

**FOUNDATION INVESTIGATION REPORT
PROPOSED HIGHWAY 17 (NEW) TWIN OVERPASS STRUCTURE
AT HURON CENTRAL RAIL (CPR)
HIGHWAY 17 (NEW) FROM ECHO RIVER TO BAR RIVER ROAD
DISTRICT 62, SAULT STE. MARIE, ONTARIO
G.W.P. 354 AND 352-94-00
SITE: 385-526/1; 385-526/2**

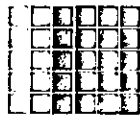
Prepared For:

MARSHALL MACKLIN MONAGHAN LTD.

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1055
August 21, 2003**



shaheen & peaker
limited

**20 Meteor Drive
Toronto, Ontario
M9W 1A4**

**Tel: (416) 213-1255
Fax: (416) 213-1260**

WEB SITE: WWW.SHAHEENPEAKER.CA

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DRAWINGS

BOREHOLE LOCATION PLANS

DRAWING NO.

1 AND 2

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**FOUNDATION INVESTIGATION REPORT
PROPOSED HIGHWAY 17 (NEW) TWIN OVERPASS STRUCTURE
AT HURON CENTRAL RAIL (CPR) INTERSECTION
HIGHWAY 17 (NEW) FROM ECHO RIVER TO BAR RIVER ROAD
SAULT STE. MARIE, ONTARIO
G.W.P. 354 AND 352-94-00
SITE: 385-526/1; 385-526/2**

1. INTRODUCTION

Realignment of Highway 17 to the north of the existing highway from the Lower Echo River to Bar River Road includes new twin bridges at the Huron Central Rail (CPR) crossing about 0.2 km north of Maple Leaf Road in Sault Ste. Marie, Ontario.

Shaheen & Peaker Limited (S&P) was retained by Marshall Macklin Monaghan Ltd. to carry out a foundation investigation for the proposed new twin bridges.

The findings of investigation are presented in this report.

2. SITE DESCRIPTION AND GEOLOGY

The site is located in MacDonald Township, about 0.2 km north of Maple Leaf Road and about 1.5 km of south of Watson Road.

According to Map 2108 published by the Ontario Department of Mines (see Appendix C), the bedrock at the site consists of Cambrian sandstone of Jacobsville Formation at the interface with Pre-cambrian Lorrain Formation which consists of quartzite, siltstone, greywacke and conglomerate.

Typically, in the higher lying areas, bedrock of undifferentiated igneous and metamorphic classifications (Southern Province) are exposed at surface forming shallow hills. These rocks are generally Pre-cambrian formations while some Cambrian unconformities are also noted. The rock generally dips rapidly to below surface and in the relatively higher lying foot-hill areas, the bedrock is covered with some glacial till and/or granular deposits. In the low-lying areas, peat muck and marl are found, covering glaciolacustrine deposits. The glaciolacustrine deposits typically consist of clay and silt, minor sand deposited in basin and quiet water environments. The depth of clay in areas can exceed 40 m.

The massive bedrock outcrop, which is exposed to the north, northeast of the railway crossing, was identified as sandstone of Cambrian origin. Some Pre-cambrian quartzite is also present in the area.

The grade in the general area falls from north to south and to a certain extent from east to west. Along the proposed alignment, the ground elevation to the north of the site falls southerly along the massive rock outcrop from about Elevation 212 m (about 370 m north of the railway crossing) to about Elevation 195 m near the toe of the outcrop and then more gradually to about Elevation 189 m at the crossing itself, within a horizontal distance of about 240 m. Further south, the grade is relatively level and drops to an elevation of about 188 m near Maple Leaf Road (i.e. about 200 m further south).

3. INVESTIGATION PROCEDURES

One borehole was drilled at each abutment location and one at a distance of about 20 m from each abutment for approaches, bringing the total number of boreholes to eight. The boreholes were numbered one to eight as shown on the Borehole Location Plan, Drawing Nos. 1 and 2. Dynamic Cone Penetration tests (DCPT) were also performed adjacent to most of the boreholes.

The boreholes were drilled during the period of November 6 to December 1, 2002, except for Borehole 4A which was put down on April 19, 2002 to enable us to give some advance preliminary information for the foundation design of the bridges.

The boreholes were advanced using solid and hollow stem continuous flight augers with track-mounted vehicles owned and operated by Colbar Resources of Sudbury, Ontario, under the supervision and direction of Geotechnical Engineers from our office. The depths of the boreholes ranged from 11.3 to 15.1 m. Sampling in the overburden was effected starting at the ground surface at 0.76 m intervals of depth by the Standard Penetration Test (SPT) method, as specified in ASTM D1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter O.D. split-spoon (split barrel) sampler into the relatively undisturbed ground. The number of blows required to drive the sampler into the ground by a vertical distance of 0.30 m is recorded as the Standard Penetration or the N-value of the soil and this gives an indication of the consistency or the compactness condition of the soil deposit.

In most cases, the sand backed-up in the boreholes (in spite of the fact that drilling was used to prevent this from happening) and/or cobbles and boulders were encountered with increasing depth of overburden in the boreholes. These necessitated washboring and/or diamond drilling methods. Because of this and due to the prevailing cold weather conditions, the drilling took considerably longer than anticipated.

The bedrock was proven by NQ rock coring at all the borehole locations, except BQ rock coring was used in Boreholes 5 and 8. The length of bedrock coring ranged from about 1 to 3 m at the approach borehole locations and 3 m at the bridge abutment locations.

As mentioned before, Dynamic Cone Penetration tests (DCPT) were performed at most (six) borehole locations. In this test, a 51 mm diameter, 60-degree apex cone, screw attached to the tip of an A-size rod, is driven into the ground, using the same driving energy as the SPT method. By recording the number of blows of the hammer to drive the cone/rod assembly into the soil every 0.3 m, a qualitative record of soil compactness condition is obtained.

For long-term observation of the groundwater conditions, piezometers were installed in Borehole Nos. 3 and 6.

The subsurface stratigraphy encountered in the boreholes, type of samples and sampling depths, N-values and DCPT results, together with the coring data are presented on the Record of Borehole Sheets, in Appendix A of this report.

Upon their completion, the boreholes were backfilled to about 8 m below the ground surface with soils brought up by augering (i.e. auger cuttings). The upper 8 m of the open boreholes were then grouted using a cement/bentonite mixture.

The borehole locations were established in the field by surveyors from Marshall Macklin Monaghan Ltd., who also provided us with ground surface elevations at the borehole locations along with borehole co-ordinates.

A laboratory testing programme, consisting of natural moisture content measurement and grain-size analysis, was performed on selected soil samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets and also in Appendix B.

4. SUBSURFACE CONDITIONS

The boreholes show, below a 0.1 to 0.2 m thick topsoil layer, the presence of granular soils consisting of sand to silty fine sand or sand and silt. These are underlain at most borehole locations by a zone of cobbles and boulders in a sand and gravel matrix, overlying the bedrock. At the borehole locations, the overburden soils extend to depths ranging from 9.8 to 12.8 m below the ground surface.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets, Appendix A. Subsurface profiles along the East Bound Lanes (EBL) and West Bound Lanes (WBL) are presented on Drawing Nos. 1 and 2, respectively. The individual strata are briefly described in the following subsections of this report.

4.1 TOPSOIL

The boreholes contacted a 0.1 to 0.2 m thick topsoil layer. It should be pointed out that at the time of our investigation the ground was mostly frozen and therefore, the description of the soil within the upper several decimeters may not be accurate.

4.2 SAND

Underlying a thin veneer of topsoil, uppermost overburden at the site consists of sand. These granular soils at borehole locations extend to depths ranging from 4.4 m/Elevation 184.2 m to 10.4 m/Elevation 178.2 m. In general, the sand is generally fine to fine to medium with traces to some gravel, although its composition varies from sand with silt to gravelly sand.

In general, there is a tendency for the sand to attain a finer texture from north to south and, as well to become somewhat coarser with increasing depth. The presence of occasional cobbles and boulders was also noted with increasing depth.

The grain-size distribution of selected samples from the sand is given in Figures B-1, B-2 and B-3. Figure B-1 shows the grain-size distribution of eight samples from the predominant fine to fine-medium sand, in an envelope form. The curves indicate a relatively uniform grading, with traces to some silt content. Figure B-2 shows the grading of five samples from the relatively coarser sand also in an envelope form. This is a relatively well-graded sand with some gravel content. Traces of silt are also indicated. Figure B-3 shows in an envelope form the grain-size distribution of three samples from zones with a high silt content.

Standard Penetration tests performed in this granular sand deposit yielded N-values ranging from zero (hammer sank under own weight) to in excess of 75 blows/0.3 m. While some of the lower N-values (i.e. N=0-4 blows/0.3 m) may have been influenced by the inevitable disturbance due to hydrostatic uplift (i.e. high water table), in spite of the fact drilling mud was used to counter balance these uplift pressures; nevertheless, the relative density of the deposit is described as being generally loose or loose-compact with some very loose and compact to dense zones. The presence of very loose or loose zones was verified by the Dynamic Cone Penetration tests (DCPT) performed adjacent to most of the boreholes. Again, there is a tendency for the soil to become less compact or loose from north to south, as well more compact to dense near the bottom of the stratum.

4.3 SAND AND SILT

Boreholes 4 and 4A contacted an approximately 4 m thick layer of well-graded soil, consisting of sand and silt with some gravel and clay. This deposit, which exhibits a glacial

till structure, was contacted below a depth of 4.4 m (Elevation 184.2 m) and extended to 8.3 m (Elevation 180.3 m).

The grain-size distribution of two samples from this deposit is given in Figure B-4 of Appendix B. This indicates 4-13% gravel, 41-44% sand, 30-53% silt and 2-13% clay-size particles.

Based on N-values which range from 6 to 9 blows/0.3 m, the upper zones of this basically granular material is described as loose becoming very dense near the bottom (i.e. below about Elevation 181.5 m), as evidenced by the recorded N-values of in excess of 100 blows/0.3 m, as well as DCPT.

4.4 ZONE OF COBBLES AND BOULDERS IN A SAND & GRAVEL MATRIX

Underlying the sand in Boreholes 2, 3, 6, 7 and 8 and the sand and silt layer in Borehole 4, a very coarse soil consisting of cobbles and boulders in a sand and gravel, some silt matrix, was contacted at depths ranging from 5.2 m (Elevation 183.3 m) to 9.6 m (Elevation 179.0 m). This deposit was found to be about 1 to 7 m thick and extended to the surface of the bedrock. It should be pointed out that in Boreholes 1 and 5 (i.e. most northerly boreholes) where this material was not encountered, the presence of occasional cobbles and boulders was inferred within the sand deposit, at greater depths, closer to the surface of the bedrock.

