
GEOCRES No:
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**Foundation Investigation Report
Musquash River,
Northbound Lanes Bridge, Site 42-46N
W.P. 207-90-01 Highway 69, District 52
Huntsville, Ontario**

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Preface

Work Project 207-90-01 is one of a series of projects for the four lane expansion of Highway 69. This project is located from 0.4 km south of the Musquash River, northerly 8.9 km to Tower Road, within the MTO Northern Region, District 52, Huntsville.

It is located in the former Townships of Gibson, Freeman and on the Whata Mohawk First Nation Lands in the present Township of Georgian Bay, District of Muskoka. This project includes:

- the construction of new Southbound Lanes
- rehabilitation of the existing highway to divided freeway standards to become the Northbound Lanes
- construction of a replacement bridge over the Musquash River for the Northbound Lanes
- construction of a bridge over the Musquash River for the Southbound Lanes
- construction of a diamond interchange at the intersection of Cranberry Marsh Road and Highway 69
- construction of a bridge over the Moon River for the Southbound Lanes
- construction of associated side roads resulting from the creation of the controlled access highway
- construction of a diamond interchange at the intersection of Muskoka Road 12 and Highway 69

The following report comments on the foundation investigation and subsequent engineering recommendations for the Northbound Musquash River Bridge.

Other associated Geotechnical, Foundation and Pavement Reports for this project include:

- Foundation Investigation Report, Approach Embankments, Southbound Lanes, Musquash River, MTO Foundation Section, March 1993
- Pavement Design report, Trow Consulting Engineers Ltd., January 1998
- Foundation Investigation, Musquash River, Southbound Lane Bridge, Site 42-465, MTO Foundation Section, March 1993
- Foundation Investigation Report, Cranberry Marsh Road Interchange, Site 42-318, Trow Consulting Engineers Ltd., January 1998
- Foundation Investigation Report, Moon River, Southbound Lane Bridge, Site 42-265, Trow Consulting Engineers Ltd., January 1998

Forthcoming reports include:

- Foundation Investigation Report, Muskoka Road 12 Interchange, Trow Consulting Engineers Ltd., Spring 1998.
- Supplemental Pavement Design Report, Trow Consulting Engineers Ltd., Spring, 1998.

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PART 1 Foundation Investigation

1.1 Introduction

This submission presents the results of a geotechnical investigation completed by Trow Consulting Engineers Ltd. (Trow) for the Musquash River Bridge, WP 207-90-01, Highway 69, District 52, Gibson Township.

A new two lane, three span bridge has been proposed for the northbound lanes of Highway 69 at the Musquash River crossing. This report applies to the proposed bridge structure and the approaches within approximately 20 metres of the bridge abutments between stations 20+360± and 20+450±.

1.2 Site Description and Geological Setting

1.2.1 Site Description

The site is located in the former Gibson Township (District of Miskoka) along Highway 69 at the Musquash River (Stations 20+360± and 20+450±). Figure 1 shows the site location plan. The existing Highway 69 consists of a two lane road which carries both north and south bound traffic across the Musquash River on an existing three (3) span bridge. The centreline of the proposed new north bound Musquash River bridge is located approximately 25 m to 35 m west of the existing bridge centreline.

The terrain in the area of the proposed river crossing is moderately undulating consisting of rock outcrops of gneissic bedrock separated by intervening marshy zones and wooded areas. The Musquash River flows west toward Georgian Bay. At the time of the present investigation, the Musquash River water level was found to fluctuate substantially with an estimated elevation range of between 194.3 m and 196.5 m. The river width was between 40 metres and 50 metres from edge of water to edge of water.

The existing natural banks to the Musquash River at the location of the proposed new construction are generally less than 2 metres above the river level and sloped at a gradient between 2:1 and 2.5:1 horizontal to vertical. Photograph 1 shows the existing Musquash River bridge from the north approach embankment of the existing highway. Photograph 2 shows the north bank of the Musquash River from the south approach embankment of the existing highway.

1.2.2 Geological Setting

According to OGS Maps 2544 and 2556, the site is located in what is known as the central gneiss belt. The bedrock at the site consists of Precambrian gneisses of metasedimentary origin. As previously noted, the topography in the area is undulating consisting of bedrock outcrops. As such, the surface soils in the area consist of intervening shallow organic deposits (peat, muck and marl), and glaciofluvial deposits consisting of gravel, silt and sand including proglacial river and deltaic deposits.

1.3 Investigative Procedures

1.3.1 General

The fieldwork for the proposed northbound Musquash River Bridge was carried out in two phases between November 6 and November 14 (1997) and March 18 and March 24 (1998).

Between November 6 and November 14, 1997, eight (8) boreholes and seven (7) cone holes in total were advanced through the overburden soils at the site using equipment owned and operated by Master Soil Investigation Ltd. Drawing 1 in Appendix B shows the site plan and the borehole locations.

Considering the highly sloping bedrock surface at the bridge piers, an additional investigation was conducted at the site between March 18 and March 24, 1998, to obtain additional subsurface information at the pier locations. For this portion of the field work, one (1) borehole and four (4) cone holes were advanced at each of the north and south piers using equipment owned and operated by Longyear Canada Inc. Drawing 1 in Appendix B shows the locations of the second phase boreholes.

All fieldwork was supervised by a member of Trow's engineering staff who directed the drilling and sampling operations, logged the factual borehole data, and retrieved soil and rock core samples for subsequent laboratory testing and identification. Borehole elevations were provided by Mr. P. Collie of Dennis Consultants, a division of R.V. Anderson (R.V.A.) and are referenced to geodetic.

The following is a discussion of the field procedures used for the boreholes drilled at the location of the bridge abutments and bridge piers, respectively.

1.3.2 Field Investigation - North and South Bridge Abutments

The geotechnical investigation for the bridge abutments was carried out on November 12 and 14, 1997, and consisted of four (4) boreholes (Boreholes 1, 4, 5 and 8 on Drawing 1) which were advanced to depths ranging from 7 metres to 22.1 metres. Two (2) boreholes were drilled near each of the north and south abutments. In addition, three (3) cone holes were

advanced through the overburden soils near each of the north and south abutments and approach embankments. The borehole and cone hole locations are shown on Drawing 1 (Appendix B). Appendix C contains the detailed borehole and cone hole logs.

The boreholes for the north and south abutments were advanced through the overburden soils using a truck mounted CME-55 drill rig equipped with hollow stem augers. Soil samples were obtained using a standard 51 mm O.D. split spoon sampler in conjunction with Standard Penetration Tests (ASTM D1586) at approximately 0.75 metre and 1.5 metre intervals. The Standard Penetration N-values were recorded and used to provide an assessment of the relative denseness of the overburden soils at the site and the soil samples were used for identification and laboratory testing.

Field vane tests were conducted at selected depths to obtain an estimate of the undrained shear strength of the soft to firm clayey soils encountered at the site during the drilling program. In addition, thin wall Shelby tube samples of the clayey soils were retrieved for subsequent laboratory consolidation testing.

Conventional rock coring techniques were used to advance the boreholes approximately 4.0, 0.45, 4.1 metres into the underlying bedrock at boreholes 1, 5, and 8, respectively. Standard N-size casings and B-size core barrels were used and core samples of the bedrock were retrieved for rock quality determinations and classification purposes.

1.3.3 Field Investigation - North and South Bridge Piers

The initial geotechnical investigation for the bridge piers was carried out on November 6 and 8, 1997, and consisted of four (4) boreholes (Boreholes 2, 3, 6 and 7) which were advanced to depths ranging from 5.6 metres to 9.1 metres. Two (2) boreholes were drilled near each of the north and south piers as indicated on Drawing 1 (Appendix B). One cone hole (cone-7) was advanced near the centre of the north pier (see Drawing 1). The detailed borehole logs and cone hole logs are attached in Appendix C.

The boreholes for the north and south piers were advanced through the overburden soils using a D-25 (Deitric) drill rig equipped with hollow stem augers. Soil samples were obtained using a standard 51mm O.D. split spoon sampler in conjunction with Standard Penetration Tests (ASTM D1586) at approximately 0.75 metre and 1.5 metre intervals when practical. The Standard Penetration N-values were recorded and used to provide an assessment of the relative denseness of the overburden soils at the site. The soil samples were used for identification and laboratory testing. Photograph 3 shows the barge and drilling equipment used to drill at the pier locations.

Conventional rock coring techniques were used to advance Boreholes 2 and 3 approximately 3.0 metres into the underlying bedrock. Boreholes 6 and 7 were advanced 0.4 metres and 1.2 metres into the bedrock, respectively. Standard B size core barrels and N size casing was

used and core samples of the bedrock were retrieved for rock quality determinations and classification purposes.

1.3.4 Additional Field Investigation - North and South Bridge Piers

An additional geotechnical investigation for the north and south bridge piers was carried out between March 18 and March 24, 1998. The purpose of this field work was to obtain additional subsurface information at the pier locations in order to optimise the foundation design for the bridge piers. This portion of the foundation investigation consisted of two (2) boreholes (Boreholes 101 and 102) which were advanced to depths ranging from 9.7 metres to 10.4 metres. One (1) borehole was drilled near the centre of each of the north and south piers as indicated on Drawing 1 (Appendix B).

In addition to the boreholes advanced at the pier locations, four (4) cone holes (cones A, B, C and D) were advanced at the north pier and four (4) cone holes (cones E, F, G and H) were advanced at the south pier (see Drawing 1). The depth of the additional cone holes varied between 4.3 metres for Cone-G to 8.9 metres for Cone-A.

The boreholes for the north (Borehole 101) and south piers (Borehole 102) were advanced through the overburden soils using a Longyear 24 diamond drill equipped with standard B size casing and B size core barrel. Soil samples were obtained using a standard 51mm O.D. split spoon sampler in conjunction with Standard Penetration Tests (ASTM D1586) at approximately 0.75 metre and 1.5 metre intervals when practical. The Standard Penetration N-values were recorded and used to provide an assessment of the relative denseness of the overburden soils at the site and the soil samples were used for identification and laboratory testing. Photograph 4 shows the barge and drilling equipment used to drill at the pier locations for the second phase investigation.

Conventional rock coring techniques were used to advance Boreholes 101 and 102 approximately 3.2 metres into the underlying bedrock. Standard B size core barrels and B size casing were used and core samples of the bedrock were retrieved for rock quality determinations and classification purposes.

Detailed borehole and cone hole logs for the north and south pier are attached in Appendix C.

1.3.5 Laboratory

The soil samples which were obtained in the field were examined in the laboratory for further verification and classification. A laboratory testing program for select soil samples consisted of the following:

- Atterberg Limits
- Natural Moisture Contents
- Unit Weights

- Oedometer Consolidation

The laboratory test results are summarized on the attached Borehole Logs (see Appendix C) and are discussed further in Sections 1.4 and 2 of this report.

1.4 Subsurface Conditions

1.4.1 General

The subsurface conditions encountered during the field investigations at the site are summarized on the attached borehole logs and cone hole logs in Appendix C. In addition, Drawings 2 through 6, inclusive, summarize the subsurface conditions at the bridge site in stratigraphical form. The following is a description of the subsurface conditions encountered during the field investigation.

1.4.2 TOPSOIL

Topsoil was encountered in Boreholes 1 and 8 at the location of the south abutment and Borehole 5 at the location of the north abutment. The thickness of the topsoil varied from 175 mm at Borehole 5 to 225 mm at Borehole 1.

1.4.3 FILL

A surficial layer of loose fill 0.5, 0.2 m 0.5 metres thick was encountered during drilling of Boreholes 1, 4, and 8, respectively. The fill generally consisted of silty clay to clayey silt at Borehole 1, silty sand at Borehole 8 and gravel at Borehole 4.

1.4.4 Upper SAND

An upper deposit of fine, light brown loose to compact sand with trace to some silt was encountered between Elevation 197.1 metres and 195.0 metres at borehole 5. This upper sand deposit was not encountered during drilling of Boreholes 1 through 4, and 6 through 8 and was found to overly clayey silt to silty clay at the location of Borehole 5, only. Standard Penetration N-values were relatively consistent ranging from 15 to 16 blows/300 mm and the moisture content was found to vary between 19% and 28%.

1.4.5 SILTY CLAY to CLAYEY SILT (CL)

Stratified silty clay to clayey silt with thin silt layers or lenses was encountered in Boreholes 1, 4, 5, 6, 7 and 8 and in Boreholes 101 and 102. This soil layer was found to be very soft to firm with an undrained shear strength, c_u , ranging from 18.0 kPa at elevation 191.6 m in Borehole 8 to 42 kPa at elevation 192.8 m in Borehole 4. Figure 2 shows the undrained shear strength versus depth for this soil layer based on the results of field vane measurements. The

natural moisture content, w_n , ranged from a minimum of 20% for silt seams to a maximum of 56% for more cohesive samples. Atterberg limits were measured in the laboratory for 5 split spoon samples and the test results are summarized in Table 1-1. In general, the soil samples tested plot just above the A-line (see Figure 3) and are classified as low plasticity clays (CL) using the Unified Soil Classification System (USCS).

The thickness of the silty clay to clayey silt soil layer ranged from approximately 1.0 metre at Borehole 101 (El. 191.7m to El. 190.7 m) to approximately 8.8 metres at Borehole 8 (El. 195.4 m to El. 186.6 m).

Consolidation tests were carried out on soil samples obtained from the silty clay to clayey silt deposit on both sides of the river. The results of these consolidation tests are plotted on Figures 4 and 5.

The preconsolidation pressure of the clayey silt to silty clay deposit (CL) was found to range from approximately 80 kPa for sample TW-4 from Borehole 5 to 120 kPa for sample TW-5 from borehole 8. These preconsolidation values are consistent with the values reported in the Foundation Investigation Report for the southbound Musquash River Bridge¹. The re-compression index, C_r , for this deposit was measured to be in the order of 0.05 based on the two consolidation tests shown in Figures 4 and 5 while the compression index, C_c , was found to be relatively consistent at approximately 0.6.

Table 1-1 Summary of Atterberg Limits.

Borehole No.	Sample	LL (Liquid Limit)	PL (Plastic Limit)	Plasticity Index
BH1	SS-4	34	17	17
BH4	SS-2	53	22	31
BH5	SS-3	28	16	12
BH6	SS-2	22	16	6
BH-7	SS-2	26	15	11

¹ FOUNDATION INVESTIGATION REPORT for the Musquash River Bridge, W.P. 208-90-01, Site 42-46, Highway 69, District 11, Huntsville, by John A. Blair P.Eng. and M. Devata, P.Eng.

1.4.6 LOWER SILTY SAND to SANDY SILT - with some GRAVEL (TILL)

Silty sand to sandy silt with some gravel, and occasional cobbles (TILL) was found below the silty clay to clayey silt soils at the site. Based on Standard Penetration Test N-values, this soil layer was generally loose to compact. N-values were generally in the order of 2 to 15 blows/300mm with occasional higher N-values in zones with gravel and/or cobbles. The moisture content was found to range from 11% to 22%.

Grain size distributions were obtained for select soil samples taken from this layer at Boreholes 1, 2, 3, 4, and 8. The results are presented on the attached borehole logs and are shown on Figures 6 through 10, inclusive. This soil layer was found to be predominantly sand and was generally found to overly bedrock at the site with a thickness ranging from 0.6 m at Borehole 3 (between Elevation 190.2m and 189.3 m) to 9.4 m at Borehole 8 (between Elevation 186.6 m and 177.2m).

1.4.7 BIOTITE-HORNBLENDE GNEISS

The bedrock was cored at boreholes 2, 3, 5, 7, 8, 101 and 102. RQD and core recovery was logged in the field and the cores were returned to Trow for identification and classification purposes. The bedrock at the site was found to be predominantly light grey to greyish black with occasional reddish seams, medium grained, unweathered, strong, hard to very hard biotite-hornblende gneiss.

RQD values ranged from 72% for rock core (RC) sample number 2 of Borehole 5 to 100%. Based on RQD values, the rock mass quality is good to excellent. A RQD value of 39% was obtained for rock core (RC) sample 3 of Borehole 3; however, the core length was short and some fractures appeared to be mechanically induced. Fractures were close to widely spaced and were generally oriented at approximately 30 - 60° to the core axis.

In general, the bedrock surface was found to be steeply sloping and highly erratic at the site. At the south abutment location, the bedrock slopes at a gradient of 0.8H:1V or more (see Drawing 3 in Appendix B). At the location of both the south and north piers, the bedrock was found to dip steeply towards the northeast (see Drawings 4 and 5). The bedrock profile was found to be less erratic at the north abutment (see Drawing 6).

1.5 Groundwater Conditions

Stabilized groundwater levels were measured in Borehole 1 near the south abutment and Borehole 4 near the north abutment after the completion of drilling (Between November 12 and 14, 1997). The water level was found to be 0.35 m below ground surface (Elevation 195.25 m) at Borehole 1 and 1.25 m below ground surface (Elevation 194.3 m) at Borehole 4. Measurements show that the groundwater table was between 0.35 m and 1.25 m below the ground surface and at or slightly above the water level of the Musquash River. The average water level of the Musquash River at the time of the initial field investigation and groundwater level readings (November 6 to 14, 1997) was 194.3 m.

Part 2 Engineering Discussions and Recommendations

2.1 Foundation - Design

It is anticipated that the approach grades adjacent to the bridge abutments will be raised by approximately 3 m on average with some sections immediately adjacent to the river banks and proposed abutment areas being raised by up to 5 metres. The settlements of the approach embankments within approximately 20 m of the abutments have been estimated in addition to the effects of the embankment construction on the abutment foundation capacities.

Since the soils overlying bedrock at the site are generally weak, spread footings founded within the overburden soils are not considered feasible given the load and serviceability requirements for the abutments. To support the bridge loads, both driven piles and caisson options have been considered for the north and south abutments. For the north and south piers, five alternative foundation options have been considered. These options include driven piles, caissons, driven or socketed piles combined with a spread footing on bedrock, and spread footings placed on a mass concrete fill. There are a number of significant practical issues and potential problems associated with the various foundation alternatives given the relatively poor overburden soil conditions and the steeply sloping bedrock at the site. The issues and problems associated with each foundation alternative are discussed in the following Sections.

