	5	SS	4	
	- 300mm silt & organics	6	SS	4	
		7	SS	6	N' = 9
	-----				N' = 8
	- some silt to silty, occ. organic layer (<5mm), wood fragment, grey, loose to compact	8	SS	3	
		9	SS	8	N' = 14
261.1	SILT - grey, dense				0 4 96
260.6 / 18.8	SAND - trace silt, dense	10	SS	38	N' = 17
		11	SS	19	N' = 41
257.9		12	WS		
21.5	End of Borehole at 21.5m Note: N' represents the number of blows per 0.3m for the last counts for a split spoon driven past 0.45m.				

Chart headings: GROUND WATER CONDITIONS; ELEVATION SCALE (279–258); DYNAMIC CONE PENETRATION RESISTANCE PLOT 60 120 180 240 300; SHEAR STRENGTH kPa: ○ UNCONFINED ✖ FIELD VANE □ QUICK TRIAXIAL ★ LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m^3

✖^{3}, ★^{3}: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 5

METRIC

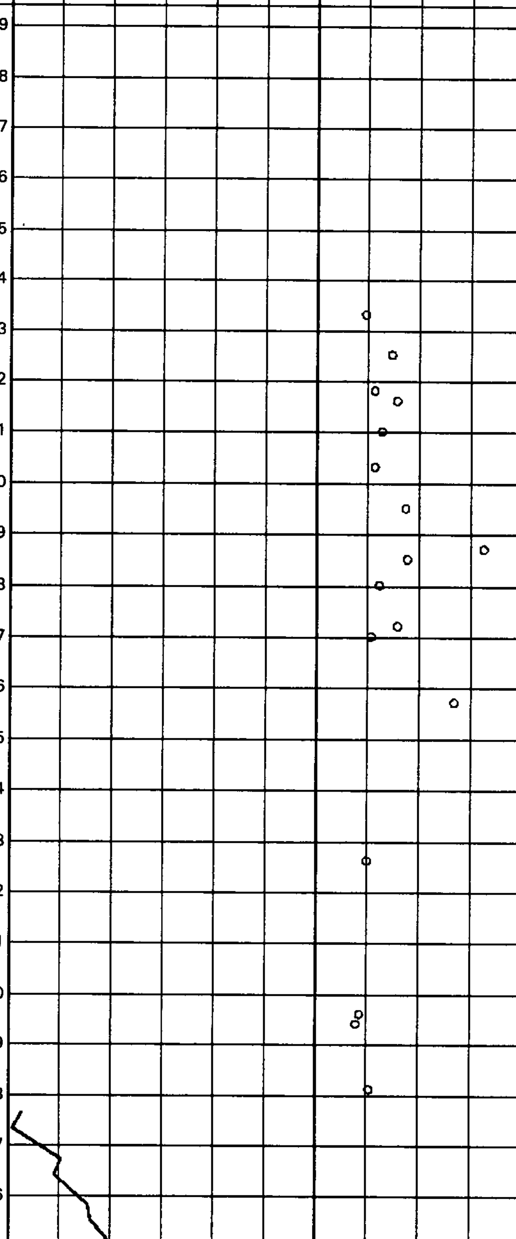
W.P. 127-88-01 | LOCATION CO-ORDINATES 5288038 N 355050 E | ORIGINATED BY G.M.

DIST 53 HWY 65 | BOREHOLE TYPE HOLLOW STEM AUGER | COMPILED BY C.S.

DATUM Geodetic | DATE February 10th, 1996 | CHECKED BY W.H.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
279.4	**ICE SURFACE**					
278.8 / 0.0	ICE				279	
0.6	WATER				278, 277, 276, 275	
273.9 / 5.5	SAND - trace to some silt, trace organics & wood, occ. peat & silt seam or pocket, grey, very loose to compact	1	SS	2	274, 273	0 97 3
		2	SS	1	272	
		3	SS	1		
		4	SS	1	271	
		5	SS	1	270	
		6	SS	2	269	
		7	SS	1		
		8	SS	4	268	N' = 5
		9	SS	3	267	N' = 4
	- 100mm silt layer				266	N' = 11
	- - - - -	10	SS	4	265	
	- occ. silt seam, compact to dense	11	WS		264	
		12	SS	31	263, 262	0 54 46
		13	WS		261	
	- 100mm silt layer	14	SS	28	260, 259	
257.9 / 21.5	End of Borehole at 21.5m Casing sanded in 3.0m DYNAMIC CONE ONLY NO SAMPLES TAKEN	15	WS		258, 257, 256	
255.0 / 24.4	End of Dynamic Cone at 24.5m Note: N' represents the number of blows per 0.3m for the last counts for a split spoon driven past 0.45m.					

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 60 120 180 240 300; SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE □ QUICK TRIAXIAL ★ LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³.



✖³, ★³: Numbers refer to Sensitivity   15 ⊕ 5 (20 / 10) (%) STRAIN AT FAILURE

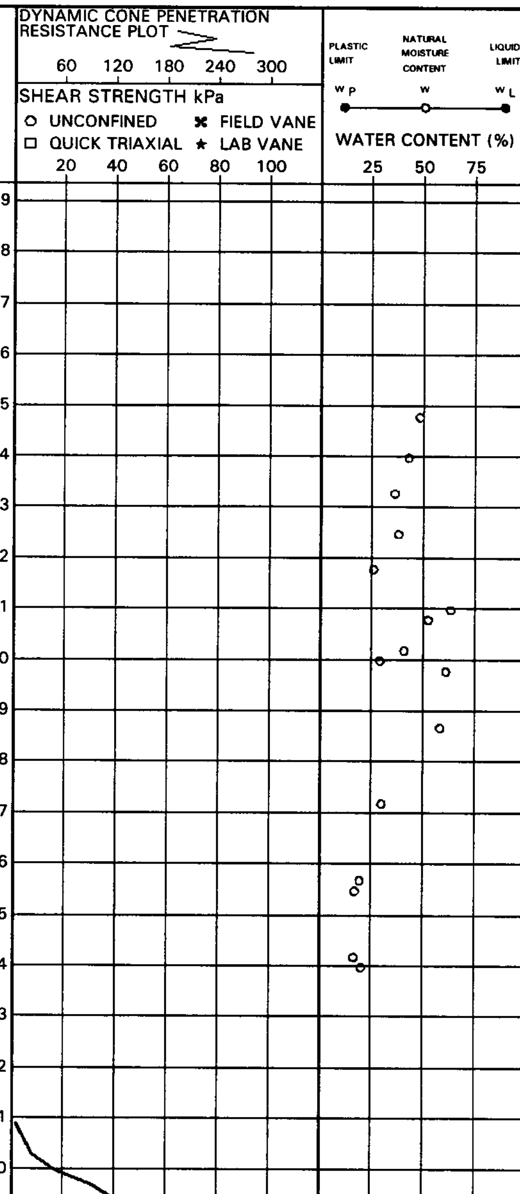
RECORD OF BOREHOLE No 6

METRIC

W.P. 127-88-01	LOCATION	CO-ORDINATES 5288046 N 355083 E	ORIGINATED BY G.M.
DIST 53 HWY 65	BOREHOLE TYPE	HOLLOW STEM AUGER	COMPILED BY C.S.
DATUM Geodetic	DATE	February 9th, 1996	CHECKED BY W.H.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
279.4	ICE SURFACE				
278.8 / 0.0	ICE				
0.6	WATER				
274.8 / 4.6	SAND - trace to silty, trace organics & rootlets, occ. organic seam <3mm, grey/brown, loose	1	SS	1	0 75 25
		2	SS	1	
		3	SS	1	
		4	SS	2	
		5	SS	2	
		6	SS	3	N' = 4
		7	SS	2	N' = 3
		8	SS	3	N' = 5
268.8 / 10.6	SILT - some sand, trace to some organics, occ. sand seam, layered, grey, loose to compact	9	SS	4	
		10	SS	5	N' = 20
265.9 / 13.5	SAND - occ. silt seams, grey, compact to dense	11	SS	21	
		12	SS	44	0 92 8
		13	WS		
260.9 / 18.5	End of Borehole at 18.5m DYNAMIC CONE ONLY NO SAMPLES TAKEN				
259.3 / 20.1	End of Dynamic Cone at 20.1m Casing sanded in 3.0m Note: N' represents the number of blows per 0.3m for the last counts for a split spoon driven past 0.45m.				

STRAT PLOT · GROUND WATER CONDITIONS · ELEVATION SCALE · UNIT WEIGHT γ kN/m^3



✖^{3}, ★^{3}: Numbers refer to Sensitivity   20 / 15 -⊕- 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 7

METRIC

W.P. 127-88-01 LOCATION CO-ORDINATES 5288067 N 355089 E ORIGINATED BY G.M.

DIST 53 HWY 65 BOREHOLE TYPE HOLLOW STEM AUGER COMPILED BY C.S.

DATUM Geodetic DATE January 13th, 1996 CHECKED BY W.H.

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
282.8	GROUND SURFACE
0.0	25mm TOPSOIL FILL - sand & gravel - organics, brown - sand, silty, trace gravel & organics, grey, very loose to loose
280.2 2.6	SAND - trace to some silt, trace organics & rootlets, grey, loose to very loose
272.8 10.0	SILT - trace to sandy, organics, grey/brown, very loose to compact - - - - - - trace clay, layered, grey, loose
266.6 16.2	SAND - trace silt to silty, grey, loose to compact - - - - - - occ. silt seams
260.4 22.4	SILT - trace to some sand, grey, compact - - - - - - occ. clay varves 25 to 50mm thick
252.8 30.0	Continued Next Page

SAMPLES

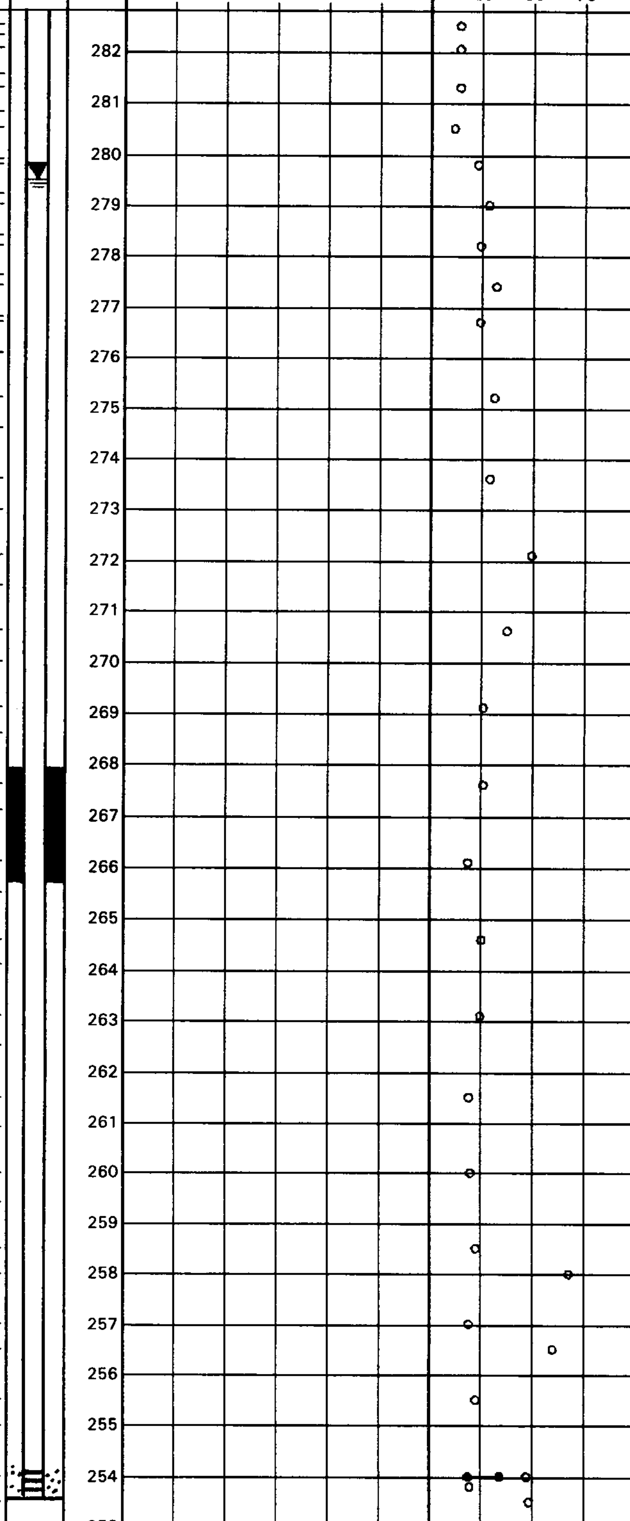
NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AS		
2	SS	8	
3	SS	4	
4	SS	8	
5	SS	7	N' = 9
6	SS	2	
7	SS	2	
8	SS	1	
9	SS	1	
10	SS	1	
11	SS	2	
12	SS	3	0 33 67 N' = 5
13	SS	2	
14	SS	14	
15	SS	6	
16	SS	8	
17	SS	21	
18	SS	11	N' = 15
19	SS	13	N' = 12
20	SS	16	N' = 20
21	SS	21	
22	SS	15	
23	SS	16	
24	SS	25	

GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³

DYNAMIC CONE PENETRATION RESISTANCE PLOT 60 120 180 240 300

SHEAR STRENGTH kPa: ○ UNCONFINED ✖ FIELD VANE □ QUICK TRIAXIAL ★ LAB VANE 20 40 60 80 100

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



✖³, ★³: Numbers refer to Sensitivity

15 –○– 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 7

METRIC

W.P. 127-88-01 LOCATION CO-ORDINATES 5288067 N 355089 E ORIGINATED BY G.M.

DIST 53 HWY 65 BOREHOLE TYPE HOLLOW STEM AUGER COMPILED BY C.S.

DATUM Geodetic DATE January 13th, 1996 CHECKED BY W.H.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 60 120 180 240 300
SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, □ QUICK TRIAXIAL, ★ LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%): 25 50 75
UNIT WEIGHT γ kN/m^3
REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
252.8	CONTINUED					
30.0	SILT - trace to some sand, occ. clay varves 10 -25mm thick, grey, compact	25	SS	17	252	
		26	SS	18	251, 250	
		27	SS	20	249	
		28	TW	PH	248, 247	0 1 83 16
	- clay layer 10mm in thickness	29	SS	18	246	
		30	SS	23	245, 244	
	- 100mm clay layer	31	SS	18	243	
		32	SS	24	242, 241	
		33	SS	28	240, 239	
238.4		34	SS	19		
44.4	SAND - grey, compact				238	
236.5		35	SS	13	237	N' = 15
46.3	End of Borehole at 46.3m Casagrande Piezometer installed to 29.1m with 2m Bentonite Seal @ 14.9m Note: N' represents the number of blows per 0.3m for the last counts for a split spoon driven past 0.45m.					

✖^{3}, ★^{3}: Numbers refer to Sensitivity

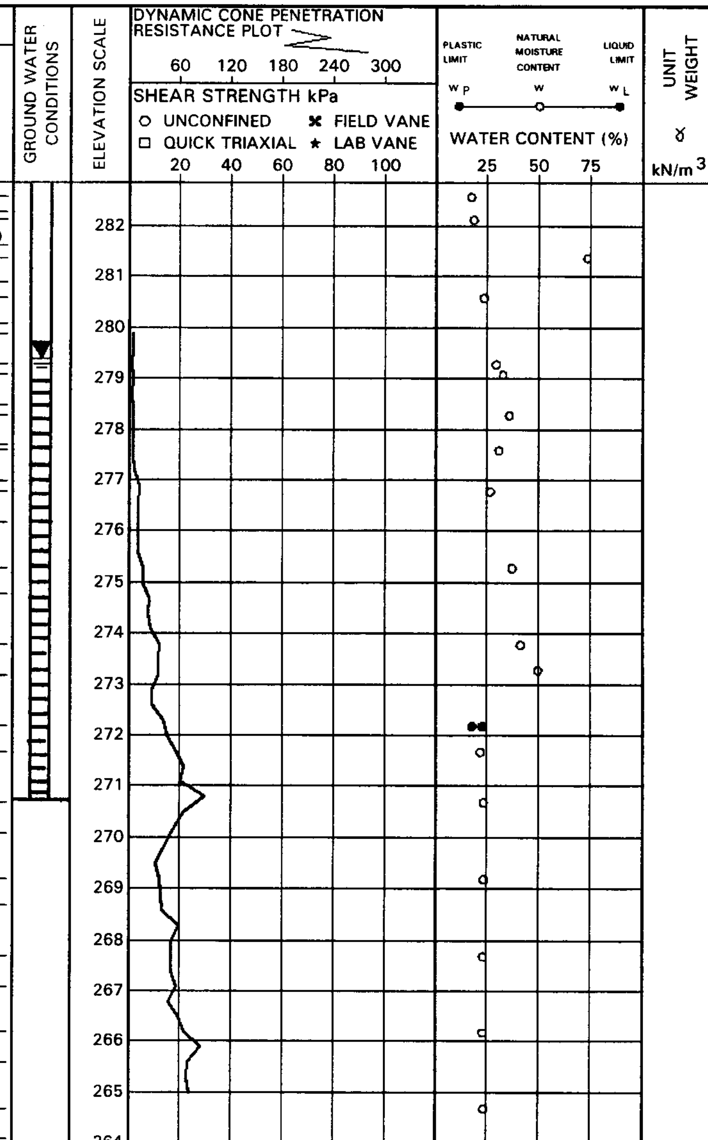
15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 8

METRIC

W.P. 127-88-01
LOCATION CO-ORDINATES 5288070 N 355103 E
ORIGINATED BY G.M.
DIST 53 HWY 65
BOREHOLE TYPE HOLLOW STEM AUGER
COMPILED BY C.S.
DATUM Geodetic
DATE January 12th, 1996
CHECKED BY W.H.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
282.9	GROUND SURFACE				
0.0	50mm TOPSOIL	1	AS		
	FILL - sand - silty, some gravel, trace organics, trace organics, brown	2	SS	10	
280.8		3	SS	21	
2.1	SAND - some silt to silty, grey, loose to very loose	4	SS	7	
		8	SS	2	
	- trace rootlets & organics	6	SS	1	0 80 20
	- trace organics	7	SS	2	
	- sand layer	8	SS	1	0 68 32
	- - - - -				
	- layered, grey/brown	9	SS	1	
		10	SS	3	
274.3 / 8.6	SILT - sandy, trace clay & rootlets, layered, grey, loose to compact	11	SS	3	0 33 59 8 N' = 4
		12	SS	13	
271.2 / 11.7	SAND - some silt to silty, occ. 50mm layers of grey silt & clay, trace clay, grey, loose to compact	13	SS	4	
		14	SS	13	
		15	SS	8	0 76 24 N' = 9
		16	SS	8	N' = 6
264.0		17	SS	8	0 87 13 N' = 14
18.9	End of Borehole at 18.9m Standpipe installed to 12.2m Note: N' represents the number of blows per 0.3m for the last counts for a split spoon driven past 0.45m.				



✖ 3, ★ 3: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 9

METRIC

W.P. 127-88-01 LOCATION CO-ORDINATES 5288068N 355092E ORIGINATED BY H.F.

DIST 53 HWY 65 BOREHOLE TYPE HOLLOW STEM AUGER - B-CASING COMPILED BY C.S.

DATUM Geodetic DATE January 11th, 1996 CHECKED BY W.H.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (60, 120, 180, 240, 300); SHEAR STRENGTH kPa (○ UNCONFINED, ✕ FIELD VANE, □ QUICK TRIAXIAL, ★ LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
282.8	**GROUND SURFACE**										
0.0	Set H casing to 9.1m. Advanced B-casing with drilling mud to 48.5m						282				
							281				
							280				
							279				
							278				
							277				
							276				
							275				
							274				
							273				
							272				
							271				
							270				
							269				
							268				
							267				
							266				
							265				
							264				
							263				
							262				
							261				
							260				
							259				
							258				
							257				
							256				
							255				
							254				
252.8 / 30.0							253				

Continued Next Page

✕[3], ★[3]: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 9

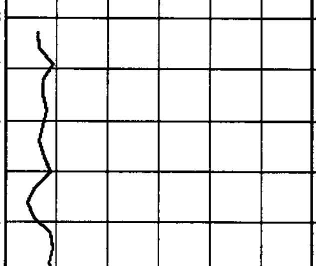
METRIC

W.P. 127-88-01	LOCATION CO-ORDINATES 5288068N 355092E	ORIGINATED BY H.F.
DIST 53 HWY 65	BOREHOLE TYPE HOLLOW STEM AUGER - B-CASING	COMPILED BY C.S.
DATUM Geodetic	DATE January 11th, 1996	CHECKED BY W.H.

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (60, 120, 180, 240, 300) | SHEAR STRENGTH kPa: ○ UNCONFINED, ✖ FIELD VANE, □ QUICK TRIAXIAL, ★ LAB VANE (20, 40, 60, 80, 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (25, 50, 75) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS (GR SA SI CL)
252.8	**CONTINUED**				
	NO SAMPLES TAKEN				
234.3 / 48.5	SAND - grey	1	RC		
232.5 / 50.3	SAND & GRAVEL - silty, occ. cobbles, compact to dense	2	SS	25	N' = 44
		3	SS	40	N' = 22
	- cobble layer 0.3m thick				
	- - - - -	4	SS	38	32 38 30
	- inferred				
222.8 / 60.0	Continued Next Page				

Elevation scale: 252, 251, 250, 249, 248, 247, 246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224, 223



✖ 3, ★ 3 : Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

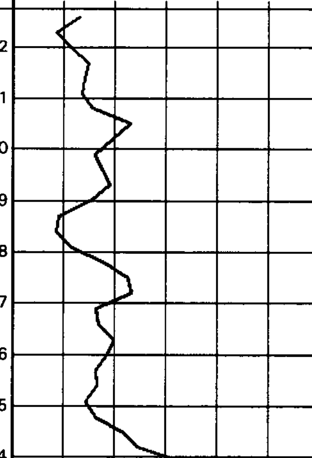
Ministry of Transportation Ontario

RECORD OF BOREHOLE No 9

METRIC

W.P. 127-88-01	LOCATION	CO-ORDINATES 5288068N 355092E	ORIGINATED BY H.F.
DIST 53 HWY 65	BOREHOLE TYPE	HOLLOW STEM AUGER - B-CASING	COMPILED BY C.S.
DATUM Geodetic	DATE	January 11th, 1996	CHECKED BY W.H.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 60 120 180 240 300 / SHEAR STRENGTH kPa ○ UNCONFINED ✖ FIELD VANE □ QUICK TRIAXIAL ★ LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
222.8	**CONTINUED**										
60.0	SAND & GRAVEL - dense to very dense						222 221 220 219 218 217 216 215 214				
213.9											
68.9	End of Borehole at 68.9m Note: N' represents the number of blows per 0.3m for the last counts for a split spoon driven past 0.45m.										



✖³, ★³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 10

METRIC

W.P. 127-88-01	LOCATION CO-ORDINATES 5287969N 354990E		ORIGINATED BY H.F.
DIST 53 HWY 65	BOREHOLE TYPE HOLLOW STEM AUGER - B-CASING		COMPILED BY C.S.
DATUM Geodetic	DATE January 13th, 1996		CHECKED BY W.H.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 60 120 180 240 300 / SHEAR STRENGTH kPa (O UNCONFINED, ✖ FIELD VANE, □ QUICK TRIAXIAL, ★ LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L / WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
283.9	**GROUND SURFACE**										
0.0	FILL - sand & gravel - inferred from cuttings						283 282 281 280				
279.6 / 4.3	ORGANICS - inferred from cuttings						279 278 277 276				
275.9 / 8.0	SILT & SAND - inferred from cuttings						275 274 273 272 271 270 269 268 267 266 265 264 263 262 261 260 259 258 257 256 255 254				
253.9 / 30.0											

Continued Next Page

✖³, ★³: Numbers refer to Sensitivity

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 10

METRIC

W.P. 127-88-01 LOCATION CO-ORDINATES 5287969N 354990E ORIGINATED BY H.F.

DIST 53 HWY 65 BOREHOLE TYPE HOLLOW STEM AUGER - B-CASING COMPILED BY C.S.