Due to the extremely coarse nature of this deposit, representative samples could not be obtained for grain-size analysis. Also owing to the coarse nature of the deposit, washboring and diamond drilling had to be utilized to penetrate it. Several Standard Penetration tests, which could be performed, gave N-values in excess of 100 blows/0.3 m (with little or no penetration). These values are unlikely to be reliable due to the presence of oversize particles. Nevertheless, the relative density of the deposit is inferred to be generally dense to very dense.

4.5 SANDSTONE BEDROCK

Bedrock, consisting of reddish brown sandstone, was contacted at depths ranging from 9.8 m (Borehole 2) or Elevation 180.8 m (Borehole 1) to 12.8 m depth/Elevation 175.6 m (Borehole 8). The borehole results indicate that the surface of the bedrock dips generally from northeast to southwest, with a maximum elevation difference of 5.2 m between Borehole 1 (Elevation 180.8 m) and Borehole 8 (Elevation 175.6 m). This represents a drop of approximately 10H:1V over a distance of about 54 m.

The bedrock was cored in all of the eight boreholes, to depths ranging from 0.9 to 3.1 m. The percentage of recovery ranged from 81% to 100%, but more generally 87-100%. The recorded RQD values ranged from 40 to 100% but generally ranged from 58 to 96%.

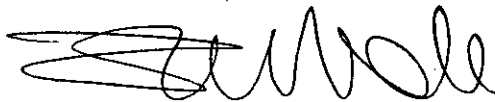
These values indicate a poor to excellent but generally fair to excellent rock quality. The lower values were recorded with the upper zones of the bedrock and therefore, the rock is described as weathered within the upper 0.6 to 1.2 m and generally sound and good to excellent quality below this upper weathered zone.

4.6 GROUNDWATER CONDITIONS

In the piezometers installed in Boreholes 3 and 6, the stabilized groundwater table was measured at 0.5 m and 0.7 m below the ground surface or at Elevation 188.0 and 187.9 m, respectively. In the remaining boreholes, the soil became wet at depths of 1.8 m (Elevation 188.9 m) and 0.9 m (Elevation 188.5 m) in Boreholes 1 and 2, respectively and between 0.6 and 0.8 m (Elevation 187.8-187.6 m) in Boreholes 4, 5, 7 and 8.

From these observations, it appears that the groundwater table at the site generally lies 0.5 to 1.0 m below the ground surface. It should, however, be pointed out that the groundwater table can be expected to fluctuate seasonally and in response to major weather events. It is to a certain extent controlled by the ditches along the railway embankment.

SHAHEEN & PEAKER LIMITED



Z.S. Ozden, P.Eng.



Ramon Miranda, P.Eng.



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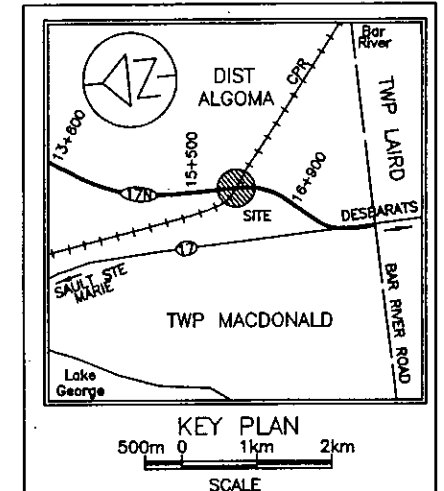
Drawings

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

GWP: 354-94-00



Shaheen & Peaker Limited



The diagram illustrates four symbols used in hydrogeological mapping:

- Bore Hole:** Represented by a solid black circle.
- Bore Hole & Cone:** Represented by a circle with a black center and a white outer ring.
- Water Level at Time of Investigation:** Represented by a circle with a horizontal line through its center and a downward-pointing arrow above it.
- Piezometer:** Represented by a vertical line with a small circle at the bottom and a downward-pointing arrow above it.

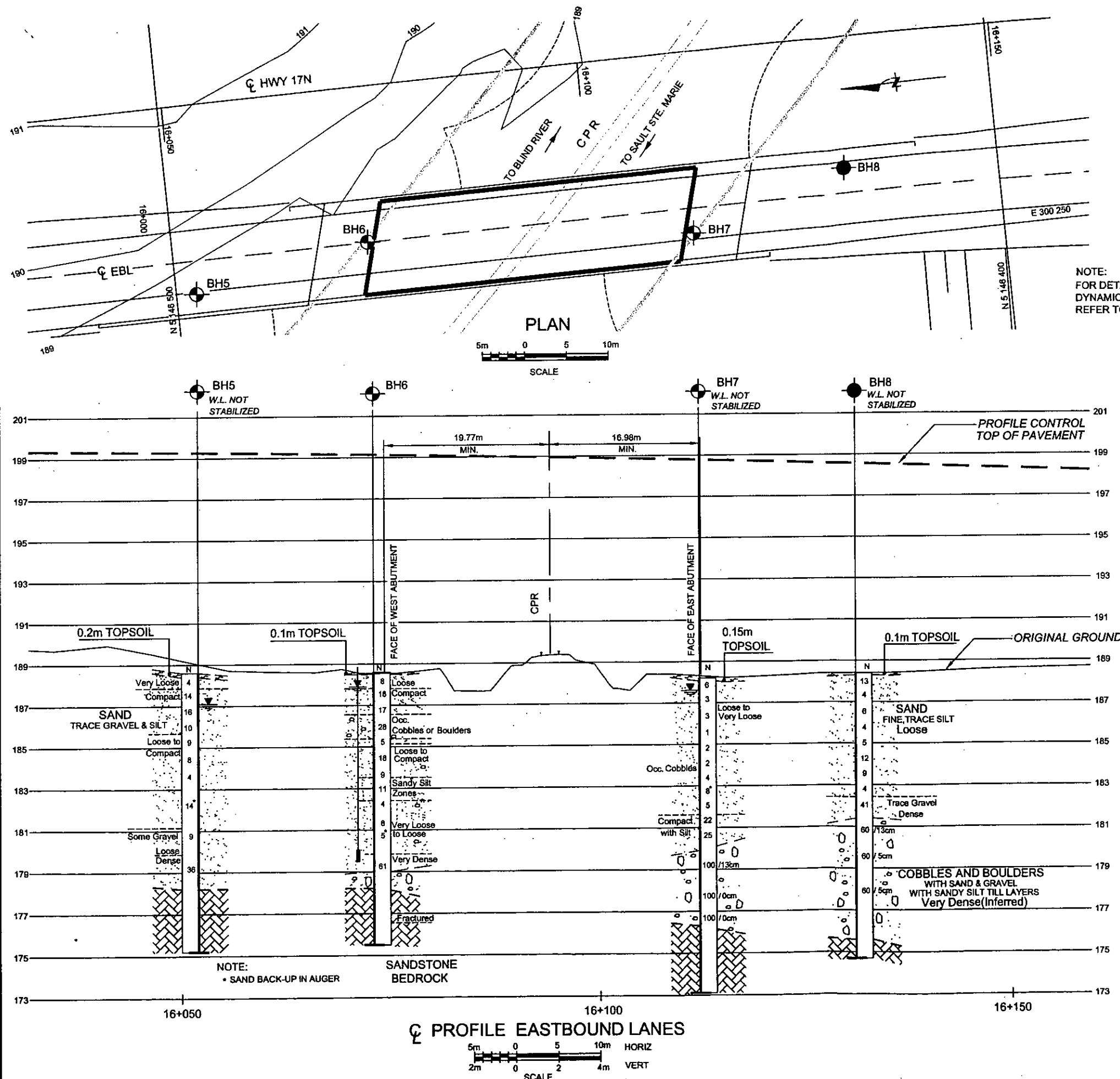
No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
BH5	188.6	5 146 497.6	300 252.3
BH6	188.6	5 146 476.6	300 256.3
BH7	188.2	5 146 437.7	300 253.1
BH8	188.4	5 146 418.7	300 258.9

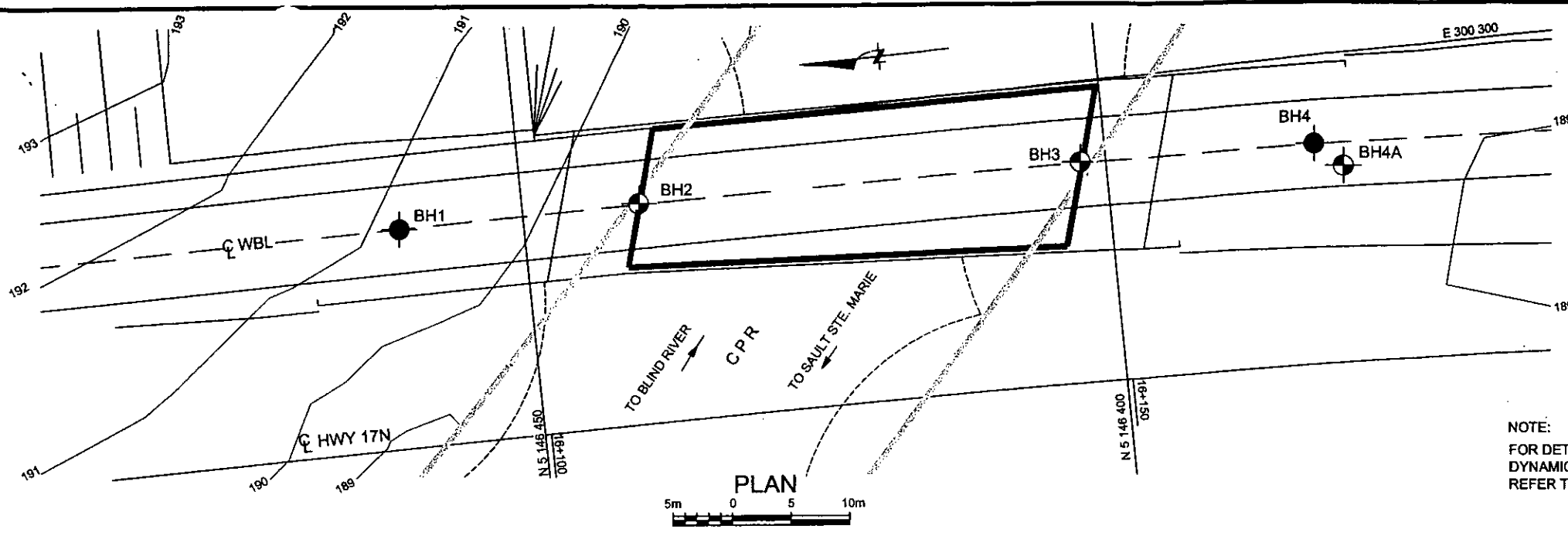
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

REV.			
	DATE	BY	DESCRIPTION

HWY No. 17 (New)			DIST 62
SUBM'D ZO	CHECKED ZO	DATE Mar, 2003	SITE 385-528/1
DRAWN JZ	CHECKED JP	APPROVED	DWG 1





METRIC

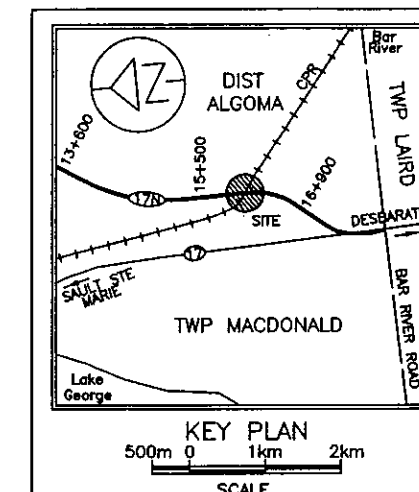
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

NOTE:
FOR DETAILED SUBSURFACE CONDITIONS AND
DYNAMIC CONE PENETRATION TESTS
REFER TO RECORD OF BOREHOLE SHEETS.

