

GEOCRES No. 30L14-42DIST. CR REGION W.P. No. 439-97-00CONT. No. W. O. No. STR. SITE No. 34-102HWY. No. 3LOCATION Mill Race Bridge
WainfleetNo of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

OVERSIZE DRAWING(S)

GEOCRES No
30614-42

Final

**GEOTECHNICAL INVESTIGATION
HWY 3/MILLRACE BRIDGE REPLACEMENT
WAINFLEET, ONTARIO
W.P. 439-97-00**

Prepared For:

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Prepared by:

SHAHEEN & PEAKER LIMITED

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**Project: SP2531
April 26, 1999**

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FACTUAL

1. INTRODUCTION

This report summarizes the results of a geotechnical investigation for the proposed Highway 3/Mill Race bridge replacement. The investigation was authorized by Paul Theil Associates Limited.

This report contains geotechnical information pertaining to structure foundations, approach embankments and related earthworks between Station 18 + 275 m and Station 18 + 340 m.

2. SITE DESCRIPTION

The site is located at the present crossing of the Mill Race creek at Highway 3, located west of Hartwell Street, in the Township of Wainfleet.

The existing crossing, built in 1930, consist of a single span rigid concrete structure with cross ties below the creek level. The clear span of the existing structure is 6.88 m.

The floor of the creek bed is about 4.0 m below the road surface. At the time of the investigation, the depth of the water in the creek was about 0.1 m.

3. PROCEDURE

The field investigation was carried out between the period of October 28 and 30, 1998. All underground utilities were cleared prior to the commencement of the fieldwork.

The fieldwork consisted of drilling four boreholes. The boreholes were advanced by solid stem augers mounted from a truck-mounted CME 75 rig operated by Groundwork Drilling Inc.

Disturbed samples were recovered by means of a 50 mm O.D. split spoon sampler driven into the soil according to the specification of the Standard penetration test (ASTM D 1587). In addition relatively undisturbed shelly tube samples were taken from the cohesive deposit. Field vane tests were carried out to determine the in-situ shear strength of cohesive soils.

The boreholes for the bridge abutment (borehole nos. 1 and 2) were advanced to depths varying from 18.8 to 31.0 m. Dynamic cone penetration tests were carried out below these depths to refusal.

Laboratory testing was carried out on representative samples to identify and determine the physical properties of the overburden including:

Natural moisture content

Grain size distribution

Atterberg Limit

Unit weight

Quick Triaxial test

Consolidation test

The elevations of the boreholes were referenced to a local geodetic benchmark No. 141, located at Station 18 + 465.030, at Elevation 177.075 m.

Piezometers were installed in boreholes 1 and 2 for monitoring long term ground water levels.

4. SITE GEOLOGY AND SUBSURFACE CONDITIONS

Geologically, the site is located on the flat Haldimand clay plain consisting of deep deposits of lacustrine clay.

The subsoil conditions essentially consist of about 16.2 m thick clay to silty clay overlying clayey silt of possible glacial till origin. The clayey silt extends to depth greater than 31.0 m. The silty clay is desiccated and forms a crust in the upper layers and becomes very weak with depth. A surficial fill layer of stiff to very stiff clayey silt varying in thickness from 1.2 to 2.0 m overlies the clay deposit. Buried organic peat was found at boreholes 3 and 4.

The boundaries of the different strata, together with the field and laboratory test results, appear on the Record of Borehole sheets appended to this report. Also refer to the sheets for the locations and elevations of the boreholes.

Stratigraphical section of the subsurface conditions is shown on Drawing 4399700-A. Detailed description of the different strata is provided below.

Clayey Silt (Fill)

Cohesive material consisting of stiff to very stiff clayey silt fill extends to depths varying from 1.2 to 2.0 m below existing grade, i.e. to elevation 174.1 to 173.0 m. At Borehole 3, the clayey silt overlies sand fill. Some organics were found in borehole 2. The fill was probably placed during the construction of the existing structure and embankments. Close to the existing structure, the depth of the fill will probably extend to the underside of the existing footings, i.e., 4.1 m. Typical gradation curve and plasticity chart are given in Appendix as Figures 1 and 2.

Organic Peat

Very loose to loose organic peat was found underneath the fill materials at boreholes 3 and 4. The N value varies from 4 to 8 blows. The thickness varies from 0.5 to 1.7 m and the bottom of the peat deposit corresponds to elevations 174.1 to 173.0 m.

Clay to Silty Clay

This lacustrine deposit underlies the fill in boreholes 1 and 2 and below the organic peat in boreholes 3 and 4. This deposit is fissured and forms a desiccated crust in the upper layers and becomes weak below 4.0 to 5.0 m depth. Generally above elevation 171.0 m, the crust is very stiff with shear strength in excess of 110 kPa. At boreholes 1 and 2, between elevation 171 and 170 m, the material becomes stiff, as evident by the quick Triaxial shear strength of 70 kPa. Below elevation 170.0 m, the deposit becomes soft to stiff with shear strength varying from 17 to 55 kPa. The findings indicate that the soil, below the crust, at borehole 2 is significantly weaker than borehole 1. This deposit extends to a depth of 16.2 to 16.3 m, i.e. bottom of deposit at elevation 159.0 m.

Groundwater Conditions

The groundwater levels in the open holes, on completion of drilling, varied from 14.5 m at borehole 2 to 23.5 m at borehole 1. These water levels were not stabilized during the course of the fieldwork. The stabilized water levels in the piezometers, measured after 3 weeks from completion of drilling, varied from 3.22m (Elevation 172.0 m) at BH1 to 2.3 m (Elevation 173.0 m) at BH2.