DATUM Geodetic DATE January 13th, 1996 CHECKED BY W.H.

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 60 120 180 240 300 / SHEAR STRENGTH kPa O UNCONFINED ✖ FIELD VANE □ QUICK TRIAXIAL ★ LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
253.9	**CONTINUED**										
30.0	SILT & SAND - inferred from cuttings						253				
							252				
							251				
							250				
							249				
	- occ. clay layer						248				
							247				
							246				
							245				
							244				
							243				
							242				
							241				
							240				
							239				
							238				
							237				
235.1							236				
48.8	SILT - several 25 to 50mm clay layers, grey, very dense		1	SS	77		235				N' = 95
							234				
							233				
			2	SS	87		232				N' = 78
							231				
	- several thin clay seams		3	SS	87		230				N' = 116
							229				
227.2							228				
56.7	SAND - silty, grey, very dense						227				
			4	SS	102		226				N' = 118
224.8							225				
59.1	SAND & GRAVEL - occ. cobbles										
223.9 60.0							224				

Continued Next Page

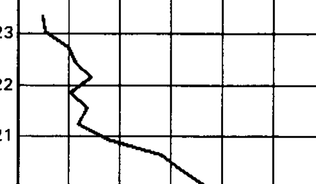
✖^{3}, ★^{3}: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 10

METRIC

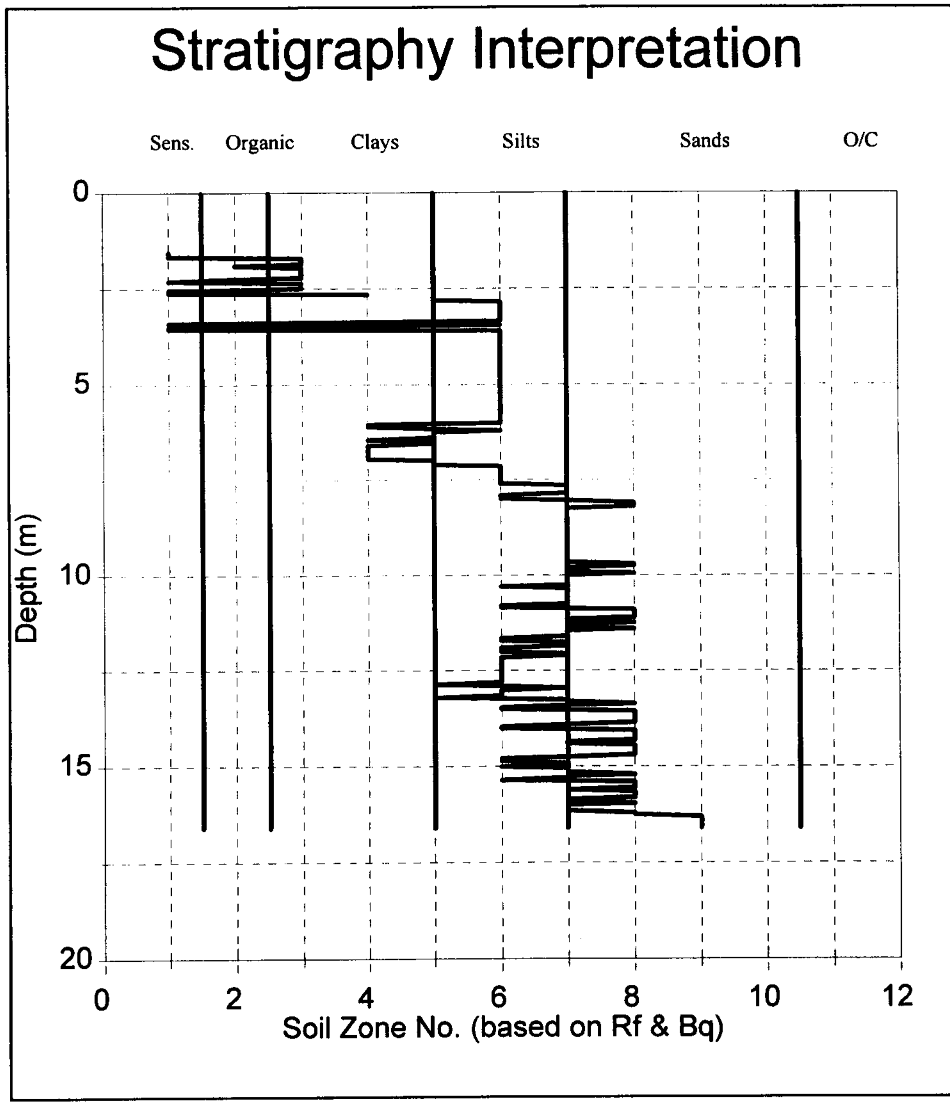
W.P. 127-88-01	LOCATION CO-ORDINATES 5287969N 354990E		ORIGINATED BY H.F.
DIST 53 HWY 65	BOREHOLE TYPE HOLLOW STEM AUGER - B-CASING		COMPILED BY C.S.
DATUM Geodetic	DATE January 13th, 1996		CHECKED BY W.H.

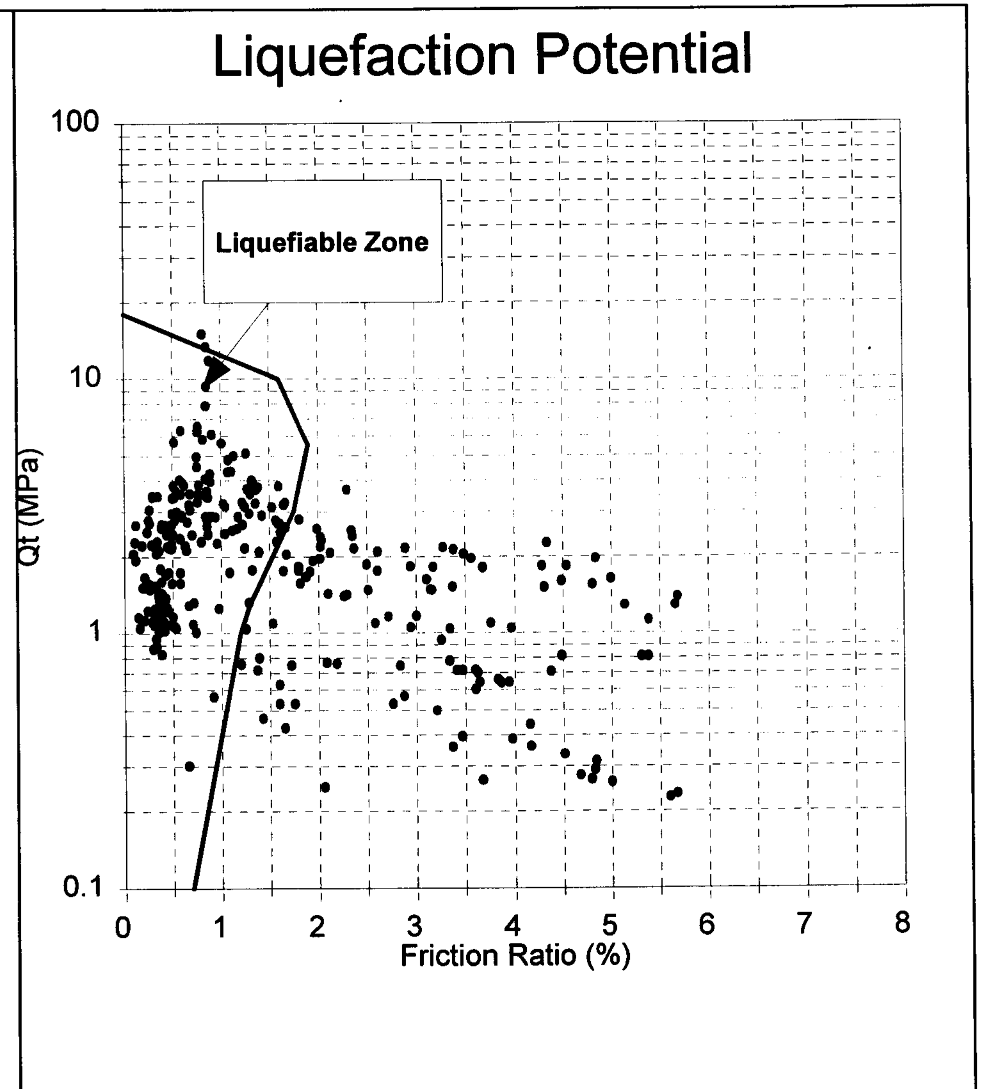
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (60 120 180 240 300); SHEAR STRENGTH kPa (○ UNCONFINED ✖ FIELD VANE □ QUICK TRIAXIAL ★ LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
223.9	**CONTINUED**										
60.0	SAND & GRAVEL - silty, compact to very dense - - - - - - inferred		5	SS	44		223 222 221				
220.0											
63.9	End of Borehole at 63.9m Note: N' represents the number of blows per 0.3m for the last counts for a split spoon driven past 0.45m.										