IT No.
GWP: 354-94-00

HIGHWAY 17 (NEW) WBL
CPR CROSSING STRUCTURE
BORE HOLE LOCATIONS & SOIL STRATA

Shaheen & Peaker Limited



LEGEND

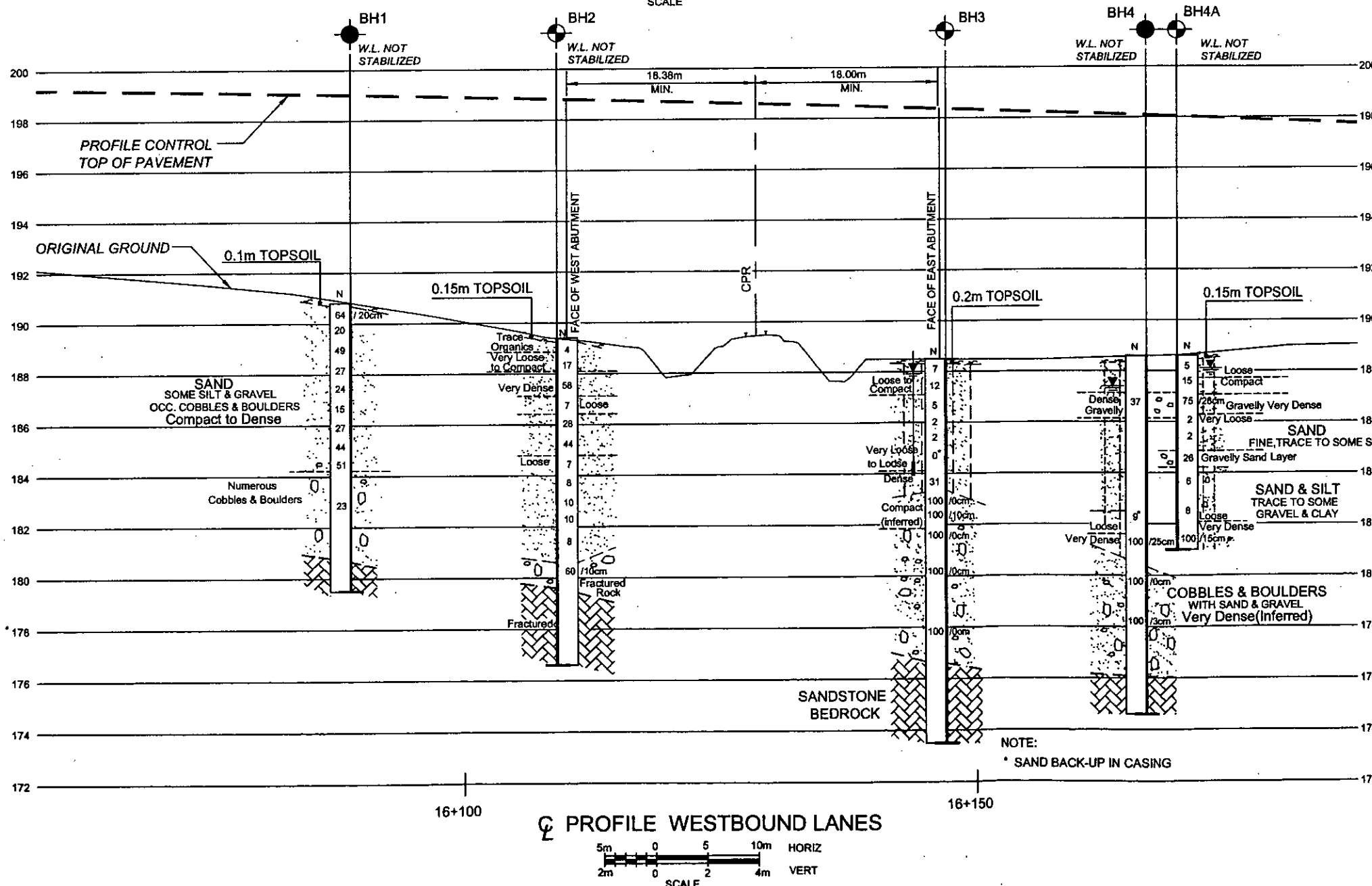
- Bore Hole
- Bore Hole & Cone
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation
April, Nov. and Dec., 2002
- Water Level in Piezometer
- Piezometer

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
BH1	190.8	5 146 460.4	300 293.7
BH2	189.4	5 146 440.4	300 293.6
BH3	188.5	5 146 402.1	300 293.1
BH4	188.6	5 146 381.8	300 292.5
BH4A	188.6	5 146 379.6	300 290.3

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 Materials Engineering and Research Office, Downsview.
Information contained in this report and related documents are
specifically excluded in accordance with the conditions of Section
GC 2.01 of OPS Gen. Cond.

REV.	DATE	BY	DESCRIPTION
Geocres No.41K-54			
HWY No. 17 (New)			DIST 62
SUBM'D ZO	CHECKED ZO	DATE Mar, 2003	SITE 385-528/2
DRAWN JZ	CHECKED JP	APPROVED	DWG 2



Appendix A

Record of Borehole Sheets

SPT1055

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

GWP 354-84-00 LOCATION Echo River to Bar River Road, Sta. 16+089, 19m LL - Coords: N 5 146 460.4; E: 300 293.7 ORIGINATED BY G.I.
DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers, Casing & Washboring & NQ Rock Coring COMPILED BY G.T.
DATUM Geodetic DATE 11/16/2002 CHECKED BY R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELFV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	20 40 60			
190.8 0.0	Ground Surface													
	0.1 m Topsoil		1	SS	64/20									
			2	SS	20									
	damp wet		3	SS	49									
			4	SS	27									
	SAND some silt and gravel occasional cobbles and boulders brown, compact to dense		5	SS	24									
			6	SS	15									
			7	SS	27									
			8	SS	44									
			9	SS	51									
	numerous boulder cobbles and boulders	boulder	10	RC	-									
		boulder	11	RC	-									
			12	SS	23									
		boulder	13	RC	-									
180.8 10.0	SANDSTONE BEDROCK reddish brown		14	NQ RC	Rec. 87%									RQD 87%
179.5 11.3	End of borehole													

SPT1055

1 OF 1

METRIC

LOCATION Echo River to Bar River Road, Sta. 16+109, 19m Lt. - Coords: N 5 146 440.4; E: 300 293.6

ORIGINATED BY G.I.

BOREHOLE TYPE Solid Stem Augers, D.C.P.T., Casing & Washboring & NQ Rock Coring

COMPILED BY G.T.