The water level in the creek is at about elevation 171.3 m.

The test results are summarized as follows:

Property	Range	Average
Natural Moisture Cont. %	25 – 55	40
Liquid Limit (%)	41 – 61	51
Plastic Limit (%)	20 – 31	26
Plasticity Index	21 – 32	27
Unit Weight (kN/cu.m)	16.6 – 19.6	18.1

From the plasticity chart (Figure 3), this deposit is classified as inorganic clay to silty clay of intermediate to high plasticity. Typical grain size distribution curves are given in Figure 4.

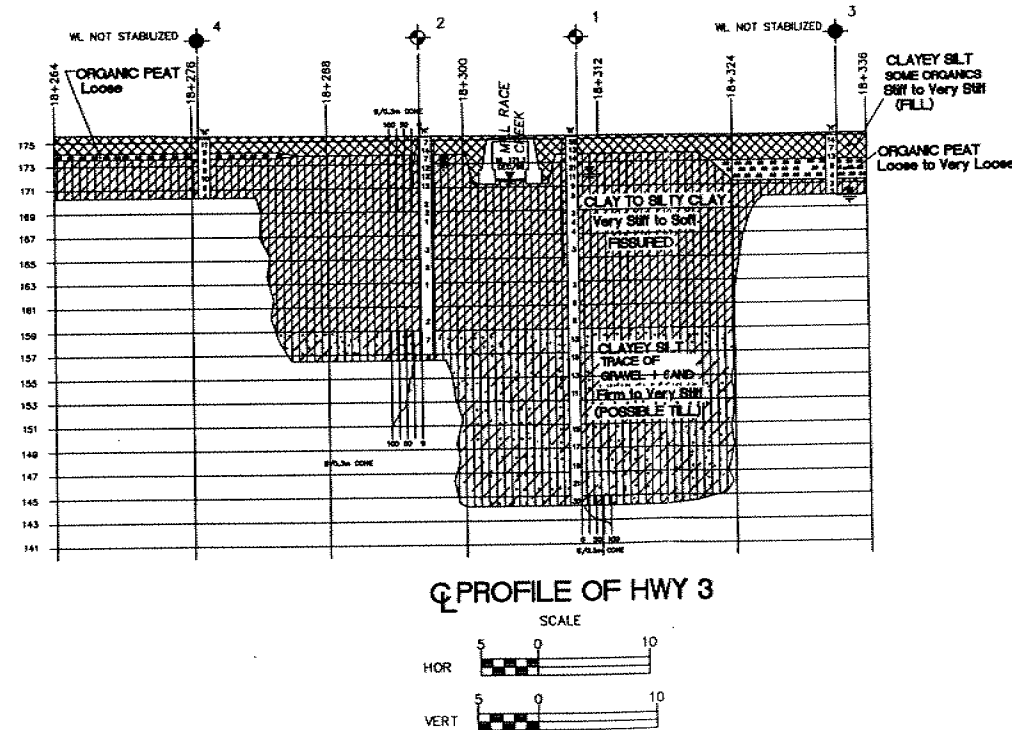
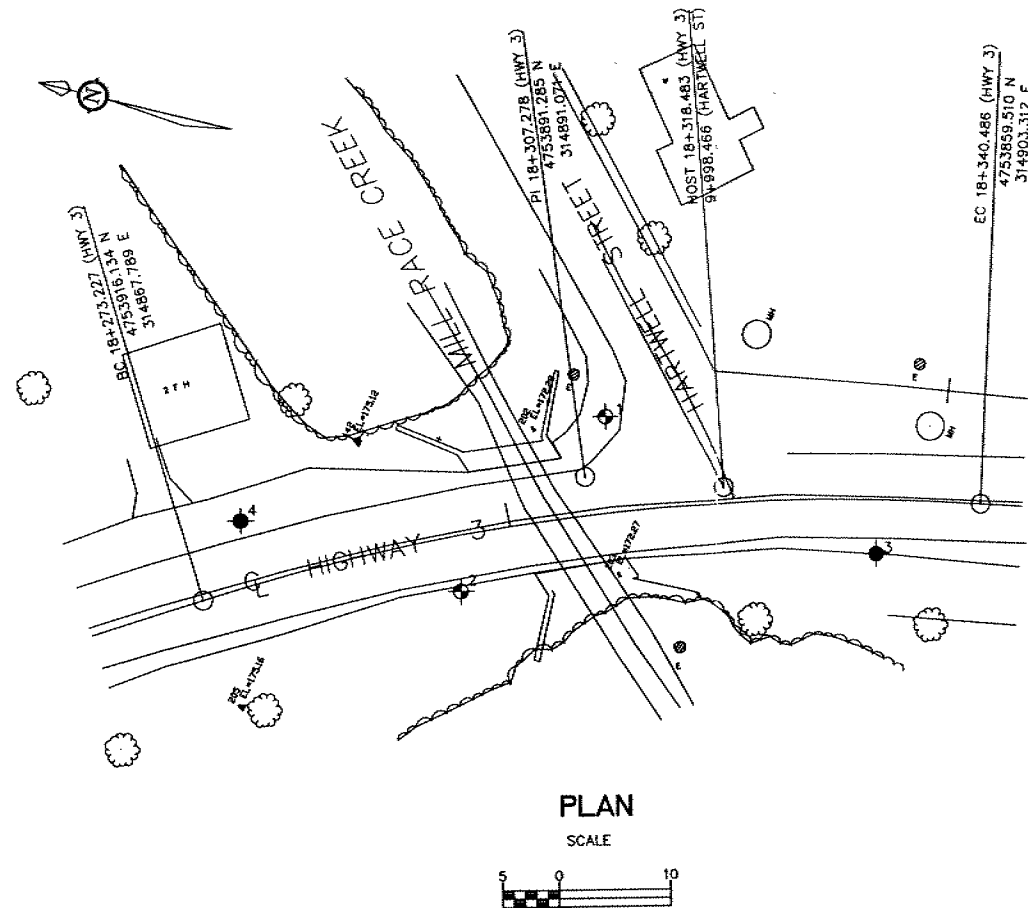
A consolidation curve, carried out on the test sample (Sample no. 11 from borehole 2), is given in Figure 5. The C_c value measured is 0.279. Based on the consolidation curve, the clay materials are interpreted to be normally consolidated.