2.2 North and South Abutments

2.2.1 Option 1 - Piled Foundations

For piles driven to end bear on bedrock or within the silty sand (till) at the abutment areas, the following Limit States design values may be assumed in accordance with the Ontario Highway Bridge Design Code (O.H.B.D.C.):

Table 2-1 Design Pile Capacities

	HP 310x79	HP 310x110
Factored Geotechnical Capacity at ULS	1150 kN	1600 kN
Ultimate Capacity for Hiley Formula	2475 kN	3450 kN

The geotechnical capacity at SLS does not apply to piles end bearing on bedrock. Based on the attached borehole logs, Table 2-2 shows a summary of the interpreted end bearing elevations at the borehole locations at or below which piles will be founded. Drawings 3 and

in Appendix B shows an interpreted cross-sectional profile of the subsurface conditions for the north and south abutments, respectively.

Table 2-2 End Bearing Elevations for Pile and Caisson Foundations

Borehole Number	Location on Structure	End Bearing Elevation (m)
BH-1	South Abutment	188.6 m
BH-8	South Abutment	177.2 m
BH-4	North Abutment	188.3 m
BH-5	North Abutment	190.8 m
BH-2	South Pier	191.8 m
BH-7	South Pier	186.4 m
BH-3	North Pier	189.2 m
BH-6	North Pier	187.6 m
BH-101	North Pier	189.4 m
BH-102	South Pier	189.6 m
Cone A	North Pier	187.6 m
Cone B	North Pier	188.7 m
Cone C	North Pier	190.2 m
Cone D	North Pier	189.1 m
Cone E	South Pier	187.9 m
Cone F	South Pier	187.2 m
Cone G	South Pier	192.3 m
Cone H	South Pier	191.8 m

It should be noted that the elevations given in Table 2-2, above, are approximate. Furthermore, based on the soil borings, the bedrock elevation at the location of the Musquash River bridge is highly variable and are expected to change sharply over very short distances.

Based on the preliminary design grades provided to Trow, it is understood that the grades immediately adjacent to the abutments will be raised by up to 5 metres. The resulting net load increase applied to the overburden soils will be approximately 90 kPa (assuming a unit weight of 18.0 kN/m³ for the approach embankment fill). Under these loading conditions, settlements of the approach embankments are expected near the abutments resulting from consolidation of the soft clayey subsurface soils, and consequently, down drag forces will be generated on the piles.

Since it will be difficult to control the consolidation settlements of the clayey silt to silty clay soils encountered at the site, it is recommended that the pile load capacities listed in Table 2-1 be reduced by 10% to account for down drag forces on the piles.

Given the relatively soft overburden soils at the abutment locations and the potential inability to develop significant pile sockets into the bedrock (see Section 2.4.3), the lateral load capacity of vertically driven H-piles will be small. As such, all lateral loads at the abutment locations should be supported using battered piles. In addition, it will be necessary to ensure that the maximum pile batter is compatible with the slope of the bedrock surface. Pile orientation must be reviewed once the foundation design has been finalized.

2.2.2 Option 2 - Caissons

Caissons have also be considered for this project; However, caissons may not be the most economical and practical method of constructing the foundation units. For caissons founded in unweathered to slightly weathered gneissic bedrock at or below the interpreted elevations given in Tables 2-2, the Factored Axial Capacity at ULS provided in Table 2-3 may be assumed in accordance with the O.H.B.D.C.:

Table 2-3 Summary of Caisson Load Capacities

	Caisson Diameter (m)		
	0.91	1.07	1.22
Factored Geotechnical Capacity at ULS	8000 kN	10750 kN	14000 kN

The geotechnical capacity at SLS will not govern for caissons founded on bedrock since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended design values in Table 2-3.

To mobilize significant lateral load capacity, the caissons must be socketed into unweathered or slightly weathered gneiss bedrock. The socket depth should not be less than one half of the caisson diameter at any point around the circumference of the caissons. This will result in the founding elevation being slightly lower than shown in Table 2-2. Coring the bedrock surface to obtain an adequate socket depth will be complicated by the hardness of the bedrock, the sloping bedrock surface which may contain rock ledges and the relatively thin soft overburden soils encountered at some borehole locations (see Drawings 3, 4, 5 and 6 in Appendix B). If rock ledges are encountered during caisson construction, the coring effort required to satisfy the socket depth may be significantly increased and in some cases coring the bedrock may not be possible.

Alternatively and/or additionally, the base of the caissons may have to be doweled-in at the bedrock surface to provide sufficient lateral resistance. This may be time consuming. A non-standard special provision (NSSP) will need to be included in the contracts for this foundation alternative.

In general, the steeply sloping and hard to very hard bedrock surface will make caisson construction difficult. The installation of caissons will be further complicated since they must penetrate a significant depth of loose sand below the groundwater table. Groundwater infiltration may have to be controlled using drilling mud or other suitable techniques making

*Caissons
option is
not selected
anyway!!
difficult*

it difficult to properly clean and inspect the caissons prior to concreting. It is unlikely that this option will be competitive with pile foundations for the north and south abutments.

2.3 North and South Piers

2.3.1 General

The foundation conditions at the north and south piers are considered to be difficult due to the highly erratic bedrock profile at the pier locations, the hard to very hard bedrock, and the relatively thin soft overburden soils at the east end of the north and south piers (see Drawings 4 and 5). The following is a discussion of the foundation alternatives which are considered to be feasible. In general, the final foundation alternative adopted for the bridge piers will be dictated by cost as each of the foundation alternatives discussed below are feasible but with some associated risks and/or practical difficulties.

2.3.2 Option 1 - Piled Foundations

At the location of the south pier, the thickness of the overburden varies from 0.6 metres at Borehole 2 to 3.6 metres at Borehole 7. Similarly, the thickness of overburden soils overlying bedrock at the north pier varies from 0.9 metres at Borehole 3 to 4 metres at Borehole 6. Drawings 4 and 5 (Appendix B) show the subsurface soil profiles at the north and south piers.

Due to the sloping bedrock and the lack of sufficient overburden soils at the east end of the north and south piers, it is recommended that the grade be raised at these locations to provide lateral support for piles during pile driving and seating. Drawing 7 (Appendix B) illustrates a suggested method of construction for both the north and south piers which is considered to be practical. The following is a brief description of the construction procedure:

1. construct a cofferdam by driving sheet pile to refusal on the underlying bedrock around the perimeter of each pier foundation (see Drawing 7),
2. backfill the sheet pile cofferdam with granular material,
3. drive piles through the backfill and overburden to end bear on the underlying bedrock,
4. construct a pile cap which also incorporates the sheet pile.

The pile load capacities listed in Table 2-1 for the abutments also apply to piles driven at the north and south pier locations. Given the relatively soft overburden soils at the pier locations, the lateral load capacity of vertically driven H-piles will be small. As such, all lateral loads at the pier locations should be supported using battered piles. The dimensions of the sheet pile cofferdam must be selected to accommodate battered piles. Table 2-2 contains a summary of the interpreted end bearing elevations for driven piles at the pier locations. In

addition, it will be necessary to ensure that the maximum pile batter is compatible with the slope of the bedrock surface. Pile orientation must be reviewed once the foundation design has been finalized. If this option is chosen, frost protection will be required for the pier pile caps, likely using insulation in conjunction with nominal soil cover.

The sheet pile cofferdam must be designed as per section 6 of the Ontario Highway Bridge Design Code to resist lateral earth pressures and unbalanced hydrostatic pressures. The soil properties to be used for the design are listed in Table 2-4 below. Depending on the stiffness of the internal bracing and the sequence of construction, the coefficient of earth pressure at rest should be used for the granular backfill to assess earth pressures rather than the active earth pressure coefficient. A stiff bracing scheme will limit displacements of the box structure and prevent mobilization of active earth pressures within the granular backfill.

The size of the sheet pile box structure will be determined by the number of piles required for the pier foundations. If a large number of piles are required, this box structure could be excessively large adversely affecting the river hydraulics. The affect of this foundation alternative on the river hydraulics will need to be assessed.

Table 2-4 Material Types and Unfactored Properties for Sheet Pile Design.

Soil Type	Unit Weight (kN/m ³)	Friction Angle (degrees)	Active Earth Pressure Coefficient, K _a	Passive Earth Pressure Coefficient, K _p	Coefficient of Earth Pressure at Rest, K _o	Horizontal Modulus of Subgrade Reaction, k, (kN/m ³)
Clayey Silt to Silty Clay (CL)	17.5	27	0.38	2.66	0.55	8,000
Native Silty Sand (till)	20.0	30	0.33	3.0	0.43	25,000
Granular 'A' Backfill	22.5	35	0.27	N/A	0.5	40,000
Granular 'B' Backfill	21.5	30	0.33	N/A	0.5	40,000
Rock Fill	18.0	35	0.27	N/A	0.43	40,000

2.3.3 Option 2 - Combination Spread Footing and Pile Group

A combined spread footing and pile group could be considered for both the north and south pier foundations. A spread footing could be constructed directly on bedrock at the east (upstream) end of the piers (see Drawings 4 and 5 in Appendix B). At the west (downstream) end of the piers, piles could be driven through the overburden soils to bedrock forming a pile

group. A grade beam could be used to bridge between the spread footing on the upstream end of the piers and the pile group on the downstream end of the piers. For spread footings founded directly on the unweathered to slightly weathered bedrock, the following Limit States design values may be assumed in accordance with the O.H.B.D.C.:

Table 2-5 Spread Footing Capacity on Bedrock

	Spread Footing
Factored Geotechnical Capacity at ULS	5000 kPa

The above Factored Geotechnical Capacity at ULS applies to spread footings placed directly on bedrock with a good Rock Mass Quality ($RQD > 75$). The geotechnical capacity at SLS will not govern for a spread footing founded on bedrock since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS. Footing sliding resistance and inclined footing loads are addressed in Section 2.4 of this report.

The pile load capacities listed in Table 2-1 for the abutments also apply to piles driven to end bearing on bedrock at the north and south pier locations. Given the relatively soft overburden soils at the pier locations, the lateral load capacity of vertically driven H-piles will be small. As such, all lateral loads at the pier locations should be supported using battered piles. As previously noted in this report, it will be necessary to ensure that the maximum pile batter is compatible with the slope of the bedrock surface. Pile orientation must be reviewed once the foundation design has been finalized.

If this foundation option is chosen, pile bent construction may be considered. Once the piles have been installed at the piers, it is recommended that hollow steel tubes with diameters slightly larger than the piles be pushed over the piles and into the bed of the river below the scour depth. The space between the pile and the hollow steel tube must then be thoroughly cleaned of all debris and concrete tremied in to fill the annular space.

The combined footing and pile group foundation option may be the most economical foundation method for the bridge piers. However, due to the sloping bedrock surface and the relatively soft thin overburden soils (approximately 3 to 5 metres) at the downstream (west) end of the piers, it will be more difficult than usual to properly seat piles on the sloping bedrock surface. At some locations, the piles will have a greater than usual tendency to skip over the bedrock surface resulting in alignment problems and deeper penetration. Somewhat longer piles may be required, and in some cases, a good portion of the piles may have to be replaced or additional piles may need to be added if proper seating cannot be achieved. Given the greater than normal tendency for pile skip to occur, additional costs associated with re-driving, replacing, and adding piles should be considered in addition to the additional costs associated with redesign of the pile cap and other elements of the foundation. Pile skip will be discussed further in Section 2.4.3 of this report.

2.3.4 Option 3 - Concrete Filled Steel Pipe Piles Socketed into Bedrock.

Alternatively, small diameter hollow steel pipe piles could be considered for the piers. With this foundation alternative, the bedrock should be cored or drilled using suitable methods to provide a socket for grouting open ended hollow steel pipe piles into the bedrock socket. The hollow steel pipe piles could be used for: (a) the entire pier foundation or (b) for a pile group at the west end of the piers combined with a spread footing at the east end of the piers.

If this alternative is chosen, the socket depth of the piles should be no less than 600 mm into bedrock. The size of the pipe pile should be chosen such that the coring effort required to install the piles is practical. However, some difficulty should be expected during coring due to the hard to very hard bedrock at the site and the steeply sloping and erratic bedrock profile. If pullout resistance is required, the pipe piles should be anchored into bedrock to provide adequate uplift resistance.

During construction of the bedrock socket, steel casing should be installed into the socket to prevent loose debris from entering into the socket hole. Prior to setting the steel pipe pile into the socket, the base of the socket hole must be cleaned using air lift or other suitable methods. The pipe pile can then be inserted into the socket hole and tapped lightly with a pile driving hammer to prove the pile capacity prior to grouting the pile into the socket. This should be practical since pile driving equipment will likely be on site for construction of the north and south bridge abutments. The annular space between the bedrock and pile must then be grouted and concrete can be tremied in to fill the pipe pile. The bedrock socket should be constructed so that the side walls of the socket are rough in order to mobilize any available side wall friction between the concrete and bedrock socket.

For hollow steep pipe piles socketed into bedrock, the following Limit States design values may be assumed in accordance with the Ontario Highway Bridge Design Code (O.H.B.D.C.):

Table 2-6 Design Pile Capacities - Socketed Steel Pipe Piles

	<u>193 mm x 9.5 mm</u>	<u>245 mm x 11 mm</u>	<u>245 mm x 13.8 mm</u>
Factored Geotechnical Capacity at ULS	1800 kN	2885 kN	2885 kN
Geotechnical Capacity at SLS	1000 kN	1450 kN	1800 kN

Given the relatively soft overburden soils at the pier locations, the lateral load capacity of vertical piles will be small. As such, all lateral loads at the pier locations should be supported using battered piles. As previously noted in this report, it will be necessary to ensure that the maximum pile batter is compatible with the slope of the bedrock surface. Pile orientation must be reviewed once the foundation design has been finalized.

2.3.5 Option 4 - Mass Concrete Fill

It is understood that excavation of the overburden soils combined with mass concrete fill was used to construct the south pier for the existing Musquash River Bridge. Similar bedrock conditions were encountered at this location. This foundation option may be considered, however, the depth of excavation required at the west (downstream) end of the piers will be in the order of 4 m through soft and loose overburden deposits.

Given the depth of excavation at the downstream end of the piers and the sloping bedrock, this option will be difficult. Some scheme such as a sheet pile box structure will be required to limit the size of the pier excavations. This structure may need to be braced internally and the material properties listed in Table 2-4 are suitable for design in accordance with Section 6 of the OHBDC.

The overburden soils should be excavated using suitable techniques and the bedrock surface should be cleaned using air lift or other suitable methods. Subsequent to cleaning the bedrock surface, concrete can be tremied into the box structure up to the underside of footing elevation. The amount of concrete required will be significant and care should be exercised during tremiing to ensure a continuous pour in order to avoid potential cement loss during stoppages in construction.

Inspection of the bedrock surface prior to placement of the concrete will be difficult and may require a diver or video methods. For a mass concrete footing bearing on the unweathered to slightly weathered bedrock, the following Limit States design values may be assumed in accordance with the O.H.B.D.C.:

Table 2-7 Mass Concrete Spread Footing Capacity on Bedrock

	Spread Footing
Factored Geotechnical Capacity at ULS	3000 kPa

The geotechnical capacity at SLS will not govern for a mass concrete spread footing founded on bedrock since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS

2.3.6 Option 5 - Caissons

Caissons may also be considered for the pier locations although, as outlined in Section 2.1.2 of this report, caisson installation will have some significant practical impediments. The caisson load capacities provided in Table 2-3 for the North and South abutments may be applied to caissons at the pier locations. As previously noted, the caisson loads listed in Table 2-3 apply to caissons founded in unweathered to lightly weathered gneissic bedrock at

or below the elevations given above in Tables 2-2. The technical issues discussed in Section 2.1.2 for caisson construction at the abutment locations also apply to the north and south piers.

2.4 Foundations General

2.4.1 Spread Footing Sliding Resistance

The computation of the sliding resistance for spread footings shall be carried out in accordance with of O.H.B.D.C. An unfactored friction angle, ϕ' , of 32 degrees can be used for sliding along the bedrock and footing base.

If the factored resistance against sliding failure is inadequate based on friction, then the footing should be anchored into bedrock by means of keys, dowels or sockets. An unfactored coefficient of passive earth pressure, K_p' , equal to 3.7 can be used for design of a passive resistance key. Given the hardness of the bedrock, sockets and keys will likely be impractical. Developing adequate resistance against sliding of spread footings founded on the sloping bedrock at the site will likely require dowels.

2.4.2 Spread Footing Inclined Loading

As per section 6-8.4.2 of the Ontario Highway Bridge design code, a reduction factor shall be applied to the Ultimate Bearing Resistance at ULS account for the effects of inclined loading on spread footings. Table 2-8 contains a summary of reduction factors for inclined loads which should also be applied for inclined loads acting on spread footings placed directly on bedrock (Table 2-5) and the mass concrete footing option given in Section 2.2.3 (Table 2-6).

Table 2-8 Reduction Factors to Account for the Effects of Inclined Loads on the Ultimate Bearing Resistance at ULS *

Ratio of Horizontal to Vertical Load	Reduction Factor
0.1	0.87
0.2	0.76
0.3	0.66
0.4	0.57

*As advised by MTO Foundation Section " Although the OHBDC provides resistance reduction due to inclined loadings for footing on bedrock, the OHBDC committee has decided that no such reduction will be required if the footing is constructed on bedrock."

Note: The structural engineer can refer to Figure 6-8.4.2 of the Ontario Highway Bridge Design Code for reduction factors corresponding to ratios of horizontal to vertical loads which are not listed above.

2.4.3 Driven Pile Foundations - Construction

All piles should be driven to bedrock. In the unlikely event that piles should end above the bedrock surface within the silty sand till, pile driving should be controlled by the Hiley Formula as per MTO standards SS103-10 or SS103-11 using the ultimate pile capacities referred to in Table 2-1.

Given the highly variable bedrock elevations at the site, the potential for irregular steeply sloping bedrock is considered to be very high and, consequently, problems may arise during pile seating. At some locations, the piles may have a tendency to skip over the bedrock surface resulting in alignment problems and deeper penetration. In the event that this problem occurs, somewhat longer piles may be required and in some cases piles may have to be added or replaced.

To minimize seating difficulties, the piles should be provided with suitable rock injector points to facilitate proper seating. It is recommended that, during pile driving and upon initial contact with the bedrock, the pile driving energy should be reduced and subsequently increased incrementally until the piles have been sufficiently seated.

2.4.4 Frost Cover

For pile caps, frost protection should consist of a minimum of 2 m of earth cover or equivalent insulation. Pile caps at the pier locations do not require frost protection.

2.5 Backfill

Backfill to abutments or retaining walls should consist of free draining granular materials such as Granular 'A', Granular 'B' or rock fill. Free draining light weight fill may also be used as backfill adjacent to the bridge abutments. Computation of earth pressures shall be in accordance with Section 6.7.4 of the Ontario Highway Bridge Design Code. Unfactored properties for backfill materials are provided in the following table:

Table 2-9: Material Types and Unfactored Properties.