$✖^3, ★^3$: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

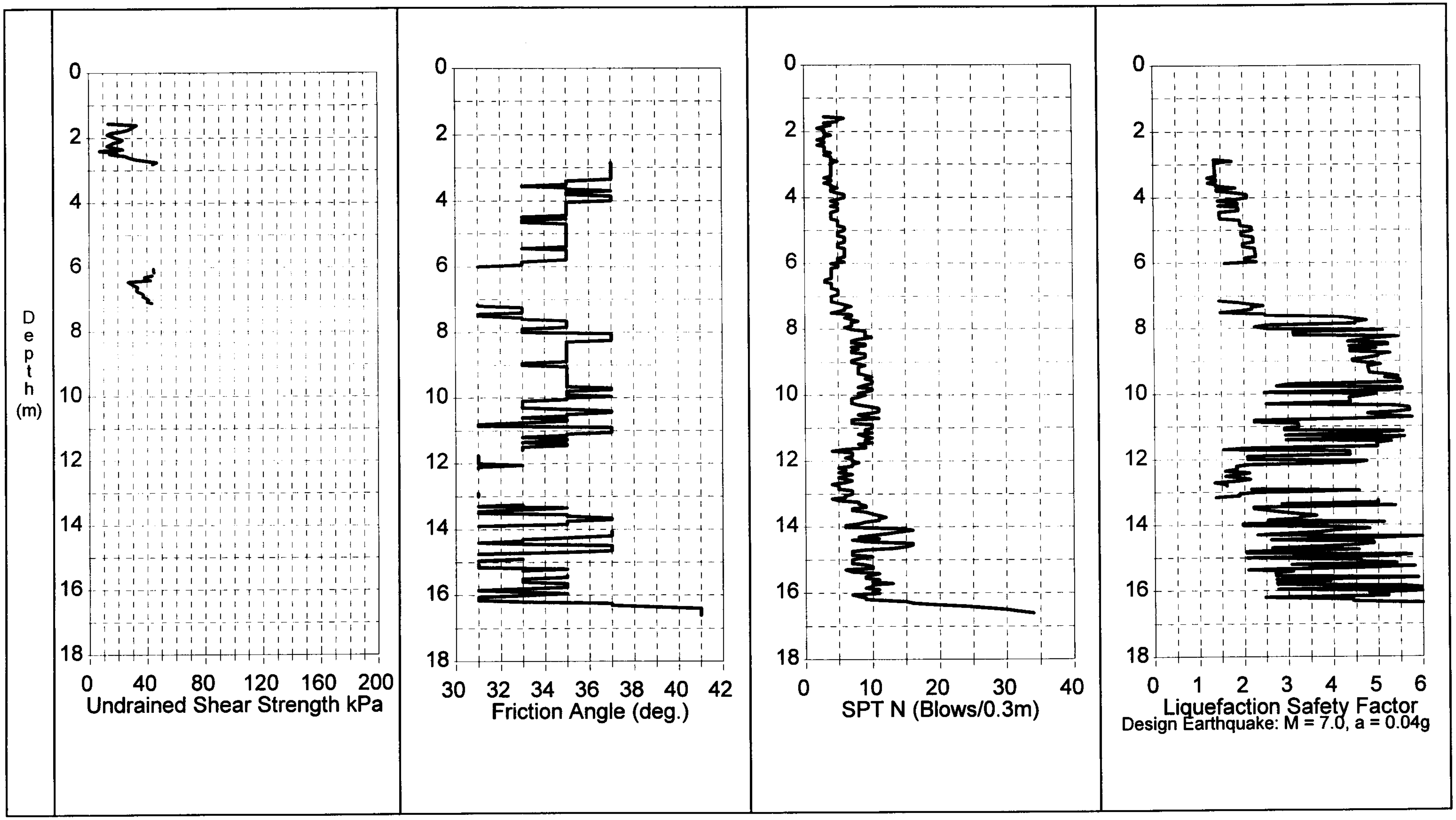
LOG OF CPTU 2A





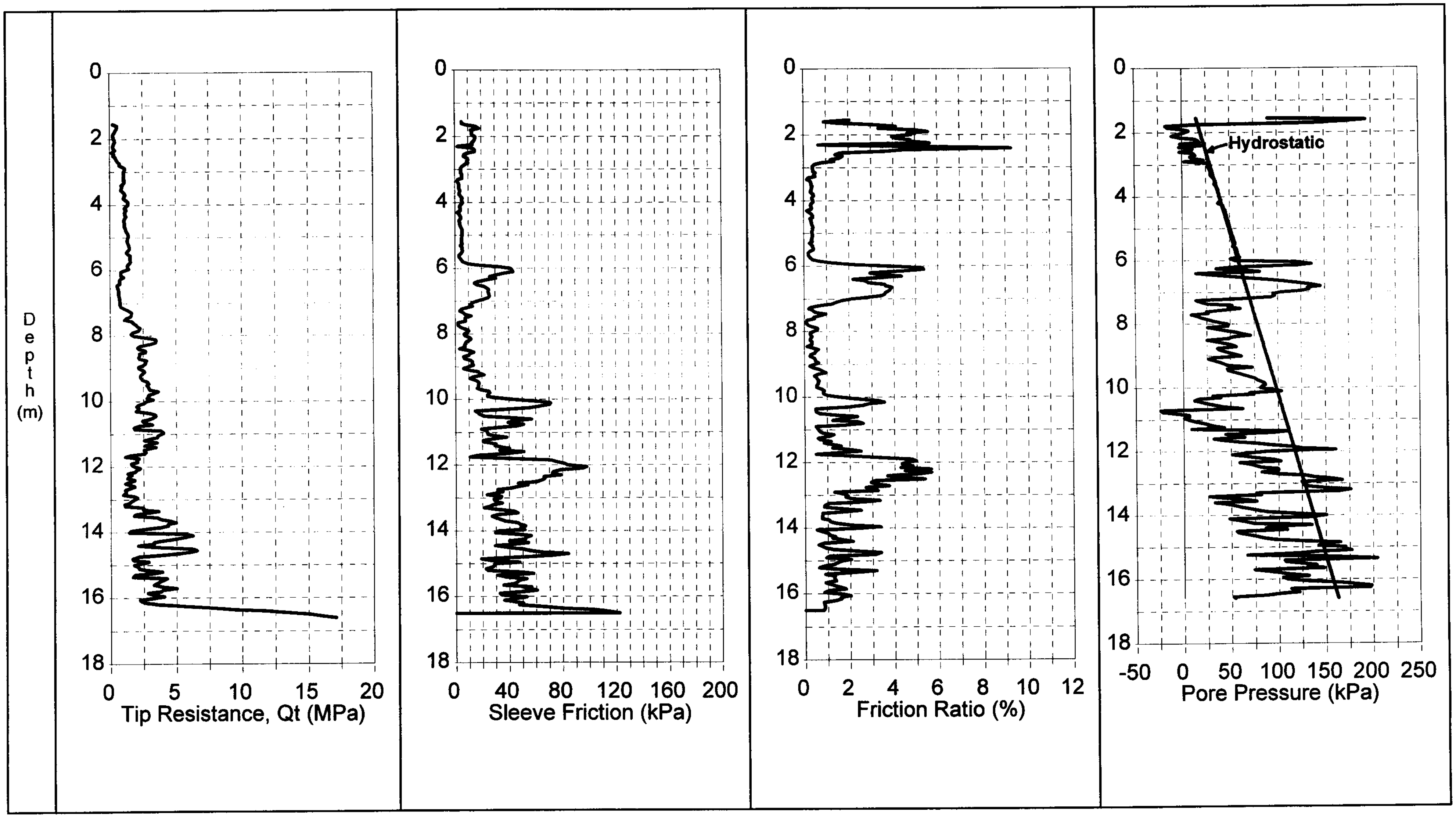
LOG OF CPTU 2A

Interpreted Parameters

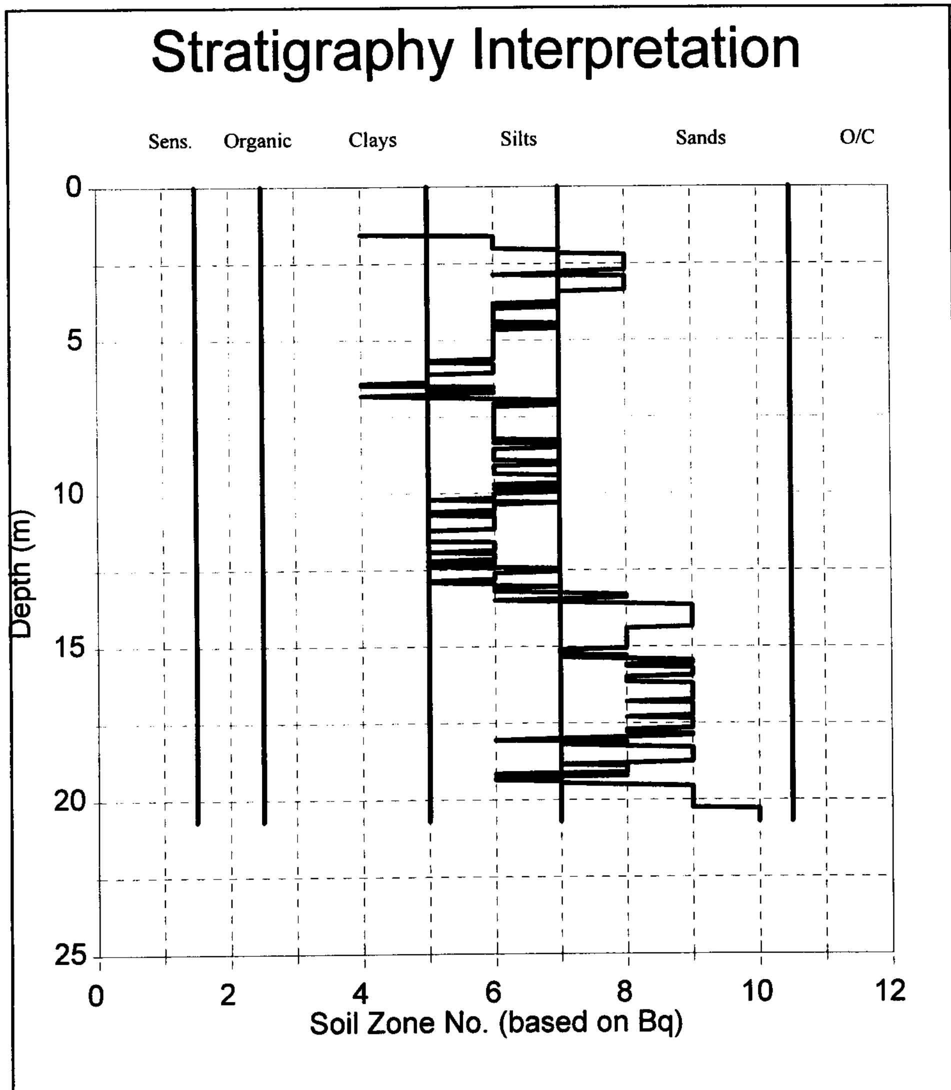


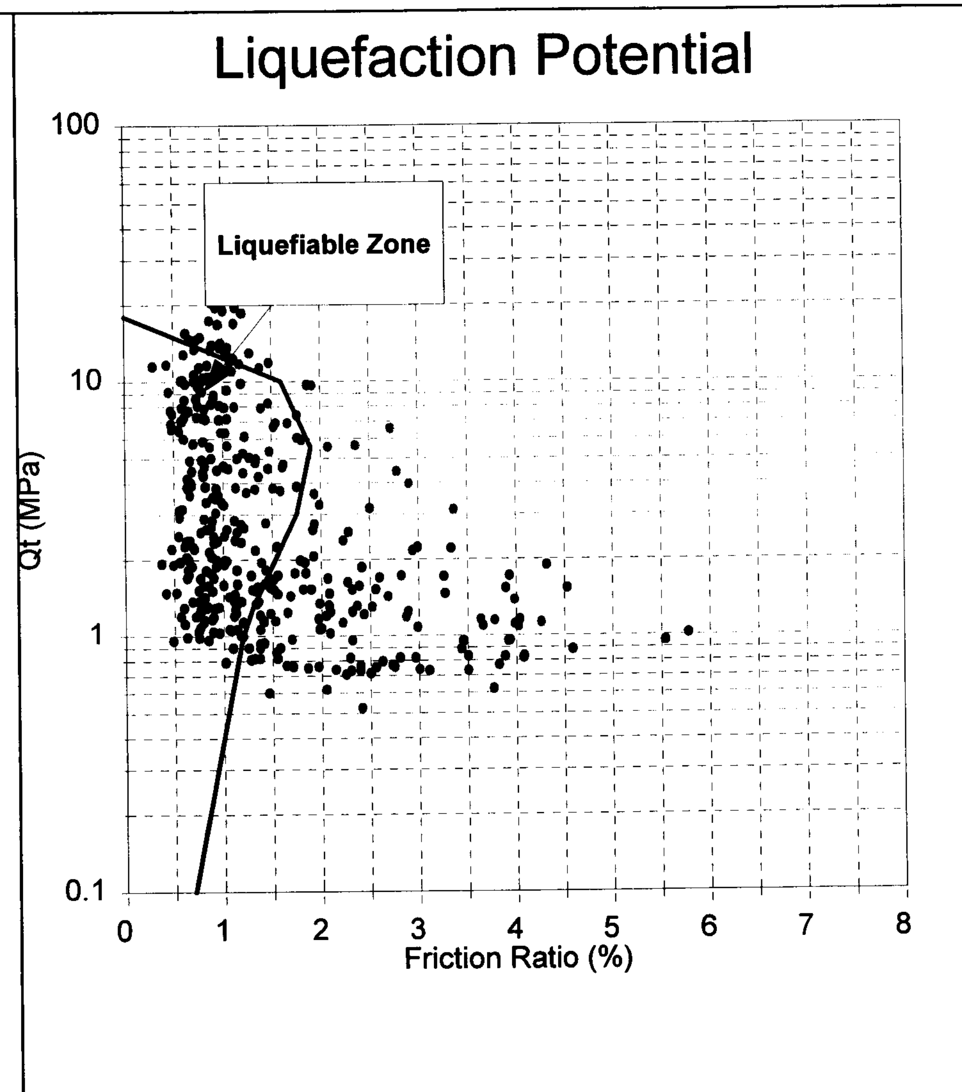
LOG OF CPTU 2A

Piezocone Measurements



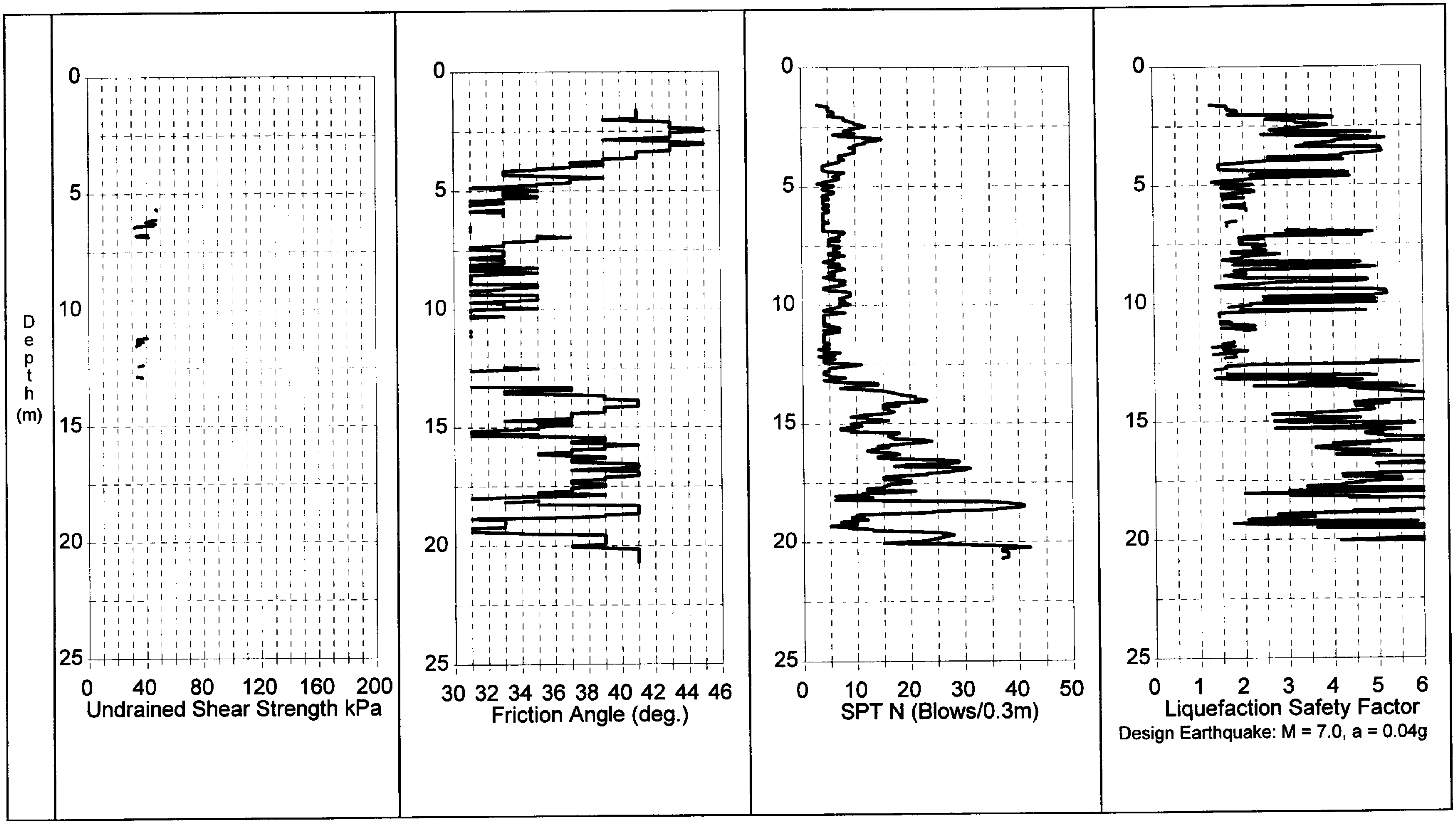
LOG OF CPTU 7A





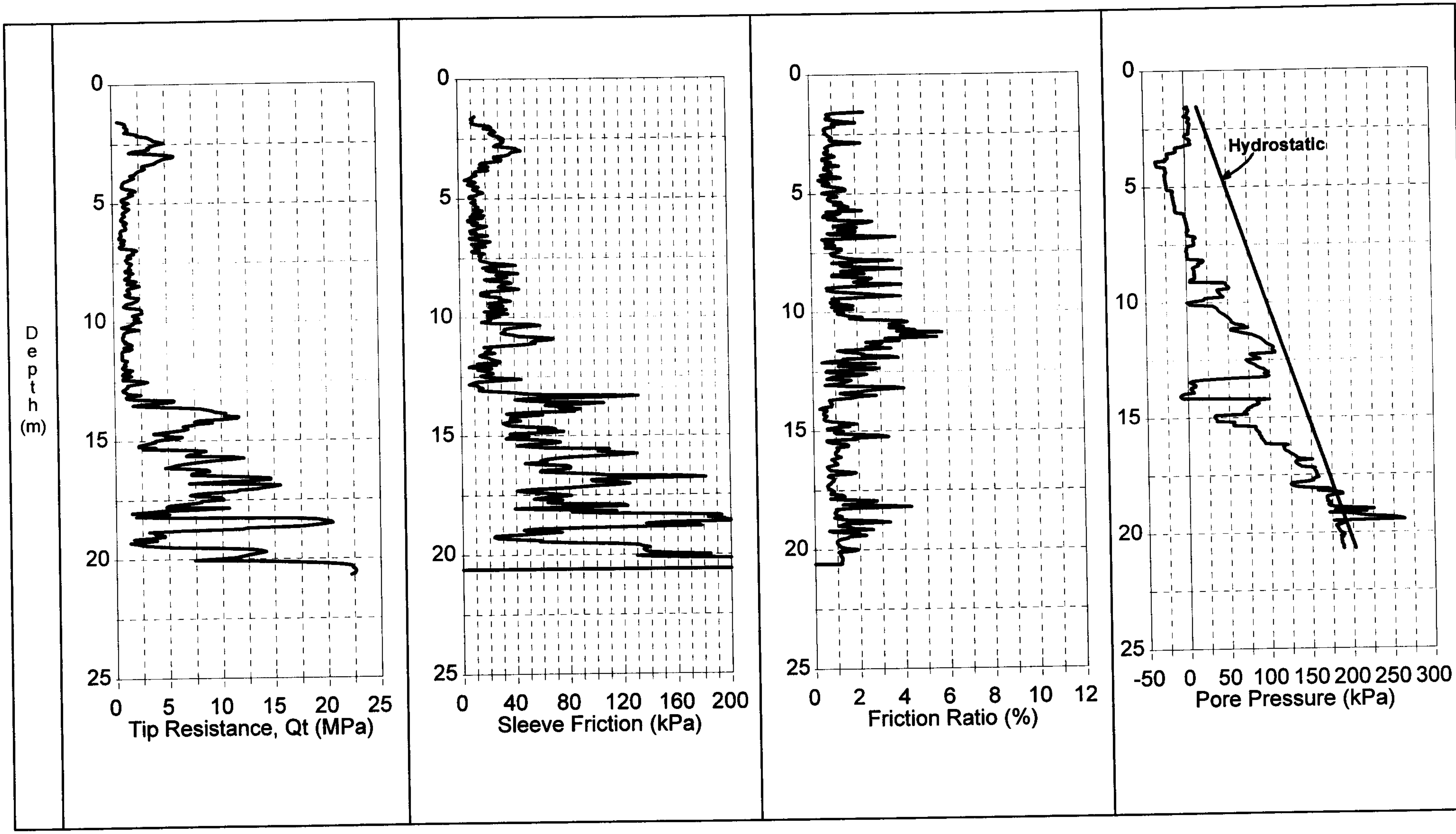
LOG OF CPTU 7A

Interpreted Parameters

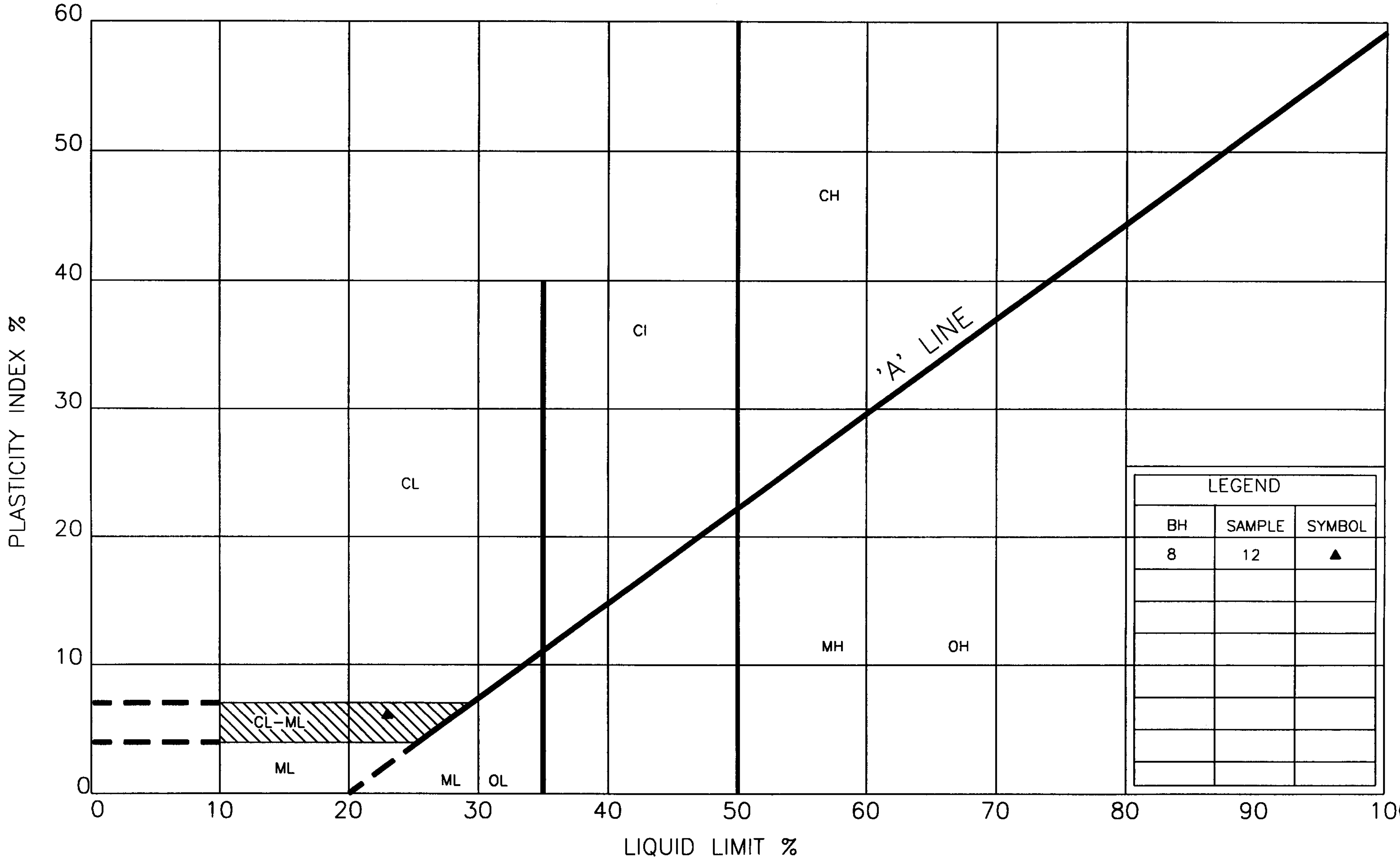


LOG OF CPTU 7A

Piezocone Measurements



ENCLOSURES



LEGEND		
BH	SAMPLE	SYMBOL
8	12	▲

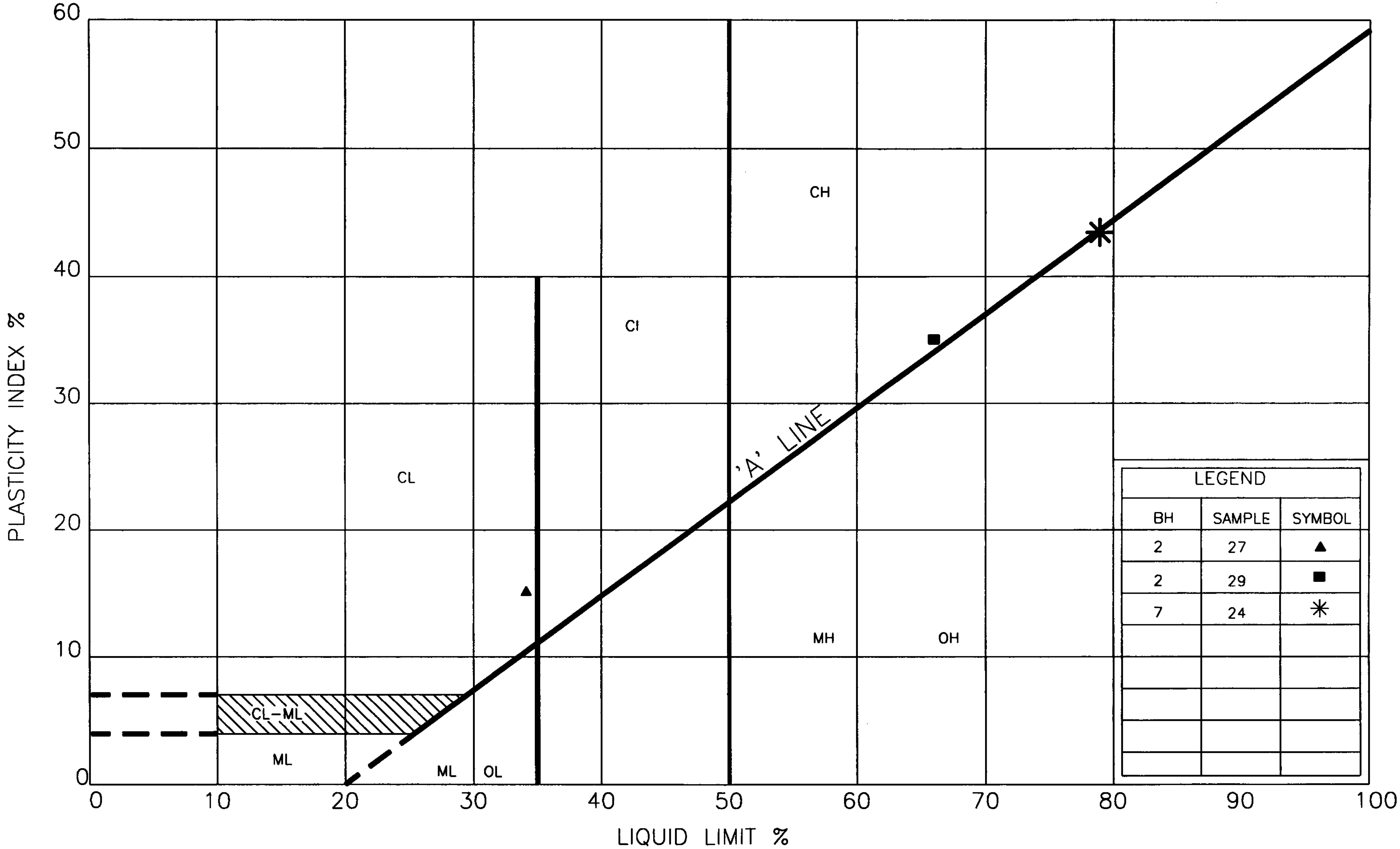
Ministry of Transportation
Ontario

PLASTICITY CHART

SILT SANDY TRACE CLAY

ENCLOSURE NO. 1

WP 127-88-00



Ministry of Transportation
Ontario

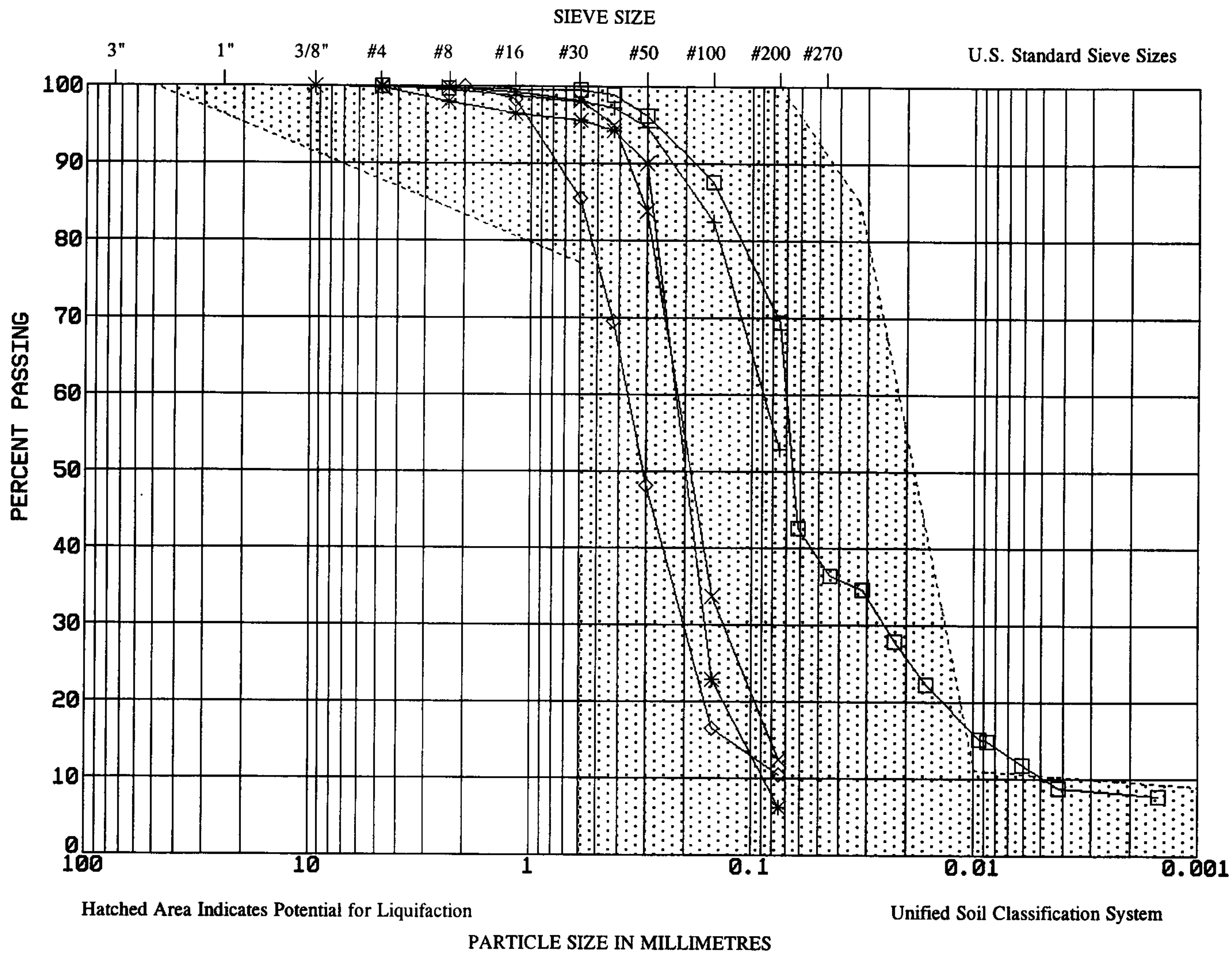
PLASTICITY CHART

CLAY VARVES

ENCLOSURE NO. 1A

WP 127-88-00

GRAINSIZE ANALYSIS



COBBLES	GRAVEL			SAND			SILT & CLAY
	coarse	medium	fine	coarse	medium	fine	

REFERENCE: Hunt, Geotechnical Engineering Techniques & Practices, 1986.

LEGEND:
- □ BOREHOLE 2 DEPTH 4.60
- ✳ BOREHOLE 2 DEPTH 25.70
- × BOREHOLE 3 DEPTH 9.10
- + BOREHOLE 3 DEPTH 13.70
- ◇ BOREHOLE 3 DEPTH 21.30

March 1996

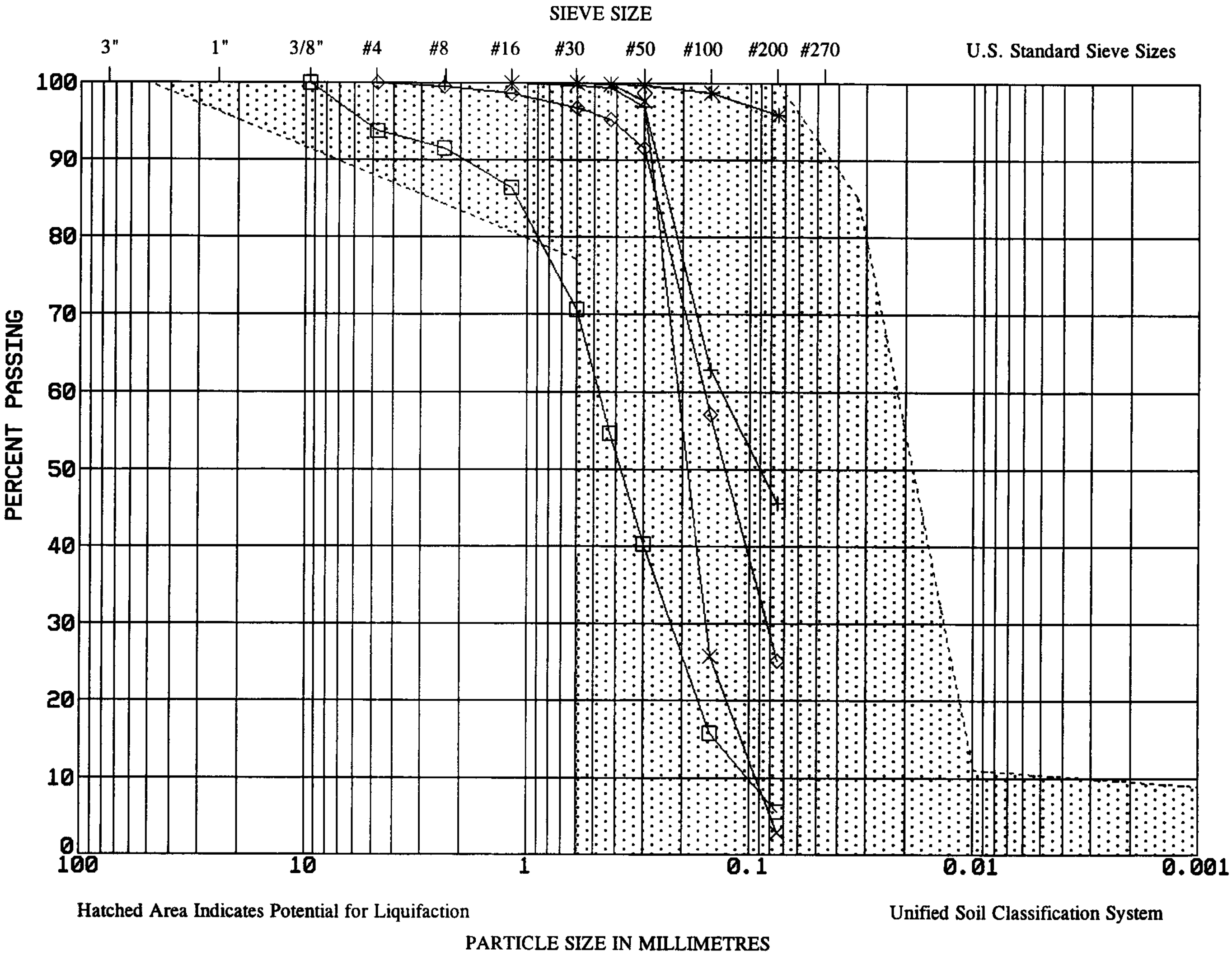
Reference No. 127-88-00

MONTREAL RIVER BRIDGE - ELK LAKE

DST CONSULTING ENGINEERING INC.

ENCLOSURE 2

GRAINSIZE ANALYSIS



COBBLES	GRAVEL			SAND			SILT & CLAY
	coarse	medium	fine	coarse	medium	fine	

REFERENCE: Hunt, Geotechnical Engineering Techniques & Practices, 1986.