DATE 11/15/2002

CHECKED BY RA

+ 3, x 3: Numbers refer to Sensitivity

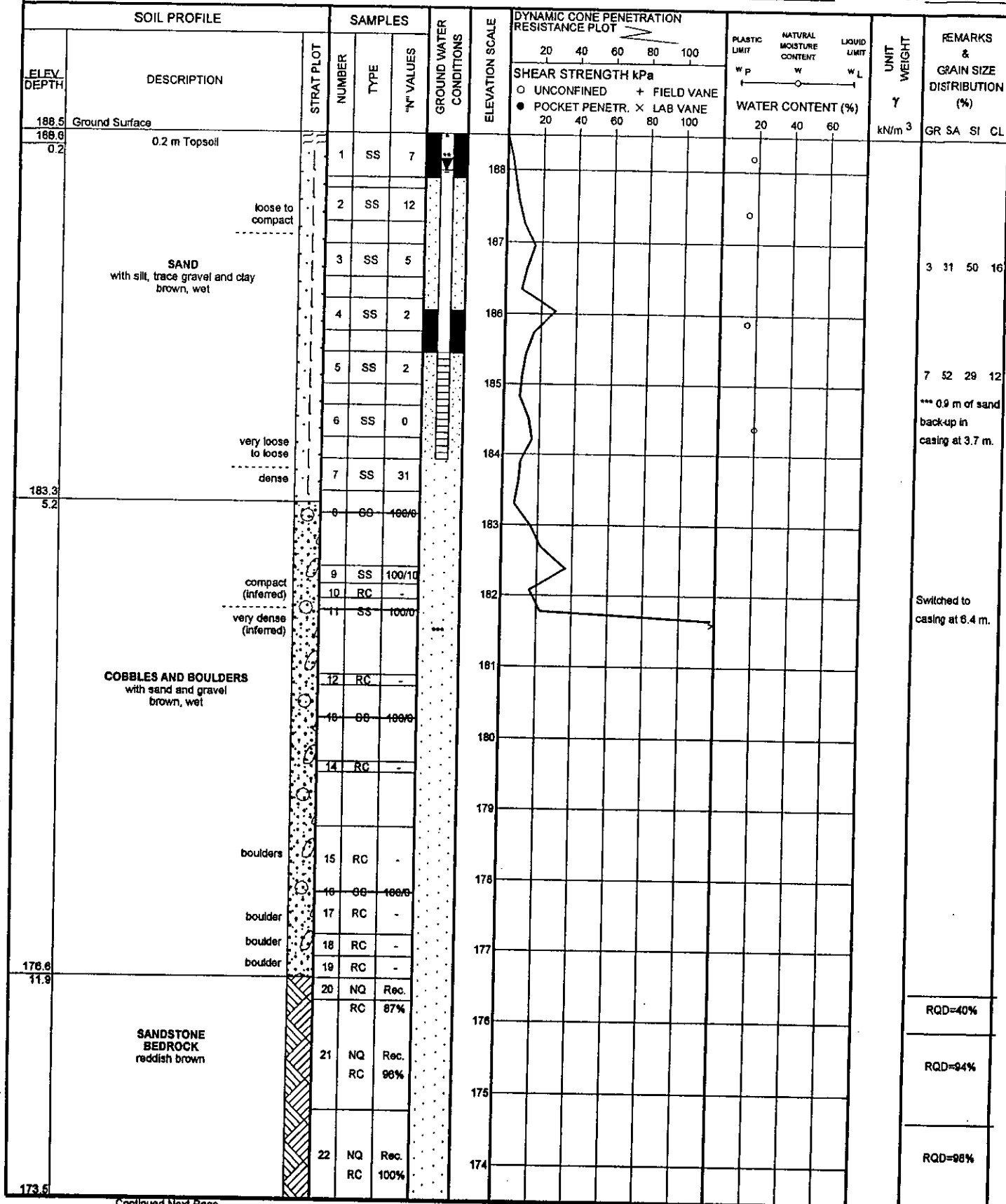
SPT1055

RECORD OF BOREHOLE No 3

1 OF 2

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sta. 16+147, 19m Lt. - Coords: N 5 146 402.1; E: 300 293.1 ORIGINATED BY Y.L.
 DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers, D.C.P.T., Casing & Washboring & NQ Rock Coring COMPILED BY M.L.
 DATUM Geodetic DATE 11/27/2002 CHECKED BY R.A.



RECORD OF BOREHOLE No 3

2 OF 2

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sta. 16+147, 19m LL - Coords: N 5 148 402.1; E: 300 293.1 ORIGINATED BY Y.L.
 DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers, D.C.P.T., Casing & Washboring & NQ Rock Coring COMPILED BY M.L.
 DATUM Geodetic DATE 11/27/2002 CHECKED BY R.A.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
FLY DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH KPa					WATER CONTENT (%)			
						20 40 60 80 100 O UNCONFINED + FIELD VANE • POCKET PENETR. X LAB VANE 20 40 60 80 100					W P W W L 20 40 60					
15.0	End of borehole * Water used to facilitate washboring and rock coring, water level at 0.2 m (not stabilized) and hole open to 4.6 m upon completion. ** Piezometer installed to 4.6 m Water Level on: December 01, 2002 0.6 m (El. 187.9) December 05, 2002 0.5 m (El. 188.0) Dynamic Cone Penetration Test (DCPT) performed from 0 to 6.8 m.					173										

SPT1055

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

GWP 354-04-00 LOCATION Echo River to Bar River Road, Sta. 16+167, 19m Lt. - Coords: N 5 146 381.8; E: 300 292.5 ORIGINATED BY YL
 DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers, Casing & Washboring & NQ Rock Coring COMPILED BY ML
 DATUM Geodetic DATE 11/30/2002 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	20 40 60			
188.8 0.0	Ground Surface													
	0.15 m Topsoil													
			1	SS	37									
	dense gravelly													
	For subsurface condition from 0 to 6.1 m refer to Record of Borehole No. 4A													
182.5 6.1			2	SS	9									
	SAND and SILT with some gravel, reddish grey, wet													
	loose													
	very dense		3	SS	100/28									
180.3 8.3			4	RC	-									
	boulder		5	SS	100/0									
	boulder		6	RC	-									
	boulder		7	RC	-									
	COBBLES AND BOULDERS with sand and gravel very dense (inferred) brown, wet		8	SS	100/0									
	boulders		9	RC	-									
	boulder		10	RC	-									
176.1 12.5			11	NQ RC	Rec. 61%									
	SANDSTONE BEDROCK reddish brown													RQD 59%
174.6 14.0	End of borehole * Water used to facilitate washboring and rock coring, water level at 1.2 m (not stabilized) and hole open to 3 m upon completion.													

+ 3, x 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT1055

RECORD OF BOREHOLE No 4A

1 OF 1

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sta. 16+170, 17m LL - Coords: N 5 146 379.6; E: 300 290.3 ORIGINATED BY Y.L.
DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers & D.C.P.T. COMPILED BY G.T.
DATUM Geodetic DATE 4/19/2002 CHECKED BY R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
188.6	Ground Surface										
0.0	0.15 m Topsoil		1	SS	5						
	brown loose										
	reddish brown compact		2	SS	15						
	reddish brown SAND										
	reddish grey Fine, trace to some slit, wet		3	SS	75/28						
	very dense gravelly										
	very loose		4	SS	2						
			5	SS	2						
	gravelly sand layer, loose to compact		6	SS	26						
184.2											
4.4			7	SS	6						
	SAND AND SILT										
	trace to some gravel and clay reddish grey, wet		8	SS	8						
	loose										
181.0											
7.6	End of borehole		9	SS	100/15						
	* Water level at 0.5 m (not stabilized) and hole open to 0.8 m on completion.										
	Dynamic Cone Penetration Test (DCPT) performed from 0 to 7.4 m.										

RECORD OF BOREHOLE No 5

1 OF 2

METRIC

GWP 354-94-00

LOCATION Echo River to Bar River Road, Sta. 16+052, 23m RL - Coords: N 5 148 497.6; E: 300 252.3

ORIGINATED BY Y.L.

DIST 62 HWY 17 (New)

BOREHOLE TYPE Hollow Stem Augers, D.C.P.T., Casing & Washboring & BQ Rock Coring

COMPILED BY M.L.

DATUM Geodetic

DATE 11/6/2002

CHECKED BY R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
188.6 0.0	Ground Surface												
	0.2 m Topsoil		1	SS	4								
	very loose, moist		2	SS	14								
	wet compact		3	SS	16								
	SAND		4	SS	10								
	trace gravel and silt		5	SS	9								
	brownish grey		6	SS	8								
	loose to compact		7	SS	4								
			8	SS	14								
			9	SS	9								
	some gravel		10	SS	38								
	loose												
	dense												
178.2 10.4	SANDSTONE BEDROCK reddish brown		11	BQ RC	Rec. 89%								
			12	BQ RC	Rec. 100%								
176.2 13.4	End of borehole.												
	* Water used to facilitate washboring and rock coring, water level at 1.5 m (not stabilized) hole open to 5.2 m on completion.												

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 5

2 OF 2

METRIC

GWP 354-84-00

LOCATION Echo River to Bar River Road, Sta. 16+052, 23m Rt. - Coords: N 5 146 497.6; E: 300 252.3

ORIGINATED BY Y.L

DIST 62

HWY 17 (New)

BOREHOLE TYPE Hollow Stem Augers, D.C.P.T., Casing & Washboring & BQ Rock Coring

COMPILED BY M.L.

DATUM Geodetic

DATE 11/6/2002

CHECKED BY R.A.

[illegible]

+³, ×³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

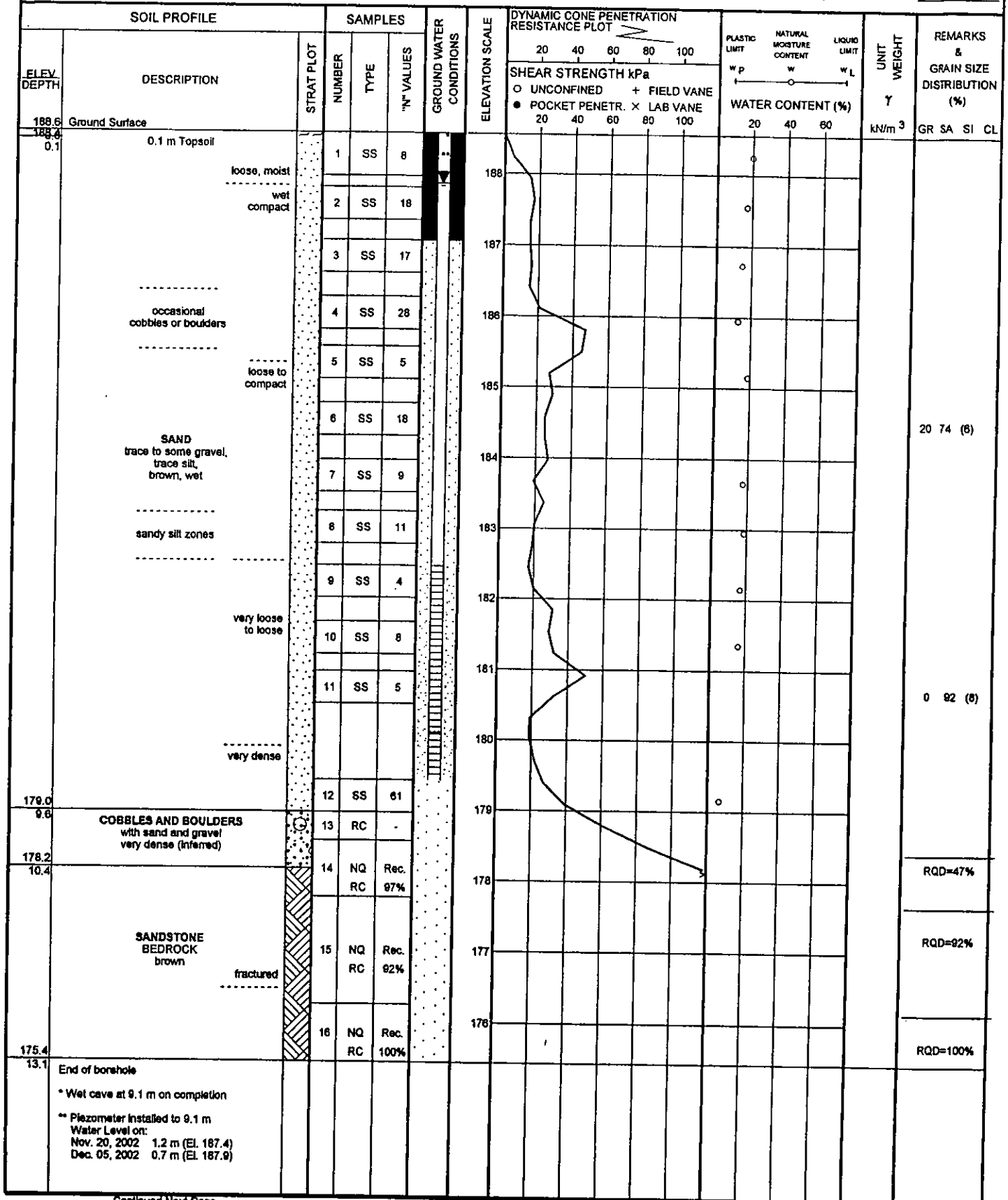
SPT1055

RECORD OF BOREHOLE No 6

1 OF 2

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sta. 16+073, 19m Rt. - Coords: N 5 146 476.8; E: 300 256.3 ORIGINATED BY G.I.
 DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers, D.C.P.T., Casing & Washboring & NQ Rock Coring COMPILED BY G.T.
 DATUM Geodetic DATE 11/17/2002 CHECKED BY R.A.