Material	Friction Angle, ϕ' , in Degrees	Coefficient of Active Earth Pressure (K_a)	Coefficient of Earth Pressure at Rest (K_o)	γ (kN/m ³)
Granular A	35 degrees	0.27	0.43	22.5
Granular B	30 degrees	0.33	0.5	21.2
Rock Fill	35 degrees	0.27	0.43	18.0

Material	Friction Angle, ϕ' , in Degrees	Coefficient of Active Earth Pressure (K_a)	Coefficient of Earth Pressure at Rest (K_o)	γ (kN/m ³)
Rock Fill	35 degrees	0.27	0.43	18.0
Light Weight Fill	35 degrees	0.27	0.43	11.5

For abutments or retaining walls founded on caissons or battered piles, the coefficient of earth pressure at-rest must be assumed for design purposes.

To reduce settlements and improve stability at the abutment locations, light weight fill (e.g., Blast furnace slag) should be used provided that the fill is free draining. A NSSP should be provided for the grain size distribution, placement, and compaction of lightweight fill. This will be discussed further in section 2.6.2.

2.6 Construction Considerations and Settlements

2.6.1 Piles and Caissons

The settlements of properly constructed piles, footings or caissons founded on bedrock are expected to be negligible.

2.6.2 Abutments and Approaches

Based on the vane shear strength profile shown in Figure 2, the maximum height of an embankment constructed with side slopes graded at a gradient of 1.5 horizontal to 1 vertical is approximately 4.6 metres at the ULS (based on a unit weight of 18 kN/m³ and factored shear strength properties). It is understood that grades within approximately 10 metres of the proposed north and south abutments will be raised by up to 5 m. As such, light weight fill must be used within approximately 10 metres of the north and south abutments to meet stability requirements.

Figure 11 shows the estimated settlements of the approach embankment based on an 8 m thick (Borehole 8) and a 4 m thick (Borehole 1) silty clayey to clayey silt deposit. Based on the calculations shown in Figure 11, the settlements are expected to vary between approximately 100 mm ($\pm 25\%$) for a 4 m thick deposit to 200 mm ($\pm 25\%$) for an 8 m thick deposit provided that the applied embankment pressure is less than about 60 kPa. This applied pressure limitation is equivalent to 5.2 m of lightweight fill, 3.3 m of rock fill, 2.7 m of Granular A and 2.8 m of Granular B.

Below an applied pressure of approximately 60 kPa, the consolidation behaviour of the approach embankments is dominated by the overconsolidated properties of the clayey silt to silty clay (CL) deposit. Only small zones of localized yielding of the foundation soil is expected below this threshold pressure. Figure 11 also illustrates that only marginal increases in applied pressure above 60 kPa will result in significant increases in settlements.

Provided that the applied embankment pressure is limited to 60 kPa, 90% consolidation will require a period of between 3 to 6 months based on laboratory consolidation tests.

As indicated above, the use of lightweight fill has been recommended to meet stability requirements within approximately 7 metres of the north and south abutments. With the use of lightweight fill at the abutment locations, the applied pressure should be limited to approximately 60 kPa. Based on the use of lightweight fill, this represents an embankment fill thickness of approximately 5.2 metres based on the unit weight provided in Table 2-8. Under these loading conditions, the settlements in the area of the abutments are expected to vary between 100 mm ($\pm 25\%$) and 200 mm ($\pm 25\%$).

The applied embankment loads will likely be close to the estimated threshold pressure of 60 kPa beyond which settlements are expected to increase significantly. As such, it is recommended that the approach embankments be built prior to constructing the north and south abutments. The approach embankments should be surcharged by approximately 18 kPa by slightly overbuilding the embankments. Based on the estimated time for 90% consolidation noted above, a settlement period of approximately 4 months should be allowed for consolidation purposes prior to construction of the abutments and establishing final design grades.

Compression of the embankment fill and underlying lower silty sand to sandy silt has been neglected in the settlement calculations since a majority of the deformations owing to these soil layers will occur during construction. It has also been assumed that the approach embankment settlements will take place due to the consolidation of the underlying silty clay to clayey silt (CL) soils at the abutment areas. As such, under the plan limits of the approach embankments and within 20 metres of the bridge abutments, all peat, topsoil and other compressible organic materials must be stripped through the full width of the proposed abutment, approach areas and under the areas of the forward slopes. The depth and extent of materials to be removed will need to be established in the field during construction; However, for preliminary planning purposes, Table 2-9 summarizes the interpreted depth of excavation at the borehole locations.

To improve the long term performance of the approach embankments and to ensure adequate stability during embankment surcharging, one third height stabilization berms should be provided within 20 metres of the Musquash River Bridge north and south abutments. The stabilization berms should have a minimum width of 6 metres and must extend along the east and west sides of the approach embankments and wrap around the abutments to the forward slope areas.

Table 2-10 Interpreted Depth of Soil Removal

Borehole Location	Estimated Depth of Soil Removal (m)
BH-1	0.23 m
BH-4	nominal
BH-5	0.18 m
BH-8	0.2 m

2.7 Excavations

All excavations must be in accordance with the Occupational Health and Safety Regulations for Construction Projects for Type 3 and Type 4 soils depending on which soil type applies. Excavations through the embankment fill and underlying clayey silt to silty clay (CL) foundation soils are expected. The native clayey silt to silty clay (CL) may be classified as Type 4 soils. All fill materials identified in the borehole logs should be classified as Type 3 soils. In the abutment areas, excavations through overlying lightweight fill and into the silty clay to clayey silt (CL) are anticipated. At these locations, excavations of up to 5 metres sloped at 2H:1V are expected to be stable in the short term. The excavation edge closest to the river should be kept at least 1m above the river level at all times during construction. This may require the construction of a small dike using the clayey silt to silty clay at the site to separate the excavation from the river. Excavations through Granular 'A', Granular 'B' or rock fill and into the underlying native clayey silt to silty clay (CL) should be sloped at 2H:1V and must not exceed 3.5 metres in depth.

For the excavations anticipated at the site, it is expected that surface water and groundwater infiltration may be handled by conventional sump and pump techniques. Should the river level rise significantly above the bottom of the abutment excavations, groundwater seepage into the excavation through more permeable silty seams may occur. Depending on the proximity of the river, this may require sheeting and/or additional dewatering requirements.

2.8 Erosion Protection/Scour

The forward slopes will require erosion protection at the abutments. This erosion protection will likely consist of rock rip rap. The extent and sizing of the protective rip rap will depend on the hydrology of the river, the final grading and the shape of the river channel both upstream and downstream of the bridge. In general, the protective rock rip rap should extend a minimum of 20 m upstream and downstream from the bridge centreline. For preliminary design purposes, at least 0.6 m of rock protection (minimum size of 0.03 m³) should be placed on the river banks up to the elevation of the design high water level. The rock protection should also extend down the river banks to the toe of the forward slopes. A suitable nonwoven geotextile should be placed as a separator between the rock rip rap and underlying native soils.

The piers must also be protected from the effects of scour and ice loading. For piles, some protection will be provided by using pile bent construction as discussed in Section 2.3.2 of this report. In general, the extent of rock protection and the type of scour protection will depend on the river hydraulics taking into account the effects of the new bridge construction. This office will be available for further consultations in this regard once the bridge design has been finalized and the river hydraulics assessed. Also, this office should be contacted to review the final design drawings for slope, channel or pier protection.

2.9 General

The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed Musquash River Bridge. The conclusions presented in this report reflect site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the borehole logs are inferred from discontinuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change.

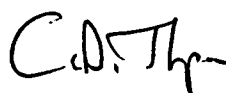
This report has been prepared by Sean Hinchberger and reviewed by S.E. Gonsalves. Eric Gonneau and Chi Ng coordinated the field investigation and Indulis Dumpus and Glenn Black performed the fieldwork.

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

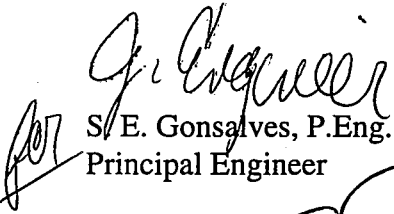
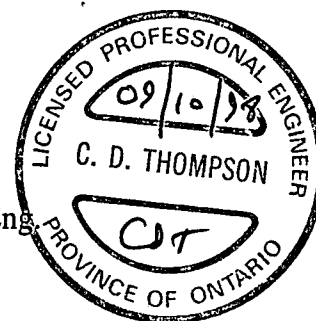
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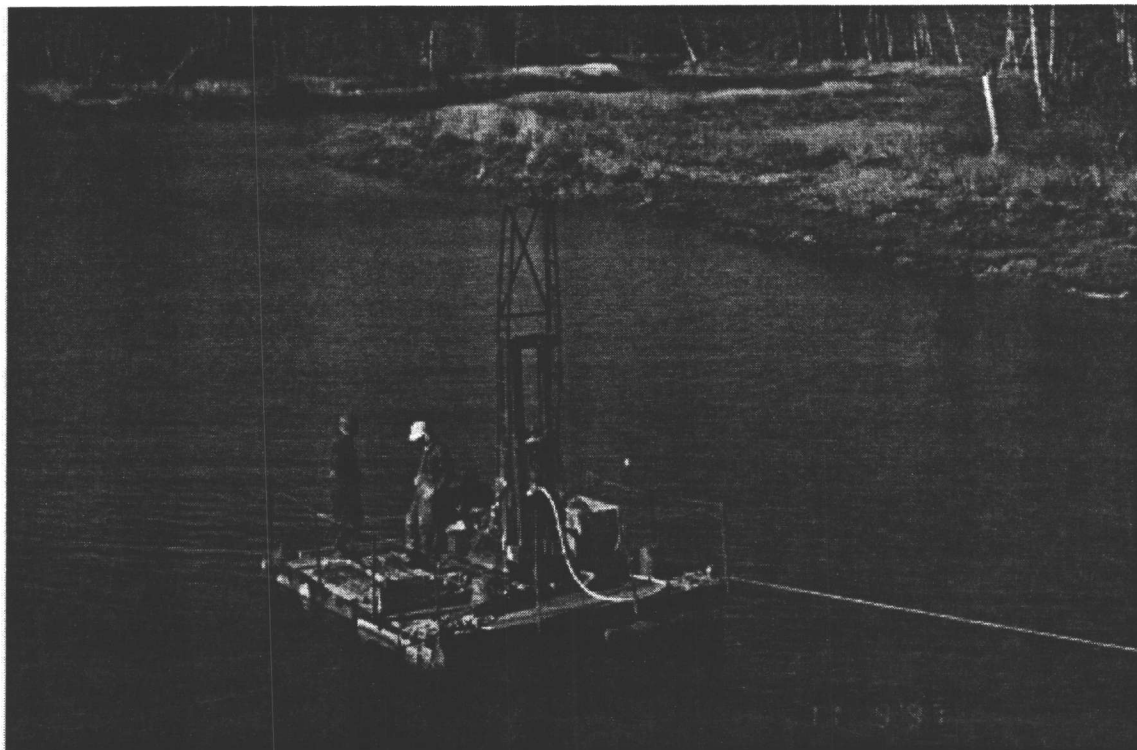
Appendix A: Photographs



PHOTOGRAPH 1 View of existing Musquash River Bridge from North Approach Embankment.



PHOTOGRAPH 2 View of Raft and North Bank of the Musquash River.



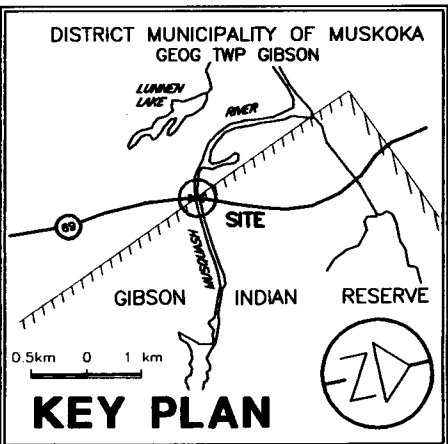
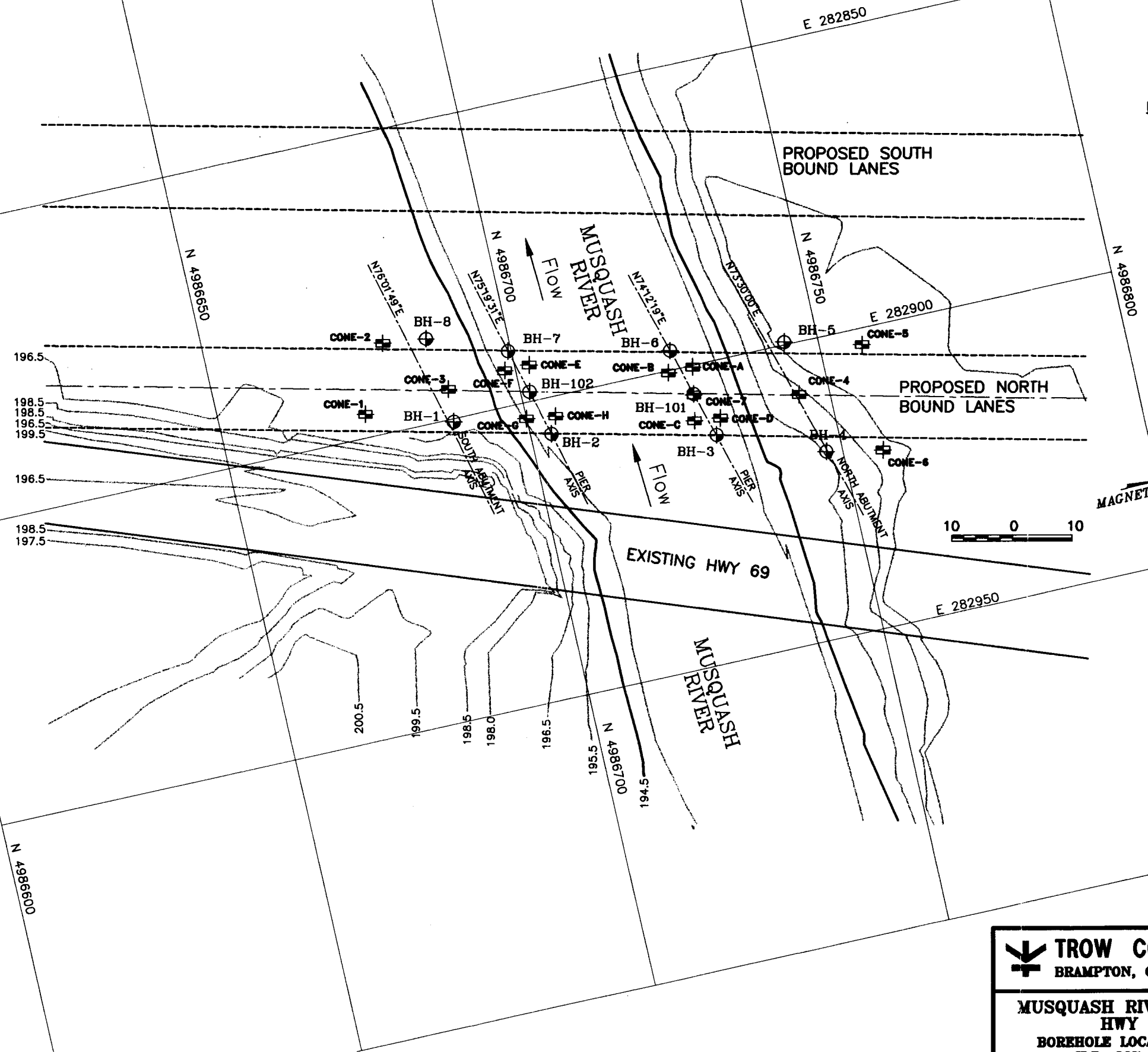
PHOTOGRAPH 3 Drill rig (D-25) and raft used to advance boreholes at pier locations between November 6 and 8, 1997.



PHOTOGRAPH 4 Drill rig (Longyear 24) and raft used to advance boreholes and cone holes at pier locations between March 18 and 24, 1998.

Appendix B: Drawings

NOTE: LOCATION OF BOREHOLES 2, 3, 6 & 7 ARE APPROX.