LEGEND:
- □ BOREHOLE 4 DEPTH 9.10
- ✳ BOREHOLE 4 DEPTH 18.30
- × BOREHOLE 5 DEPTH 6.10
- + BOREHOLE 5 DEPTH 16.80
- ◇ BOREHOLE 6 DEPTH 4.60

March 1996

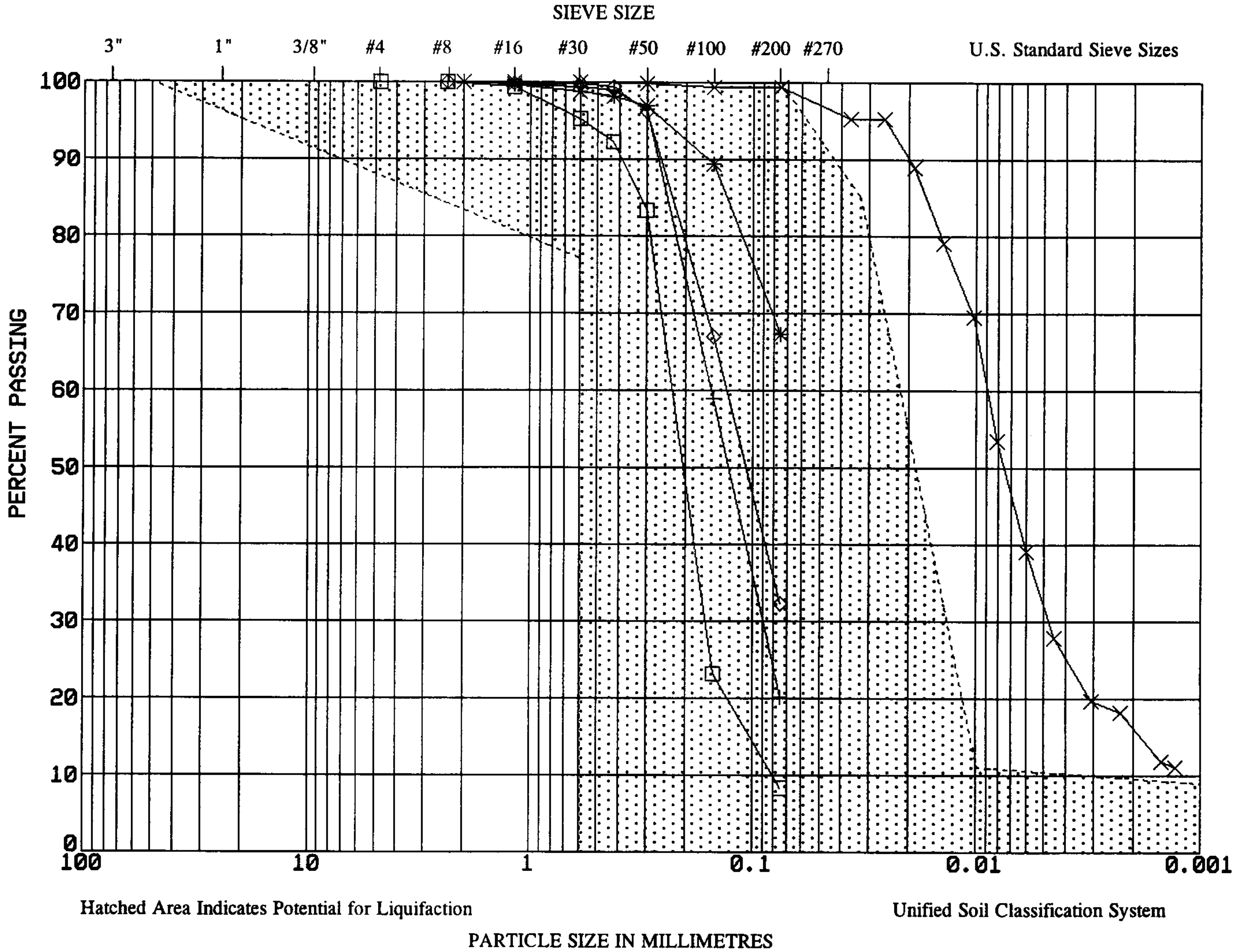
Reference No. 127-88-00

MONTREAL RIVER BRIDGE - ELK LAKE

DST CONSULTING ENGINEERING INC.

ENCLOSURE 3

GRAINSIZE ANALYSIS



COBBLES	GRAVEL			SAND			SILT & CLAY
	coarse	medium	fine	coarse	medium	fine	

REFERENCE: Hunt, Geotechnical Engineering Techniques & Practices, 1986.

LEGEND:
- □ BOREHOLE 6 DEPTH 15.20
- ✳ BOREHOLE 7 DEPTH 10.70
- × BOREHOLE 7 DEPTH 34.80
- + BOREHOLE 8 DEPTH 3.80
- ◇ BOREHOLE 8 DEPTH 5.30

March 1996

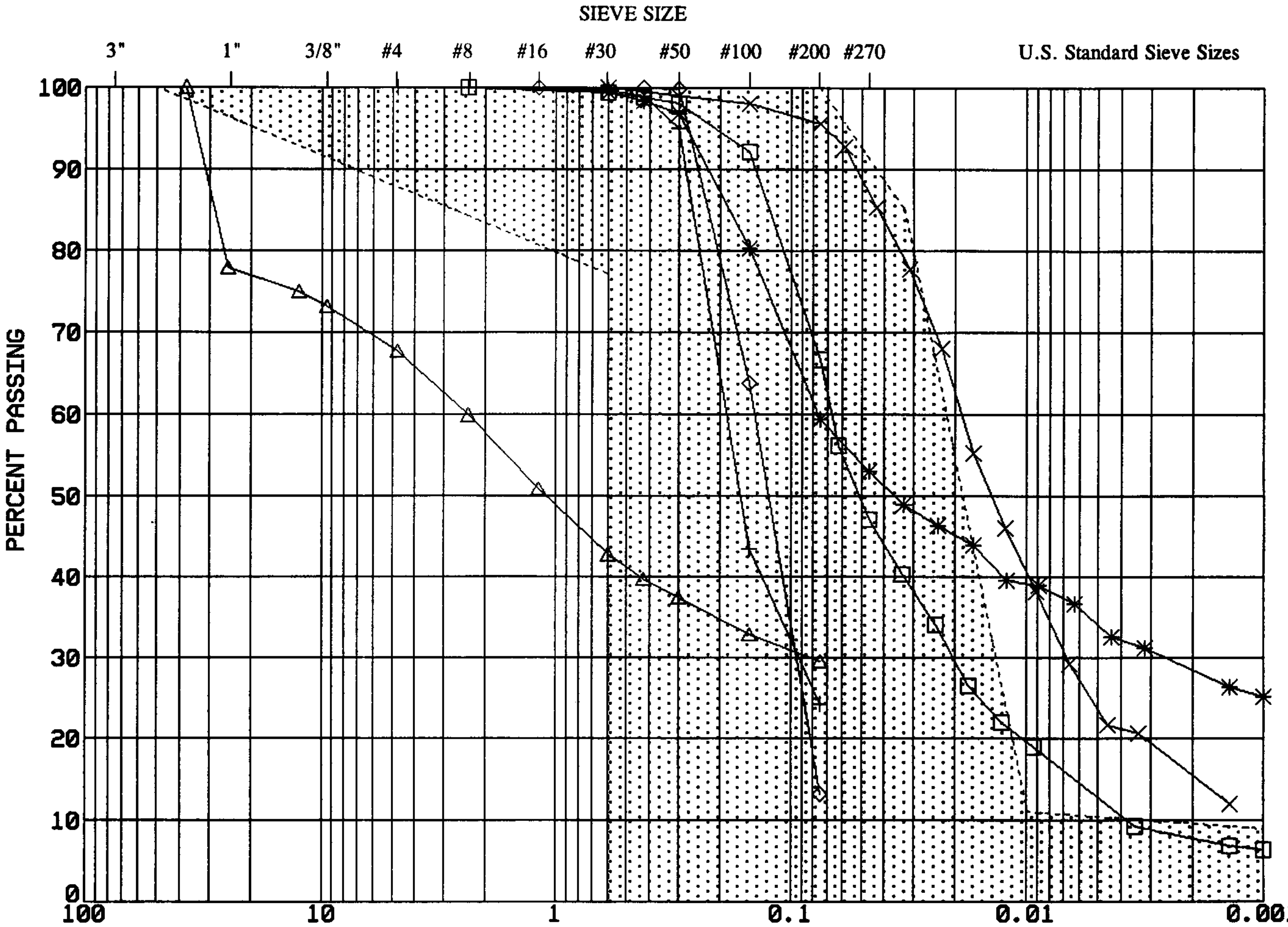
Reference No. 127-88-00

MONTREAL RIVER BRIDGE - ELK LAKE

DST CONSULTING ENGINEERING INC.

ENCLOSURE 4

GRAINSIZE ANALYSIS



Hatched Area Indicates Potential for Liquifaction

Unified Soil Classification System

PARTICLE SIZE IN MILLIMETRES

COBBLES	GRAVEL			SAND			SILT & CLAY
	coarse	medium	fine	coarse	medium	fine	

REFERENCE: Hunt, Geotechnical Engineering Techniques & Practices, 1986.

LEGEND:
- □ BOREHOLE 8 DEPTH 9.10
- ✳ BOREHOLE 8 DEPTH 12.10
- × BOREHOLE 8 DEPTH 12.25
- \+ BOREHOLE 8 DEPTH 15.20
- ◇ BOREHOLE 8 DEPTH 18.20
- △ BOREHOLE 9 DEPTH 54.90

May 1996

Reference No. 127-88-00

MONTREAL RIVER BRIDGE - ELK LAKE

DST CONSULTING ENGINEERING INC.

ENCLOSURE 5

Ministry of Transportation
Reference No.: TG96001

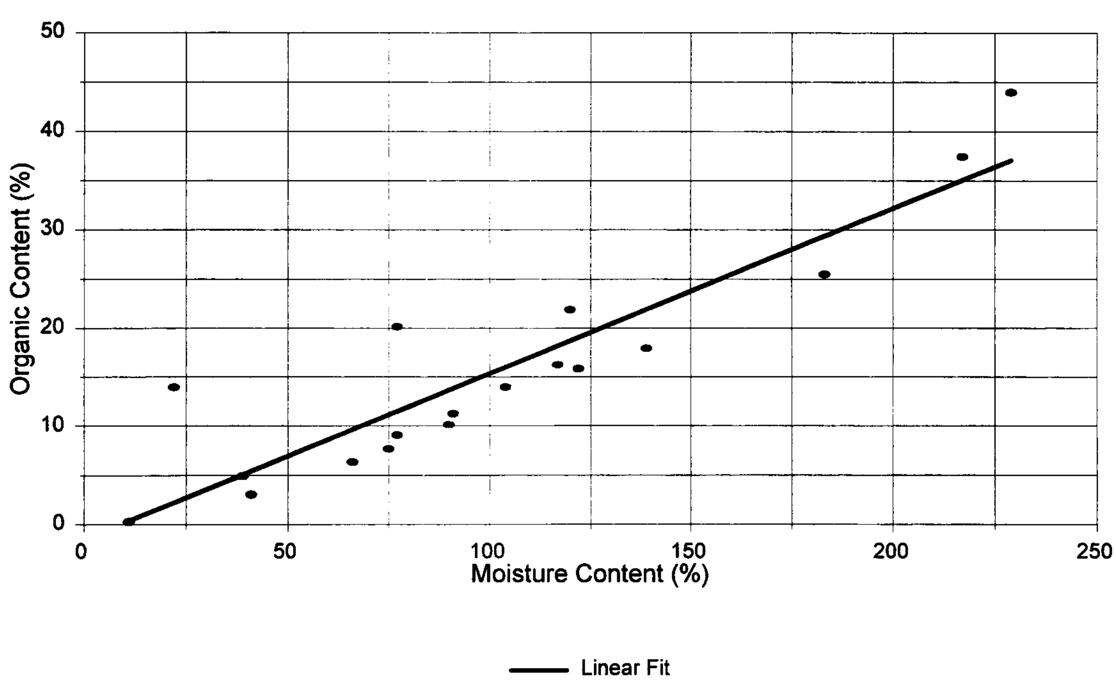
SUMMARY OF ORGANIC CONTENTS

MONTREAL RIVER BRIDGE

ELK LAKE, ONTARIO

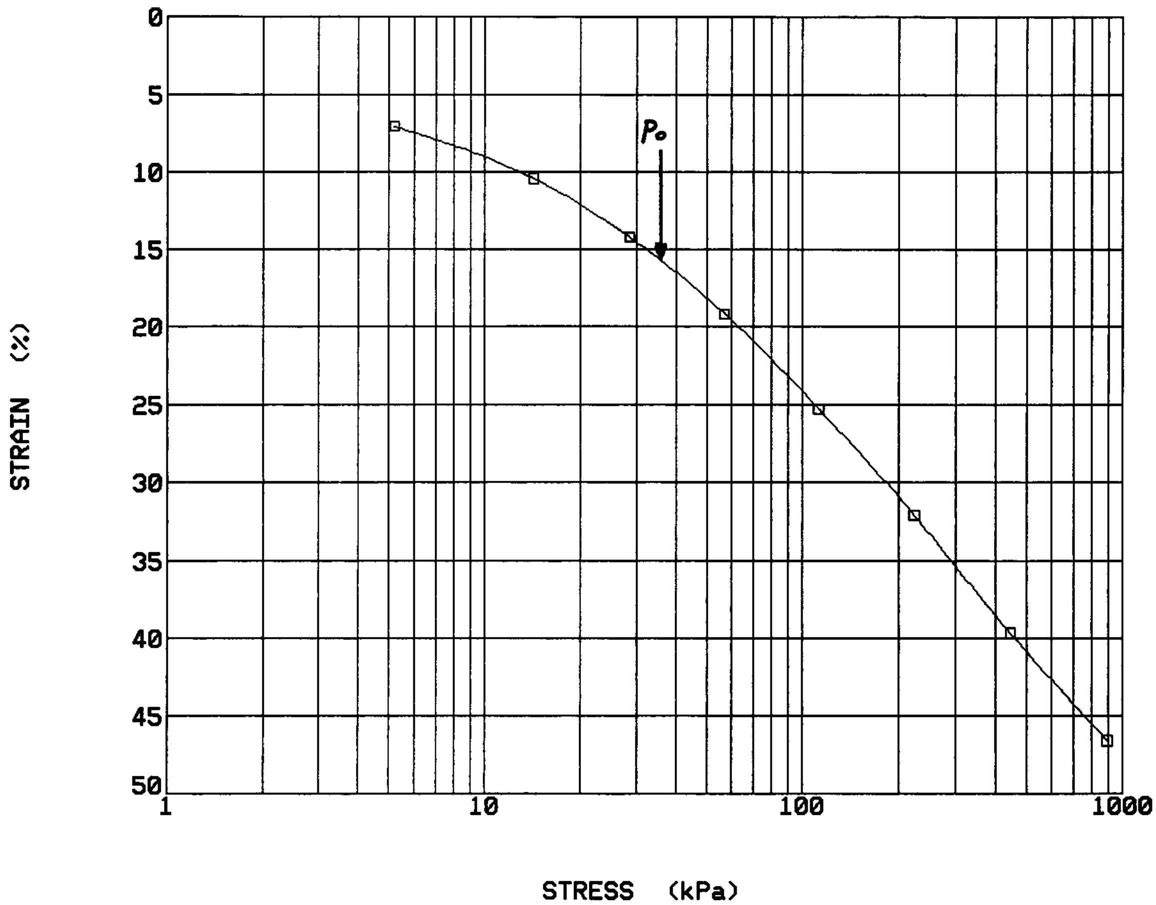
BOREHOLE	DEPTH (m)	ORGANIC CONTENT (%)	MOISTURE CONTENT (%)
1	2.3	10.2	90
1	3	37.5	217
1	3.2	9.1	77
1	3.8	11.3	91
1	4.6	7.7	75
1	5.3	6.4	66
1	6.1	0.3	11
1	6.2	3.1	41
2	3.8	16.3	117
2	4	18	139
2	4.6	15.9	122
2	5.3	44	229
2	5.4	20.2	77
2	5.5	21.9	120
2	5.7	14	104
2	6.1	25.5	183
2	10.6	14	22
8	9.1	5	39

Organic Content vs Moisture Content



Enclosure 6

CONSOLIDATION TEST



□ LEGEND: BOREHOLE 2 DEPTH 4.60

Liquid Limit = %
Plastic Index =
Natural Moisture Content = 123 %

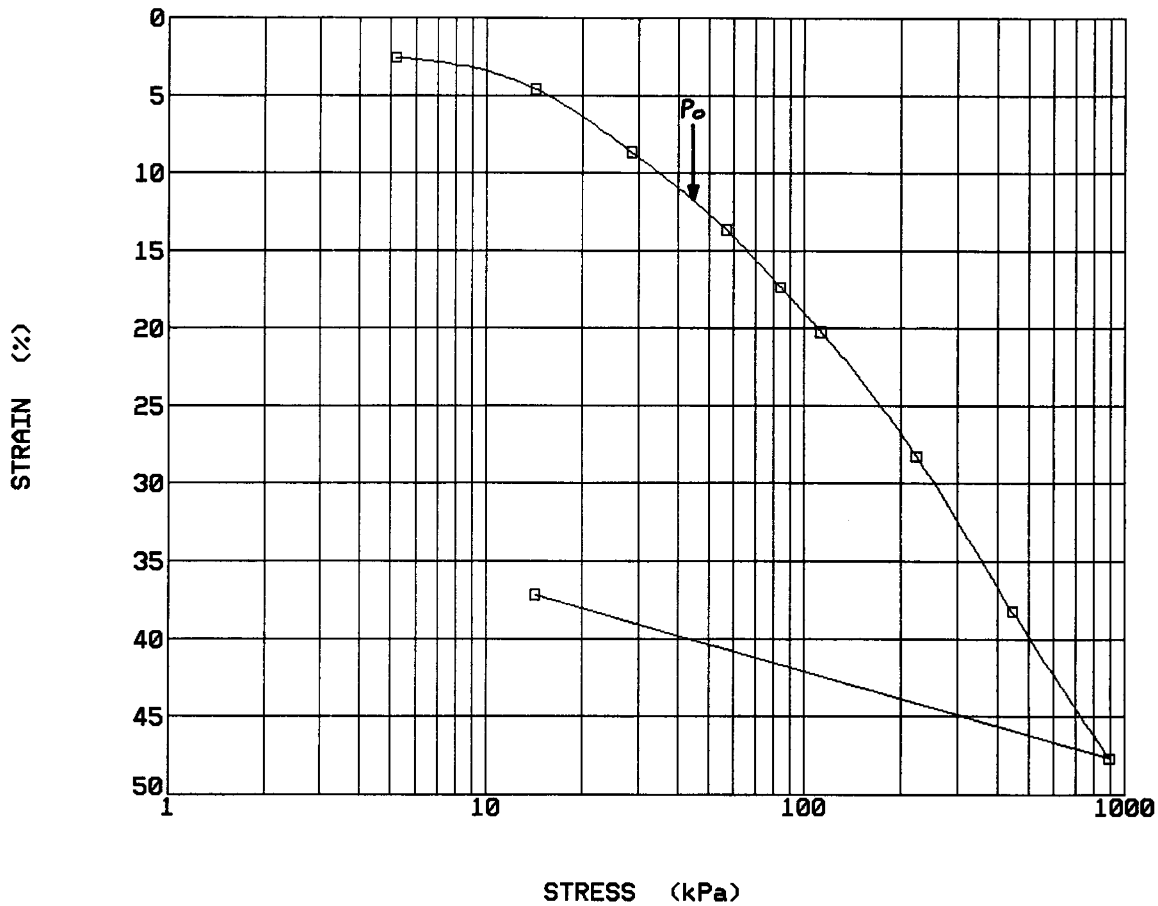
ORGANICS

Reference No.: 127-88-00 March 1996

MONTREAL RIVER BRIDGE - ELK LAKE

DST CONSULTING ENGINEERS INC. Enclosure 7

CONSOLIDATION TEST



□ LEGEND: BOREHOLE 2 DEPTH 5.30

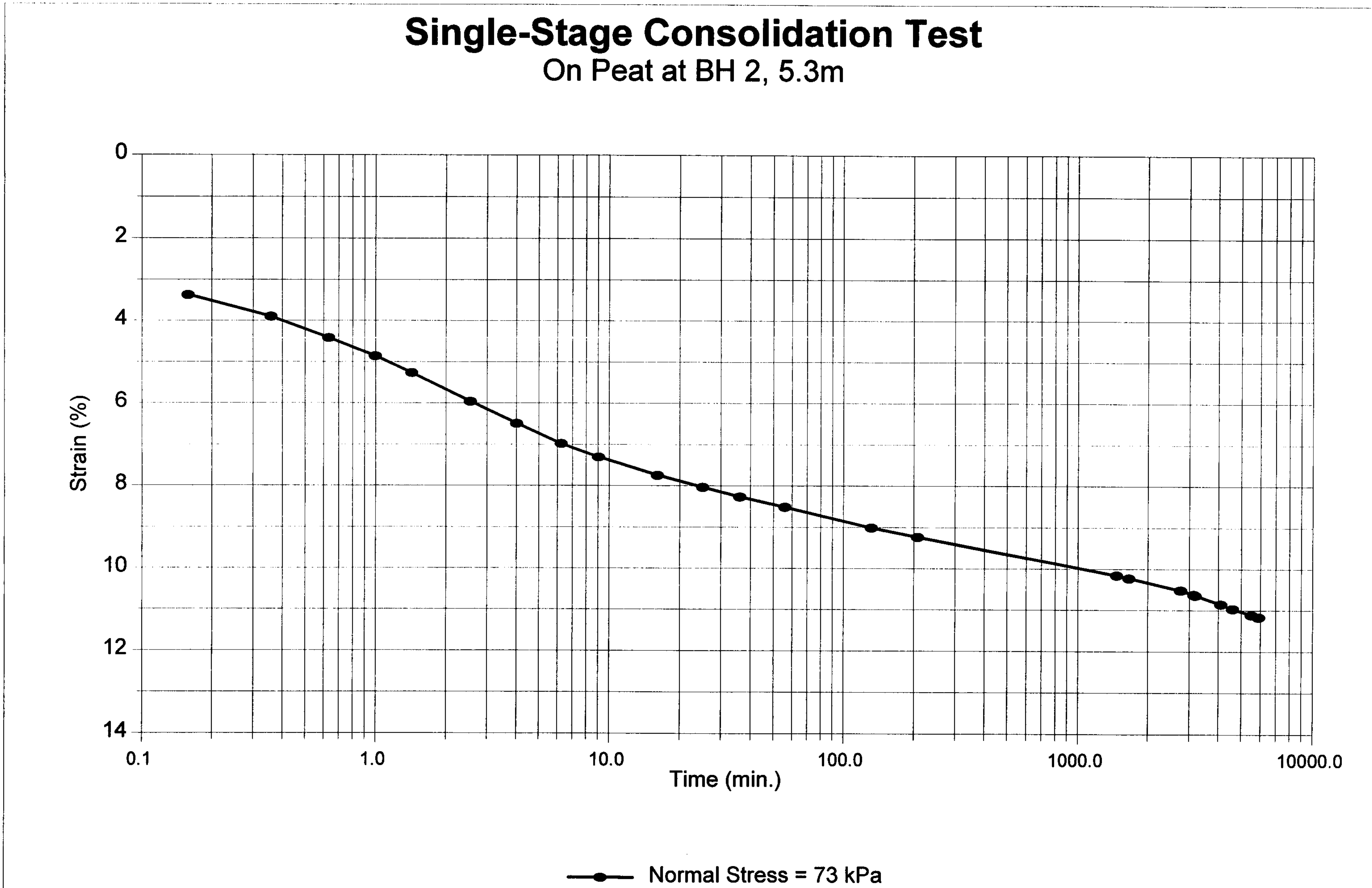
Liquid Limit = %
Plastic Index =
Natural Moisture Content = 139 %