Continued Next Page

+³, x³: Numbers refer to Sensitivity

20
15 10 5
10 (%) STRAIN AT FAILURE

SPT1055

RECORD OF BOREHOLE No 6

2 OF 2

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sta. 16+073, 19m RL - Coords: N 5 146 476.6; E: 300 256.3 ORIGINATED BY GL
DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers, D.C.P.T., Casing & Washboring & NQ Rock Coring COMPILED BY GT.
DATUM Geodetic DATE 11/17/2002 CHECKED BY RA.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
	*** 2 m of sand back up in auger at 7.5 m. Commence casing and washboring. Dynamic Cone Penetration Test (DCPT) performed from 0 to 10.4 m.																

+ 3 . x 3: Numbers refer to
Sensitivity

20
15 10
(%) STRAIN AT FAILURE

1, SPT1055

1 OF 2

METRIC

GWP 354-04-00 LOCATION Echo River to Bar River Road, Sta. 16+112, 22m RI. - Coords: N: 5 146 437.7; E: 300253.1 ORIGINATED BY Y.L.
DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers, D.C.P.T., Casing & Washboring & NQ Rock Coring COMPILED BY M.L.
DATUM Geodetic DATE 11/25/2002 CHECKED BY R.A.

[illegible]

Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 7

2 OF 2

METRIC

GWP 354-04-00 LOCATION Echo River to Bar River Road, Sta. 16+112, 22m RL - Coords: N: 5 146 437.7; E: 300253.1 ORIGINATED BY YL
 DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers, D.C.P.T., Casing & Washboring & NQ Rock Coring COMPILED BY ML
 DATUM Geodetic DATE 11/25/2002 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								20 40 60 80 100							
								○ UNCONFINED + FIELD VANE							
								● POCKET PENETR. x LAB VANE							
								20 40 60 80 100							

SPT1055

RECORD OF BOREHOLE No 8

1 OF 1

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sta. 16+131, 18m Rt. - Coords: N 5 146 418.7; E: 300 258.9 ORIGINATED BY G.I.
 DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers, Casing & Washboring & BQ Rock Coring COMPILED BY G.T.
 DATUM Geodetic DATE 11/19/2002 CHECKED BY R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N° VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE								
							20	40	60	80	100	20	40	60		
188.4 0.0	Ground Surface															
	0.1 m Topsoil		1	SS	13	*										
	moist															
	wet		2	SS	4											
			3	SS	6											
	SAND		4	SS	4											
	Fine, trace silt, brownish grey loose		5	SS	5											
	reddish brown		6	SS	12											
			7	SS	9											
			8	SS	4											
	trace gravel		9	SS	41											
	dense															
181.4 7.0			10	RC	-											
	boulders		11	SS	60/13											
			12	RC	-											
	boulders		13	RC	-											
			14	SS	60/3											
	COBBLES AND BOULDERS		15	RC	-											
	with Sand and Gravel		16	RC	-											
	with Sandy Silt till layers		17	SS	60/3											
	reddish brown		18	RC	-											
	very dense (inferred)		19	RC	-											
	boulders		20	RC	-											
	boulder		21	RC	-											
176.6 12.6	SANDSTONE BEDROCK		22	BQ	REC.											
	reddish brown			RC	97%											
174.7 13.7	End of borehole.															
	* Water used to facilitate washboring and rock coring, water level not stabilized and hole open to 6.4 m on completion.															