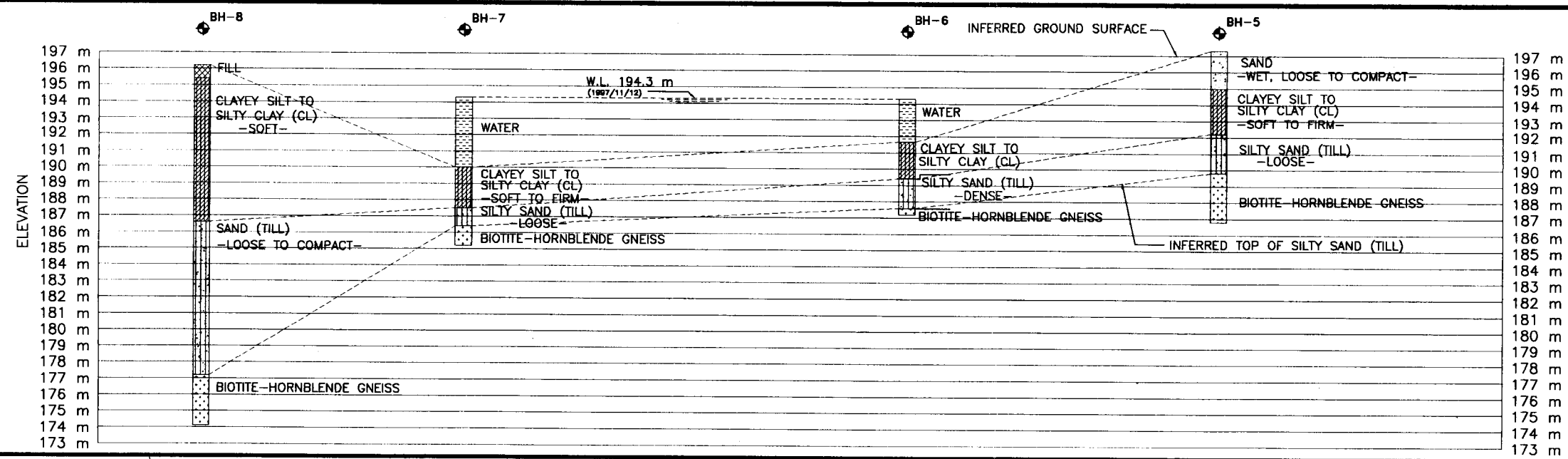


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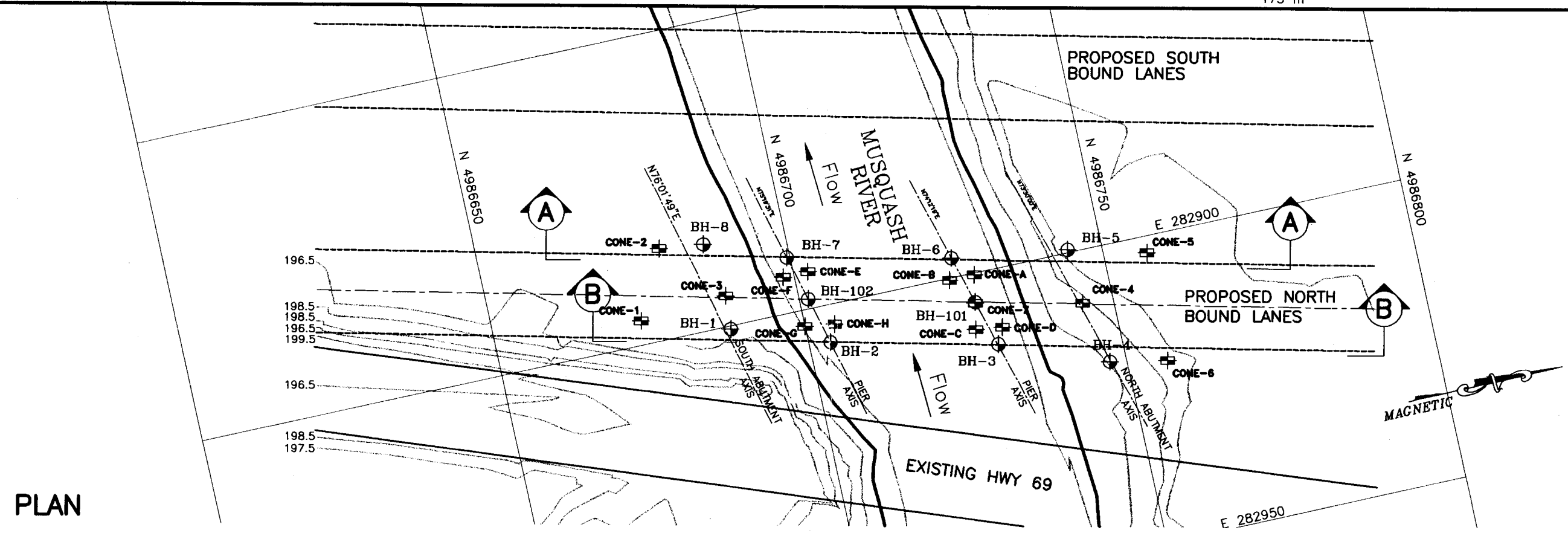
**MUSQUASH RIVER BRIDGE
HWY 69
BOREHOLE LOCATION PLAN
W.P. 207-90-01**

TORONTO ONTARIO

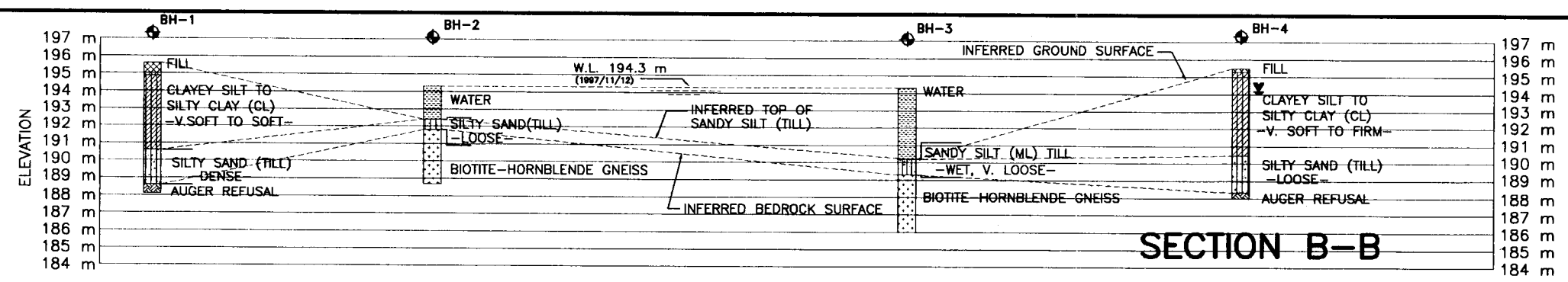
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CHECKED BY:	S.H.
DATE:	OCT., 1998
DRAWING NO.:	1



SECTION A-A



PLAN



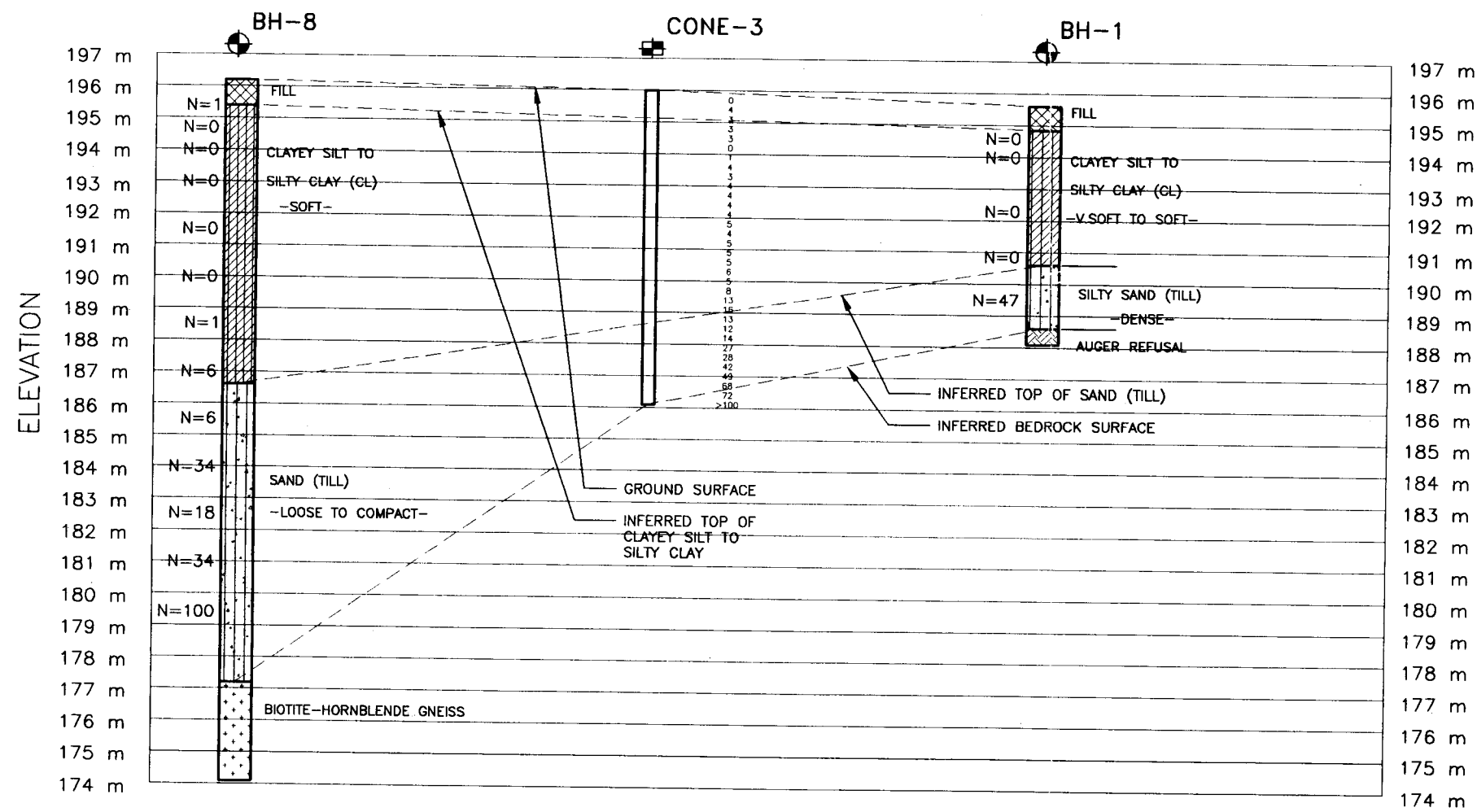
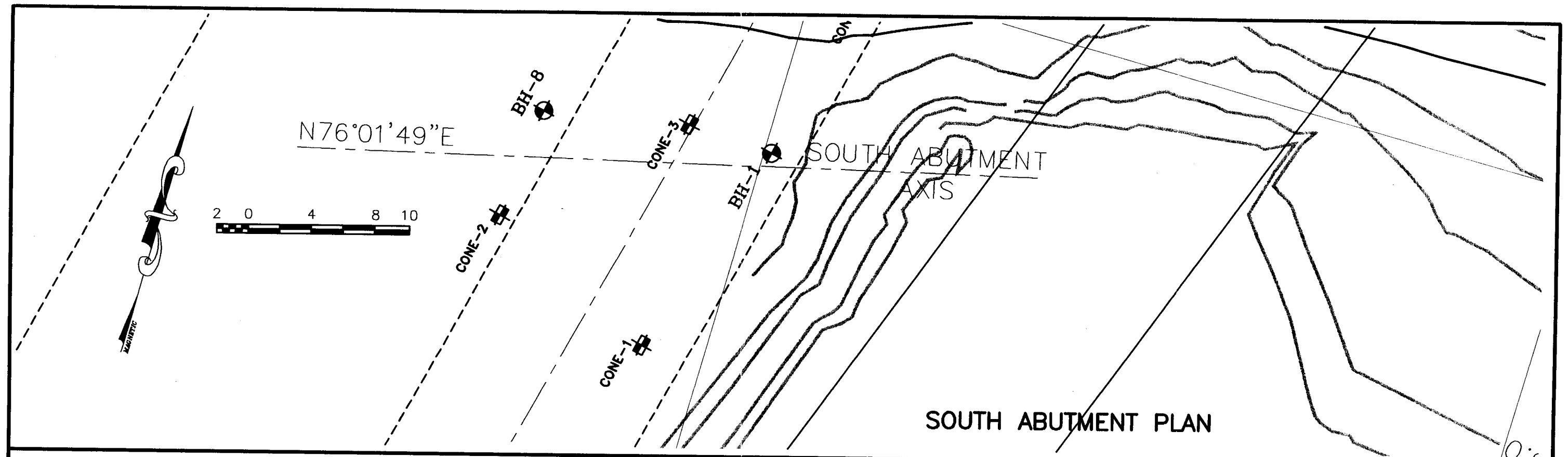
SECTION B-B

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MUSQUASH RIVER BRIDGE
HWY 69
SUBSURFACE PROFILES
W.P. 207-90-01

PROJECT NO.: BR-11546-A/G
SCALE: AS SHOWN
DRAWN BY: S.S.
CHECKED BY: S.H.
DATE: OCT., 1998
DRAWING NO.: 2

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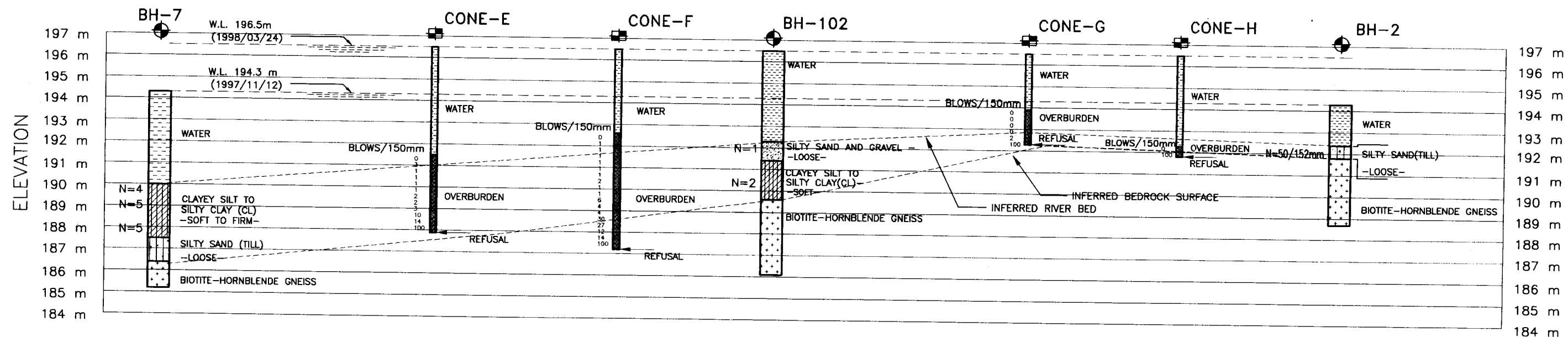
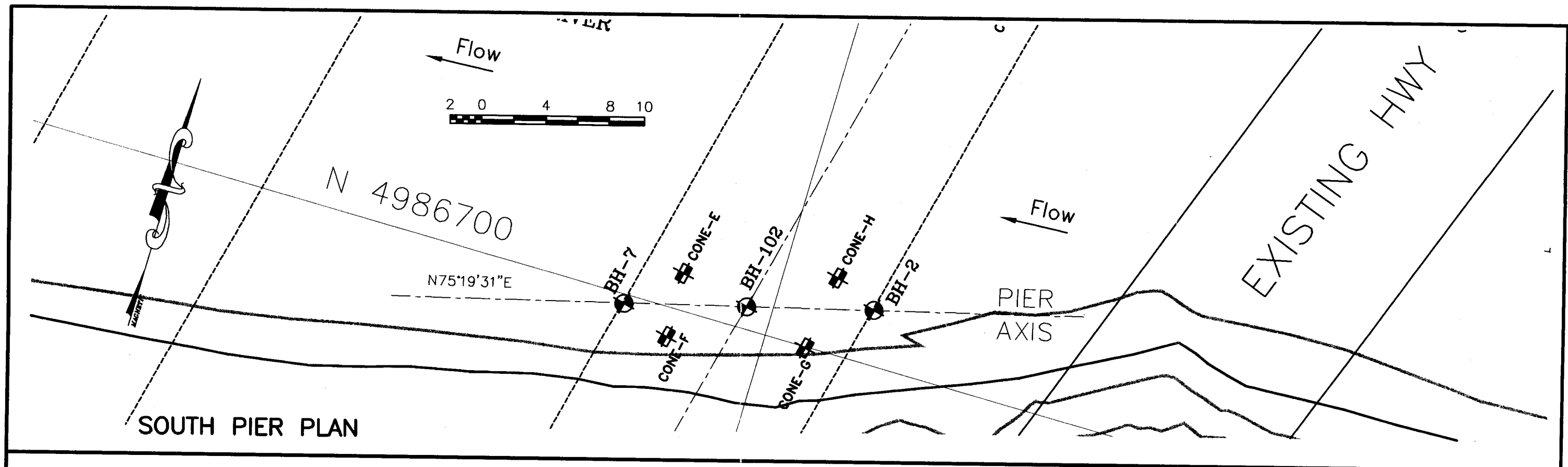
SOUTH ABUTMENT CROSS-SECTION

VERTICAL SCALE: REFER TO GRID
HORIZONTAL SCALE: NOT TO SCALE
(FOR HORIZONTAL DISTANCES SCALE ONLY
THE PLAN VIEW)

NOTE:

BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.

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MUSQUASH RIVER BRIDGE HWY. 69 - SOUTH ABUTMENT PLAN AND CROSS-SECTION W.P. 207-90-01	
PROJECT NO.: BR-11546-A/G	SCALE: AS SHOWN
DRAWN BY: S.S.	CHECKED BY: S.H.
DATE: OCT., 1998	DRAWING NO.: 3
TORONTO	ONTARIO



SOUTH PIER CROSS-SECTION

VERTICAL SCALE: REFER TO GRID
HORIZONTAL SCALE: NOT TO SCALE
(FOR HORIZONTAL DISTANCES SCALE ONLY
THE PLAN VIEW)

NOTE:

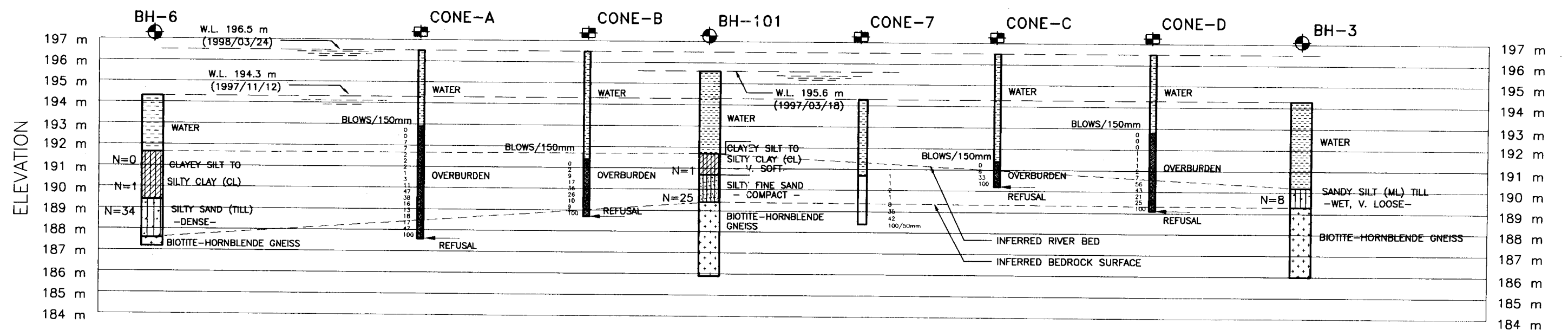
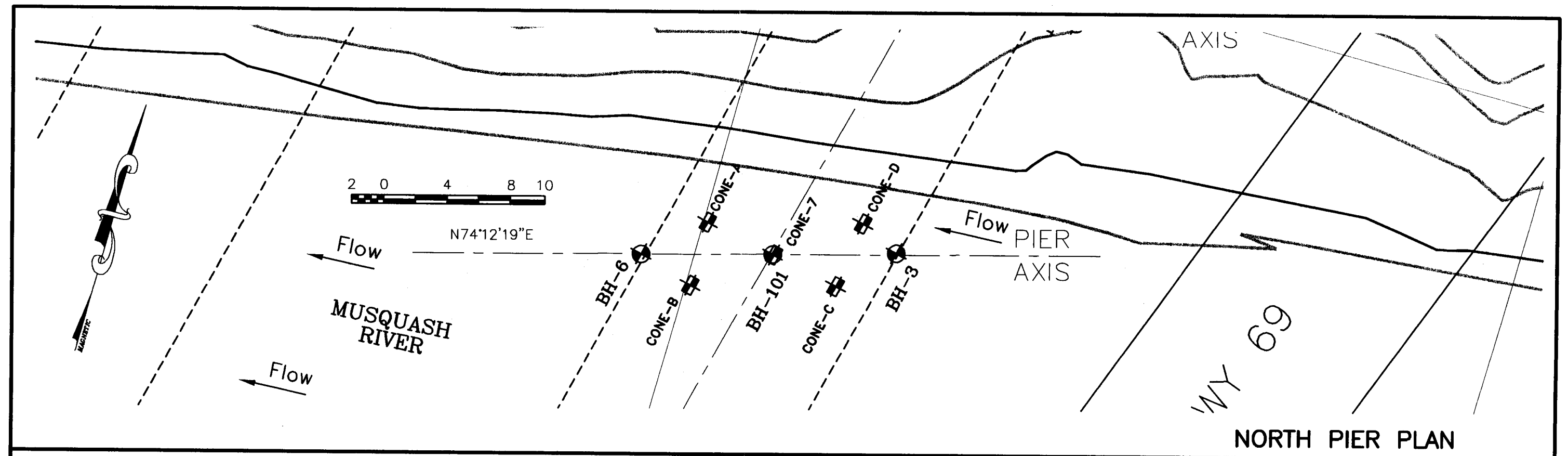
BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.