ORGANICS

Reference No.: 127-88-00 March 1996

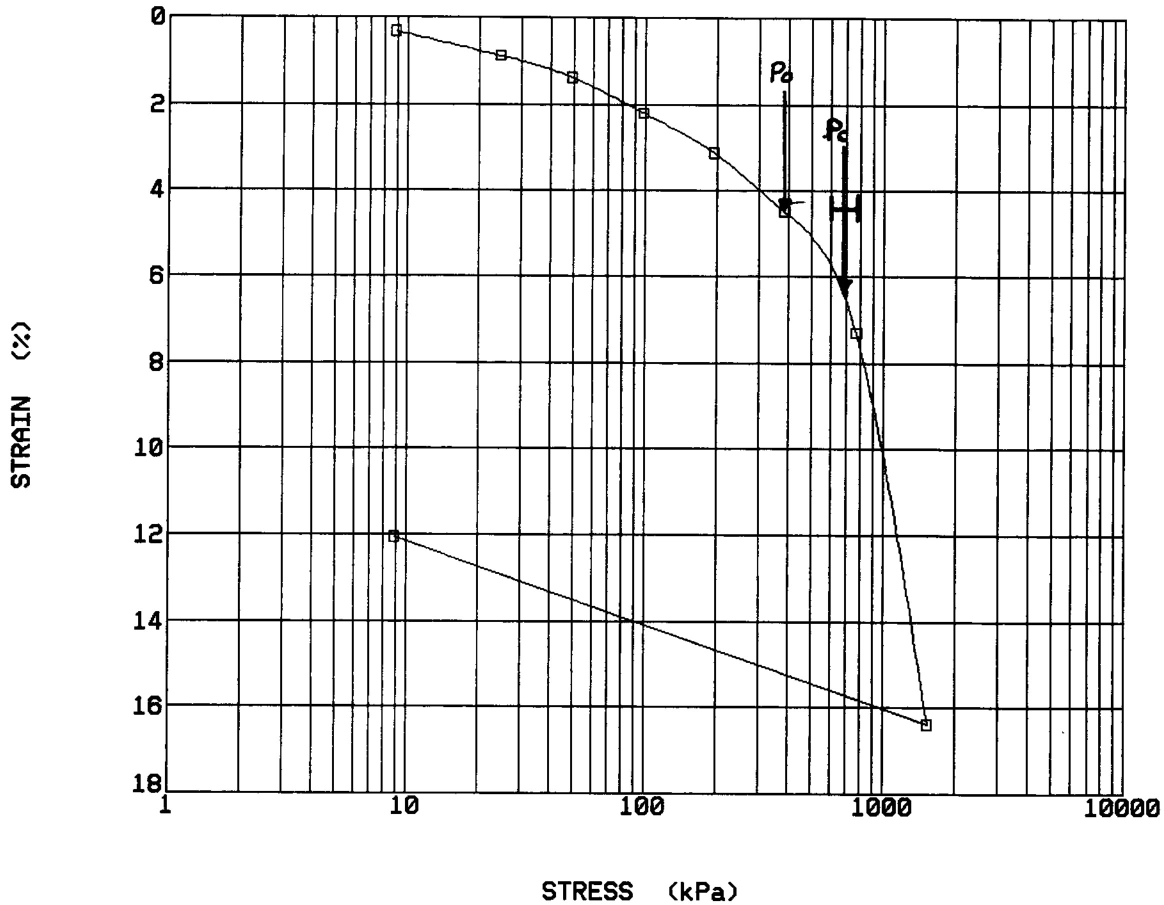
MONTREAL RIVER BRIDGE - ELK LAKE

DST CONSULTING ENGINEERS INC. Enclosure 8


Single-Stage Consolidation Test
On Peat at BH 2, 5.3m
Strain (%)
0
2
4
6
8
10
12
14
0.1
1.0
10.0
100.0
1000.0
10000.0
Time (min.)
Normal Stress = 73 kPa

Enclosure 9

CONSOLIDATION TEST



□ LEGEND: BOREHOLE 2 DEPTH 39.60

Liquid Limit = 66 %
Plastic Index = 35
Natural Moisture Content = 48 %

CLAY

Reference No.: 127-88-00

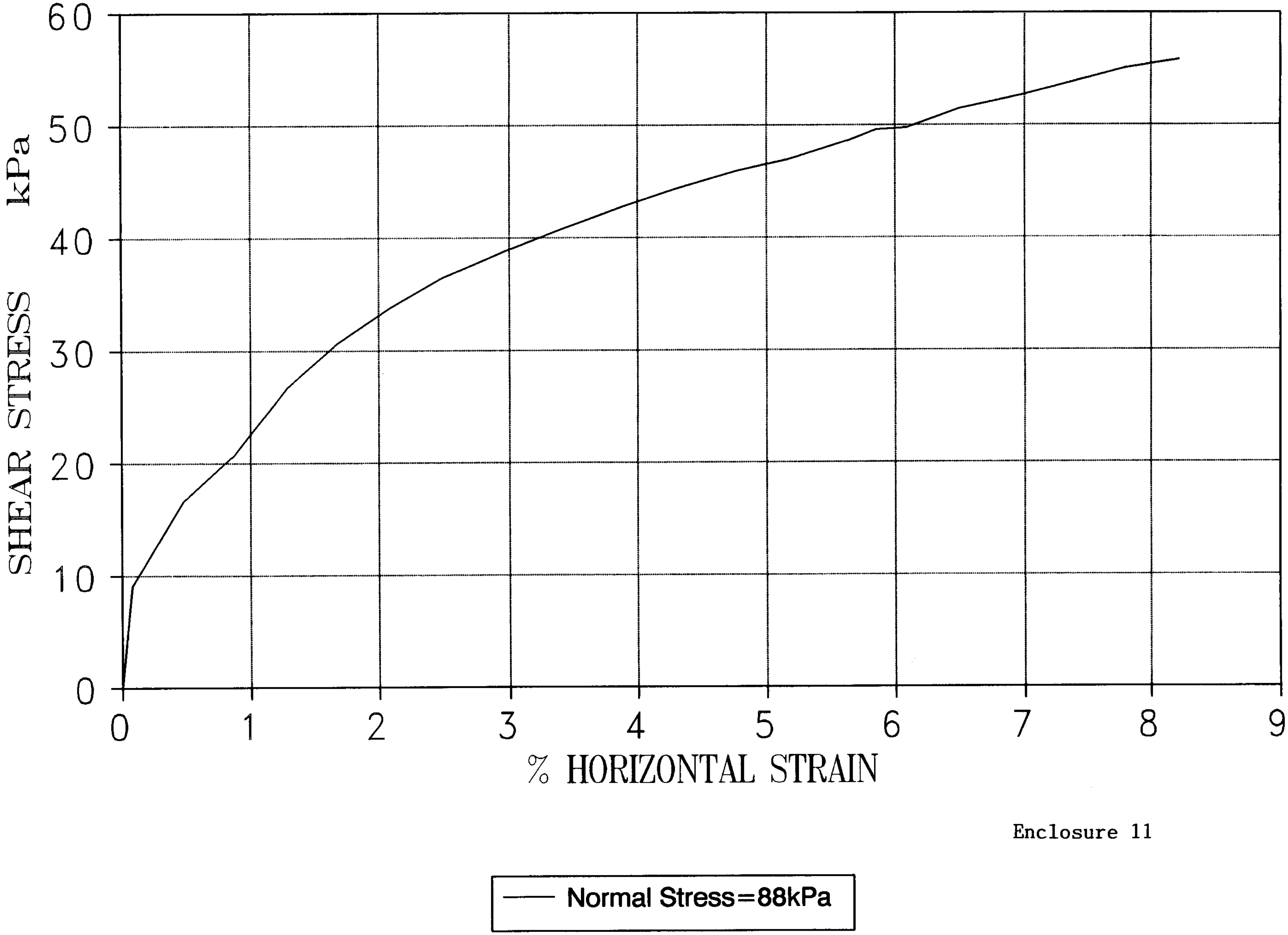
March 1996

MONTREAL RIVER BRIDGE - ELK LAKE

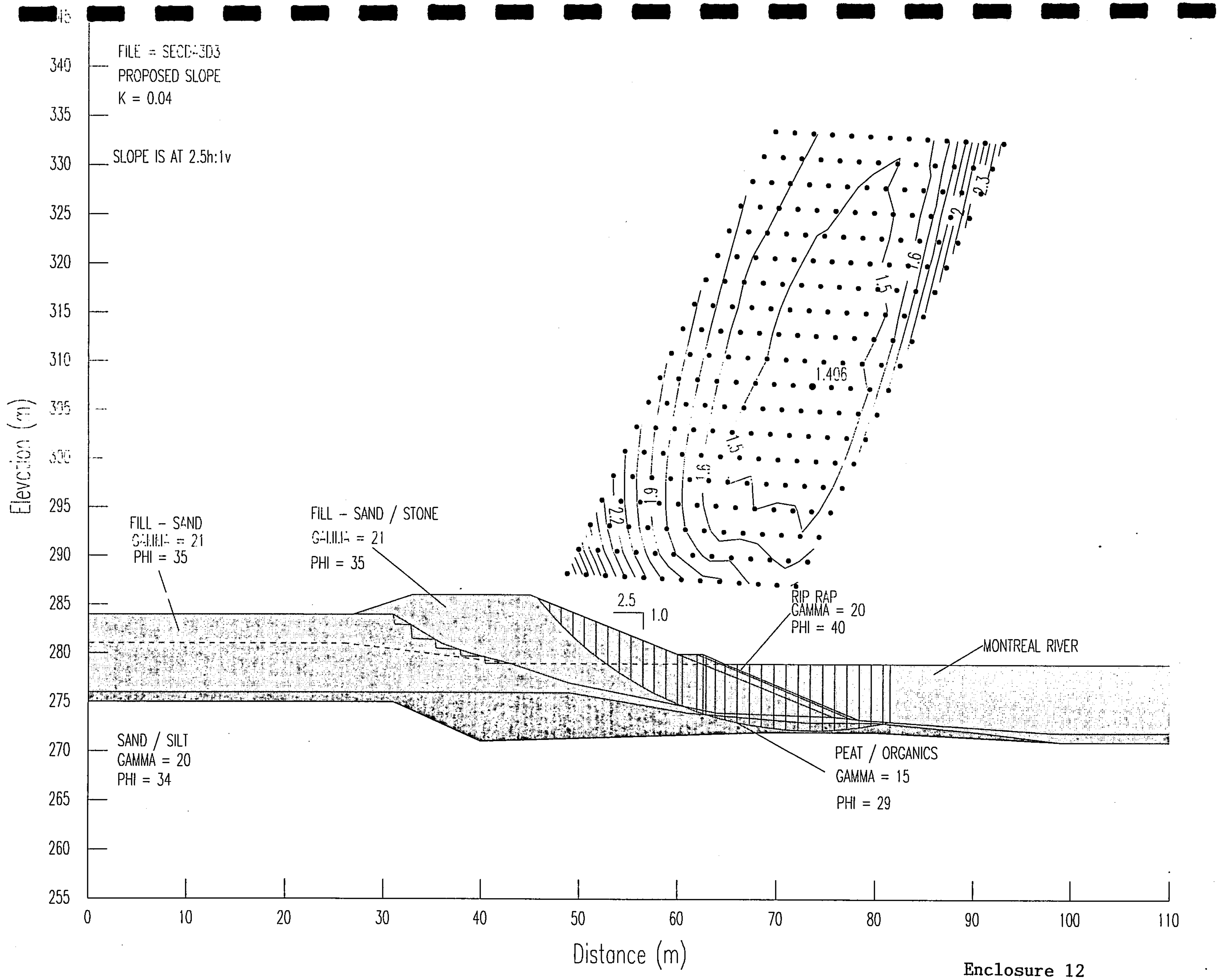
DST CONSULTING ENGINEERS INC.

Enclosure 10

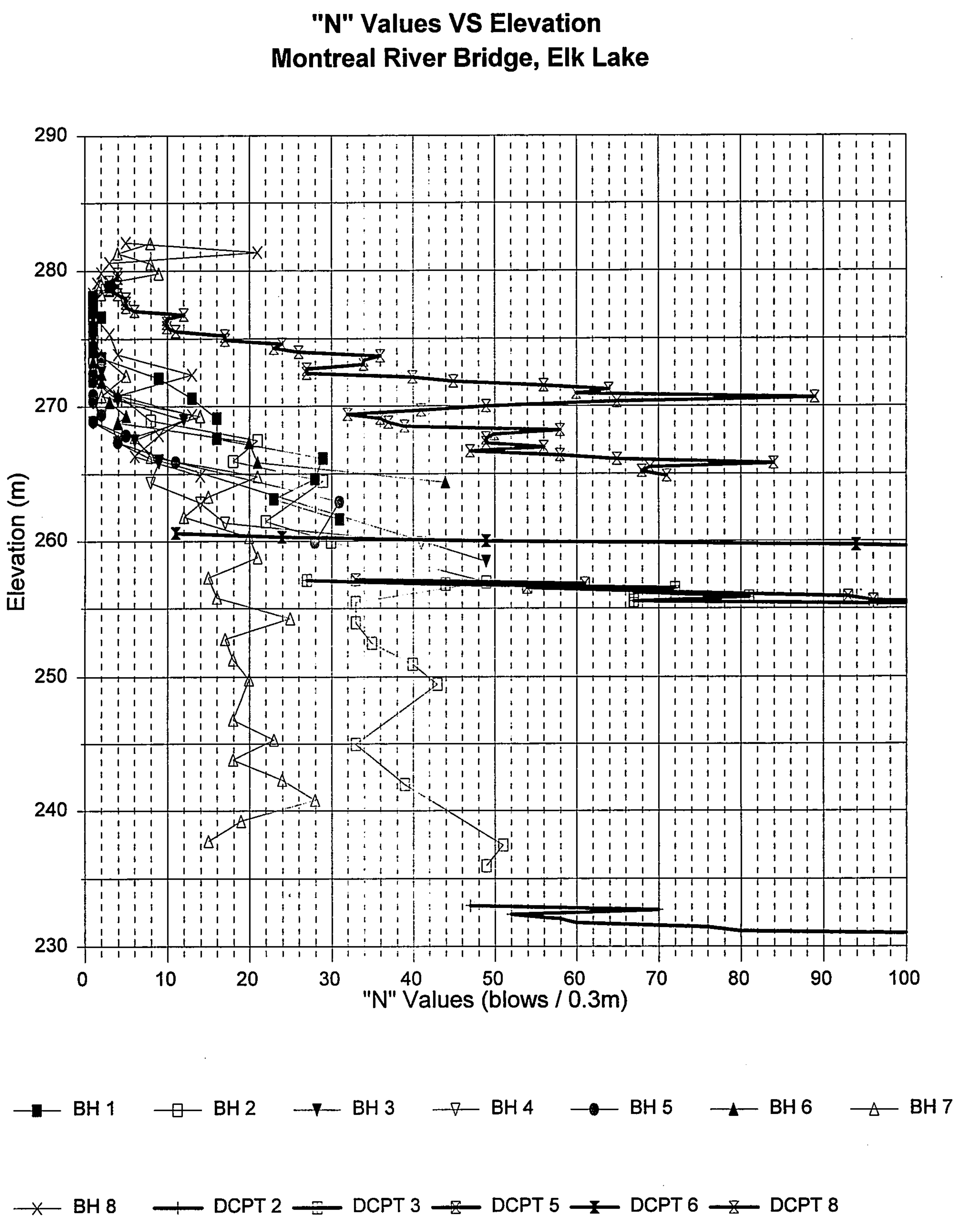
Consolidated Undrained Direct Shear
Test – Organic Soil, BH 2 @ 3.8m



Enclosure 11



Enclosure 12



Note: Where available, corrected N values (N') are inserted (see report).

Enclosure 13

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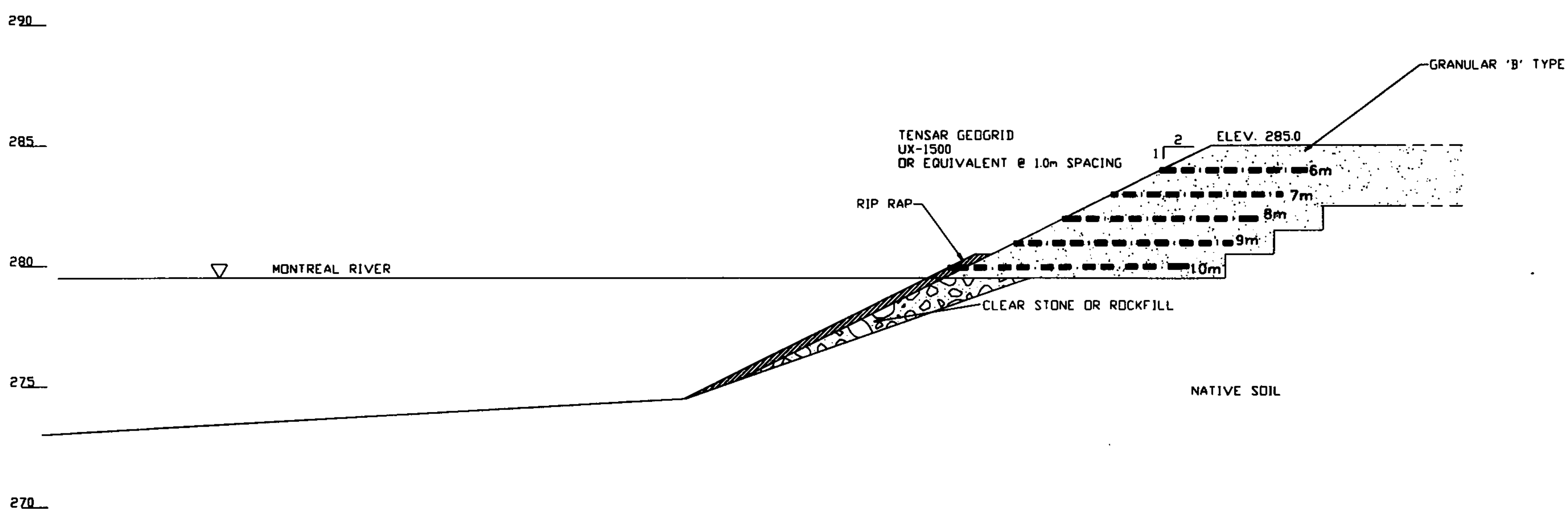
Enclosure 14

GEOGRID TO BE INSTALLED
IN ACCORDANCE WITH
MANUFACTURES REQUIREMENTS



ENCLOSURE NO. 15

0 10m
METRES

DST CONSULTING ENGINEERS

TITLE

GEOGRID PLACEMENT
EAST APPROACH EMBANKMENT
MONTREAL RIVER BRIDGE

DATE	SCALE
MAY 96	AS SHOWN
DRAWN BY / APPROVED BY	PROJ.
C.S.S. / G.M.	TG96001
CLIENT MTO	
DWG. REF. NO. MTO	

CONT No
WP No 127-88-01

MONTREAL RIVER

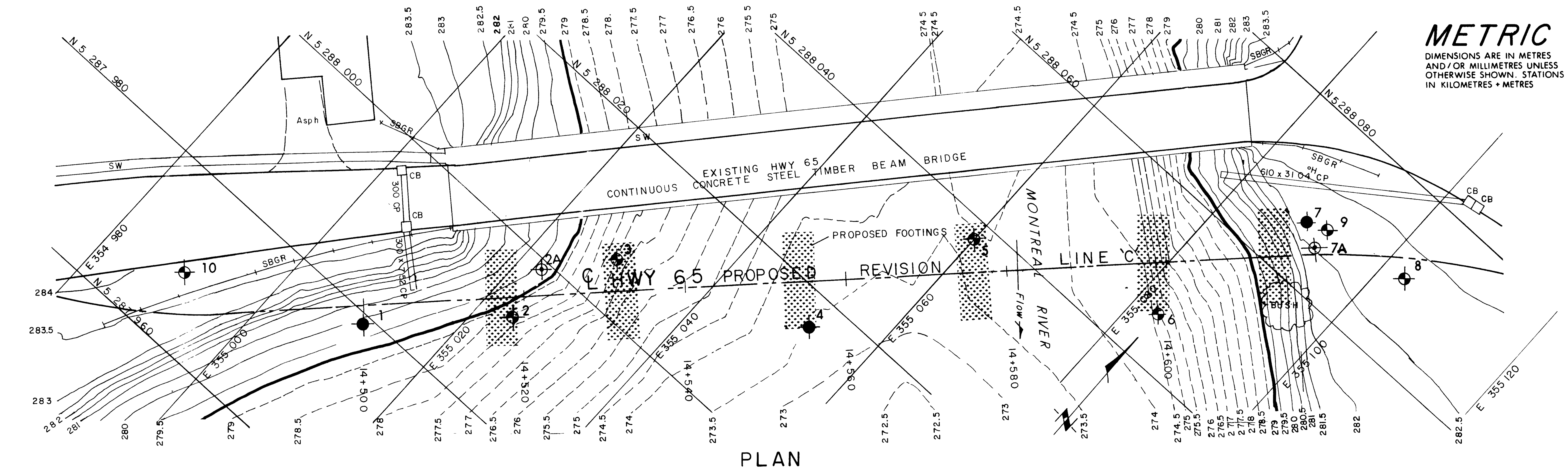
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

DST CONSULTING ENGINEERS INC.

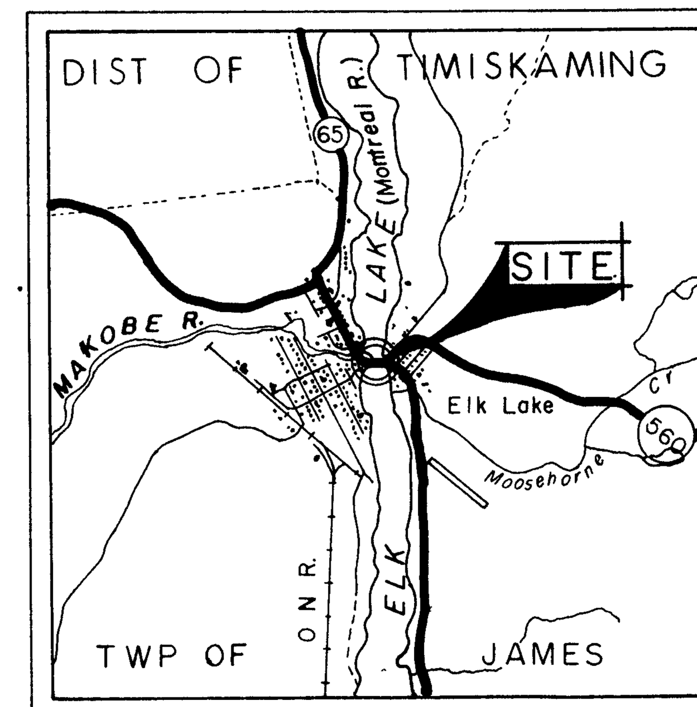
METRIC

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



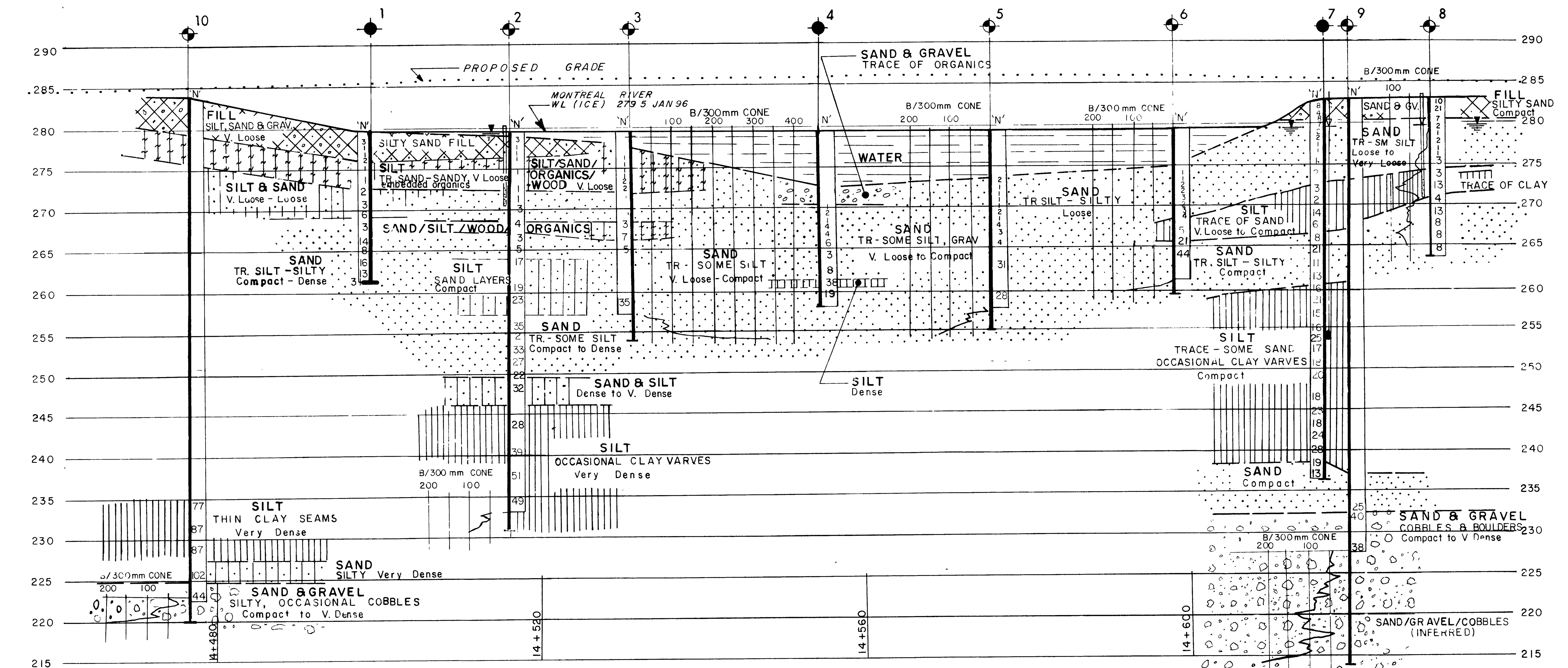
PLAN

SCALE 10 5 0 10 20 METRES



KEY PLAN

SCALE 0.5 0 1 2 Kms



℄ PROFILE HWY 65 LINE 'C'

SCALE 10 5 0 10 20 METRES

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
- Electronic Piezocone
- Head
- Piezometer
- Standpipe

No	ELEVATION	CO-ORDINATES NORTH	CO-ORDINATES EAST
1	279 6	5 287 979	355 011
2	279.5	5 287 992	355 024
3	279.5	5 288 006	355 029
4	279.4	5 288 016	355 052
5	279 4	5 288 038	355 060
6	279.4	5 288 046	355 083
7	282.8	5 288 067	355 089
8	282.9	5 288 070	355 103
9	282.8	5 288 068	355 092
10	283.9	5 287 969	354 990

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 41P-23

REF No E326-65-01, 1994 07 30

HWY No 65 LINE 'C' REVISION			DIST 53
SUBM'D WH	CHECKED	DATE 1996 05 06	SITE 47-36
DRAWN AM	CHECKED	APPROVED	DWG 1278801-A

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FAX: (905) 474-9267

Dpae

W.P. 127-88-01

REPORT
on

WEST EMBANKMENT
STABILITY AND SETTLEMENT ANALYSIS
MONTREAL RIVER BRIDGE
ELK LAKE, ONTARIO

Ref. No. G-96-1202
April 1997

Prepared for:
Armbro Construction Ltd.
25 Van Kirk Drive, Unit 8
Brampton, Ontario
L7A 1A6

Distribution

3 Copies	-	Armbro Construction Ltd.
1 Copy	-	Geo-Canada Ltd.

Ref. No. G-97.0104

CONTENTS

APPENDICES

1.0 INTRODUCTION

This report presents a summary of additional field investigations and associated stability and settlement analyses related to the design of the west embankment of the proposed Montreal River Bridge in Elk Lake, Ontario.