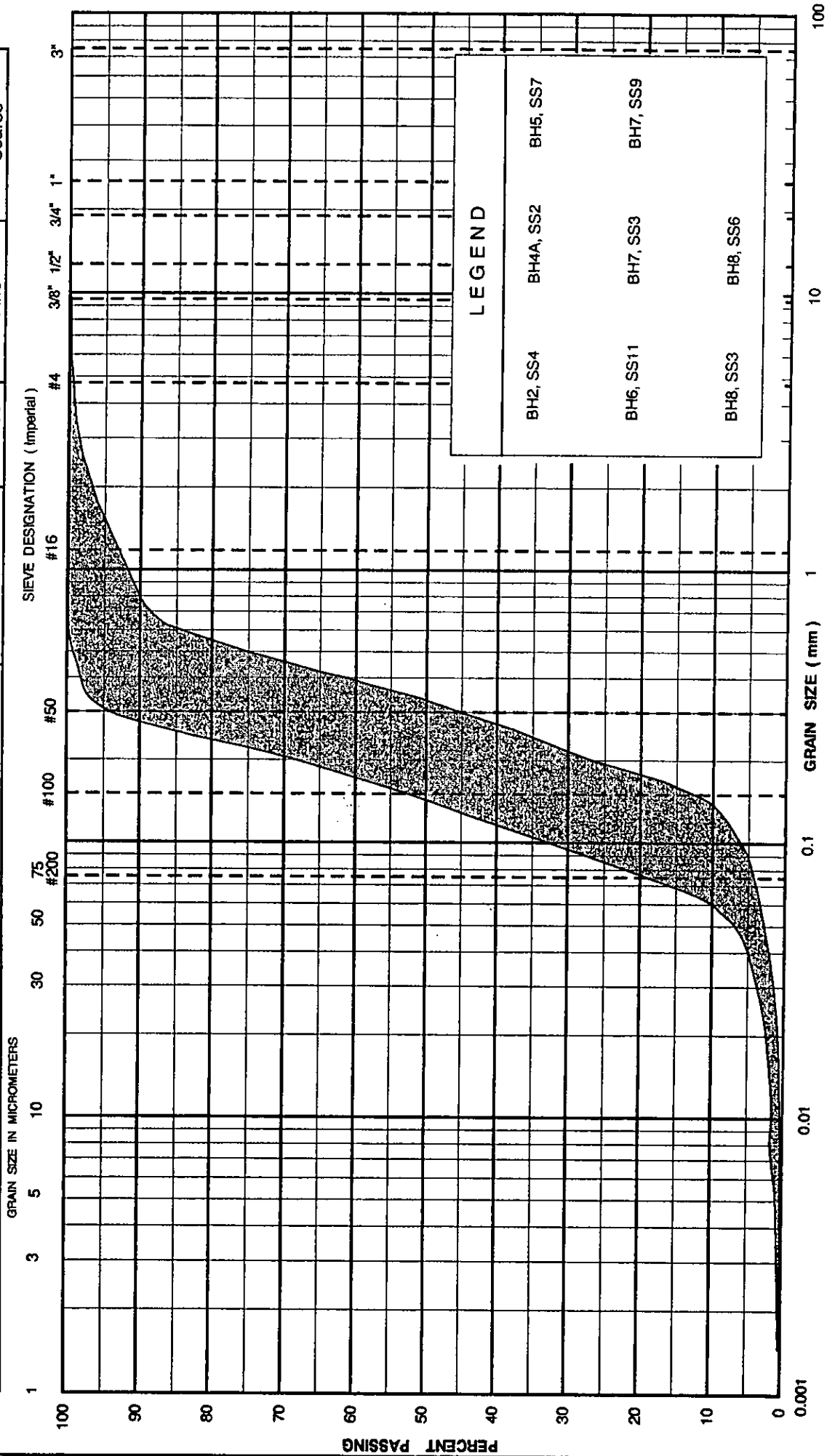
+ 3, x 3: Numbers refer to Sensitivity

20
15
10
(%) STRAIN AT FAILURE

Appendix B1

Laboratory Test Results

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



GRAIN SIZE DISTRIBUTION
SAND, Fine, traces to some silt

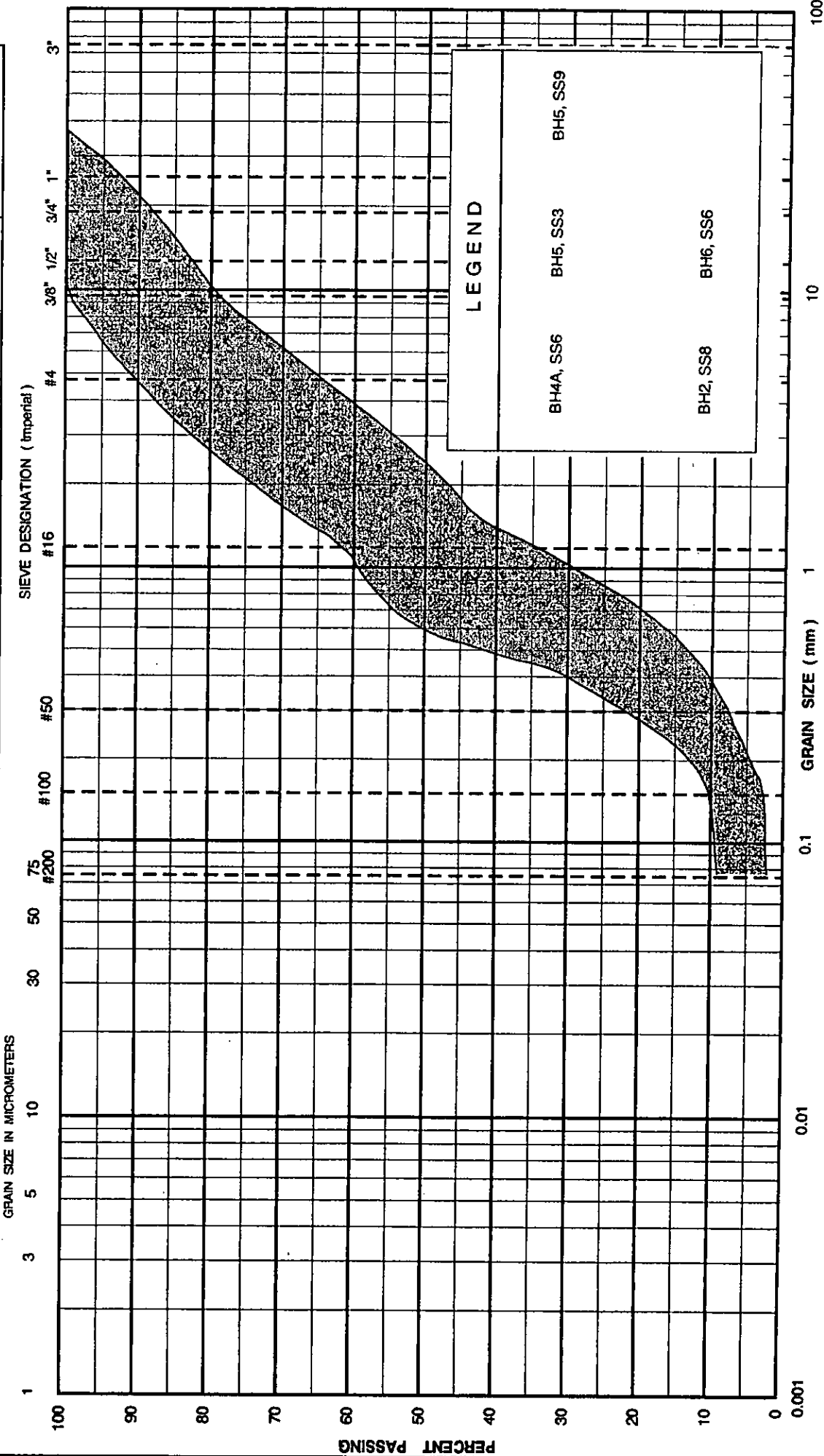
FIG. No. B-1

REF. No. SPT 1055

G.W.P. 354-94-00

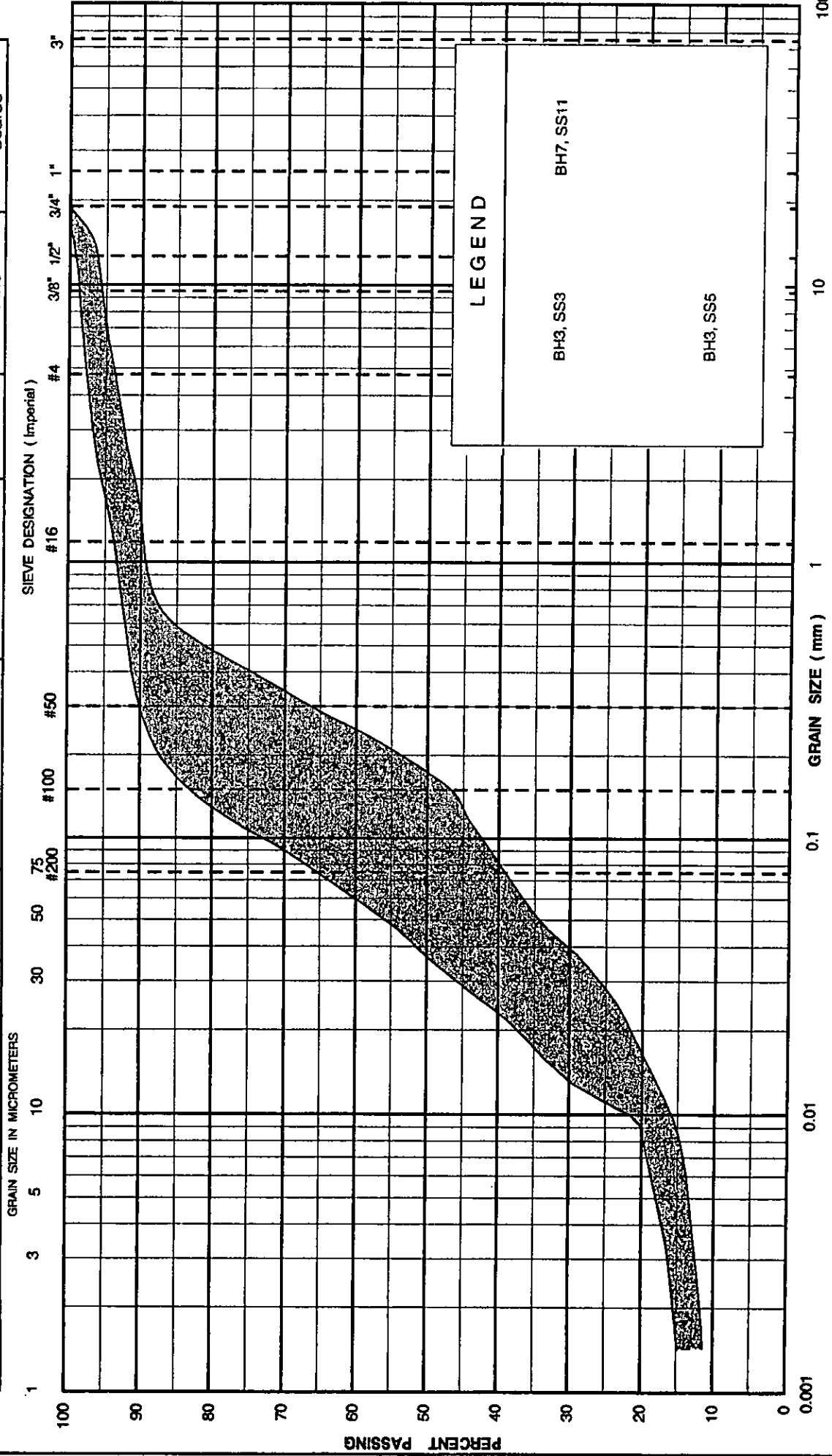
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



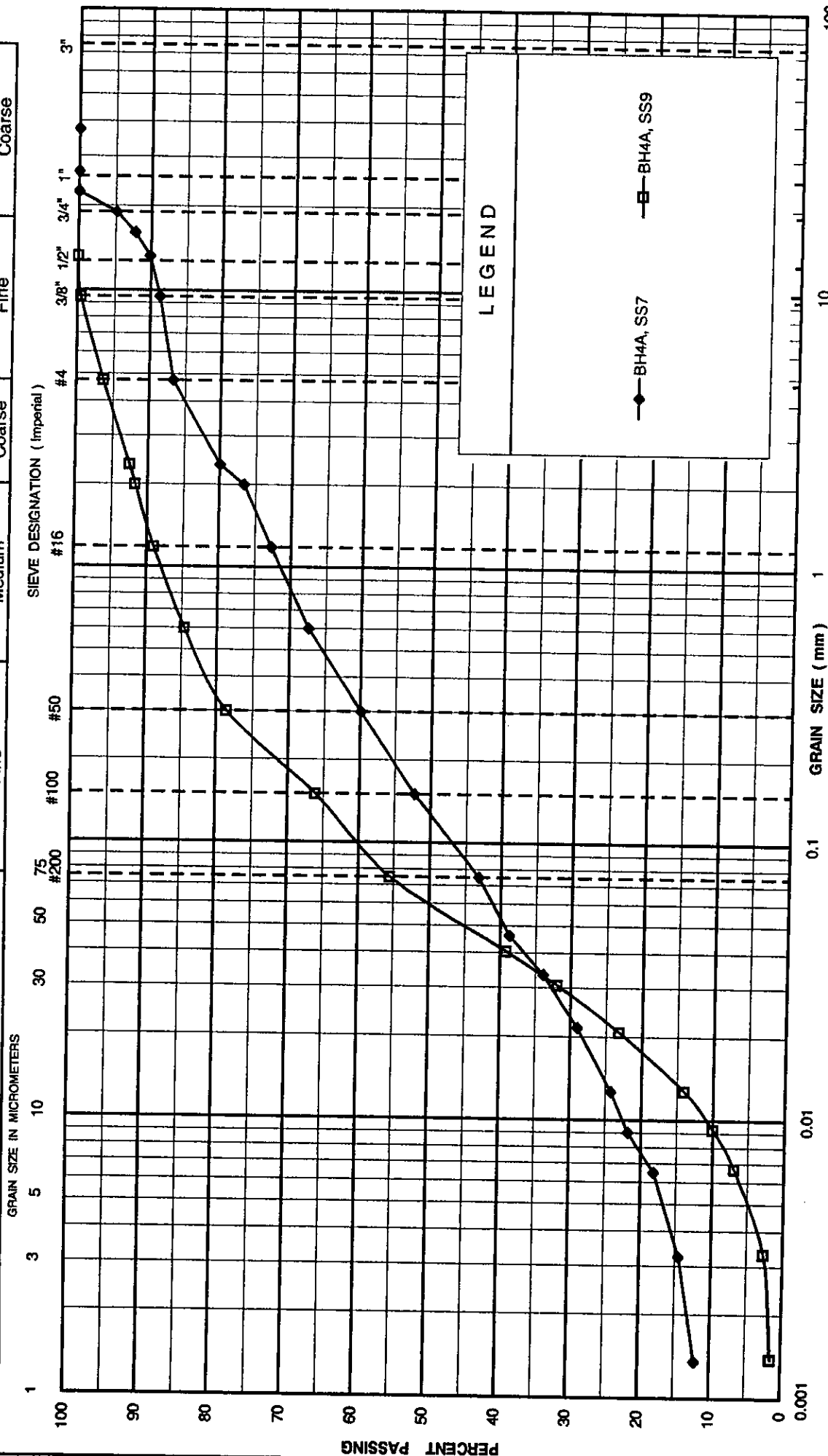
SHAHEEN & PEAKER LIMITED

GRAIN SIZE DISTRIBUTION
SAND with SILT, trace gravel and clay

FIG. No. B-3
REF. No. SPT 1055
G.W.P. 354-94-00

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	



SHAHEEN & PEAKER LIMITED

GRAIN SIZE DISTRIBUTION
SAND and SILT, some gravel and clay

FIG. No. B-4

REF. No. SPT 1055

G.W.P. 354-94-00

Appendix B2

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_r	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_u	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = c_u / c'