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**MUSQUASH RIVER BRIDGE
HWY. 69 - SOUTH PIER
PLAN AND CROSS-SECTION
W.P. 207-90-01**

PROJECT NO.: BR-11546-A/C
SCALE: AS SHOWN
DRAWN BY: S.S.
CHECKED BY: S.H.
DATE: OCT., 1998
DRAWING NO.: 4

TORONTO, ONTARIO



VERTICAL SCALE: REFER TO GRID
 HORIZONTAL SCALE: NOT TO SCALE
 (FOR HORIZONTAL DISTANCES SCALE ONLY
 THE PLAN VIEW)

NOTE:

BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.

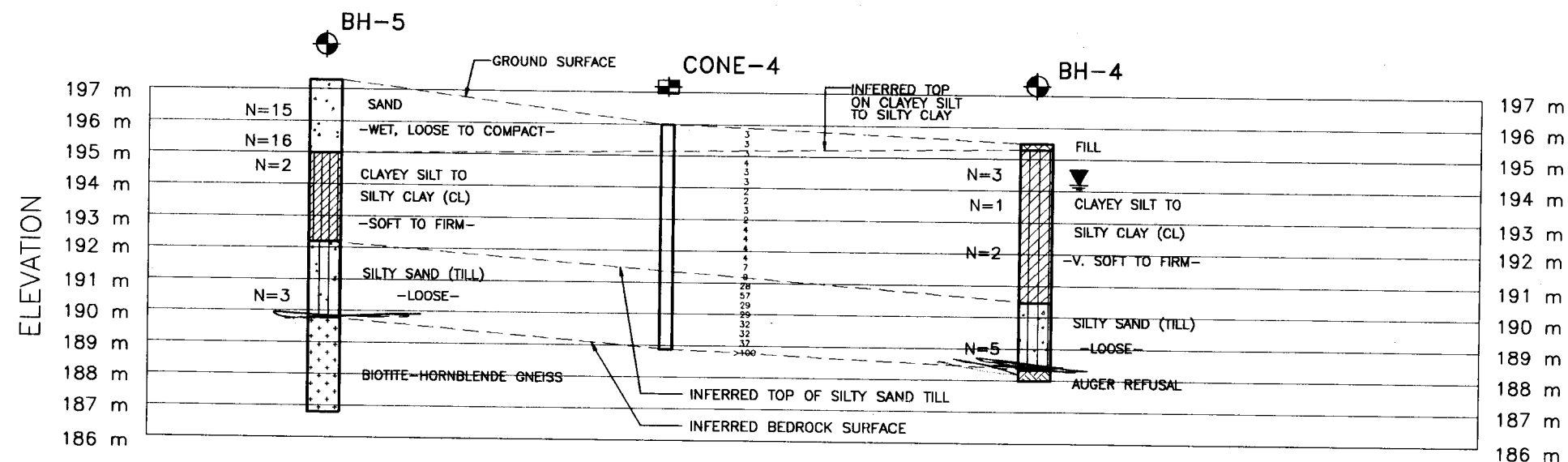
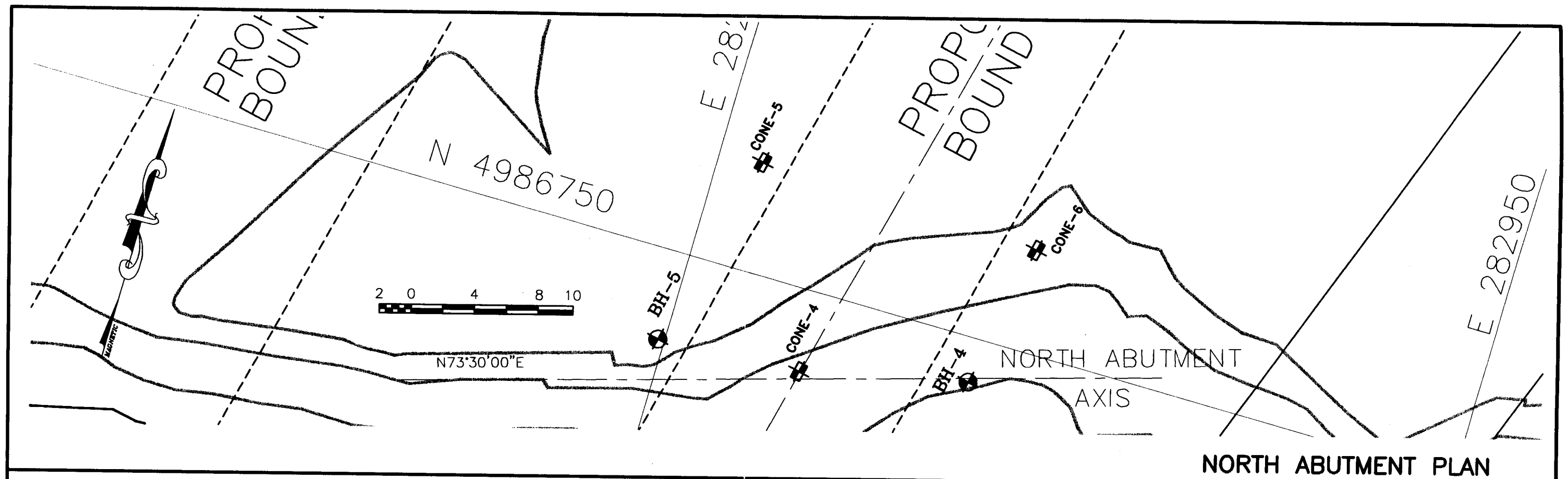
TROW CONSULTING ENGINEERS LTD.
 BRAMPTON, ONTARIO

MUSQUASH RIVER BRIDGE
HWY. 69 - NORTH PIER
PLAN AND CROSS-SECTION
W.P. 207-90-01

TORONTO

ONTARIO

PROJECT NO.:	BR-11546-A/G
SCALE:	AS SHOWN
DRAWN BY:	S.S.
CHECKED BY:	S.H.
DATE:	OCT., 1998
DRAWING NO.:	5



VERTICAL SCALE: REFER TO GRID
 HORIZONTAL SCALE: NOT TO SCALE
 (FOR HORIZONTAL DISTANCES SCALE ONLY
 THE PLAN VIEW)

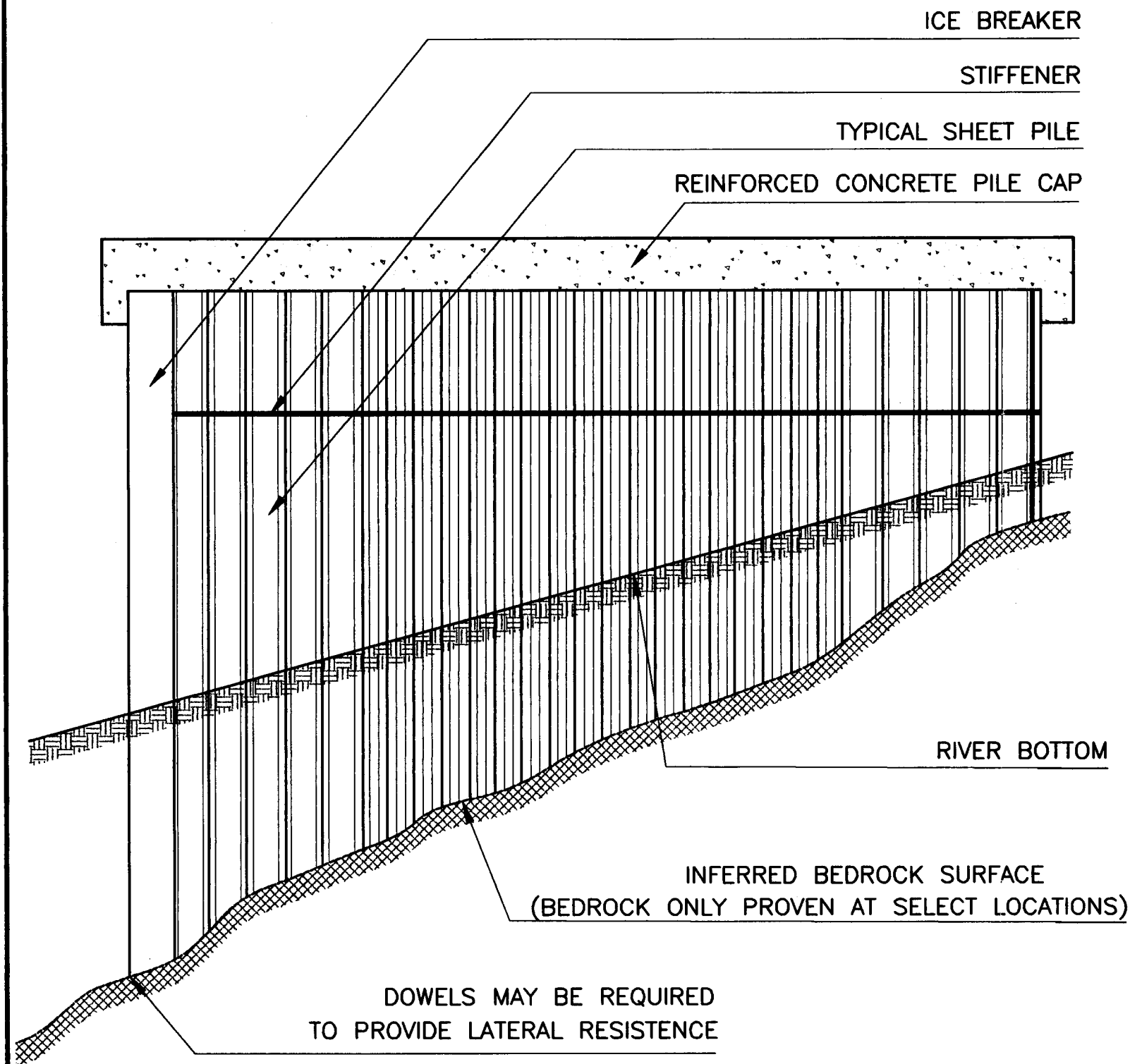
NOTE:
 BOUNDARIES BETWEEN STRATA AT BOREHOLES ARE ESTIMATED FROM NON-CONTINUOUS SAMPLES. STRATA BOUNDARIES BETWEEN BOREHOLES ARE PLOTTED TO AID IN THE INTERPRETATION OF GENERAL STRATIGRAPHY. ACTUAL BOUNDARIES WILL NOT EXACTLY CORRELATE WITH THOSE SHOWN.

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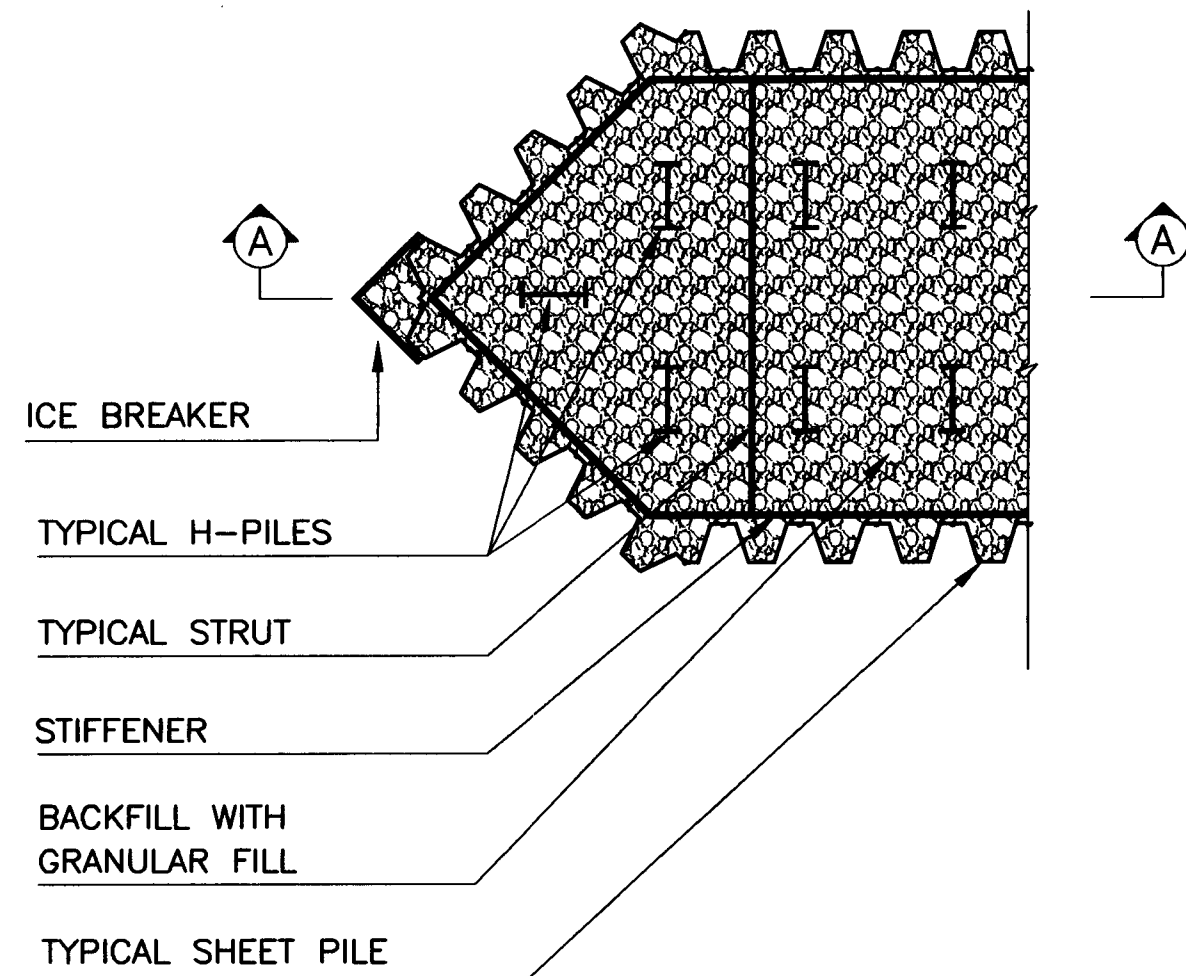
MUSQUASH RIVER BRIDGE
 HWY. 69 - NORTH ABUTMENT
 PLAN AND CROSS-SECTION
 W.P. 207-90-01

PROJECT NO.: BR-11546-A/G
 SCALE: AS SHOWN
 DRAWN BY: S.S.
 CHECKED BY: S.H.
 DATE: OCT., 1998
 DRAWING NO.: 6

TORONTO ONTARIO



SECTION A-A



PLAN VIEW

TROW CONSULTING ENGINEERS LTD.
BRAMPTON, ONTARIO

MUSQUASH RIVER BRIDGE
HWY 69
PIER FOUNDATION - SCHEMATICS
W.P. 207-90-01

TORONTO

ONTARIO

PROJECT NO.:	BR-11546-A/C
SCALE:	AS SHOWN
DRAWN BY:	S.S.
CHECKED BY:	S.H.
DATE:	OCT., 1998
DRAWING NO.:	7

Appendix C: Borehole Logs & Cone Logs

RECORD OF BOREHOLE 1

MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 207-90-01 LOCATION SOUTH ABUTMENT - 4 986 688.ON 282 900.5E ORIGINATED BY G.B.
 DIST 52 HWY 69 BOREHOLE TYPE SPT STANDARD AUGERS COMPILED BY S.H.
 DATUM GEODETIC DATE November 12, 1997 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp	w	wl		
195.6	GROUND SURFACE															
0.0	~ 228 mm Topsoil over FILL - consists of silty clay, reddish															
194.9	CLAYEY SILT to SILTY CLAY (CL) - Low plasticity, reddish changing to grey below El. 194 m, wet, very soft to soft		1	SS	0		195								15.60	
0.8			2	SS	0		194								16.70	
			3	TW			193									
			4	SS	0		192									
							191									
190.6			5	SS	0		190									
5.1	SILTY SAND - with GRAVEL, occ. cobbles, moist, dense (TILL).						189									
	AUGER REFUSAL - POSSIBLE BEDROCK OR BOULDER AT 7.0 m		6	SS	47										22% 55% 23%	
188.6	End of borehole															
7.0																

METRIC

CHECKED BY C.N.

MTOMUSQ 11546 04/06/98



1 OF 1

METRIC

W.P. 207-90-01

LOCATION WEST END - NORTH PIER - APPROX. 20+421

ORIGINATED BY G.B.

DIST 52 HWY HWY 69

BOREHOLE TYPE STD AUGERS-BQ CORE SPT

COMPILED BY S.H.

DATUM GEODETIC

DATE November 6, 1997

CHECKED BY C.N.