Clayey Silt (Possible Till)

Underneath the clay to silty clay, the clayey silt deposit was encountered in both the deep boreholes. It is possibly derived from the glacial till origin. This deposit extends to the full depth of both the boreholes, to the maximum depth of 31.0 m, i.e. to elevation 144.2 m. The field shear strength varies from 30 to 110 kPa with N values varying from 7 to 30 blows. The consistency varies from firm to very stiff.

The moisture content varies from 19 to 30 percent. The liquid limit varies from 33 to 39 percent with the plasticity index varying from 7 to 9. The plasticity chart and the grain size distribution curves are given in Figures 6 and 7.

Drawings



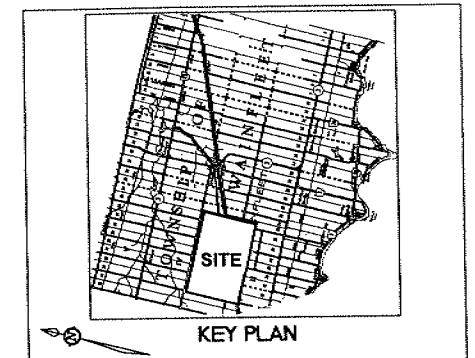
METRIC
DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETERS + METERS.

CONT No
WP No 439-97-00

HWY 3 & MILL RACE BRIDGE
WAINFLEET, ONT.
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

SHAHEEN & PEAKER LIMITED



LEGEND

- Bore Hole
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- W L at time of investigation Dec 1998
- PIEZOMETER

No	ELEVATION	STATION	OFFSET
1	175.2	18+308.8	42.3 LT
2	175.3	18+295.0	25.9 RT
3	175.0	18+331.7	25.5 RT
4	175.3	18+278.1	28.0 LT

NOTE

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

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

DATE	BY	DESCRIPTION
Geocres No		
FWY No	3	DIST 4
SUBMD	8 B	CHECKED
DRAWN	M Z	CHECKED
DATE	Dec 12, 1998	SITE 34-102
		DWG 4398700-A

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

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_f	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_r	kPa	REMOULDED SHEAR STRENGTH
S_f	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
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

PHYSICAL PROPERTIES OF SOIL

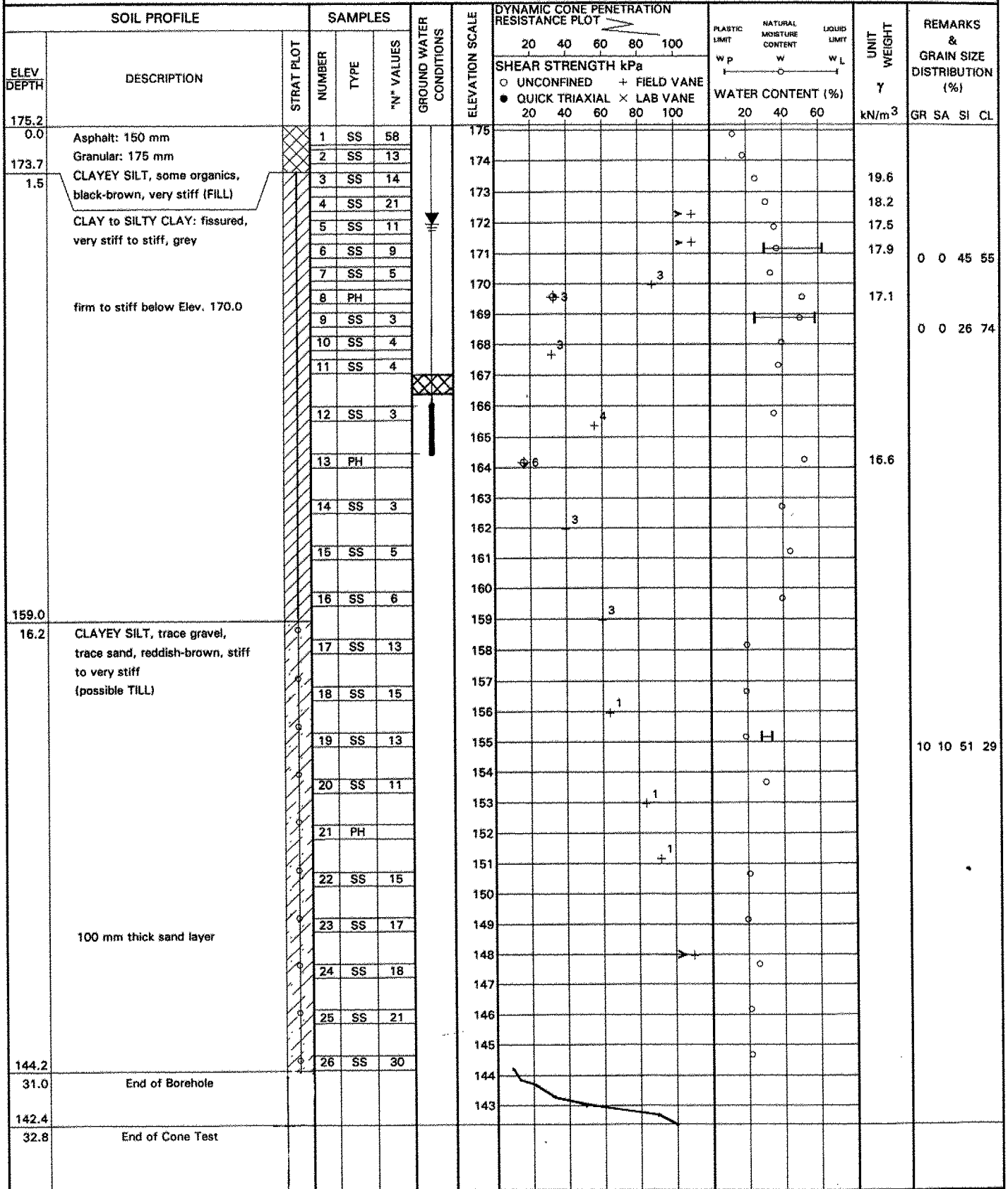
ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT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