PHYSICAL PROPERTIES OF SOIL

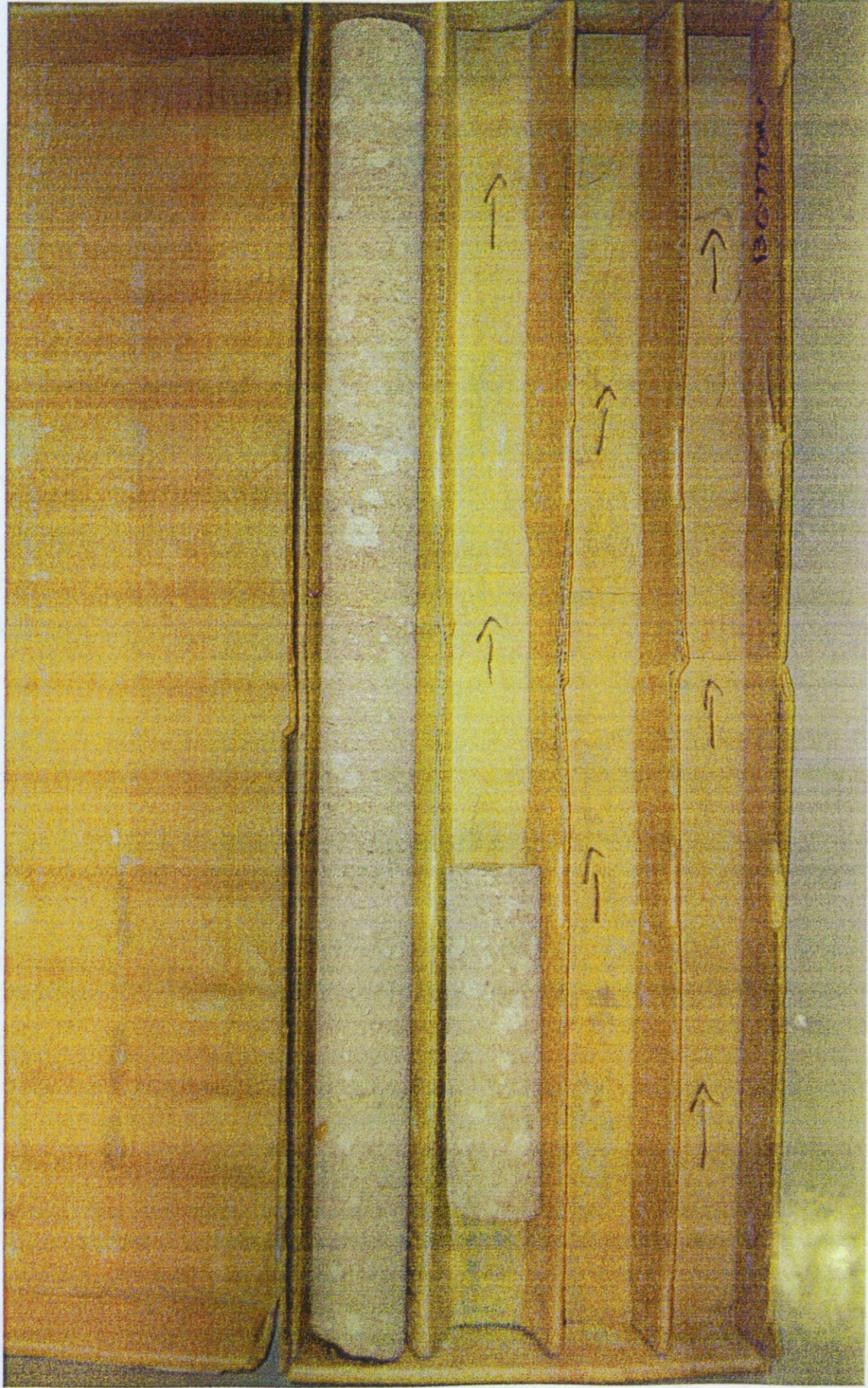
ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_b	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER				D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	w	1, %	WATER CONTENT	D_u	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	S_r	%	DEGREE OF SATURATION	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_L	%	LIQUID LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_p	%	PLASTIC LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	L	1	LIQUIDITY INDEX = $(w - w_p) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	I_c	1	CONSISTENCY INDEX = $(w_L - w) / I_p$	j	kN/m ²	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE			

Appendix C

Geology Map

Appendix D

Photograph



PHOTOGRAPH OF BEDROCK CORE
Borehole 6, Station 16+073, 19m Rt, Sandstone Bedrock

**FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 17 (NEW) TWIN OVERPASS STRUCTURE
AT HURON CENTRAL RAIL (CPR)
HIGHWAY 17 (NEW) FROM ECHO RIVER TO BAR RIVER ROAD
DISTRICT 62, SAULT STE. MARIE, ONTARIO
G.W.P. 354 AND 352-94-00
SITE: 385-526/1; 385-526/2**

Prepared For:

MARSHALL MACKLIN MONAGHAN LTD.

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1055
August 21, 2003**



**shaheen & peaker
limited**

**20 Meteor Drive
Toronto, Ontario
M9W 1A4**

Tel: (416) 213-1255

Fax: (416) 213-1260

WEB SITE: WWW.SHAHEENPEAKER.CA

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5.1 Foundations	8
5.1.1 Steel H-Piles	9
5.2 LATERAL EARTH PRESSURES	12
5.3 Approach Embankments	14
5.4 Construction Comments	15
5.5 Frost Protection	15
6.0 CLOSURE	15

APPENDICES

APPENDIX E: LIMITATIONS OF REPORT

**FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 17 (NEW) TWIN OVERPASS STRUCTURE
AT HURON CENTRAL RAIL (CPR) INTERSECTION
HIGHWAY 17 (NEW) FROM ECHO RIVER TO BAR RIVER ROAD
SAULT STE. MARIE, ONTARIO
G.W.P. 354 AND 352-94-00
SITE: 385-526/1; 385-526/2**

5. DISCUSSION AND RECOMMENDATIONS

The proposed Highway 17 N twin bridges over the Huron Central Railway (CPR) will be approximately 37 m long, single span structures. As shown on Drawing Nos. 1 and 2, at this crossing the Highway will be at a skew to the railway. Each structure (carrying the proposed two-lane east and west bound lanes of the highway) will be approximately 12 to 14 m wide with approximately 26 m wide space (median) in between. The proposed elevation of the highway along the westbound lanes(WBL) is about 199.0 m and 198.5 at the north and south abutments, respectively. As the existing grades generally range from about 189.5 m on the north abutment to about 188.5 m on the south abutment locations, the height of the approach fills at the abutment locations will generally be about 10 m. Along the eastbound lanes (EBL) bridge the proposed and existing ground elevations are approximately 199.0 m and 188.5 m, respectively at the north abutment location while at the south abutment location they are about 0.5 m lower. The anticipated height of embankments at the abutment locations of the EBL bridge is, therefore, about 10.5 m.

The boreholes revealed that beneath a veneer of topsoil, the site is underlain by a sand deposit, which has a wide range of grain size distribution from sand with silt to gravelly sand but is generally fine sand. The sand is water bearing and is generally loose to compact with some very loose and dense zones. The sand extends to depths ranging from 4.4 to 10.4 m. In Boreholes 1 and 5, the sand deposit extends to the surface of bedrock at depths of 10.0 and 10.4 m, respectively while in the remaining boreholes, underlying the sand, there is a very coarse granular layer, immediately above the bedrock. This zone is about 1 to 7 m thick and consists of cobbles and boulders in a matrix of sand and gravel with some silt. Sandstone bedrock was encountered at depths ranging from 9.8 m or at Elevation 180.8 m to 12.8 m (Elevation 175.6 m).

The groundwater table at the site generally lies 0.5 to 1.0 m below the ground surface, but can be expected to fluctuate.

5.1 FOUNDATIONS

Standard Penetration tests performed in the granular sand deposit yielded N-values ranging from a very low value of 1 to in excess of 50 blows/0.3 m, but the recorded values are generally in the 2 to 15 blows/0.3 m range. The use of normal spread footing foundations, including foundations on compacted fill, is considered to be somewhat risky even after a period of surcharging for the structures, which will carry a major highway, because of the possibility of greater than normal differential settlements between the abutments. For this reason, the use of normal spread footing foundations is not recommended, including the use of footings on engineered fill (i.e. on compacted Granular 'A' pad), based on reliability. Densifying the soil by such methods as dynamic compaction can be considered but will probably be difficult to implement immediately adjacent to a railway line. Similarly, the use of compacted base concrete piles (Franki type) will also be impractical. Auger press piles are advantageous for use in water bearing sand deposits, but are costly and offer very little resistance to lateral loading. Drilled caisson foundations bearing in the bedrock could also be considered. Caissons extended at least 0.6 m into the bedrock could be designed for a Factored Bearing Resistance at ULS of the order 15,000 kPa and Bearing Resistance at SLS need not be considered. In this case, however, because of the presence of coarse granular soils, with frequent cobbles and boulders overlying the bedrock, as well as the prevailing high water table, the use of drilled caisson foundations is not considered to represent an economic solution.

At this site, the use of driven steel piles appear to be more appropriate. Because of the presence of dense zones, along with cobbles or boulders, low displacement piles (i.e. H-piles) are better suited in comparison with tube piles. In any event, if an integral abutment bridge type structure is to be considered, then H-piles are required to support the abutment foundations. Because of the presence of cobbles and boulders, the use of heavy pile section such as HP 310 x 110, with reinforced tips, will need to be used. Driven piles at this site suffer from the disadvantage that it is very difficult to determine whether all or some of the piles will reach the bedrock surface, as well as determining the refusal depths in the overburden, due to the presence of frequent cobbles and boulders.

The relative merits and disadvantages of various foundation support types are summarized in Table 1.

Table 1
Summary of Foundation Alternatives

Foundation Type	Comments	Recommendations
<ul style="list-style-type: none"> Normal Spread Footings Spread footings on compacted Granular 'A' pad. 	Excessive differential settlements could occur due to variable and generally very loose to compact subgrade to depths ranging between 4 and 8 m below the existing ground surface.	Not recommended based on reliability.
<ul style="list-style-type: none"> Normal Spread footing or spread footings on compacted Granular 'A' pad after in-situ densification of foundation soils. 	Considered impractical due to the close proximity of the abutment support locations to the existing railway line.	Not recommended based on cost and impracticality of in-situ soil densification application.
<ul style="list-style-type: none"> Compacted Base (Franki) type concrete piles 	Considered impractical due to the close proximity of the abutment support locations to the existing railway line.	Not recommended based on cost and impracticality of application.
<ul style="list-style-type: none"> Drilled caissons extending into bedrock (to approximately 10-13 m below the existing ground surface) 	Difficult to reach bedrock due to the presence of cobbles and boulders in the overburden immediately above bedrock. Also difficult to install due to waterbearing granular overburden.	Not recommended based on reliability.
<ul style="list-style-type: none"> Concrete Piles Timber Piles 	Not recommended due to presence of cobbles and boulders within the overburden, which could damage the piles.	Not recommended based on cost and reliability.
<ul style="list-style-type: none"> Steel H-piles 	Suffers from the disadvantage that pile tips could be damaged, as well as unpredictable pile lengths due to cobbles and boulders.	Considered best option based on a combination of reliability and cost factors.
<ul style="list-style-type: none"> Steel Tube Piles 	Suffers from the same disadvantages as Steel H-piles plus they are high displacement piles and, therefore, less reliable for reaching desired depths.	Not recommended based on reliability.