[illegible]

RECORD OF BOREHOLE 4

MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 207-90-01 LOCATION NORTH ABUTMENT - 4 986 747.2N 282 918.9E ORIGINATED BY G.B.
 DIST 52 HWY 69 BOREHOLE TYPE STANDARD AUGERS SPT - VANE COMPILED BY S.H.
 DATUM GEODETIC DATE November 14, 1997 CHECKED BY C.N.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER TYPE BLOWS/0.3m			20 40 60 80	20 40 60 80						20 40 60 80
195.5	GROUND SURFACE												
195.0	FILL - consists of gravel												
0.2	CLAYEY SILT to SILTY CLAY (CL) - Low plasticity, reddish to brown, grey below El. 194.0 m, wet, very soft to firm		1 SS 3		195								
			2 SS 1		194							16.20	
			3 VS		193								
			4 SS 2		192								
			5 TW		191							20.40	
190.4	Changing to silt, wet, loose				190								
5.1	SILTY SAND - with some gravel, brown, wet, loose (TILL).				189								
	AUGER REFUSAL - PROBABLE BEDROCK OR BOULDER AT 7.15 m		6 SS 5									6% 90% 3%	
188.3	End of borehole												
7.2													

RECORD OF BOREHOLE 5

MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 207-90-01 LOCATION NORTH ABUTMENT - 4 986 744.4N 282 899.5E ORIGINATED BY G.B.
 DIST 52 HWY 69 BOREHOLE TYPE STD AUGERS-BQ CORE SPT-VANE COMPILED BY S.H.
 DATUM GEODETIC DATE November 14, 1997 CHECKED BY C.N.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) \otimes CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wl	WATER CONTENT (%)		
197.3	GROUND SURFACE														
0.0	"178 mm topsoil over SAND - fine, light brown, wet, loose to compact		1	SS	15										
			2	SS	16										
195.0	CLAYEY SILT to SILTY CLAY (CL) - reddish to brown, changing to grey at El. 194 m, wet, soft to firm.		3	SS	2									17.00	
2.3			4	TW											
			5	SS											
192.2	SILTY SAND - light brown, wet, loose (TILL)		6	SS	3										
5.1															
189.8	BIOTITE-HORNBLende GNEISS - light grey, medium to coarse grained, strong, unweathered to slightly weathered.		7	RC											Rec. 100%RQD. 100%
7.5			8	RC											Rec. 100%RQD. 72%
186.8	End of borehole														
10.5															

METRIC

ORIGINATED BY G.B.

COMPILED BY S.H.

CHECKED BY C.N.



METRIC

ORIGINATED BY G.B.

COMPILED BY S.H.

CHECKED BY C.N.

MTOMUSQ 11546 04/06/98
















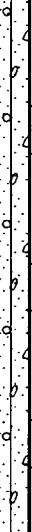








RECORD OF BOREHOLE 8 MUSQUASH RIVER BRIDGE

1 OF 2

METRIC

W.P. 207-90-01 LOCATION SOUTH ABUTMENT - 4 986 686.5N 282 885.9E ORIGINATED BY G.B.
 DIST 52 HWY HWY 69 BOREHOLE TYPE STD. AUGERS-BQ CORE SPT - VANE COMPILED BY S.H.
 DATUM GEODETIC DATE November 12, 1997 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)  CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp  w  wl				
								SHEAR STRENGTH: Cu, KPa  UNCONFINED QUICK TRIAXIAL  FIELD VANE LAB VANE				WATER CONTENT (%) 20 40 60 80				
196.2	GROUND SURFACE															
0.0	~ 200 mm Topsoil over FILL - consists of silty sand, light brown						196									
195.4	CLAYEY SILT to SILTY CLAY (CL) - Low Plasticity, reddish, changing to grey below El. 194.8 m, wet, firm to soft below El. 192.5 m		1	SS	1		195								18.90	
0.8			2	SS	0		194		6.0							
			3	SS	0		193								15.30	
			4	SS	0		192		6.0							
			5	TW	0		191									
			6	SS	0		190									
			7	SS	1		189		4.0							
			8	SS	6		188									
186.6	SAND - with some gravel and silt, light brown, loose to compact (TILL).						187							20.00		
9.6			9	SS	6		186									
			10	SS	34		185							16.40		
			11	SS	18		184								18% 74% 8%	
							183									
					182											

RECORD OF BOREHOLE 8 MUSQUASH RIVER BRIDGE

2 OF 2

METRIC

W.P. 207-90-01 LOCATION SOUTH ABUTMENT - 4 986 686.5N 282 885.9E ORIGINATED BY G.B.
 DIST 52 HWY HWY 69 BOREHOLE TYPE STD. AUGERS-BQ CORE SPT - VANE COMPILED BY S.H.
 DATUM GEODETIC DATE November 12, 1997 CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80				wp ——— w ——— wl				
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
						○	●	⊗	⊙	○	○	○				
						UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB VANE			20	40	60	80	kN/m³	GR SA (SI & CL)	
196.2																
			12	SS	34			⊗					○			
			13	SS	100											
		</														

RECORD OF BOREHOLE 101

MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 216-90-01 LOCATION NORTH PIER ORIGINATED BY G.B.
 DIST 52 HWY 69 BOREHOLE TYPE BW CASING - BQ CORE DIAMOND DRILL COMPILED BY S.H.
 DATUM GEODETTIC DATE March 18, 1998 CHECKED BY S.E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80				wp ----- w ----- wl					
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)					
							UNCONFINED QUICK TRIAXIAL FIELD VANE LAB VANE										
							20 40 60 80				20 40 60 80				kN/m³	GR SA (SI & CL)	
195.6 0.0	WATER SURFACE																

Rec 100% RQD 86%

Rec 96% RQD 61%

RECORD OF BOREHOLE 102

MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 216-90-01 LOCATION SOUTH PIER ORIGINATED BY G.B.
 DIST 52 HWY 69 BOREHOLE TYPE BW CASING - BQ CORE DIAMOND DRILL COMPILED BY S.H.
 DATUM GEODETIC DATE March 24, 1998 CHECKED BY S.E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	WATER CONTENT (%)	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa									
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB VANE								
196.5 0.0	WATER SURFACE					20	40	60	80	20	40	60	80				
	</																

RECORD OF CONE 1 MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 207-90-01

LOCATION SOUTH ABUTMENT - 4 986 674.1N 282 896.0E

ORIGINATED BY G.B.

DIST 52 HWY 69

BOREHOLE TYPE CONE

COMPILED BY S.H.

DATUM GEODETIC

DATE November 11, 1997

CHECKED BY C.N.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER								
196.7 0.0	GROUND SURFACE SOIL NOT SAMPLED										
					196						
					195						
					194						
					193						
					192						
					191						
					190						
					189						
					188						
					187						
					186						
					185						
					184						
183.6 13.1											

RECORD OF CONE 2 MUSQUASH RIVER BRIDGE

2 OF 2

METRIC

W.P. 207-90-01 LOCATION SOUTH ABUTMENT - 4 986 679.5N 282 885.1E ORIGINATED BY G.B.
 DIST 52 HWY 69 BOREHOLE TYPE CONE COMPILED BY S.H.
 DATUM GEODETIC DATE November 11, 1997 CHECKED BY C.N.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60					
196.8															
180.3 16.5						181									

RECORD OF CONE 3 MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 207-90-01

LOCATION SOUTH ABUTMENT - 4 986 688.4N 282 895.0E

ORIGINATED BY G.B.

DIST 52

HWY 69

BOREHOLE TYPE CONE

COMPILED BY S.H.

DATUM GEODETIC

DATE November 11, 1997

CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
196.0 0.0	GROUND SURFACE SOIL NOT SAMPLED						196									
							195									
							194									
							193									
							192									
							191									
							190									
							189									
							188									
							187									
186.0 10.0																

RECORD OF CONE 4 MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 207-90-01 LOCATION NORTH ABUTMENT - 4 986 744.9N 282 908.6E ORIGINATED BY G.B.
 DIST 52 HWY 69 BOREHOLE TYPE CONE COMPILED BY S.H.
 DATUM GEODETIC DATE November 14, 1997 CHECKED BY C.N.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60						80	wp
198.0 0.0	GROUND SURFACE					198											
	SOIL NOT SAMPLED																
						195											
						194											
						193											
						192											
						191											
						190											
						189											
188.7 7.3																	

RECORD OF CONE 5 MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 207-90-01 LOCATION NORTH ABUTMENT - 4 986 756.9N 282 902.7E ORIGINATED BY G.B.
 DIST 52 HWY 69 BOREHOLE TYPE CONE COMPILED BY S.H.
 DATUM GEODETIC DATE November 14, 1997 CHECKED BY C.N.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL FIELD VANE LAB VANE 20 40 60 80 20 40 60 80							
197.3 0.0	GROUND SURFACE SOIL NOT SAMPLED														
197															
196															
195															
194															
193															
192															
191															
190.3 7.0															

METRIC

ORIGINATED BY G.B.

COMPILED BY S.H.

CHECKED BY C.N.

MTOMUSQ 11546 04/06/98



RECORD OF CONE B MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 216-90-01 LOCATION NORTH PIER ORIGINATED BY G.B.
 DIST 52 HWY 69 BOREHOLE TYPE CORE DIAMOND DRILL COMPILED BY S.H.
 DATUM GEODETIC DATE March 23, 1998 CHECKED BY S.E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa							
							20	40	60	80	20	40	60	80	
196.5 0.0	WATER SURFACE														
191.4 5.2	OVERBURDEN SOIL														
188.7 7.9	REFUSAL AT 188.7 m ON PROBABLE BOULDER OR BEDROCK														

RECORD OF CONE C MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 216-90-01 LOCATION NORTH PIER ORIGINATED BY G.B.
DIST 52 HWY 69 BOREHOLE TYPE CORE DIAMOND DRILL COMPILED BY S.H.
DATUM GEODETIC DATE March 23, 1998 CHECKED BY S.E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE BLOWS/0.3m			SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL * FIELD VANE LAB VANE				WATER CONTENT (%) wp —○— wl				
196.5	0.0	WATER SURFACE					20	40	60	80	20	40	60	80	
191.4	5.1	OVERBURDEN SOIL				191									
190.2	6.3	REFUSAL AT 190.2 m ON PROBABLE BOULDER OR BEDROCK													

RECORD OF CONE D MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 216-90-01 LOCATION NORTH PIER ORIGINATED BY G.B.
DIST 52 HWY 69 BOREHOLE TYPE CORE DIAMOND DRILL COMPILED BY S.H.
DATUM GEODETIC DATE March 23, 1998 CHECKED BY S.E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa							
						UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB VANE	WATER CONTENT (%)							
196.5	0.0	WATER SURFACE						20	40	60	80	20	40	60	80
196.5	0.0	WATER													
192.8	3.8	OVERBURDEN SOIL													
189.1	7.4	REFUSAL AT 189.1 m ON PROBABLE BOULDER OR BEDROCK													

RECORD OF CONE E MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 216-90-01

LOCATION SOUTH PIER

ORIGINATED BY G.B.

DIST 52

HWY 69

BOREHOLE TYPE CORE DIAMOND DRILL

COMPILED BY S.H.

DATUM GEODETIC

DATE March 24, 1998

CHECKED BY S.E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER TYPE			BLOWS/0.3m	20					
196.5 0.0	WATER SURFACE											
191.5 5.1	OVERBURDEN SOIL											
187.9 8.7	REFUSAL AT 187.9 m ON PROBABLE BOULDER OR BEDROCK											

RECORD OF CONE F MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 216-90-01 LOCATION SOUTH PIER ORIGINATED BY G.B.
DIST 52 HWY 69 BOREHOLE TYPE CORE DIAMOND DRILL COMPILED BY S.H.
DATUM GEODETIC DATE March 24, 1998 CHECKED BY S.E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa							
							20	40	60	80	20	40	60	80	
196.5 0.0	WATER SURFACE WATER														
192.6 3.9	OVERBURDEN SOIL														
187.2 9.4	REFUSAL AT 187.2 m ON PROBABLE BOULDER OR BEDROCK														

MTOMUSQ MUSQ2 04/03/98



RECORD OF CONE G MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 216-90-01 LOCATION SOUTH PIER ORIGINATED BY G.B.
 DIST 52 HWY 69 BOREHOLE TYPE CORE DIAMOND DRILL COMPILED BY S.H.
 DATUM GEODETIC DATE March 24, 1998 CHECKED BY S.E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
196.5 0.0	WATER SURFACE														
193.9 2.7	OVERBURDEN SOIL														
192.3 4.3	REFUSAL AT 192.3 m ON PROBABLE BOULDER OR BEDROCK														

RECORD OF CONE H MUSQUASH RIVER BRIDGE

1 OF 1

METRIC

W.P. 216-90-01 LOCATION SOUTH PIER ORIGINATED BY G.B.
 DIST 52 HWY 69 BOREHOLE TYPE CORE DIAMOND DRILL COMPILED BY S.H.
 DATUM GEODETIC DATE March 24, 1998 CHECKED BY S.E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE BLOWS/0.3m			SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL * FIELD VANE LAB VANE				WATER CONTENT (%) wp — w — wl				
196.5 0.0	WATER SURFACE	WATER					20	40	60	80	20	40	60	80	
192.3 4.2	OVERBURDEN SOIL	[Pattern]				196									
191.8 4.8	REFUSAL AT 191.9 m ON PROBABLE BOULDER OR BEDROCK	[Pattern]				195									
						194									
						193									
						192									

MTOMUSQ MUS02 04/03/98



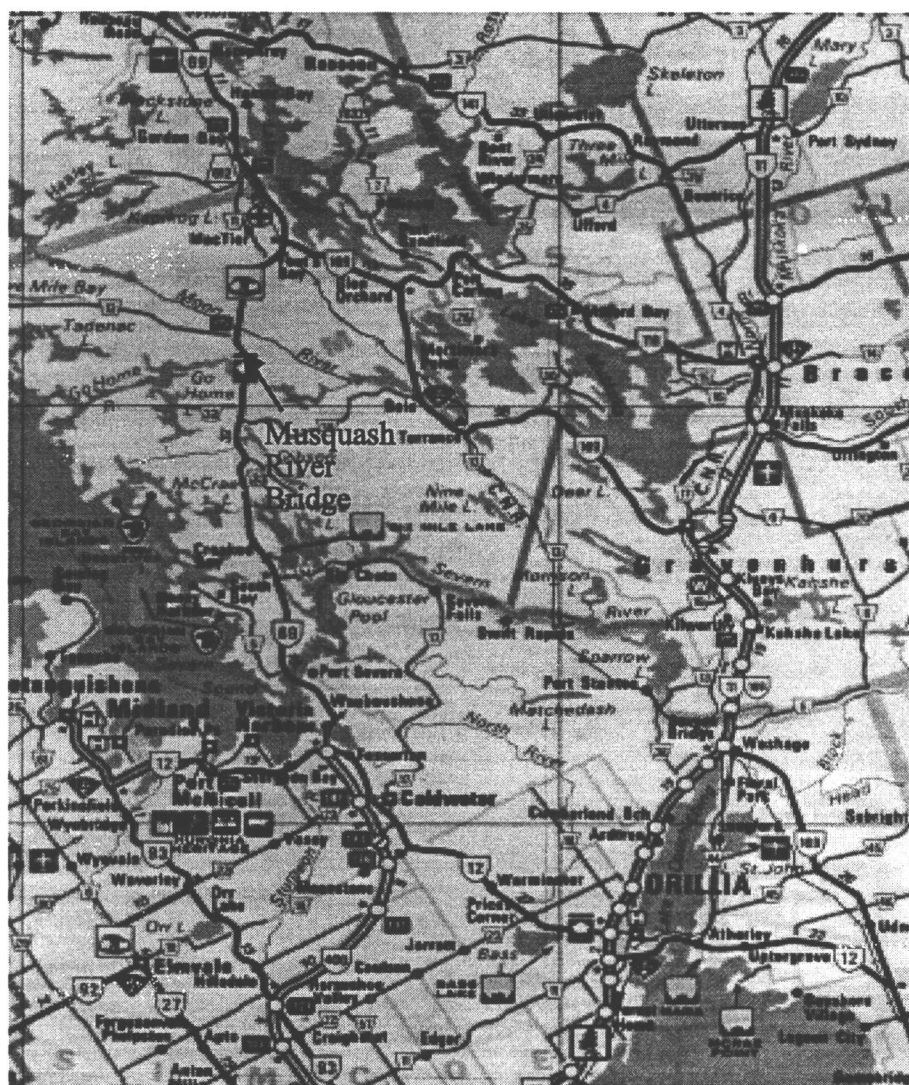


Figure 1: Site Location Plan - Musquash River Bridge

January, 1998

BR-11546A/G



TROW CONSULTING ENGINEERS LTD.

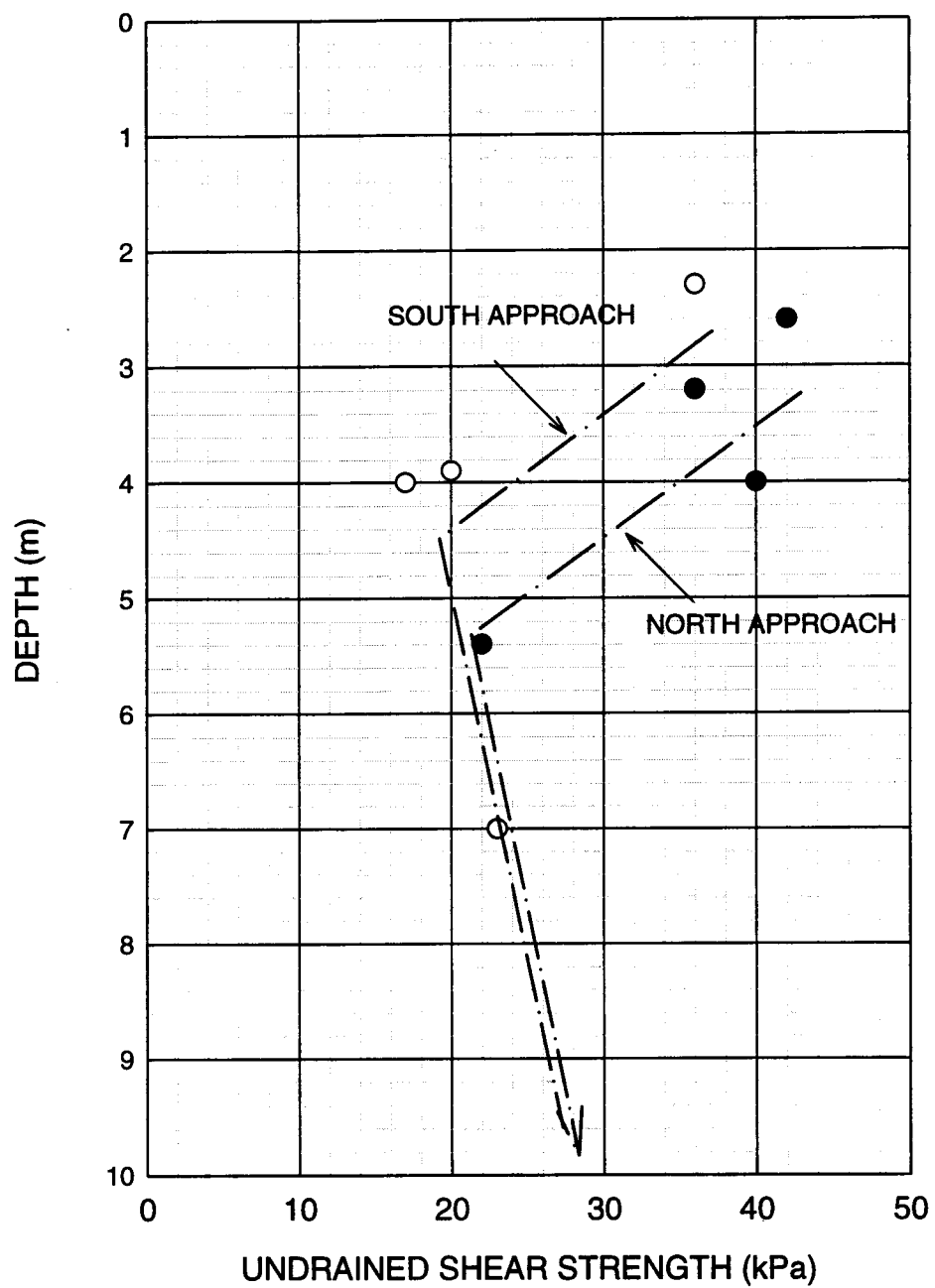


FIGURE 2: UNDRAINED SHEAR STRENGTH PROFILE
ADJACENT TO NORTH AND SOUTH ABUTMENTS.