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 439-97-00 LOCATION Proposed Bridge Replacement, Wainfleet ORIGINATED BY GI
DIST 4 HWY 3 BOREHOLE TYPE Solid Stem Augers COMPILED BY EP
DATUM Geodetic DATE 10.28.98 & 10.29.98 CHECKED BY SB



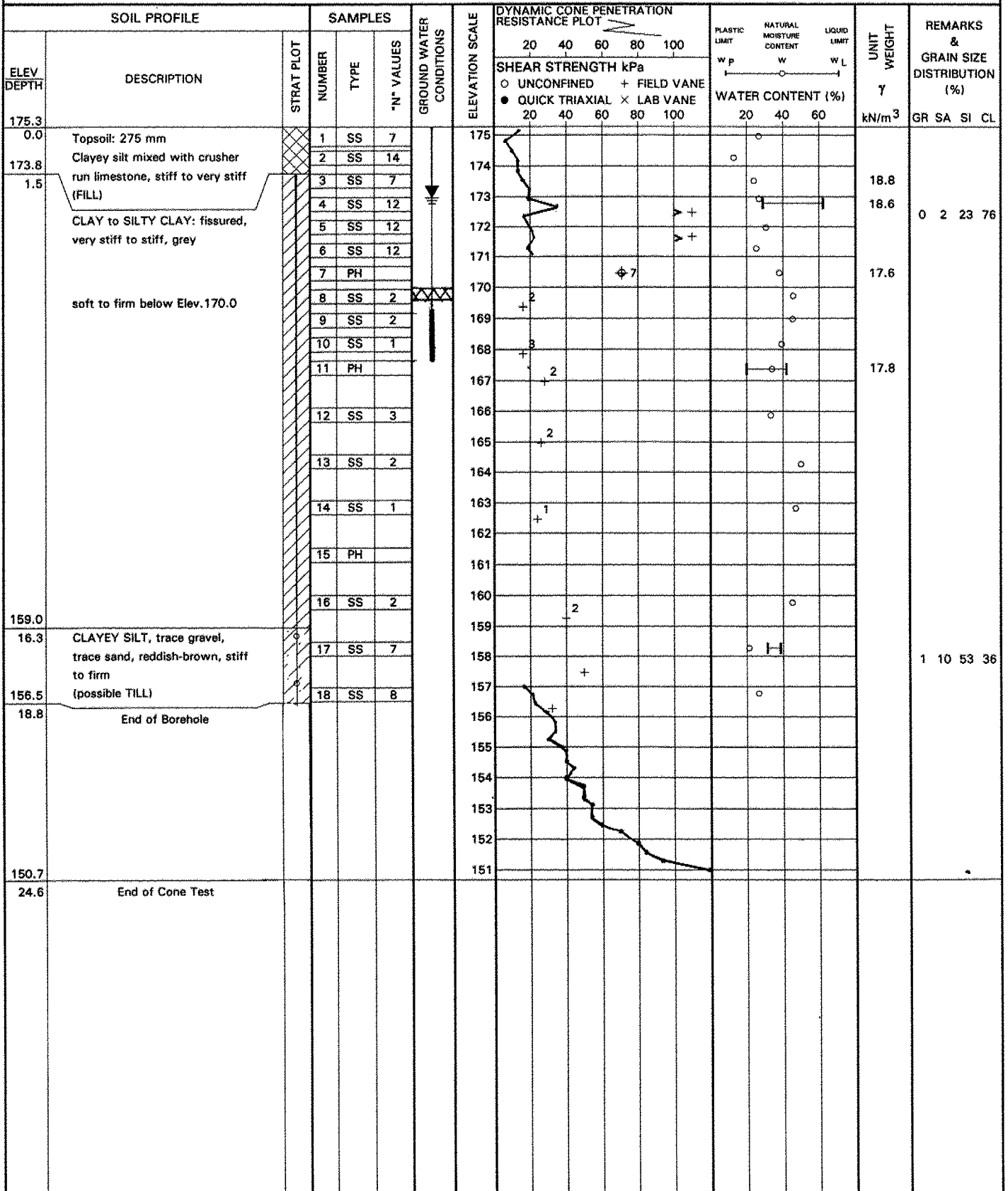
+ 3, x 3: Numbers refer to 20
Sensitivity 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 439-97-00 LOCATION Proposed Bridge Replacement, Wainfleet ORIGINATED BY GI
DIST 4 HWY 3 BOREHOLE TYPE Solid Stem Augers COMPILED BY EP
DATUM Geodetic DATE 10.30.98 & CHECKED BY SB