The work presented within was completed in general accordance with our proposal letter to Armbro Construction Ltd. dated February 20, 1997.

2.0 BACKGROUND

As part of the successful Design/Build team for the proposed Montreal River Bridge, Geo-Canada Ltd. (GCL) was requested to investigate the feasibility of eliminating a 2m wide berm at the midheight of the west embankment that had been recommended in the Geotechnical Report provided with the proposal documents. The results of that analysis based on data provided in the Geotechnical Report as prepared by DST Consulting Engineers, of Thunder Bay, Ontario, and reported in our letter to Delcan Corporation dated January 15, 1997, indicated that it should be feasible to eliminate the berm. However, to finalize the analysis, two (2) additional boreholes were recommended to confirm the subsurface conditions along a profile at 45° to bridge centre line, as shown on the attached Figure 1.

These two boreholes were subsequently completed from the surface of the frozen river on February 5, 1997, and found a greater thickness of organic silt at the bottom of the river than was previously indicated from the data presented in the DST report. Upon discussing the results of these two boreholes with the Design/Build team (Armbro and Delcan Corporation as the design/build prime consultants), a further field and laboratory investigation program was undertaken to better quantify the thickness, shear strength, and compressibility of the organic deposit. The factual results arising from these additional investigations, and subsequent analyses and recommendations are presented within.

.../...

3.0 INVESTIGATION METHODOLOGY AND RESULTS

3.1 Field Investigation

The field investigation consisted of putting down four boreholes at the approximate locations shown on the attached Figure 1. The field program was carried out in two phases:

Phase 1

Boreholes 101 and 102 were put down on February 5, 1997, using a light weight, sonic soil sampling rig working from the frozen surface of the river. This drilling technique consisted of vibrating a standard, 50 mm outside diameter split spoon sampler into the ground in 0.76m increments, to obtain continuous samples of the soil. Representative samples of the encountered soils were retrieved after each push, and the approximate SPT 'N' value estimated based on the rate of advance of the split spoon sampler. After each sample, in situ vane shear tests were carried out in the organic silt layer to evaluate its undrained shear strength. A summary of the encountered soil stratigraphy, equivalent SPT 'N' values and results of the vane shear tests are shown on the borehole logs presented in Appendix B.

Phase 2

Boreholes 103 and 104 were put down on February 25, and 26, 1997, using a bombardier mounted power auger drill rig. Since at that time the ice thickness was insufficient to support the weight of the drill rig, the boreholes were put down close to the edge of the water. During drilling of these boreholes, split spoon samples in association with SPT tests or shelby tube samples were taken at 1.5m intervals of depth. In-situ vane shear tests were performed in the organic silt below each sample. A summary of the encountered soil stratigraphy, SPT 'N' values and the vane shear strengths are summarized on the borehole logs presented in Appendix B.

.../...

3.2 Laboratory Tests

The Laboratory testing consisted of natural moisture content tests, grain size distribution analyses and Atterberg limit tests on representative split spoon samples, and drained direct shear and consolidation tests on shelby tube samples. The results of the soil index tests are summarized on the borehole logs with full grain distribution curves presented on Figures C-1 and C-2 of Appendix C. The results of the strength and compressibility tests are shown in Figures C-3 to C-7.

The direct shear tests were performed by Golder Associates Ltd. on samples of the organic silt taken from ST 4 of Borehole 103. The tests were completed by first consolidating the samples to 25, 75 and 125 kPa vertical pressure, and then shearing at a slow rate to ensure full drainage.

Consolidation tests were performed by GCL on samples of the organic silt taken from ST 3 of Borehole 103. In the first test, a maximum load of the 383 kPa (4 tsf) was applied in eight increments. The duration of each load increment was 1 to 2 hours, which was sufficient for 100% primary consolidation to occur. The second specimen was loaded to 72 kPa (0.75 tsf) in two increments, with the higher load being applied for a duration of 1 week to provide data on the applicable coefficient of secondary consolidation for the deposit.

3.3 Discussion

3.3.1 General

The stratigraphy encountered in Boreholes 101 to 104, as summarized on the attached Figure 2, is in general agreement to that of the DST report, being composed of lake sediment or fill overlying organic silt, which in turn overlies silty sand. However, the thickness of the organic silt is considerably greater than that presented in the DST report. Key properties of the Organic Silt and underlying Silty Sand layers are discussed below. However, for more detailed information, the reader should consult the factual data presented on the borehole logs of Appendix B, and Laboratory Test data of Appendix C.

.../...

3.3.2 Organic Silt

Encountered immediately below the surficial fill on land or within the base of the river, the inferred stratigraphy along a section at 45° to the centre line of the new bridge as presented on the attached Figure 2, indicates up to 7m of organic silt, which extends to as deep as Elevation 266m. This information is at variance to the summarized soil stratigraphy along the bridge centre-line as presented on Drawing 1278801-A of the DST report, which shows an average of 4m of organic silt extending from elevation 276 to 272m.

The organic silt deposit is brown in colour, sandy and also contains some partially decomposed wood. The results of three grain size curves for this material as presented on Figure C-1 of Appendix C, indicate compositions of between 32 to 51% sand, 43 to 63% silt, and 5 to 8% clay. The results of two Atterberg limit tests indicate liquid limits of 38% and 51%, plastic limits of 29% and 37%, with associated plasticity indices of 9 and 14. Measured natural moisture contents of the organic silt varied from 43 to 159%, which are generally above the measured liquid limits.

Measured in-situ undrained shear strengths as summarized on the attached Figure 3, varied from 6.4 to 70.8 kPa, and based on this data, the layer is described as very soft to stiff. However, the data presented on the attached Figure 3 clearly shows a trend of increasing strength with depth, from about 10 to 20 kPa near the surface, to over 60 kPa at 8m depth. For comparison, DST reported a shear strength range for this deposit of between 30 to 40 kPa throughout. The undrained shear strength profile adopted in the stability analyses is shown on Figure 3.

The results of the direct shear tests, as summarized on Figures C-3 and C-4 of Appendix C, indicate effective cohesion and effective angle of shearing resistance for the organic silt of 17.5 kPa and 33.9°, respectively. This measured value for the angle of shearing resistance is slightly higher than the range of 28 to 32° recommended by DST. For the design of the embankment, an effective shearing angle of 32° and zero cohesion have been adopted.

.../...

Based on the results of the consolidation tests completed by GCL (Figures C-4 and C-5 of Appendix C) and the results reported by DST, a mean coefficient of consolidation of 0.004 cm^2/sec, and a coefficient of secondary consolidation of 1% per log cycle of time are considered appropriate as input to the settlement analysis of the west embankment.

3.3.3 Silty Sand

Encountered below the organic silt, this material consists of interbedded silty sands and silts, with occasional organic rich layers. The results of three grain size analyses on bulk samples of the material as presented on Figure C-2 of Appendix C, indicate compositions of between 58% to 96% sand, 4 to 34% silt, and 0 to 8% clay. Measured natural moisture contents of the sand varied from 23 to 42%. Measured SPT 'N' values varied from 3 to 8 which indicate very loose to loose conditions.

4.0 STABILITY ANALYSIS

4.1 General

Using the soil profile of Figure 3, and summarized strength parameters presented in the preceding section, stability analyses for the west embankment were carried out for the following conditions.

- Long Term Stability
- Stability under River Rapid Drawdown
- End of Construction or Short Term Stability.

The assumed "base case" embankment profile for each of these conditions is an embankment constructed with a side slope of 2.5 horizontal to 1 vertical (2.5H:1V), crest elevation of 286 m and mean river elevation of 179.5m..

.../...

4.2 Long Term

Case 1 - Drained Conditions

This case consisted of the analysis of the stability of the base case embankment profile (2.5 : 1 slope to finished grade (286m±) and no berm) with a 12 kPa traffic load surcharge and earthquake loads equivalent to 4.5% gravitational acceleration, assuming the organic silt acts as a fully drained frictional material with an effective shearing angle (ϕ') of 32°. Under these conditions a minimum Factor of Safety of 1.33 was obtained as shown on the attached Figure 4, which is considered to be satisfactory.

Case 2 - Undrained Condition.

This case was similar to case 1), but assumes that the organic silt acts as in an undrained mode, which could happen under conditions of rapid earthquake loading. Stability analyses for this condition as presented for Case 3 of Section 4.3, indicates an acceptable Factor of Safety of 1.29 provided there is a 10 kPa increase in the undrained strength in the upper 4m of the organic silt. While data presented on Figure 2 indicates that the required amount of strength is not currently available in the upper 4m of this layer, it is expected that this portion of the layer will achieve this required increase in strength after the organic silt has fully consolidated under the weight of the embankment. In fact, this strength gain is expected to occur under the influence of the weight of an embankment constructed to Elevation 282.0m. Accordingly, provided the embankment can be successfully constructed to its design height in the short term as addressed in Section 4.3, the long term stability under this condition is considered to be satisfactory.

Case 3 - Rapid Drawdown.

For this case, the stability of the west embankment under the influence of a rapid drawdown in the level of the River from Elevation 283m to 281m was considered. A key component of this evaluation is the assumed rate of pore water pressure equalisation within the soils below

.../...

and within the river bank, relative to that of the river, which will be a function of the permeability of the native soils and the rate of rise and fall of the river.

With respect to the rate of rise and fall of the river, review of the Hydrology Report for the project, indicates that the Montreal River system is assessed as having a lot of natural buffering due to the presence of swamps and lakes which tend to attenuate the flows, which is beneficial for this case as it will allow additional time for pore-water pressure equalisation in response to changing river levels. With respect to the permeability of the native soils, the fine grained nature of the dominant organic silt layer, suggests that the pore water regime will be very slow in adjusting to changing levels within the river, and will probably not achieve full equalisation over the relatively short flood event when the river is expected to crest and subside. This assessment tends to suggest that stability analysis allowing for a full 2m of excess head differential between the pore water regime in the native soils versus that of the river would be overconservative.

This preliminary conclusion was substantiated by stability analyses of the existing river bank profile, which indicated a Factor of Safety significantly less than 1.0 if 2m of excess pore water pressure remains in the organic silt and the underlying sand, after the river level has dropped, which is inconsistent with the generally stable river banks at the site. Further analyses resulted in a Factor of Safety of 0.98 being obtained when it was assumed that there is no excess pore water pressure in the upper 2m of the organic silt, with excess pore water pressures of 1.6m below this depth. Subsequent analysis for the proposed west embankment profile using this back calculated pore water pressure regime, gave a minimum F.S. of 1.30 as presented on Figure 5, which is deemed satisfactory. Please note that this analysis did not include earthquake forces.

4.3 Short Term

Case 1 - Full Height Construction - No Strength Gain in Organic Silt

Under this case, a 2.5 : 1 slope to final grade with traffic surcharge but no earthquake load

.../...

was analysed for an assumed undrained or short term response of organic silt with undrained shear strengths of 16, 28 and 44 kPa in its top 2m, 2 to 4m and below 4m, respectively. This case is appropriate for the short term (end of construction) condition, with the implication that an earthquake is unlikely to occur during construction.

F.S.'s of 1.03, 1.08 and 1.13 were obtained assuming no berm, 2m wide berm and 4m wide berms respectively, as indicated on the attached Figure 6. These results indicate that if the west embankment is constructed to its full height in one stage, and that the organic silt is undrained during this period of construction, a 4m wide berm is necessary in order to achieve a minimum acceptable end of construction F.S. of 1.10.

Case 2 - Construction to Elevation 282.0m - No Strength Gain in Organic Silt

This case was analysed to check the stability of an embankment with 2.5 : 1 side slopes without a berm, raised to Elevation 282.0m, assuming no strength gain in the underlying organic silt. Under this condition, a minimum F.S. of 1.19 was obtained which indicates that the embankment can be safely raised to Elevation 282m in one stage.

Case 3 - Full Height Construction - Strength Gain in the Organic Silt.

This case was similar to Case 1 above, but assumes a strength gain of 10 kPa in the upper 4m of the organic silt layer which is anticipated to occur if the embankment is constructed to Elevation 282.0m and allowed to fully consolidate, prior to raising the embankment to its final design height. Under this condition a minimum F.S. of 1.29 was obtained as shown on Figure 7 for a 2.5H:1V slope without a berm, which is considered to be satisfactory. However, it should be noted that this will require scheduling of the construction works to obtain at least 90% consolidation of the organic silt, prior to raising the embankment beyond Elevation 282.0m. This aspect of the works is discussed in greater detail in Section 5.0.

4.3 Summary of Results

The results of the stability analyses as summarized in Table 1 below , indicate the following:

.../...

1) Embankment Construction utilizing side slopes of not greater than 2.5H : 1 V without a berm is satisfactory in the long term.

2) To satisfy the Short Term or End of Construction stability condition, the embankment should be stage constructed by first raising it to Elevation 282.0, followed by construction to its full height after the underlying organic silt has achieved at least 90% consolidation. The anticipated time for the consolidation to occur is addressed in Section 5. The degree of consolidation should be confirmed by monitoring of pore water pressures and settlements, prior to raising the embankment.

TABLE 1
SUMMARY OF STABILITY ANALYSIS RESULTS

Fill Elevation (m)	Traffic Surcharge	Seismic Load	F.S.	Remarks
286.0	12 kPa	0.045 g	1.33	long term condition
286.0	12 kPa	0	> 1.30	rapid drawdown
282.0	12 kPa	0	1.19	short term, stage construction condition
286.0	12 kPa	0	1.29	Short Term or End of Construction condition, assuming at least 10kPa increase in undrained shear strength of the upper 4m of the organic silt

5.0 SETTLEMENT ANALYSIS

Settlement analysis of the proposed west embankment based on the results of the

.../...

consolidation tests and measured thickness of the organic silt deposits (Figure 2) indicates that up to 1.2m of total primary settlements could occur. However, actual settlements are expected to be highly variable, especially perpendicular to road centre line, with less settlements expected to occur immediately adjacent to and over any portion of the existing embankment. Secondary settlement is estimated to be 50 mm in first five years with a further 50 mm over the next twenty-five years following construction. Pavement construction should be delayed until the primary settlements are complete. Secondary settlement can be anticipated to result in increased pavement maintenance.

The estimated time for the primary settlements to be complete is 10 months, based on the measured coefficient of consolidation values obtained from the consolidation tests. While the actual field rate of consolidation is expected to be shorter, it is highly unlikely that it would be less than 5 months. Even this optimistic rate is unacceptable to permit the project to be completed by the Fall of 1997, if stage construction and settlement requirements are taken into account. Accordingly, unless project completion can be delayed until the early summer of 1998, measures to enhance the rate of development of settlements within the underlying organic silt are recommended.

For this site, the installation of a series of sand or wick drains which pass through the organic silt layer are considered appropriate, although sand drains are preferred from a strength enhancement perspective, which is also crucial to this project. A summary of the influence of 0.3m diameter sand drains on a triangular grid pattern to achieve 90% and 100% of the primary consolidation is presented below in Table 2.

TABLE 2

EFFECT OF SAND DRAINS ON PRIMARY CONSOLIDATION TIMES IN DAYS

% Consolidation	No Sand Drains	0.3m Diameter Sand Drains @ 3m centres	0.3m Diameter Sand Drains @ 1.8m centres	0.3m Diameter Sand Drains @ 1.5m centres

.../...

90	250	90	45	25
100	300	120	80	40

Based on our current understanding of the project schedule, it appears that the installation of sand (or wick) drains on a 1.8m triangular pattern will be required to achieve a project completion by fall of 1997. In addition, to improve the post construction performance of the embankment and pavement, especially during the first 5 years, it is recommended that the embankment be surcharged with an additional 1.5m of fill and pavement construction delayed until at least 100 % consolidation and related settlements have occurred at this higher load. However, please note that maximising the time of embankment surcharge application beyond this minimum requirement, will be of further benefit.

The sand (or wick) drains should be installed to the maximum extent possible from the top surface of the embankment at Elevation 282. The proposed drains should fully penetrate the organic silt layer and terminate not less than 1.0m into the underlying silty sand. Backfill to the sand drains should consist of concrete sand meeting the gradation requirements of CDN/CDA-A23.1/2-M90.

During construction of embankment fill, the magnitude of pore water pressures and settlement of the fill should be monitored during and immediately after fill placement.

Please note that the anticipated settlement of the organic silt layer will result in negative skin friction forces on the proposed foundation piles. Accordingly, a negative skin friction force of 150 kN and 120 kN should be applied for the ULS and SLS respectively, for a W310 H pile. Alternatively, the piles can be isolated from negative impacts of the settling organic silt layer, by pre-augering and casing through the organic silt layer, followed by pile installation through the casing, following by backfill of the pre-augered hole using a weak bentonite grout

.../...

upon extraction of the casing.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The west embankment may be constructed to 2.5 : 1 (H : V) slope without a berm, by constructing in two stages, namely to Elevation 282m followed by full height construction when the organic silt layer has achieved at least 90 % consolidation under the influence of the first stage embankment.

Sand drains should be installed in the organic silt layer to improve drainage and reduce the consolidation time. Based on our current understanding of the project schedule, a sand drain spacing of 1.8m on a triangular grid seems appropriate.

The constructions sequence is envisaged as follows:

1. Install settlement and pore-water measuring systems
2. Place rock fill to elevation 282m. The F.S. at this stage should be at least 1.19. Embankment construction below the water level should be completed using rock fill with a maximum particle size of not greater than 75 mm, so as to minimise the interference with pile driving and sand drain installation. Above the water level, Granular 'A' materials placed in 150 mm lifts and compacted to at least 95% of their standard Proctor maximum dry density should be used.
3. Install 0.3m diameter sand drains in a triangular pattern at 1.8m spacings under the footprint area of the embankment to the maximum extent practical. Backfill to the sand drains should meet the CSA requirement for concrete sand.
4. Drive piles for abutments; construct abutment.

.../...

5. When settlement and porewater monitoring results indicate 90% consolidation is complete, raise embankment fill to full height + 1.5m surcharge, while undertaking regular monitoring of the pore water pressures (at least daily)
6. Maintain surcharge load until at earliest, monitoring results indicate consolidation is 100% complete. However, the length of time for the surcharge load should be maximized.

7.0 STATEMENT OF LIMITATION

The contents of this report should be read in conjunction with the attached Statement of Limitation presented in Appendix A.

8.0 CLOSURE

We trust the information provided within is sufficient to permit the design of the west embankment to be completed. If you have any questions regarding this report, or require clarification on any matter, please do not hesitate to contact us.

GEO-CANADA LTD.

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JAMES S. M. NG
Apr 18/97
PROVINCE OF ONTARIO

James Ng, P.Eng.

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I. CORBETT
Apr 18 '97
PROVINCE OF ONTARIO

Ivan Corbett, P.Eng.