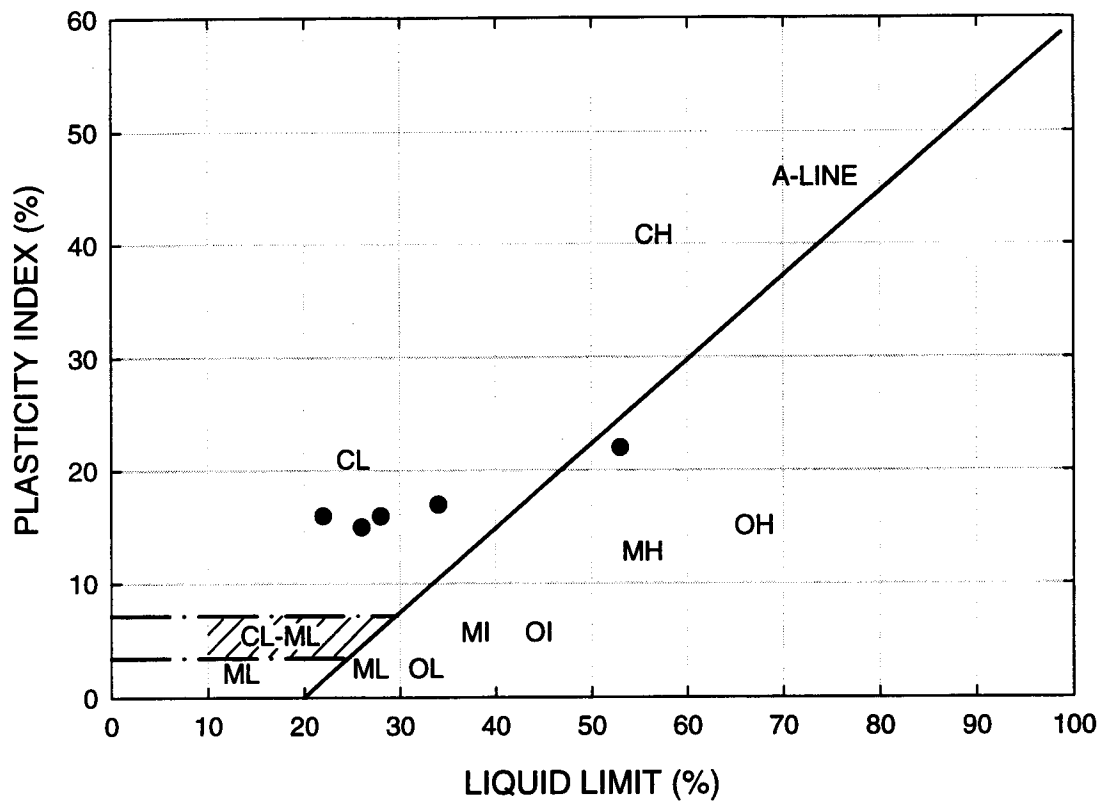
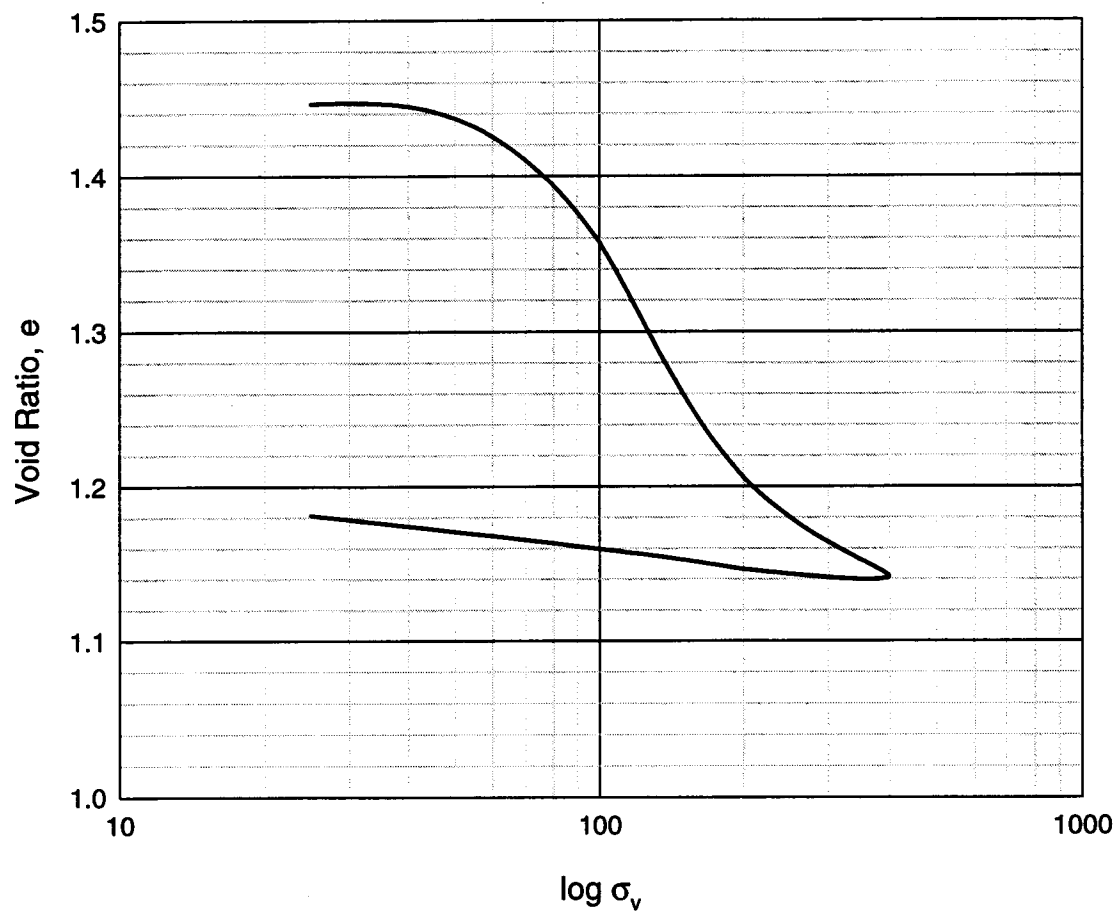


FIGURE 3 SUMMARY OF ATTERBERG LIMITS



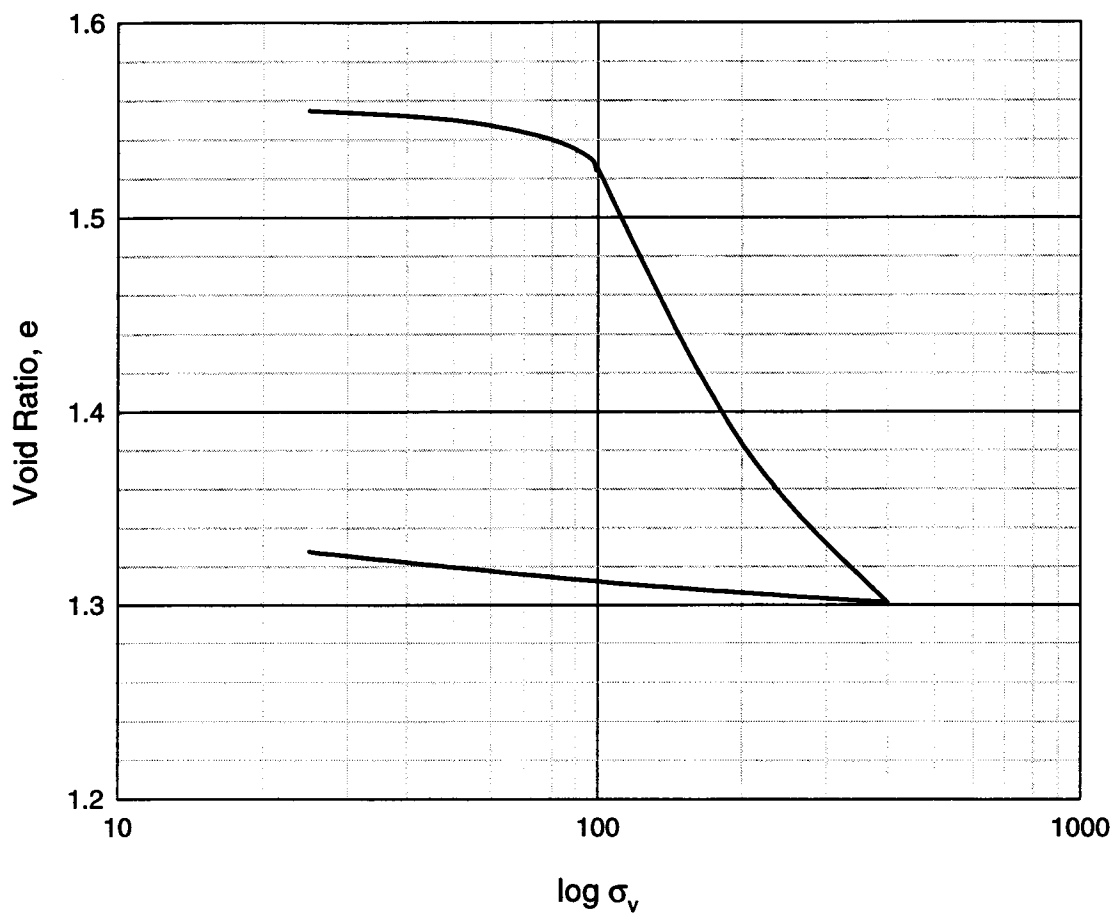
BOREHOLE NO. 5
 DEPTH 3.65m
 MOISTURE CONTENT 54%
 LIQUID LIMIT _____
 PLASTIC LIMIT _____
 PRECONSOL. PRESSURE 80 kPa

SAMPLE DESCRIPTION

CLAYEY SILT TO SILTY CLAY (CL)
 SOFT TO V. SOFT, BROWN, V. MOIST.

C_C 0.6
 C_R 0.04
 INITIAL VOID RATIO 1.45

FIGURE 4 OEDOMETER CONSOLIDATION TEST RESULTS FOR
 SHELBY TUBE SAMPLE TW4, BOREHOLE 5



BOREHOLE NO. 8
 DEPTH 4.8m
 MOISTURE CONTENT 53.5%
 LIQUID LIMIT _____
 PLASTIC LIMIT _____
 PRECONSOL. PRESSURE 120 kPa

SAMPLE DESCRIPTION

CLAYEY SILT TO SILTY CLAY (CL)
 SOFT TO V. SOFT, BROWN, V. MOIST.

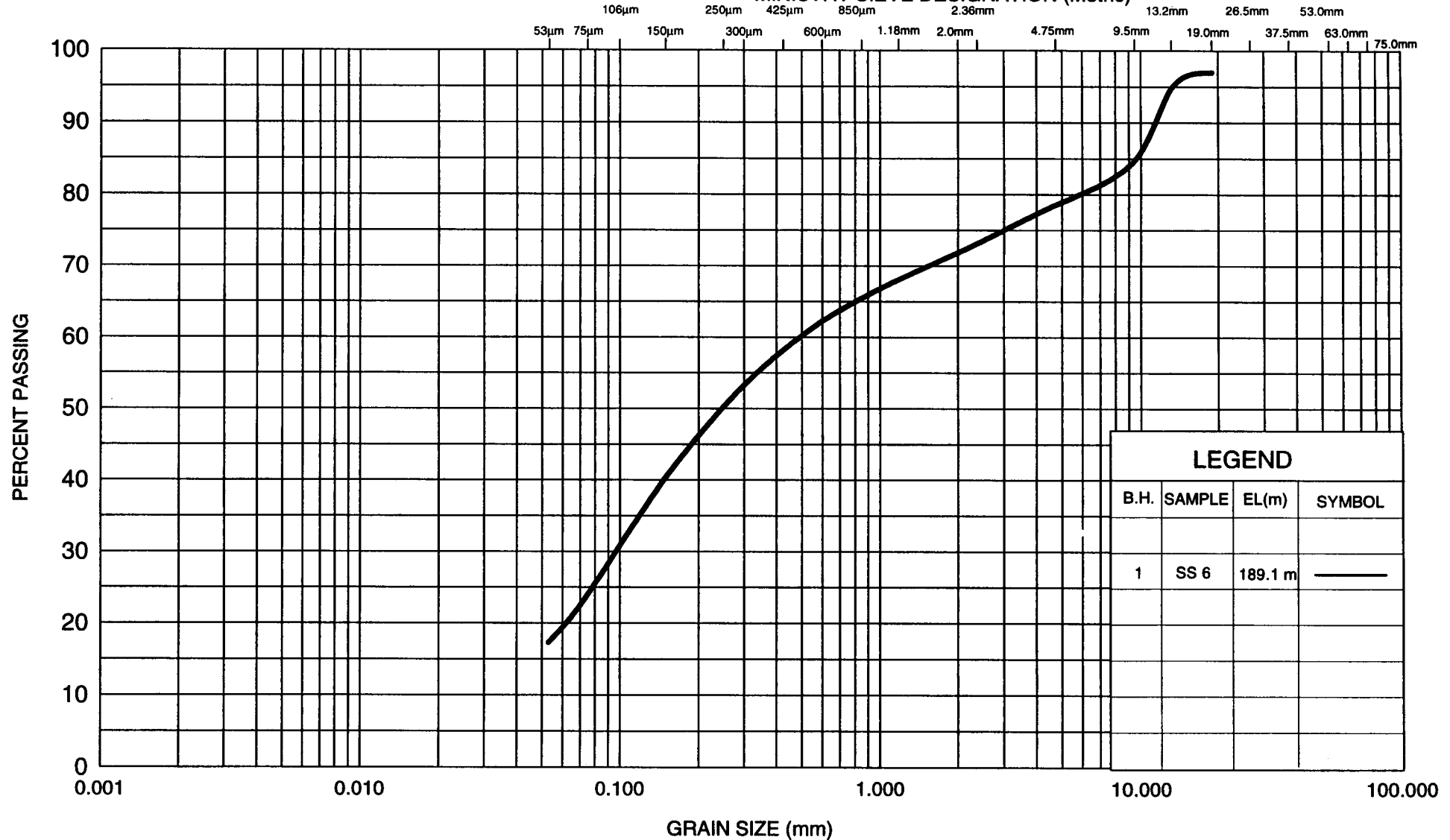
C_C 0.6
 C_R 0.06
 INITIAL VOID RATIO 1.6

FIGURE 5 OEDOMETER CONSOLIDATION TEST RESULTS FOR
 SHELBY TUBE SAMPLE TW-5, BOREHOLE 8

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

MINISTRY SIEVE DESIGNATION (Metric)



LEGEND

B.H.	SAMPLE	EL(m)	SYMBOL
1	SS 6	189.1 m	—

Ministry of
Transportation

METRIC

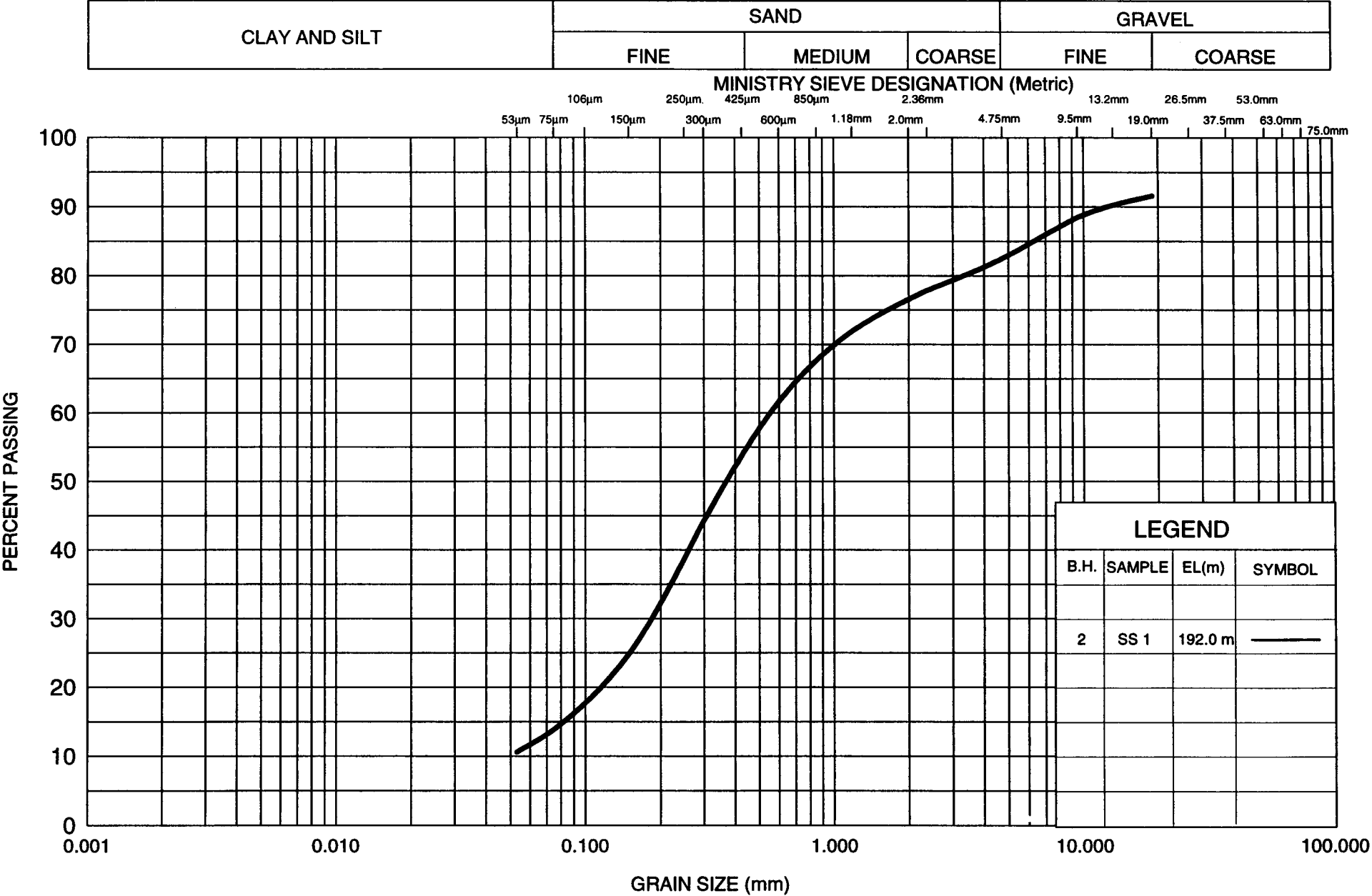
GRAIN SIZE DISTRIBUTION

B.H. 1 - SAMPLE 6: SILTY SAND - with GRAVEL, occ. cobbles, moist,
dense (TILL).