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

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 439-97-00 LOCATION Proposed Bridge Replacement, Wainfleet ORIGINATED BY GI
DIST 4 HWY 3 BOREHOLE TYPE Solid Stem Augers COMPILED BY EP
DATUM Geodetic DATE 10.30.98 & CHECKED BY SB

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										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
175.0							20	40	60	80	100									
0.0	Crusher Run Limestone: 450 mm clayey silt overlying sand, stiff to very stiff (FILL)		1	SS	14															
			2	SS	7															
173.0			3	SS	13															
2.0	ORGANIC PEAT, black, loose to very loose		4	SS	8															
171.3			5	SS	4															
3.7	SILTY CLAY, pieces of wood in upper 0.3 m, grey, firm		6	SS	3															
169.9			7	SS	3															
5.0	End of Borehole																			
<div>Note: 1) *WL not stabilized</div>																				

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

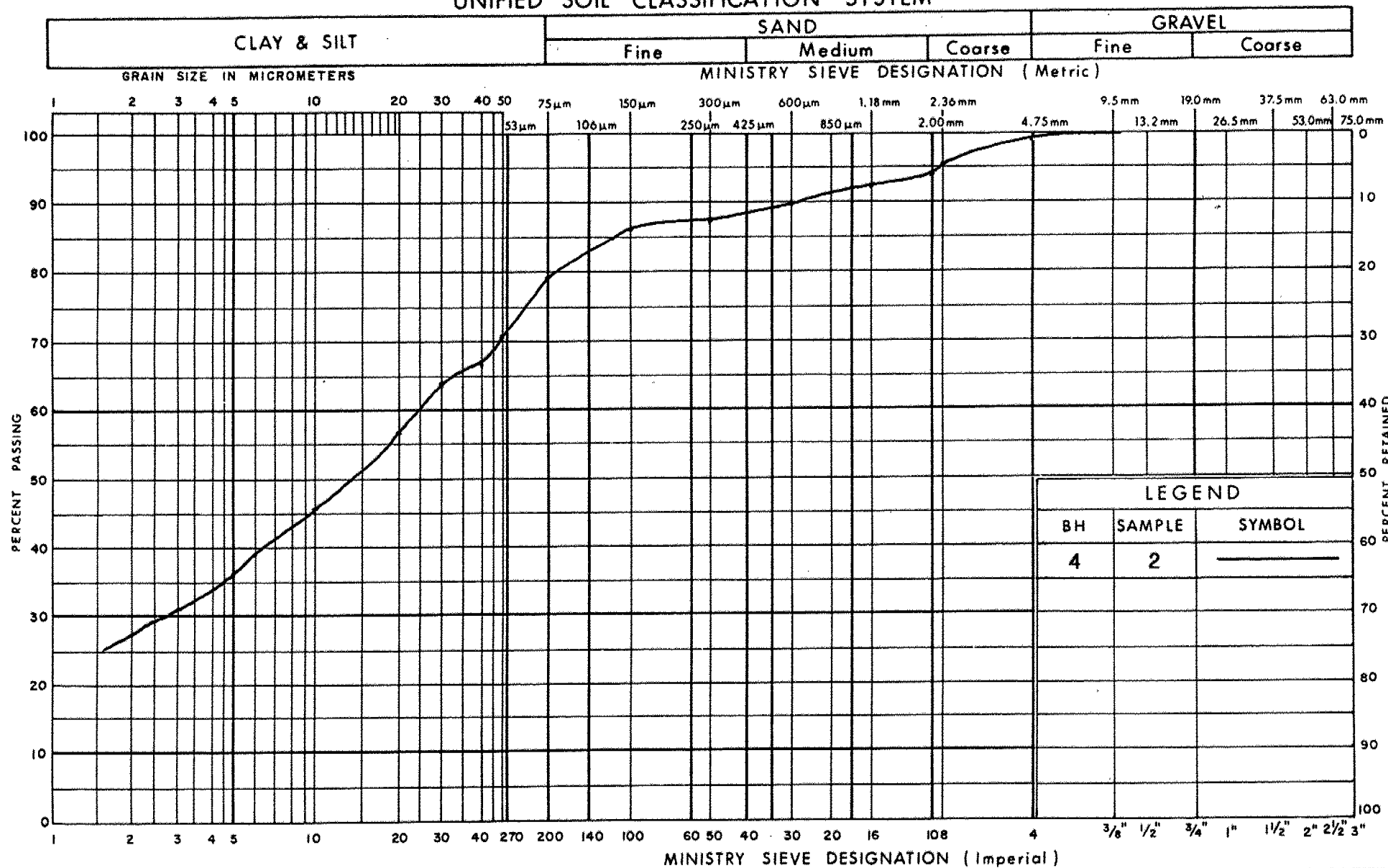
W.P. 439-97-00 LOCATION Proposed Bridge Replacement, Wainfleet ORIGINATED BY GI
DIST 4 HWY 3 BOREHOLE TYPE Solid Stem Augers COMPILED BY EP
DATUM Geodetic DATE 10.29.98 & CHECKED BY SB