JN/IC/ic

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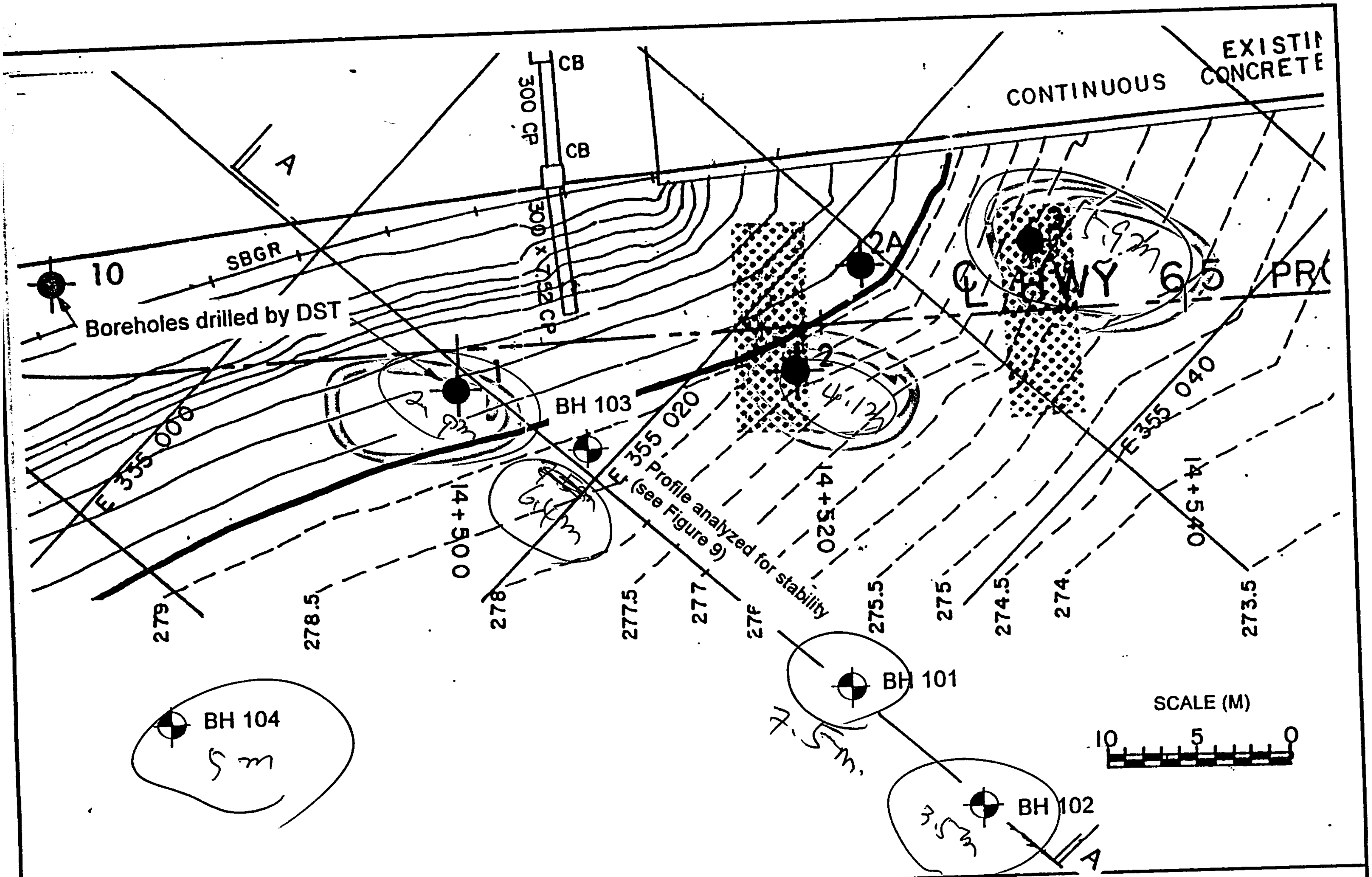
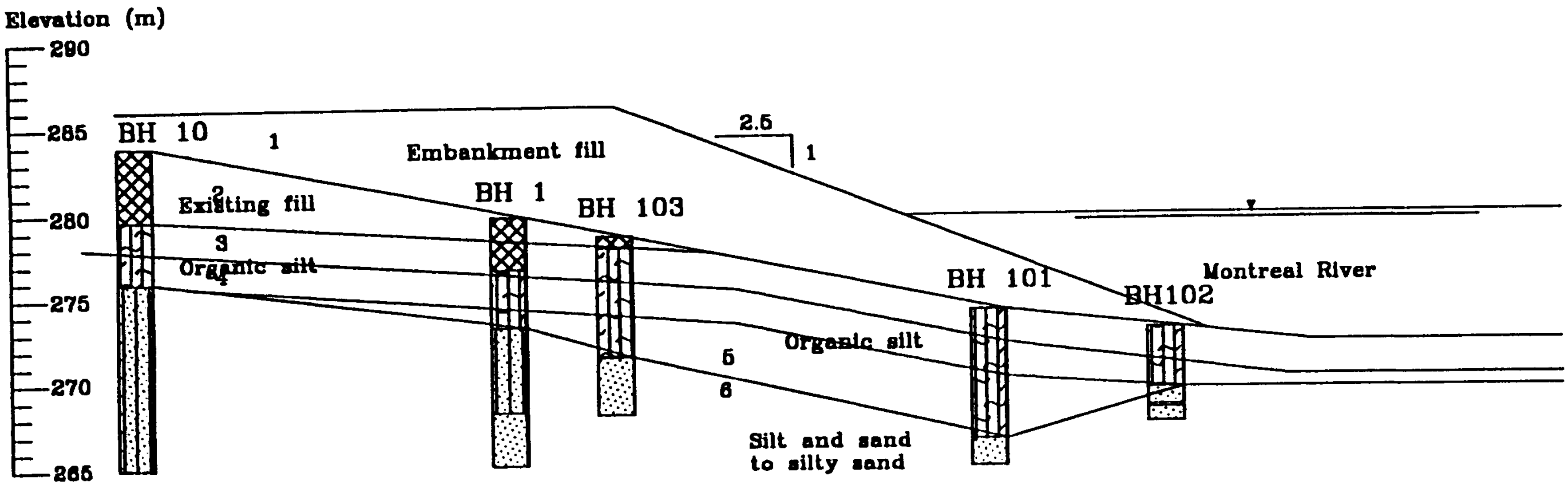


MONTREAL RIVER BRIDGE
WEST EMBANKMENT

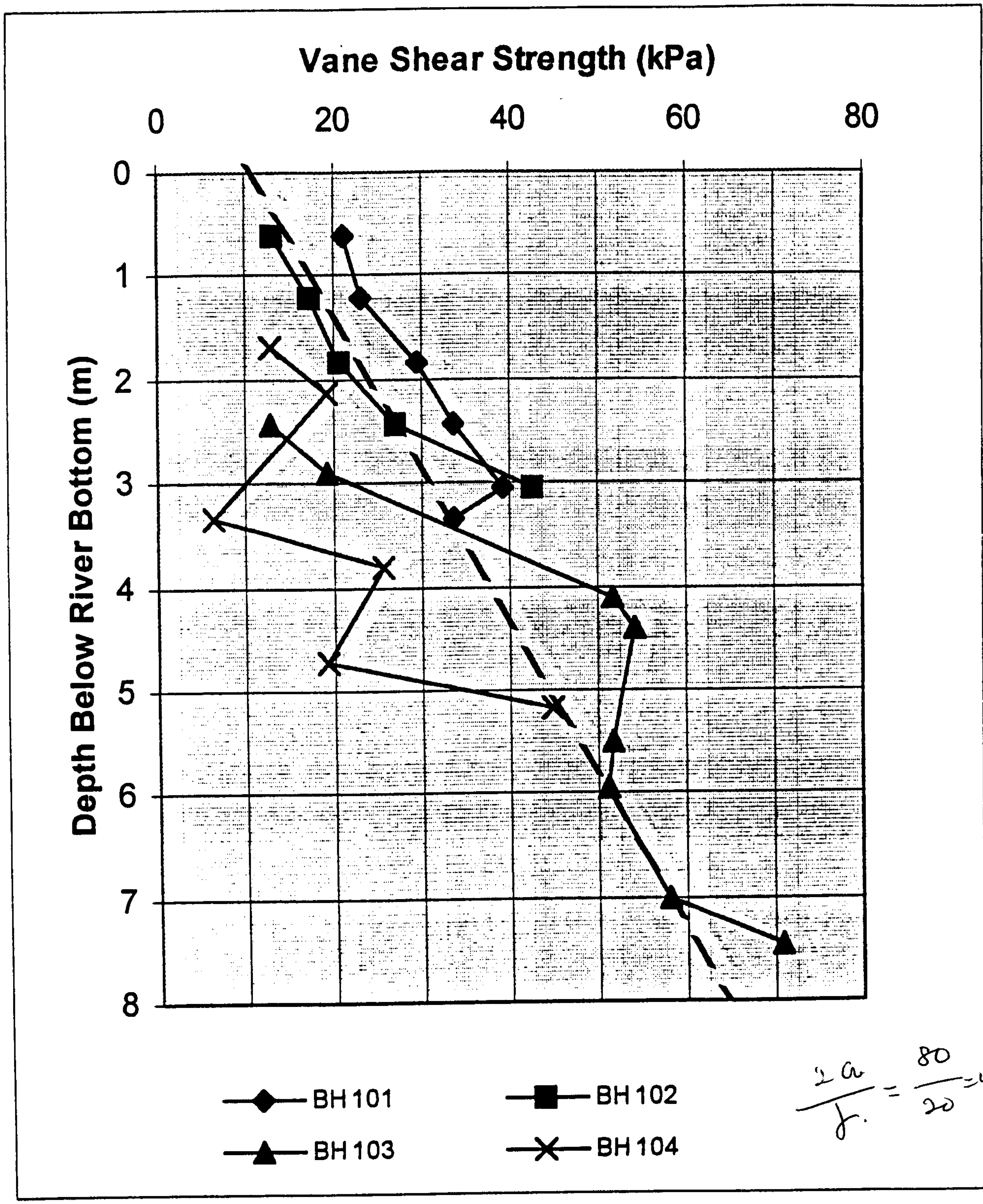
G-96.1202
FIGURE No. 1
March 1997



Note : Refer to Figure 1 for location of section in plan.

Scale: 1:400

MONTREAL RIVER BRIDGE WEST EMBANKMENT SIMPLIFIED SOIL PROFILE	G–96.1202 Figure 2 March 1997



$\frac{2a}{\gamma} = \frac{80}{20} = 4m$

Note: dotted line is the shear strength profile used in the stability analyses

MONTREAL RIVER BRIDGE WEST EMBANKMENT	G-96.1202 Figure 3 March 1997

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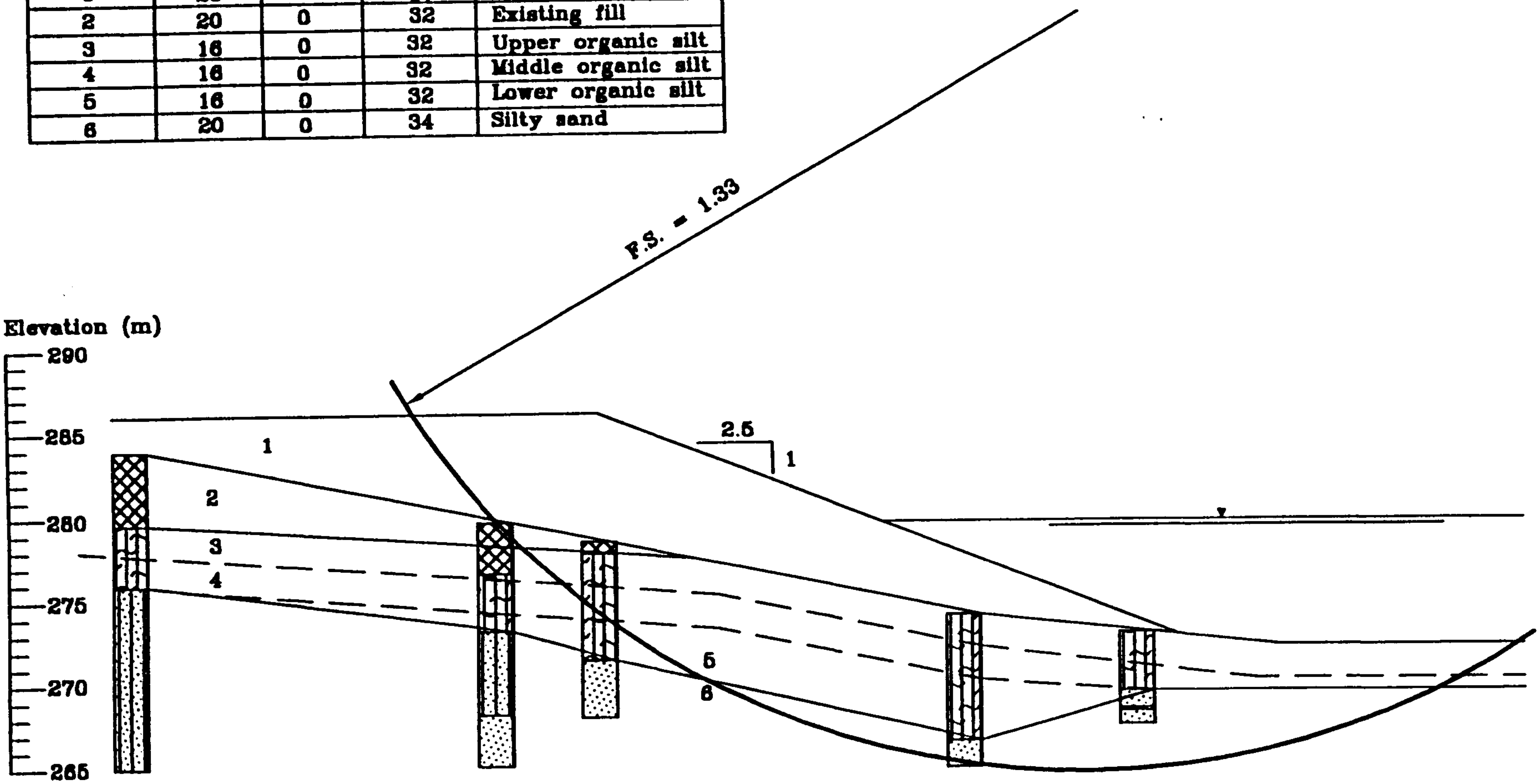
Stratum	γ kN/m³	Cu kPa	φ' degrees	Description
1	20	0	37	Proposed fill
2	20	0	32	Existing fill
3	16	0	32	Upper organic silt
4	16	0	32	Middle organic silt
5	16	0	32	Lower organic silt
6	20	0	34	Silty sand

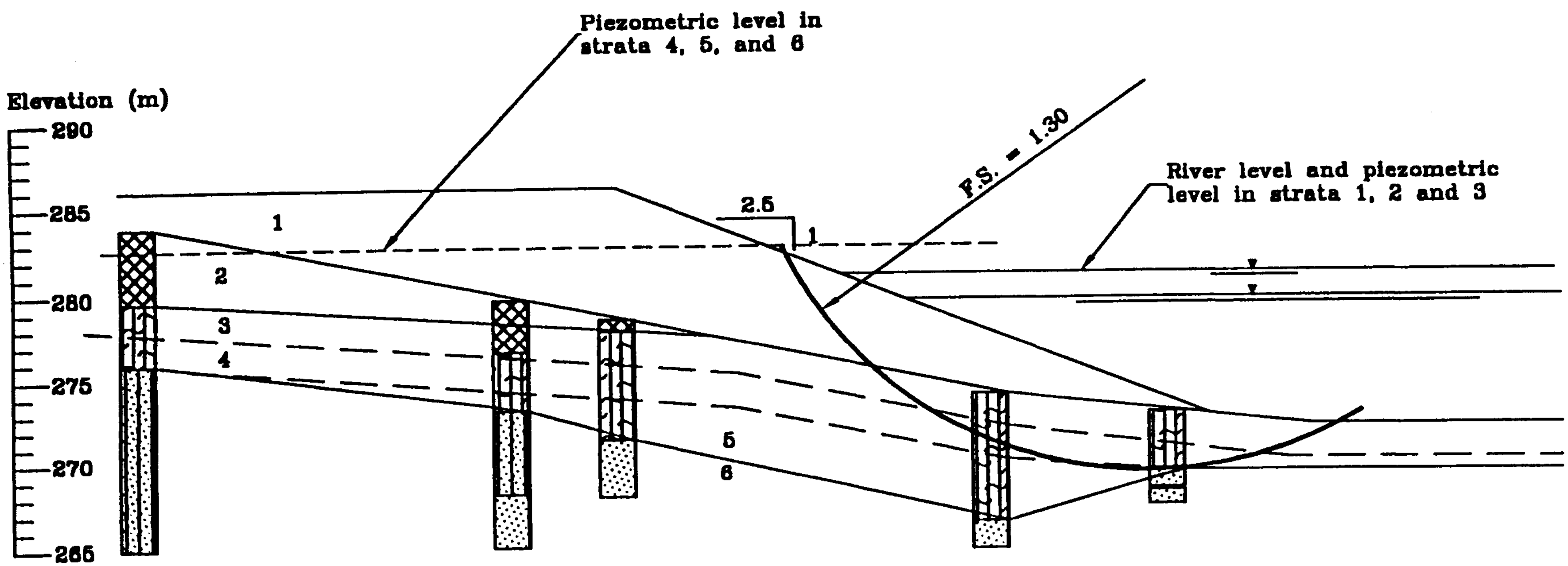


Scale: 1:400

MONTREAL RIVER BRIDGE WEST EMBANKMENT STABILITY ANALYSIS
LOCATION OF LONG TERM CRITICAL FAILURE SURFACE

G–96.1202
Figure 4
March 1997

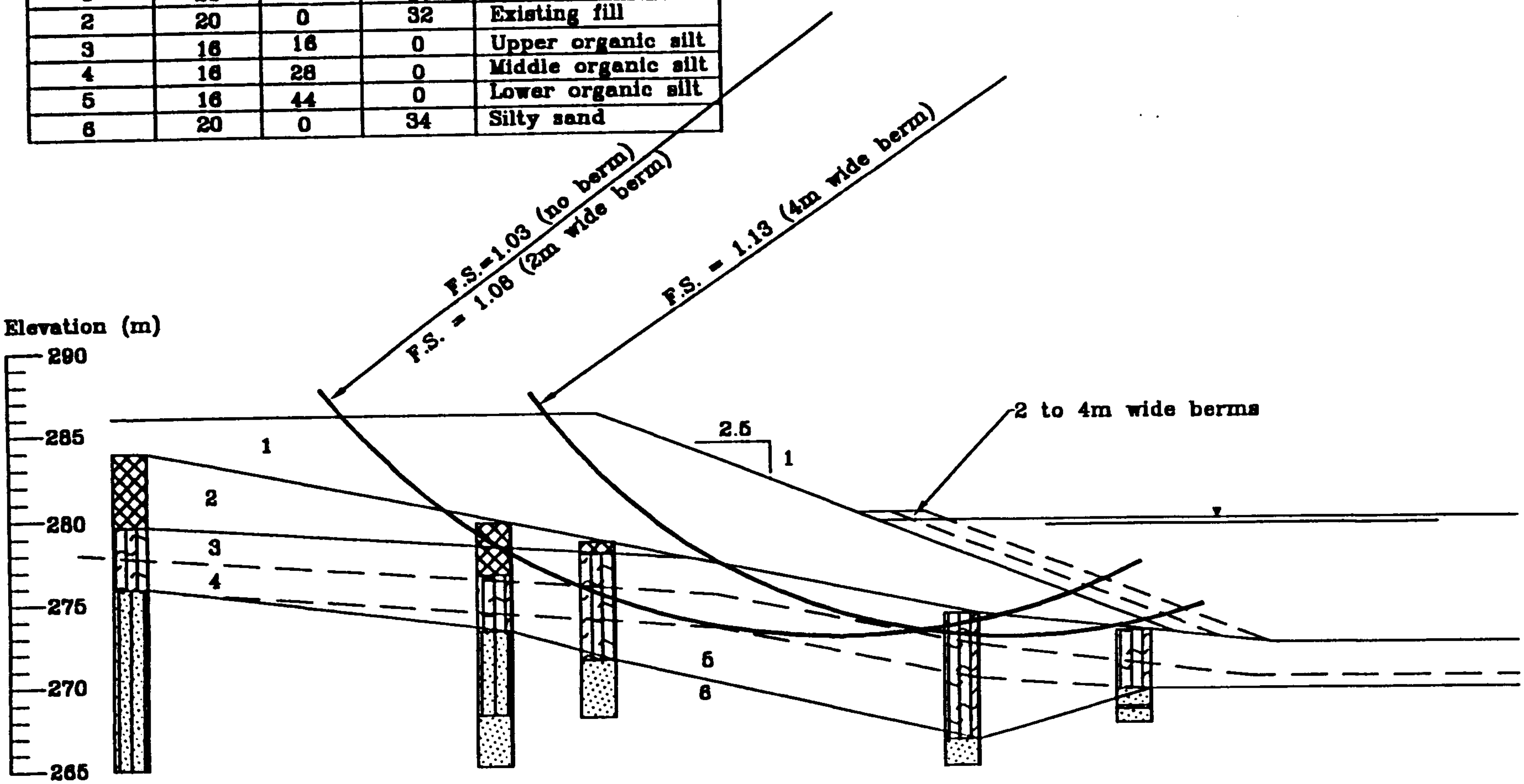
Stratum	γ kN/m³	Cu kPa	φ' degrees	Description
1	20	0	37	Proposed fill
2	20	0	32	Existing fill
3	16	0	32	Upper organic silt
4	16	0	32	Middle organic silt
5	16	0	32	Lower organic silt
6	20	0	34	Silty sand



Scale: 1:400

MONTREAL RIVER BRIDGE WEST EMBANKMENT STABILITY ANALYSIS STABILITY UNDER RAPID DRAWDOWN CONDITION	G-96.1202 Figure 5 March 1997

Stratum	γ kN/m³	Cu kPa	φ' degrees	Description
1	20	0	37	Proposed fill
2	20	0	32	Existing fill
3	16	16	0	Upper organic silt
4	16	28	0	Middle organic silt
5	16	44	0	Lower organic silt
6	20	0	34	Silty sand



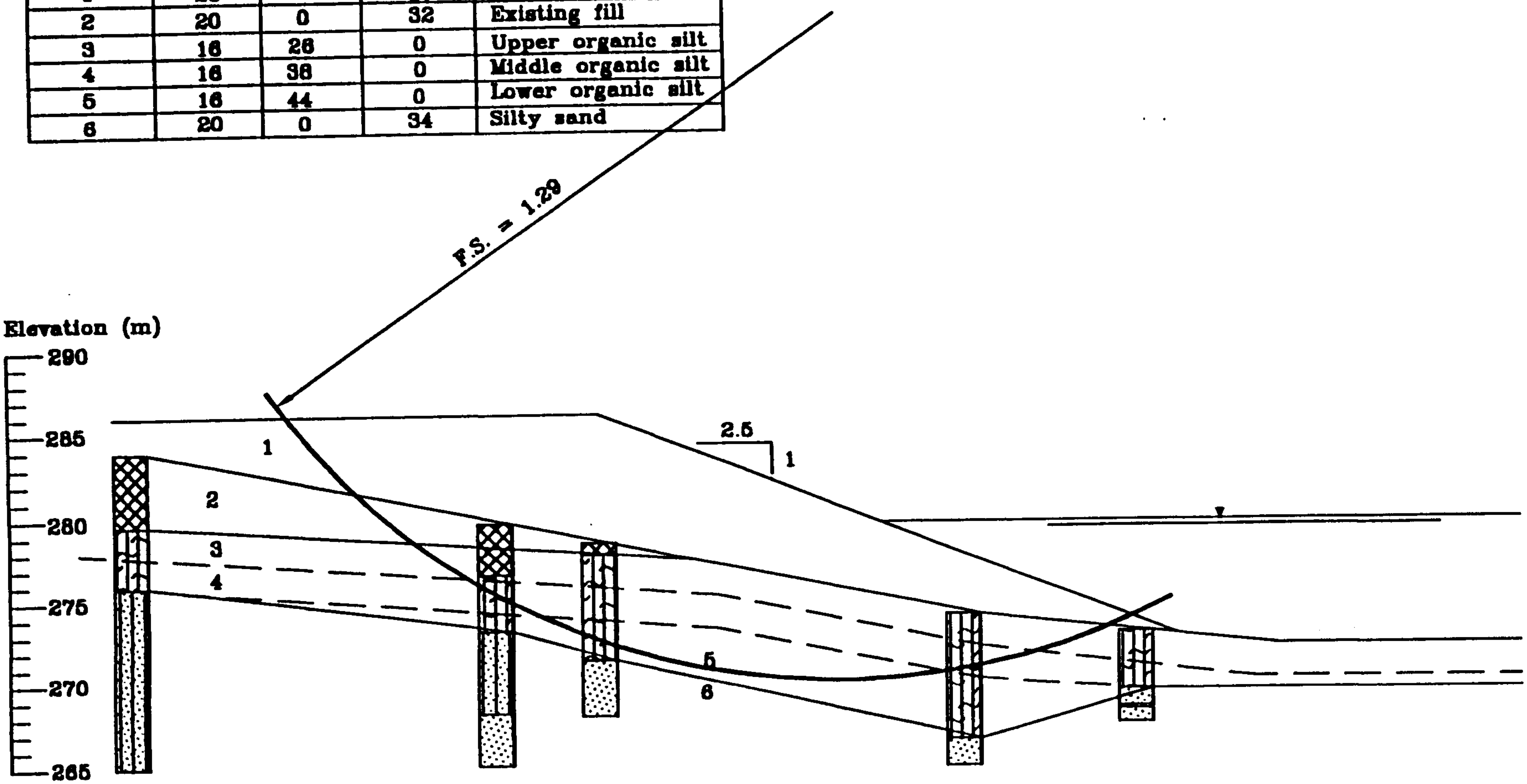
Key assumptions:
1. No increase in shear strength of organic silt
2. Single stage embankment construction

Scale: 1:400

MONTREAL RIVER BRIDGE WEST EMBANKMENT STABILITY ANALYSIS
LOCATION OF CRITICAL FAILURE SURFACES AT END OF CONSTRUCTION

G-96.1202
Figure 6
March 1997

Stratum	γ kN/m³	Cu kPa	φ' degrees	Description
1	20	0	37	Proposed fill
2	20	0	32	Existing fill
3	16	26	0	Upper organic silt
4	16	36	0	Middle organic silt
5	16	44	0	Lower organic silt
6	20	0	34	Silty sand



Assumption:
10 kPa increase in shear strength of organic silt
at end of construction

Scale: 1:400

MONTREAL RIVER BRIDGE WEST EMBANKMENT STABILITY ANALYSIS
LOCATION OF CRITICAL FAILURE SURFACE AT END OF CONSTRUCTION

G–96.1202
Figure 7
March 1997

APPENDIX "A"

APPENDIX
"A"
Statement of Limitation

The conclusions and recommendations in this report are based on information determined at the borehole locations. Soil and groundwater conditions between and beyond the boreholes may differ from those encountered at the borehole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the soil investigation.