FIGURE 6

W.P. 207-90-01

UNIFIED SOIL CLASSIFICATION SYSTEM



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Transportation

METRIC

GRAIN SIZE DISTRIBUTION

B.H. 2 - SAMPLE 1: SILTY SAND - with some GRAVEL, light brown, wet,
loose (TILL).

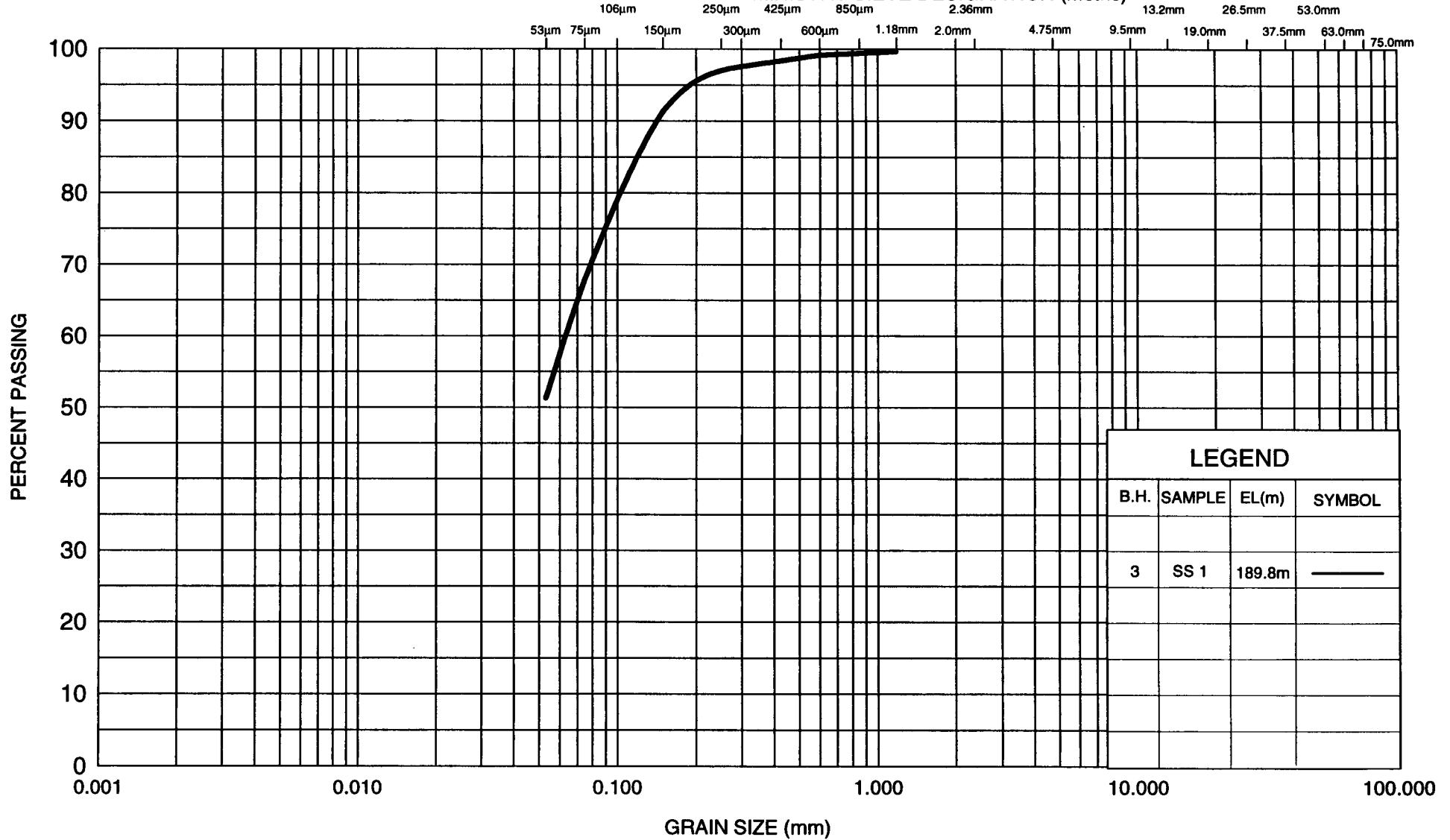
FIGURE 7

W.P. 207-90-01

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

MINISTRY SIEVE DESIGNATION (Metric)



LEGEND

B.H.	SAMPLE	EL(m)	SYMBOL
3	SS 1	189.8m	—

Ministry of
Transportation

METRIC

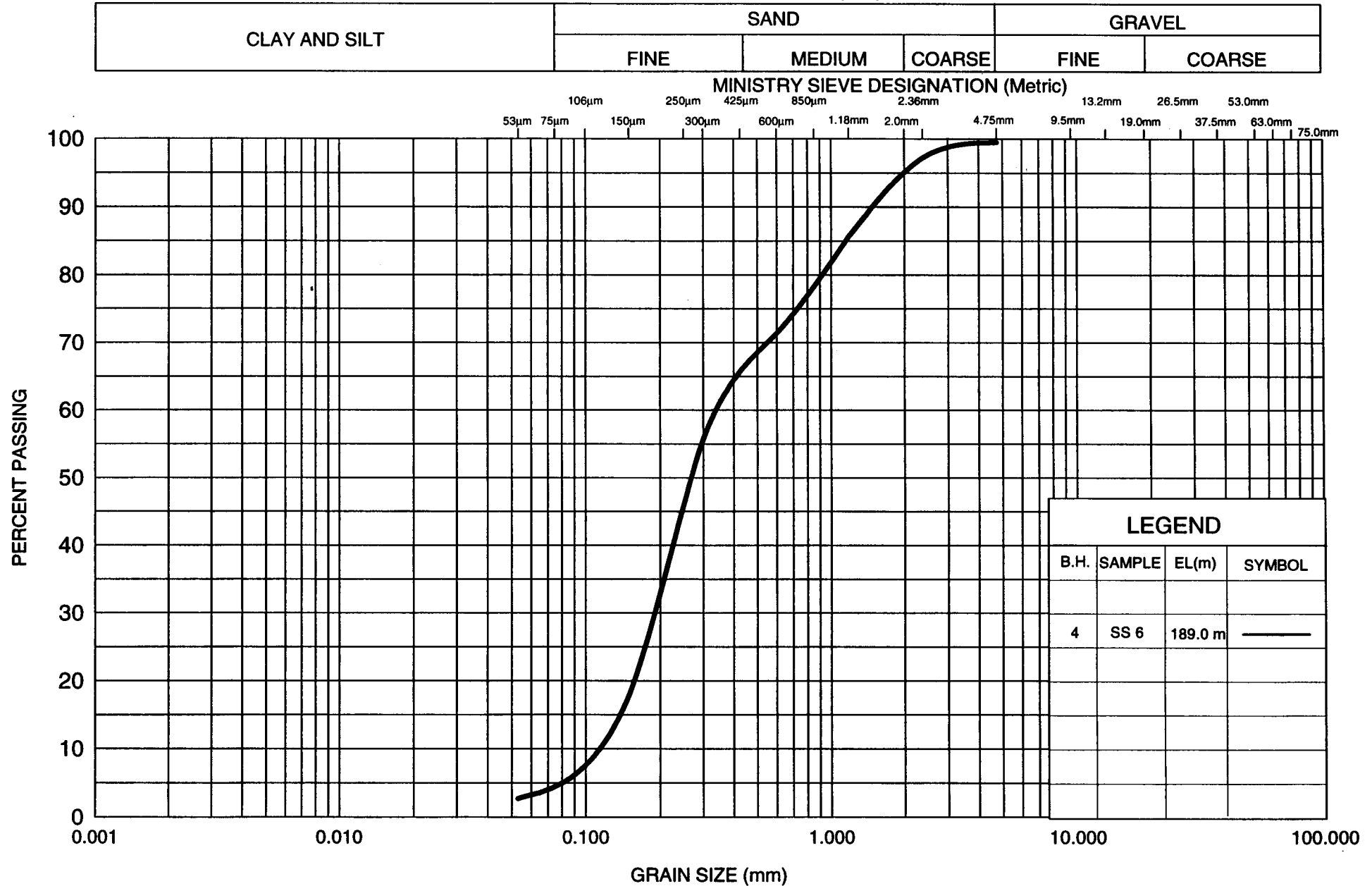
GRAIN SIZE DISTRIBUTION

B.H. 3 - SAMPLE 1: SANDY SILT (ML) - trace of clay, some rock frgements,
red to brown, wet, very loose.

FIGURE 8

W.P. 207-90-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

METRIC

B.H. 4 - SAMPLE 6:

GRAIN SIZE DISTRIBUTION

SAND - trace of SILT, stratified, brown, wet,
loose.

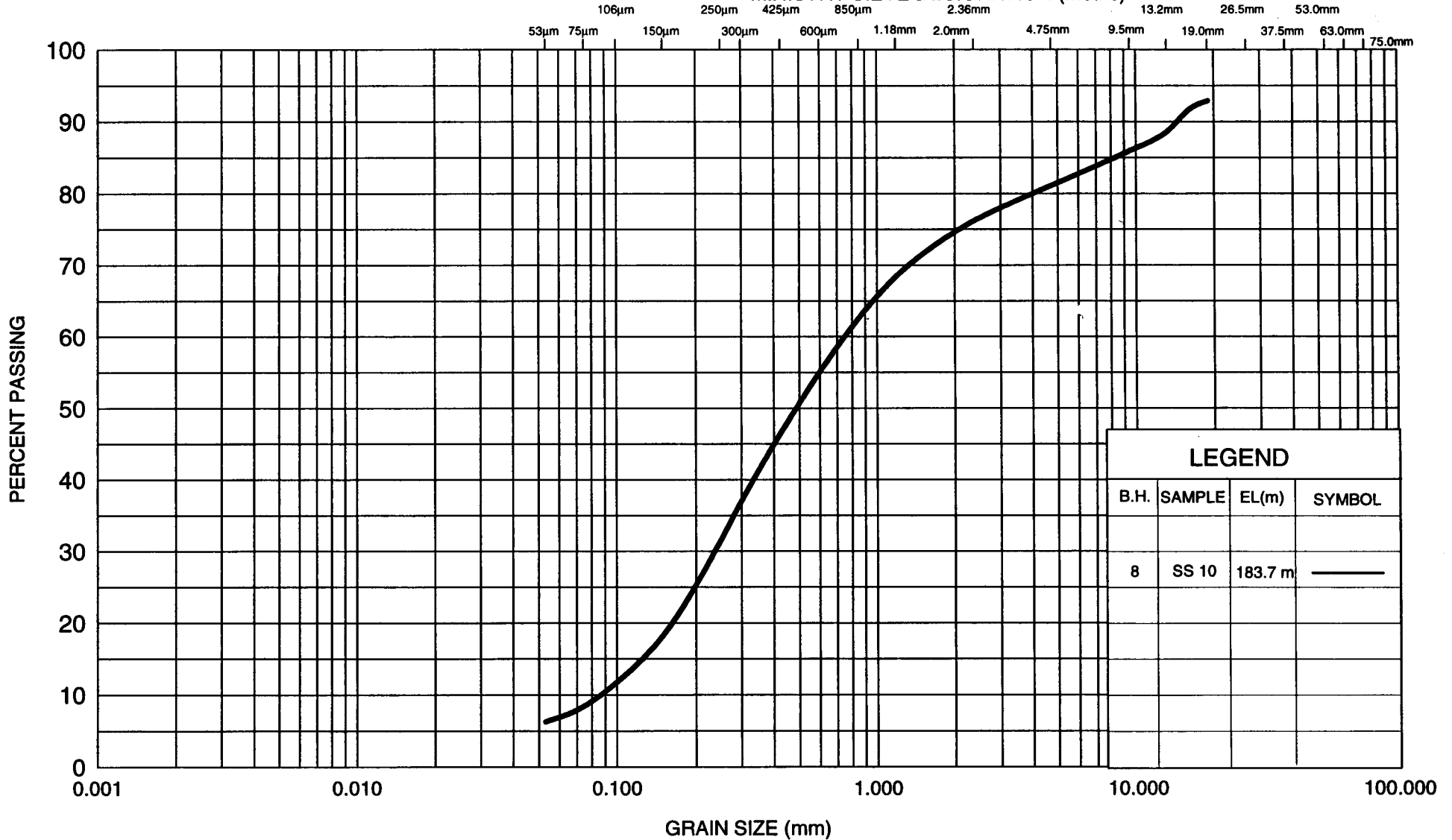
FIGURE 9

W.P. 207-90-01

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of
Transportation

METRIC

GRAIN SIZE DISTRIBUTION

B.H. 8 - SAMPLE 10: SAND - with some GRAVEL, trace of SILT, brown, wet,
loose to compact (TILL).

FIGURE 10

W.P. 207-90-01

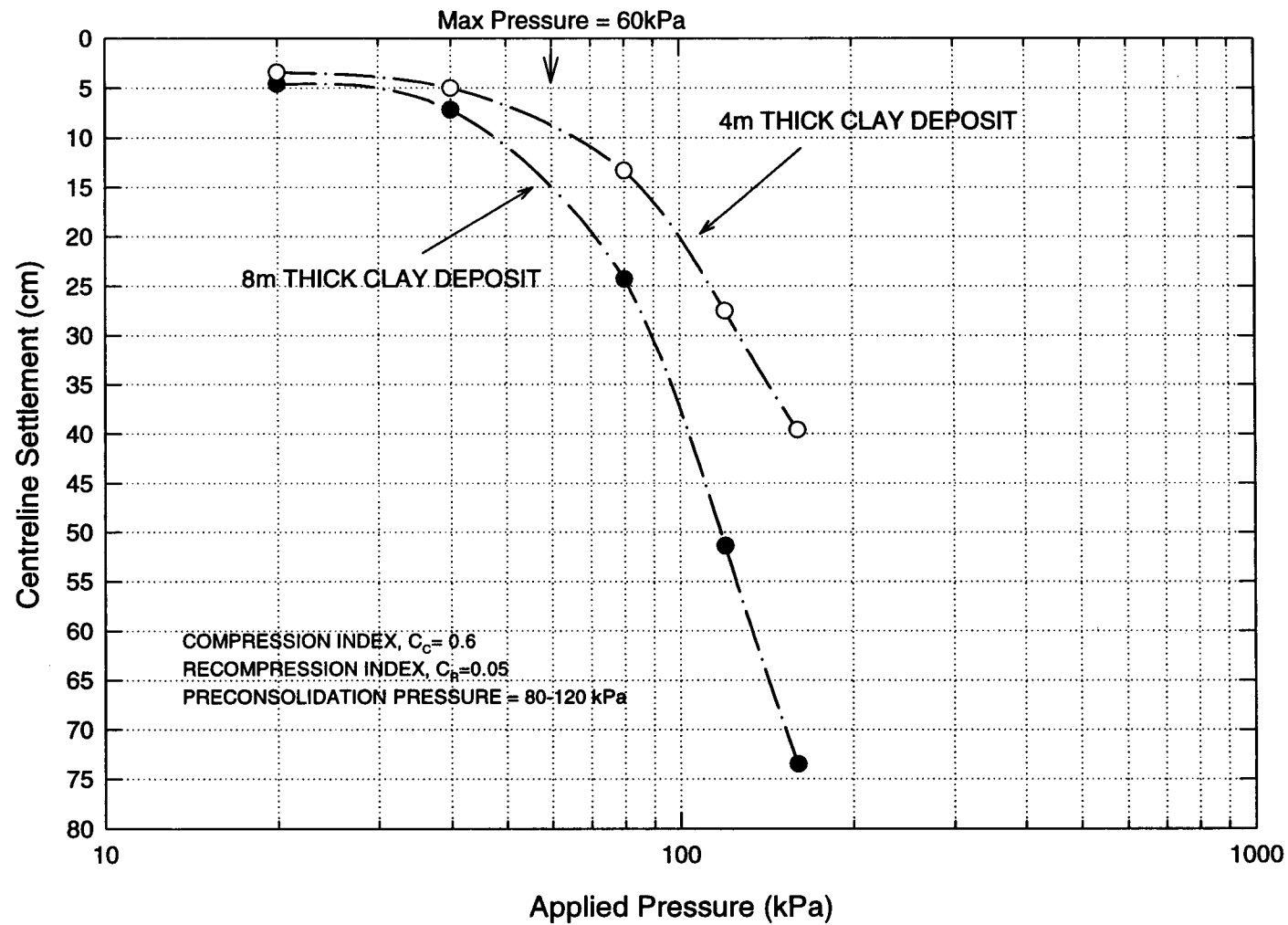


FIGURE 11 PREDICTED EMBANKMENT SETTLEMENTS FOR
 FOR 4m AND 8m THICK SOFT CLAY DEPOSIT.