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100		
175.3	Asphalt: 125 mm		1	SS	11	175							
174.1	Granular base: 150 mm		2	SS	9	174						18.7	1 20 52 27
1.2	clayey silt, stiff (FILL)		3	SS	8								
173.6	ORGANIC PEAT, loose		4	SS	9	173						18.8	
1.7	SILTY CLAY, fissured, grey, very stiff to stiff		5	SS	10	172						17.6	
	stiff		6	SS	8	171							
170.3	End of Borehole		7	SS	4								
5.0													

Note:

- 1) Borehole dry upon completion of drilling
- 2) *WL not stabilized during the course of fieldwork

UNIFIED SOIL CLASSIFICATION SYSTEM

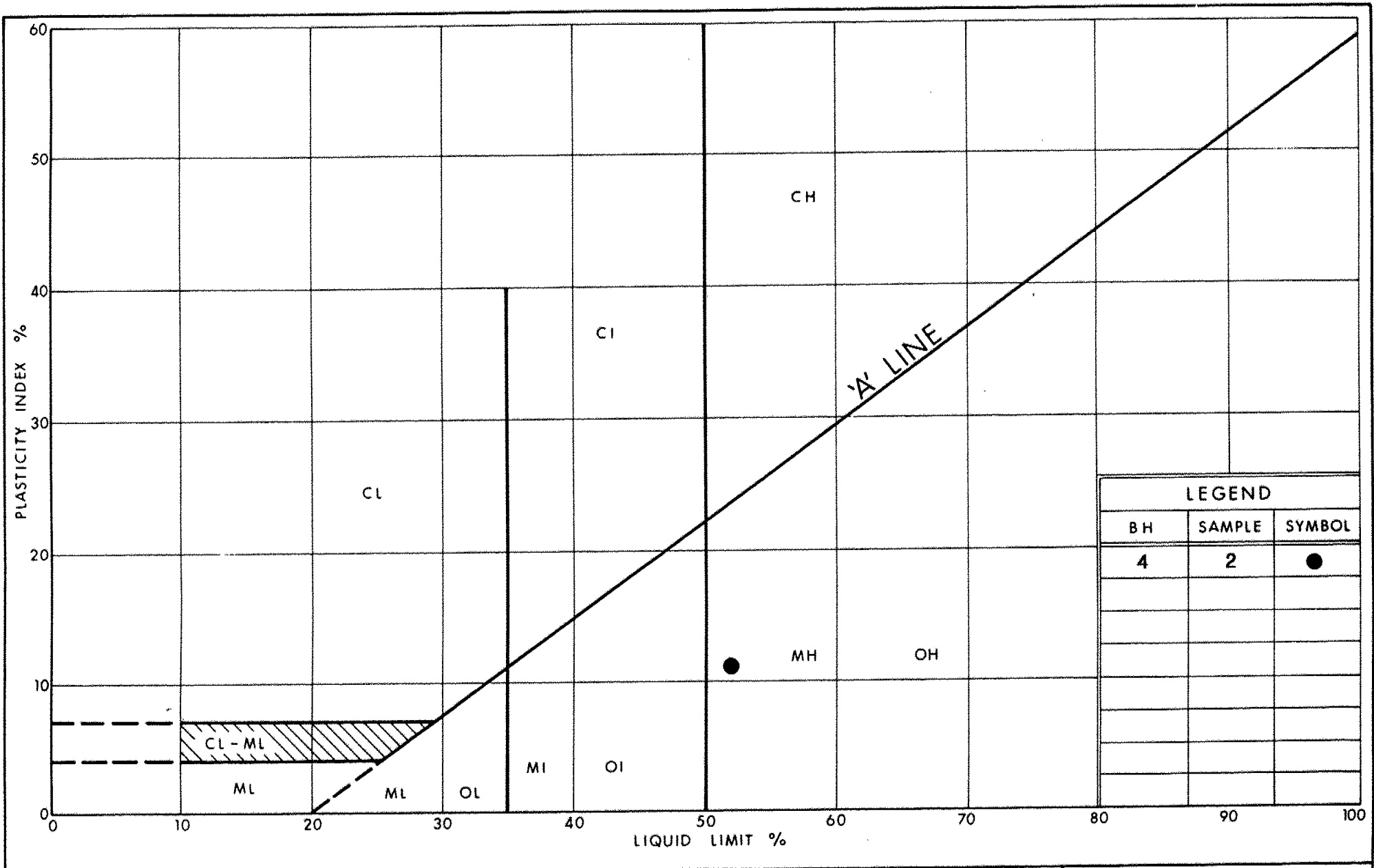


Ministry of
Transportation

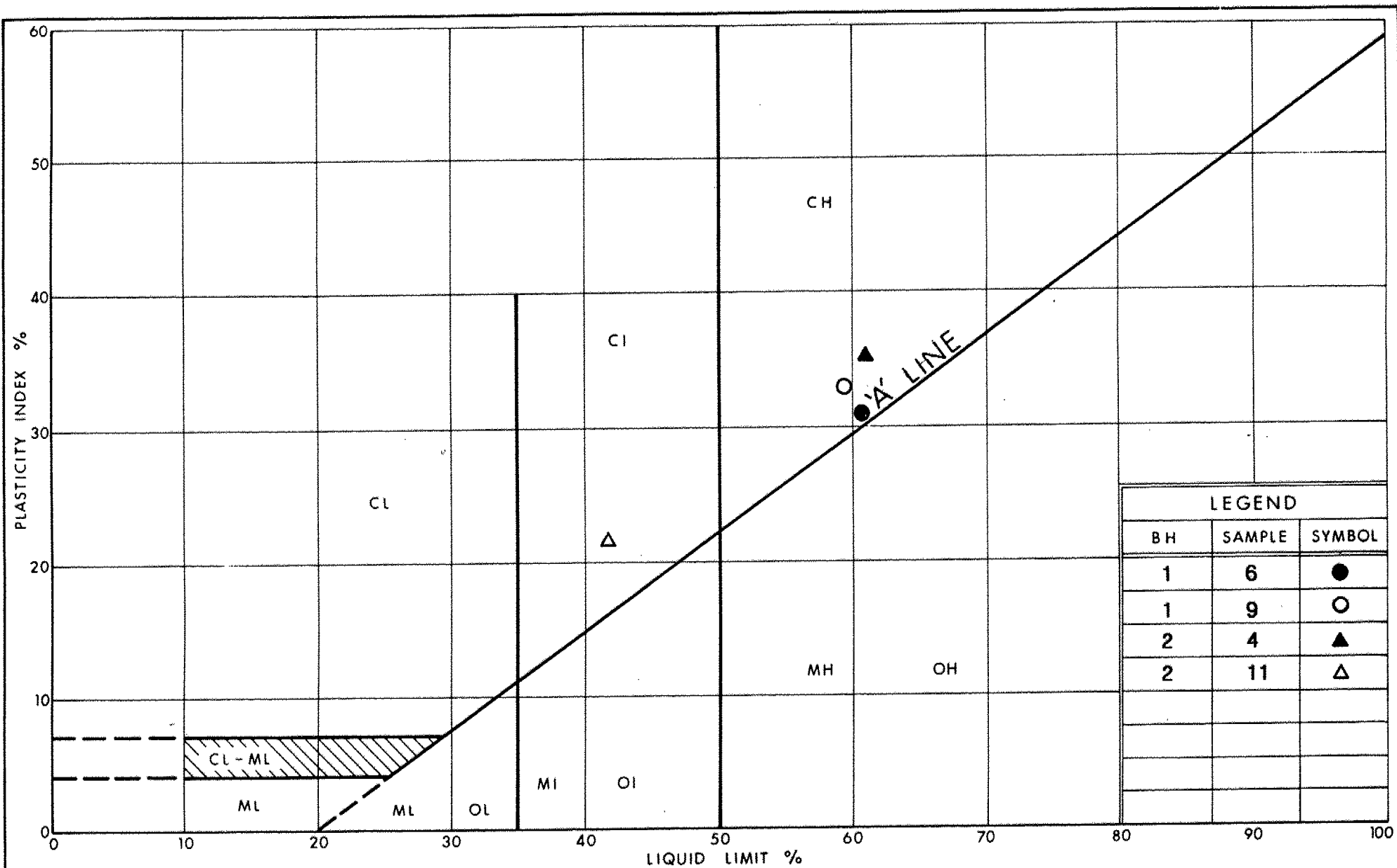
GRAIN SIZE DISTRIBUTION CLAYEY SILT (FILL)