The design recommendations given in this report are applicable only to the project described in the text, and then only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known to us, in our analysis certain assumptions had to be made. The actual conditions may, however, vary from those assumed, in which case changes and modifications may be required to our recommendations.

We recommend, therefore, that we be retained during the final design stage to review the design drawings and to verify that they are consistent with our recommendations or the assumptions made in our analysis. We recommend also that we be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the boreholes. In cases where these recommendations are not followed, the company's responsibility is limited to interpreting accurately the information encountered at the boreholes.

The comments given in this report on potential construction problems and possible methods are intended only for the guidance of the design engineer. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods and costs. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work.

APPENDIX B

Borehole Logs

LOG OF BOREHOLE 101

CLIENT : Armbro Construction
PROJECT : Montreal River Bridge
LOCATION : Elk Lake, Ontario
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Continuous sampling
Diameter : 50mm
Date : 1997.02.05

REF. NO. : G-96.1202
ENCL. NO. : B-1

(m) ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRATA PLOT	SAMPLES: NUMBER	TYPE	BLOWS / 0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X LAB VANE) (10 20 30 40 50)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
273.9	River Bottom										River Level at 279.2m
0.0	ORGANIC SILT some partly decomposed wood wet grey/brown soft to firm (possibly stiff at depth)		1	SS	Hand pushed			S=1.6 +		97	
			2	SS	Hand pushed			S=1.9 +		83	
			3	SS	Hand pushed		272	S=2.3 +		96	
			4	SS	Hand pushed			S=2.2 +		93	
								S=2.2 +			
			5	SS	Hand pushed			S=4.0 +			
			6	SS	Approx. 5-8		270				Could not push vane into organic silt below El 270m
			7	SS	Approx. 5-8						
			8	SS	Approx. 5-8		268			91	
			9	SS	Approx. 6-8					103	
266.4			10	SS	Approx. 5-8					81	0 42 50 8
7.5	SAND some silt wet loose.		11	SS	Approx. 6-10		266				0 58 34 8
			12	SS	Approx. 6-10						
264.8	some org. silt										
9.1	END OF BOREHOLE										

LOG OF BOREHOLE ___

CLIENT : Armbro Construction
PROJECT : Montreal River Bridge
LOCATION : Elk Lake, Ontario
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Continuous Sampling
Diameter : 50mm
Date : 1997.02.05

REF. NO. : G-96.1202
ENCL. NO. : B-2

(m) ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" BLOWS / 0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (+ FIELD VANE)	WATER CONTENT (%)	UNIT WEIGHT	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
272.8	River Bottom									
0.0	ORGANIC SILT some partly decomposed wood, wet, brown, soft to firm.	1	SS	hand pushed			S=2.0			River Level at 279.2m
		2	SS	hand pushed		272	S=3.0			
		3	SS	hand pushed			S=4.2			
		4	SS	hand pushed		270	S=3.4 S=3.3			
269.3		5 A / B	SS	hand pushed				94		0 51 43 6
3.5	SAND some silt, wet brown loose.	6 A / B	SS	Approx. 5–10				o		0 96 4 –
268.3										
4.6	ORGANIC SILT									
268.1 / 4.7	SAND some silt, wet, brown, loose.	7	SS	Approx. 5–10		268		o		
267.3										
5.5	ORGANIC SILT	8	SS					o		
266.7 / 6.1	END OF BOREHOLE									

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LOG OF BOREHOLE

CLIENT : Armbro Construction
PROJECT : Montreal River Bridge
LOCATION : Elk Lake, Ontario
DATUM ELEVATION : Geodetic

DRILLING DATA
Method : Hollow stem augering
Diameter : 200mm
Date : 1997.02.25

REF. NO. : G-96.1202
ENCL. NO. : B-3

(m) ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 M	ELEVATION SCALE	SHEAR STRENGTH kPa (+ field vane)	UNIT WEIGHT	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
278.4	River bottom							
277.6 / 0.0	SAND some silt, gravel brown very loose	1	SS	2	278			River level at 279.2m 0.3m ice
0.8	ORGANIC SILT some sand some partially decomposed wood brown very soft to stiff	2	SS	hand pushed			156	
		3	ST				159	Consolidation test – see Fig. 6 and 7
							81	
					276	S=5.0; S=3.8		0 32 63 5
		4	ST					Direct shear test – see Fig. 4 and 5
					274	51.5; 54.0		
	6.4 m	5	ST				82	
						51.5; 51.0		
271.2		6	ST		272	57.9; 70.8		
7.2	SILTY SAND some silt seams some 3 to 5 cm thick organics seams brown very loose	7	ST					0 72 28 –
		8	SS	4	270			
267.9		9	SS	4	268			
10.5	END OF BOREHOLE							

Geo-Canada Ltd.

LOG OF BOREHOLE 104

CLIENT : Armbro Construction	DRILLING DATA	REF. NO. : G-96.1202
PROJECT : Montreal River Bridge	Method : Hollow stem augering	ENCL. NO. : B-4
LOCATION : Elk Lake, Ontario	Diameter : 200mm	
DATUM ELEVATION : Geodetic	Date : 1997.02.26	

(m) ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRATA PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" BLOWS / 0.3 M	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 10 20 30 40 50)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%)	UNIT WEIGHT	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
277.7	Ground Surface										
0.0 277.2	SAND brown, very loose										Lake level 279.2m 0.5m ice
0.5	ORGANIC SILT some sand some partially decomposed wood brown very soft tofirm		1	SS	1						
							276	S=5.0 +			
								S=6.0 +			
			2	SS	<1						
								S= +			
							274	S=3.0 +			
			3	SS	<1						
								S=3.0 +			
272.7								S=2.3 +			
5.0	SILTY SAND some thin organic seams brown very loose		4	SS	3		272				
			5	SS	7						
							270				
268.7			6	SS	8						
9.0	END OF BOREHOLE										

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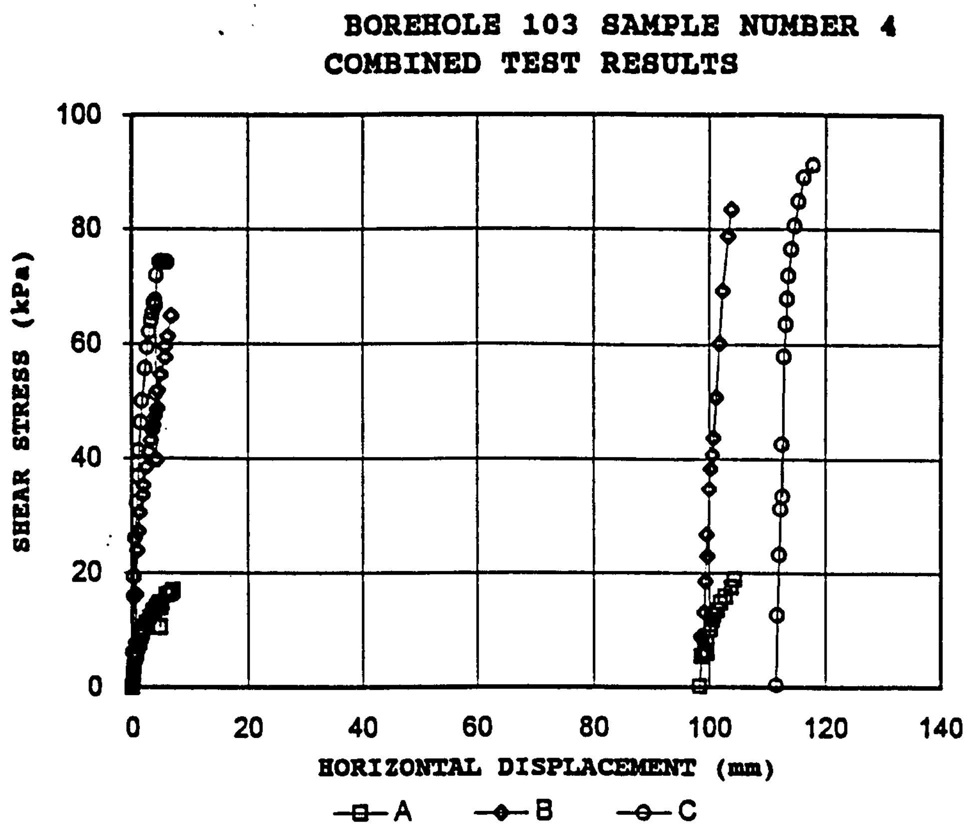
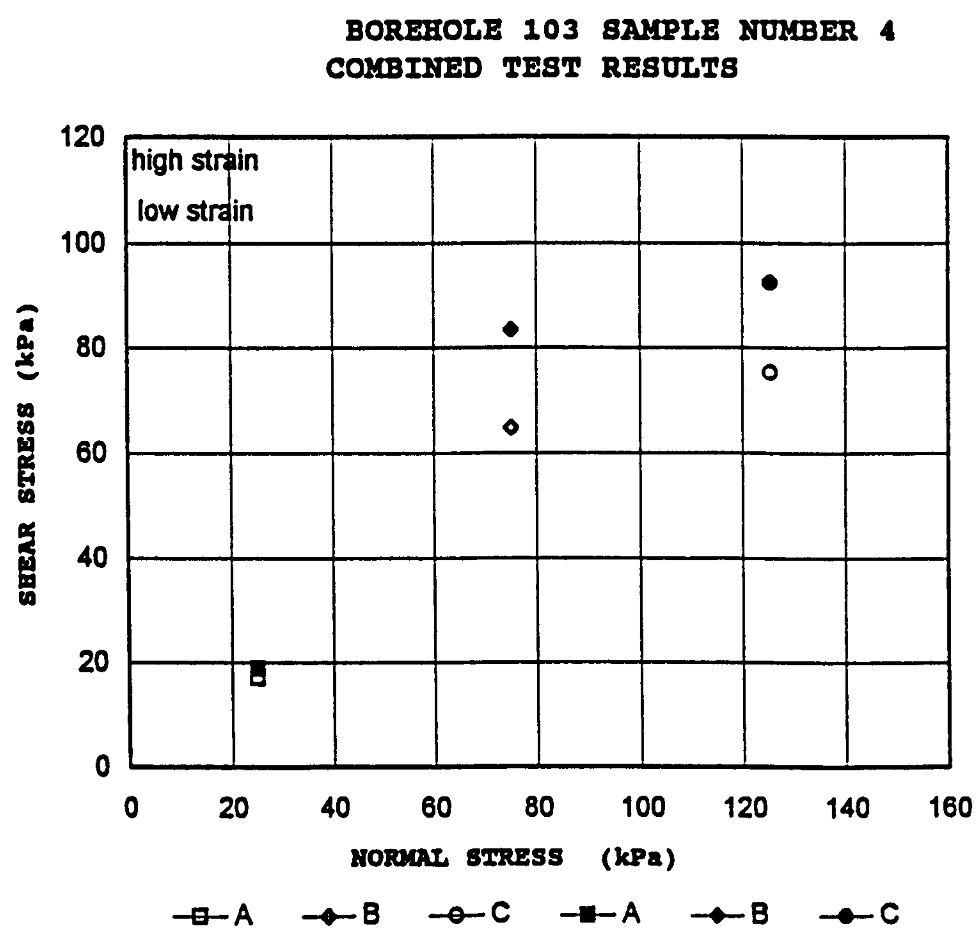
APPENDIX C

Laboratory Test Data

CONSOLIDATED DRAINED DIRECT SHEAR TEST	G-96.1202 Figure C3





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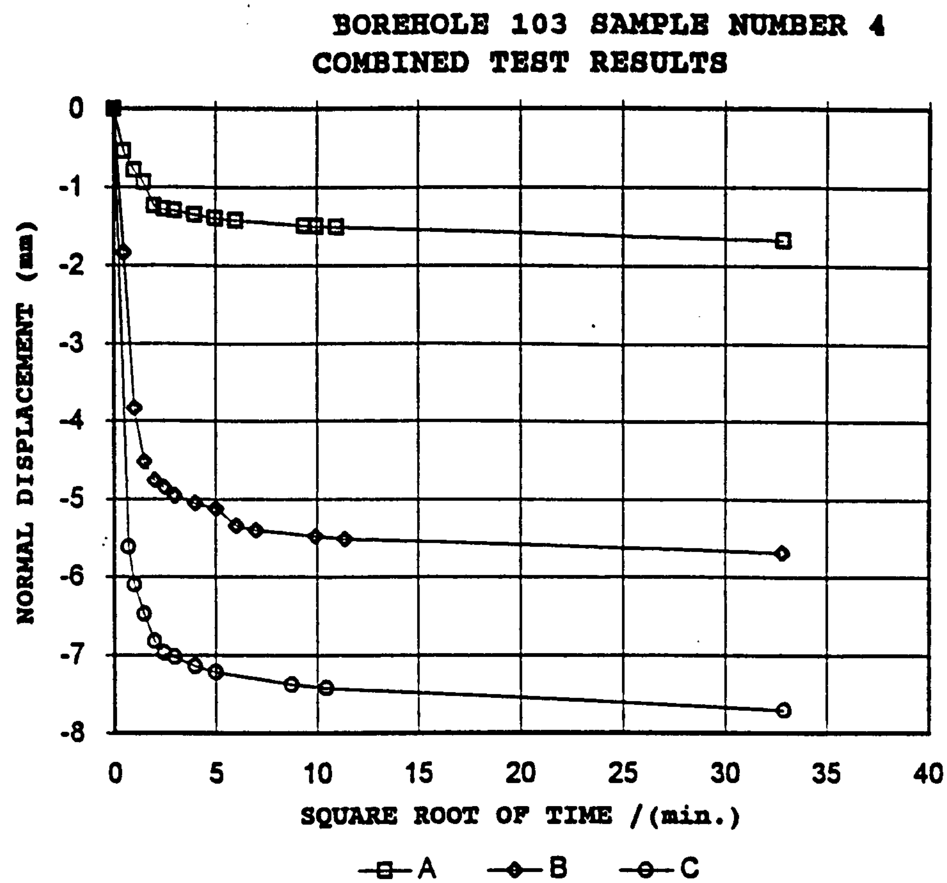
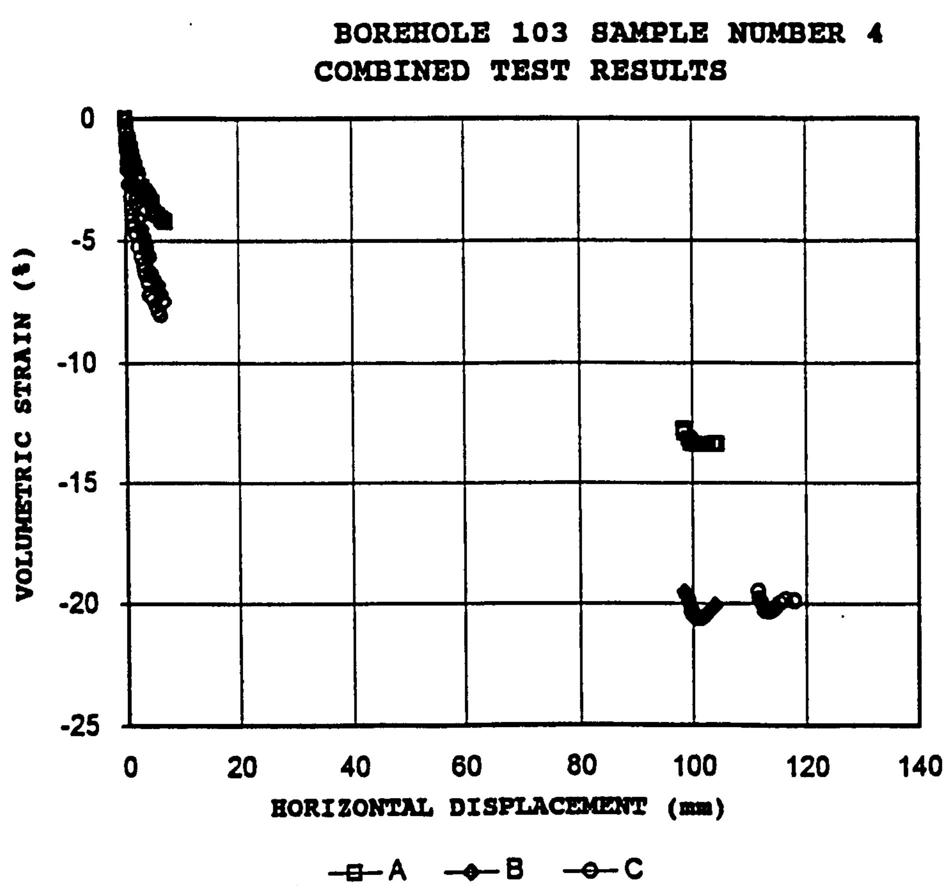
CONSOLIDATED DRAINED DIRECT SHEAR TEST	G-96.1202 Figure C4





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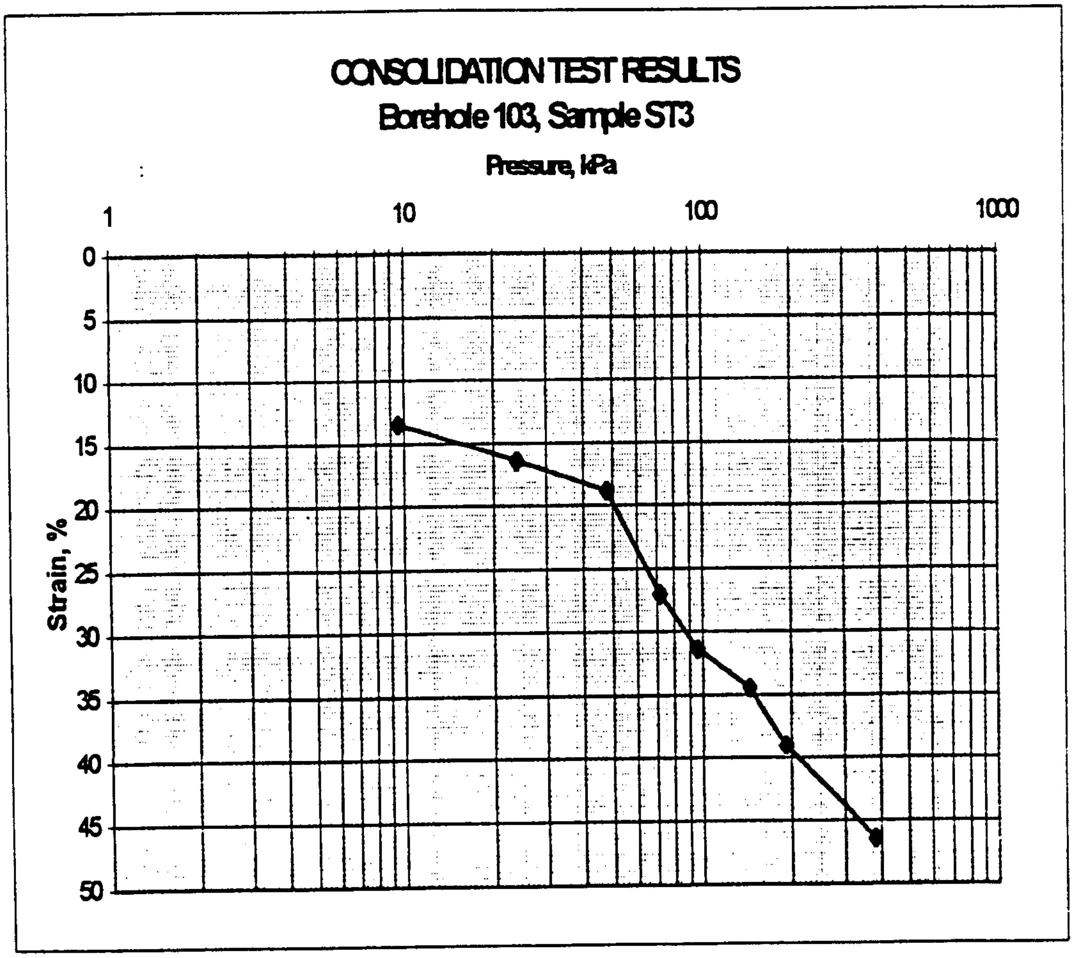

MONTREAL RIVER BRIDGE
WEST EMBANKMENT

G-96.1202
Figure C-5
March, 1997

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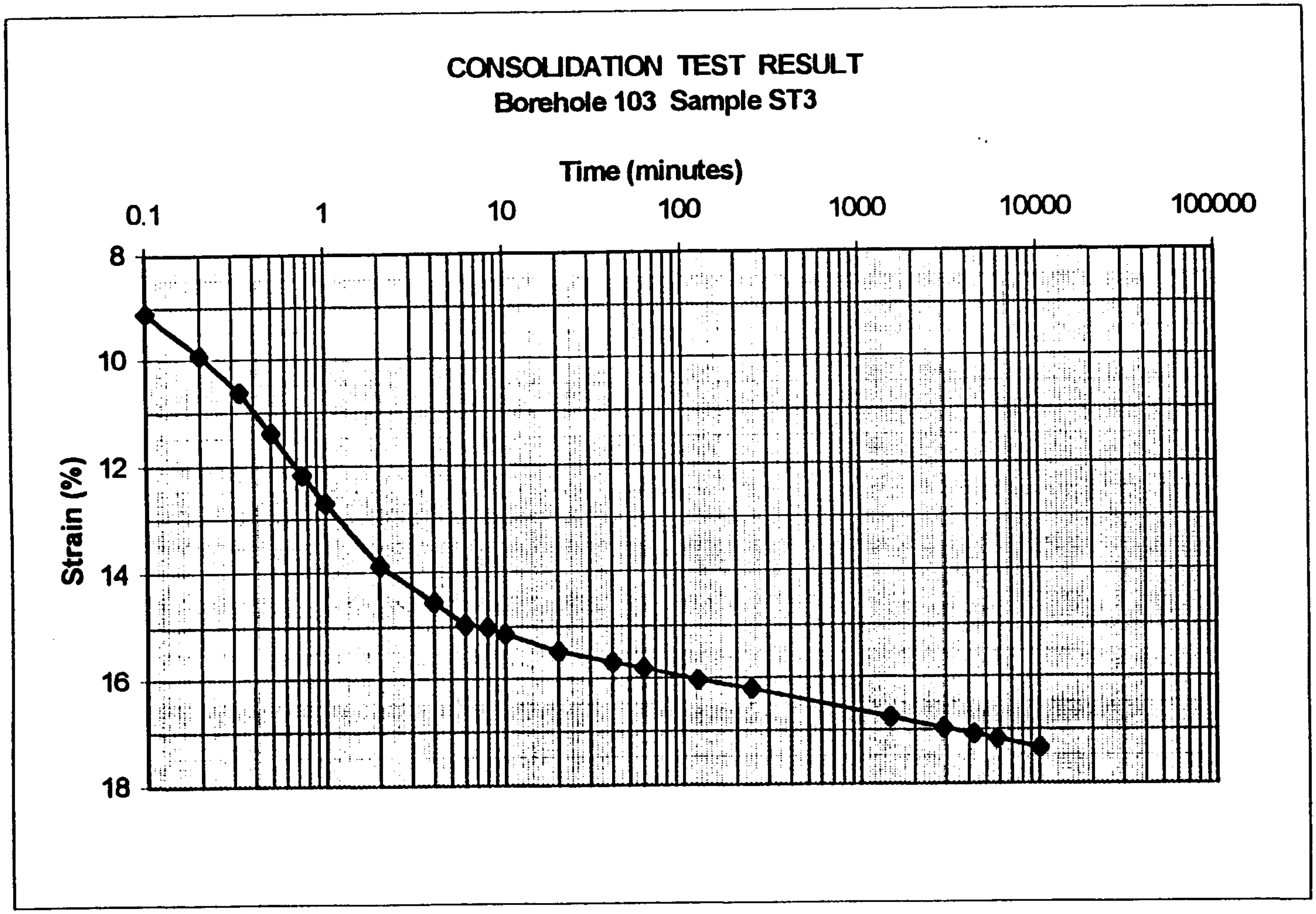

Pressure = 72 kPa

MONTREAL RIVER BRIDGE
WEST EMBANKMENT

G-96.1202
Figure C-6
March 1997