5.1.1 STEEL H-PILES

As mentioned before, driven steel H-piles are considered to be the best alternative at this site. Because of the presence of intermediate dense zones, along with cobbles and boulders, low displacement piles (i.e. steel H-piles) are better suited in comparison with steel tube piles. As also mentioned before, driven piles at this site suffer from the disadvantage that it is very difficult to determine whether all or some of the piles will reach

the surface of the bedrock, as well as determining the refusal depths in the overburden, due to the presence of frequent cobbles and boulders.

Because of the presence of cobbles and boulders, the use of a heavy pile section such as HP 310x110 with reinforced tips, as per MTO Specification (OPSD 3301.00) is recommended. In addition, the pile tips should be reinforced with H-bearing points such as Titus HPP-S-12 or approved equivalent.

For HP 310x110 steel H-piles which are driven to practical refusal in the bedrock normally a value of 2000 kN value is utilized for Factored Axial Resistance at ULS and Axial Resistance at SLS need not be considered. Normally, when piles are driven to a set capacity in consistently very dense soils, values of 1700 kN at ULS and 1200 kN at SLS are used. The anticipated pile tips depths/Elevations for HP 310x110 H-piles driven to practical refusal (or to required resistance as per Hiley Formula) are given in Table 1.

Table 1
Anticipated Pile Tip Depths/Elevations

Support Location	Reference Borehole No.	Existing Ground Surface Elevation (m)	Estimated Depth of Pile Tip Below Existing Ground Surface (m)	Estimated Pile Tip Elevation (m)	Stratum
North Abutment WBL Bridge	2	189.4	9.9	179.5	Bedrock
South Abutment WBL Bridge	3	188.5	9.8	178.7	Cobbles and boulders
North Abutment EBL Bridge	6	188.6	10.4	178.2	Bedrock
South Abutment EBL Bridge	7	188.2	10.2	178.0	Cobbles and boulders

In view of the fact that some of the piles may be 'hung-up' on boulders before reaching the bedrock surface we recommend the following reduced values be used for HP 310x110 steel H-piles.

	South Abutments (both bridges-Boreholes 3 & 7)	North Abutments (both bridges-Boreholes 2 & 6)
Factored Axial Resistance at ULS	1600 kN	1800 kN
Axial Resistance at SLS	1200 kN	1500 kN

The piles should be driven using a suitably heavy hammer capable of delivering a rated energy of at least 55 kilojoules/blow, but not more than 70 kilojoules/blow.

The driving of the piles should be controlled by a recognized dynamic pile driving formula such as the Hiley Formula. The piles should be driven to about 3 m above the design elevation and driving should then be monitored and controlled by employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standards SS103-10 and SS103-11.

For piles terminating in the overburden, the estimated ultimate resistance as per Hiley Formula is 3600 kN. For practical refusal on bedrock, a refusal criterion of 5 blows for 6 mm for three consecutive sets is recommended. Alternatively, 16 blows for 20 mm or 20 blows for 25 mm penetration can also be used. These values are based on typical hammer energy of 60 kilojoules/blow, with an energy transfer (efficiency) of 40%.

The minimum spacing between the piles should be in accordance with the Canadian Highway Bridge Design Code.

Pile lengths may be different than the estimated values and therefore this aspect will have to be considered in the contract documents and when ordering the piles. If difficulties are encountered to penetrate the pile to sufficient depth then pre-augering may need to be resorted to.

For frost protection, all pile caps should have a permanent earth cover of at least 1.8 m or be provided with an equivalent thickness of extruded rigid exterior-grade polystyrene insulation.

In cohesionless soils, the coefficient of horizontal subgrade reaction can be estimated from:

$$k_s = n_h z / d$$

where

k_s = coefficient of horizontal subgrade reaction (kN/m³)

z = depth (m)

d = pile width (m)

n_h = coefficient related to soil density

Recommended n_h values in the soil (overburden) are 2,500 kN/m³ to about Elevation 180 m and 11,000 kN/m³ below Elevation 180 m.

For preliminary design purposes, the recommended horizontal resistances for HP310x110 steel H-piles are as follows:

Factored Horizontal Resistance at U.L.S. = 110 kN/pile
Horizontal Resistance at S.L.S. = 40 kN/pile

If integral abutments are not constructed (e.g. due to skew of the abutments, etc.) then the lateral resistance of the piles can be supplemented, if desired, by horizontal components of battered piles. In this instance, we recommend that the batter be limited to no more than 4:1 as in practice greater batter is difficult to install.

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven.

In order to minimize future settlements of the embankment fill, we recommend that the approach embankments be placed prior to driving the piles. We also recommend a surcharge of 0.6 m above the final embankment (road) grade, also prior to driving. A surcharge period of six weeks is considered sufficient. If it is not possible to build the abutment fills to their full height plus 0.6 m surcharge at least six weeks prior to driving the piles, then the embankment fills should be constructed at least six weeks before driving the piles as high as possible but not less than at 50% of the final embankment fill height (i.e. about 5 m for an approximately 10 m high embankment).

The abutment wall should be designed and constructed with due consideration to the horizontal clearance between the abutment wall and the railway tracks.

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone. In essence where a false RSS type abutment is to be constructed, the current MTO standard for the flex zone consists of an annular space in between two concentric corrugated steel pipes (CSP's). One of the CSP's surrounds the H-pile (i.e. has a diameter of about 600 mm surrounding the pile) while the second CSP has a somewhat larger diameter; typically 800 mm for a 310 mm H-pile. The annular space in between the CSP's is the 3 m long flex zone. After the pile is driven, the space between the H-pile and the inner CSP is filled with cement bentonite or sand. If a retained soil system is not constructed, then in accordance with MTO Structural Office requirements (Report SO-96-01), the flex zone is provided by augering a 600 mm diameter hole 3000 mm deep and filling it with uniform sand.

5.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free draining granular materials in accordance with the Ontario Ministry of Transportation Standards.

Free-draining backfill materials (i.e. Granular A or Granular B) and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of

earth pressures should be in accordance with C.H.B.D.C. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A'

Unit Weight = 22 kN/m³

Coefficient of Lateral Earth Pressures:

Ka = 0.27

Ko = 0.43

Compacted Granular 'B'

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressures:

Ka = 0.31

Ko = 0.47

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used as per Clause 6.9.2 of CAN/CSA S6-00 C.H.B.D.C. current edition. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6.9.2 of CAN/CSA S6-00 C.H.B.D.C.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

As an alternative to conventional retaining walls, MTO's Retained Soil System may be used. The following should be included in the Contract Documents:

- identify longitudinal extent in plan of the Retained Soil System
- identify in plan transverse space constraints (top of wall and bottom of wall)
- identify elevation of top of wall and bottom of wall
- include NSSP for Retained Soil Systems in Contract Documents

The Retained Soil System (RSS) should be of high performance and high appearance.

The design of the RSS, including the foundation for the facing wall of the RSS, is the responsibility of the RSS provider. It should, however, be pointed out that in order to provide adequate foundation support for the foundation of the facing wall of the RSS, the

upper one metre or so of the existing soil may need to be removed and replaced with compacted suitable on-site excavated soils, at the discretion of the RSS provider. In this event, depending on the site conditions at the time of construction, some dewatering can be expected during the soil replacement/compaction process. Perimeter gravity drainage and pumping from filtered sumps should be suitable for this purpose.

5.3 APPROACH EMBANKMENTS

Based on the borehole results, no foundation failures are anticipated for the proposed 8 to 10.5 m high embankments, provided that all organic soils, weak or otherwise unsuitable materials are removed within the footprint of the embankments, as per MTO Standards, before placing the fill. Assuming properly compacted, acceptable inorganic earth fill material is used (e.g. SSM) 2 horizontal in 1 vertical side slopes can be used for embankment heights of up to 10.5 m. Proper mid-height berm should be provided for slopes in excess of 8 m, as per OPSD and MTO Northwestern Region Practice.

In order to substantially reduce the settlement of the embankments, it is recommended that embankments be built to their final elevation together with about 0.6 m of surcharge, about six weeks or more before driving the piles. This is due to the fact that upper very loose to compact sand deposits can be expected to undergo greater than normally acceptable settlements, as well as to effect the settlement of the embankment fills under their own weight to a large extent prior to the paving of the roadway adjacent to the bridge abutments.

The average thickness of the unsuitable soils to be stripped under the abutment embankments can be expected about 0.2 m. This process and the rolling of the underlying subgrade from the surface may require some dewatering. Depending on the site conditions, this can be accomplished by gravity drainage and pumping from sumps. After stripping, the exposed subgrade should be inspected, approved and properly compacted from the surface, using a suitable compactor.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. select subgrade materials or Granular 'B' – OPSS 1010). As mentioned before, oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven. The embankment fills should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. The selection, placement and compaction of the fill should be carried out under supervision of the Quality Verification Engineer (QVE). The settlement of embankment fills, prepared as described above and surcharged for a period of not less than six weeks as discussed before, should not exceed 25 mm.

Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by prompt seeding or sodding (OPSS 572).

5.4 CONSTRUCTION COMMENTS

At the time of our investigation, the groundwater table was recorded at a depth of about 1.0 to 1.5 m at the north approach and proposed north abutment locations of the WBL (i.e. Boreholes 1 and 2, where the existing grade is relatively higher) and between 0.5 and 0.8 m below the ground surface in the remaining areas (i.e. along the EBL and south abutment and south approach of the WBL structure). As was discussed before, this high water table may necessitate dewatering after stripping, in order to properly roll and surface compact the exposed subgrade, depending on the site conditions at the time of construction. If dewatering is required, this can be achieved by gravity drainage from dug ditches and pumping from strategically placed sumps. It is our opinion that pumping from the existing ditches along the railway would be very useful for this purpose.

5.5 FROST PROTECTION

Design frost penetration for the general area is 1.8 m. Therefore, a permanent soil cover of not less than 1.8 m or its thermal equivalent is required for frost protection of pile caps.

6.0 CLOSURE

The Limitations of Report, as quoted in Appendix D, are an integral part of this report.

SHAHEEN & PEAKER LIMITED



Z.S. Ozden, P.Eng.



Ramon Miranda, P.Eng.



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Appendix E

Limitations of Report

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. 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, excavating, 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 the details stated in this report.

The comments made 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. For example, 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 conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

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