FIG No 1

W P 439-97-00



LEGEND		
BH	SAMPLE	SYMBOL
4	2	●



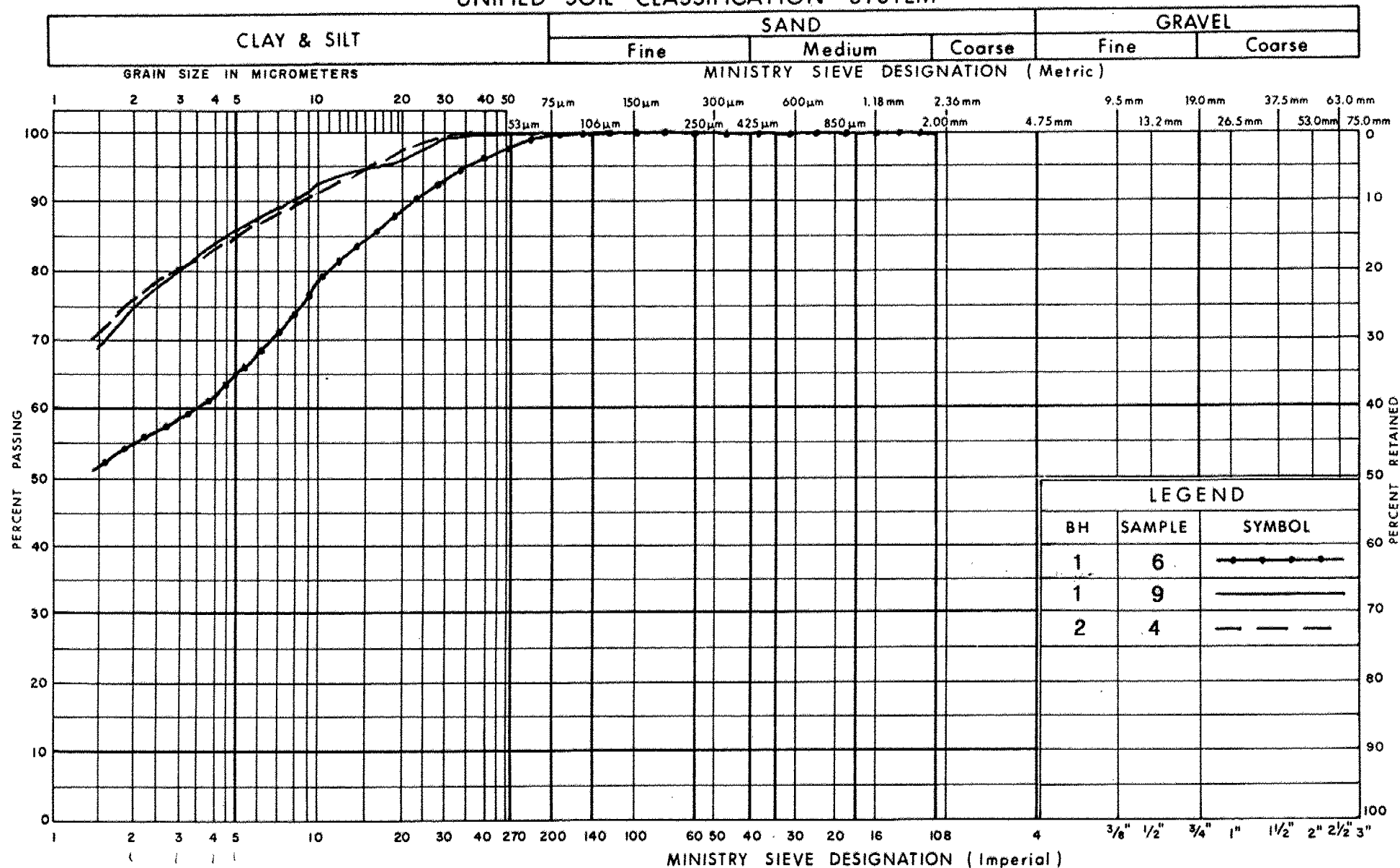
Ministry of
Transportation
Ontario

PLASTICITY CHART CLAY to SILTY CLAY

FIG No 3

W P 439-97-00

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

CLAY to SILTY CLAY

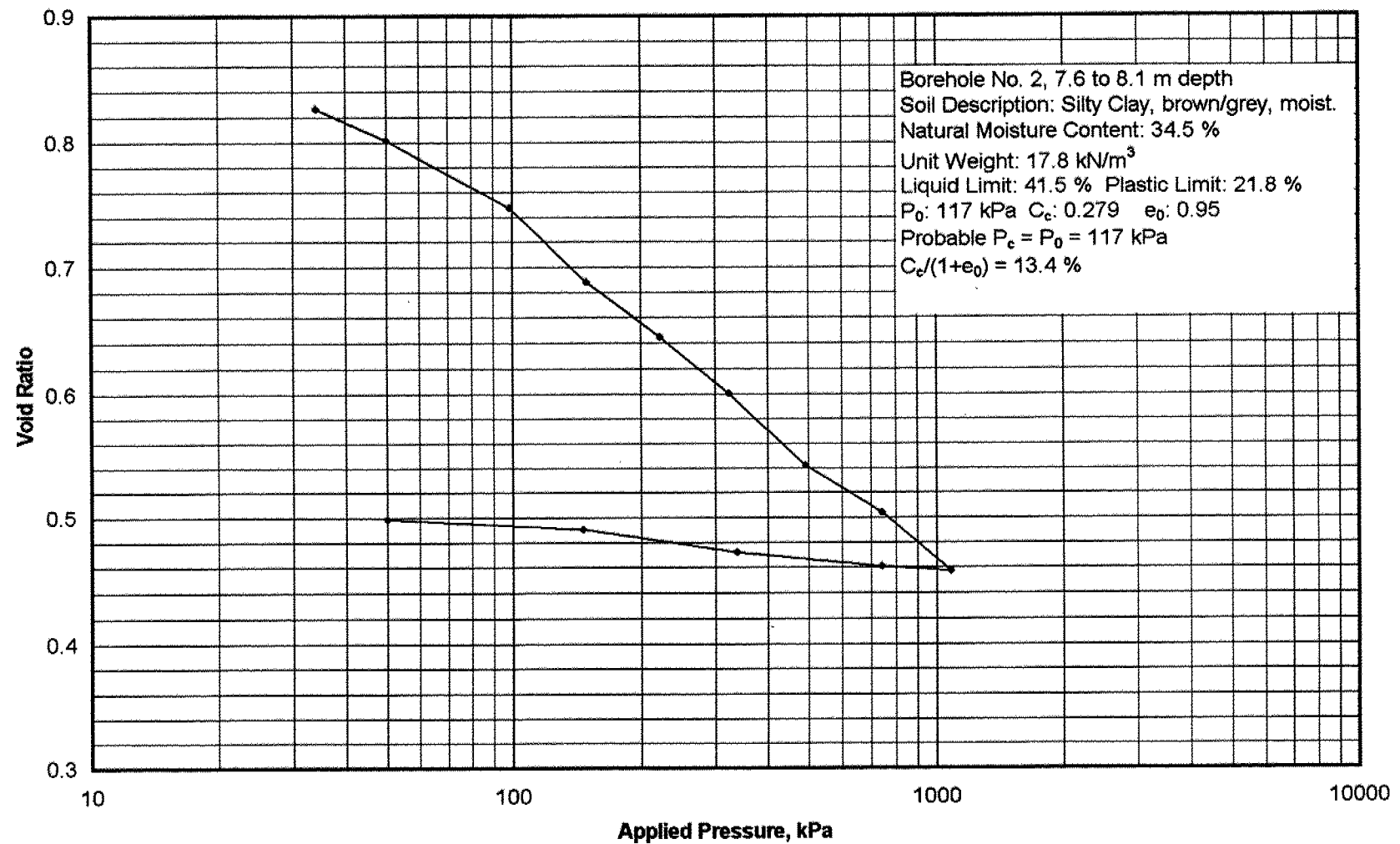


Ministry of
Transportation

FIG No 4

W P 439-97-00

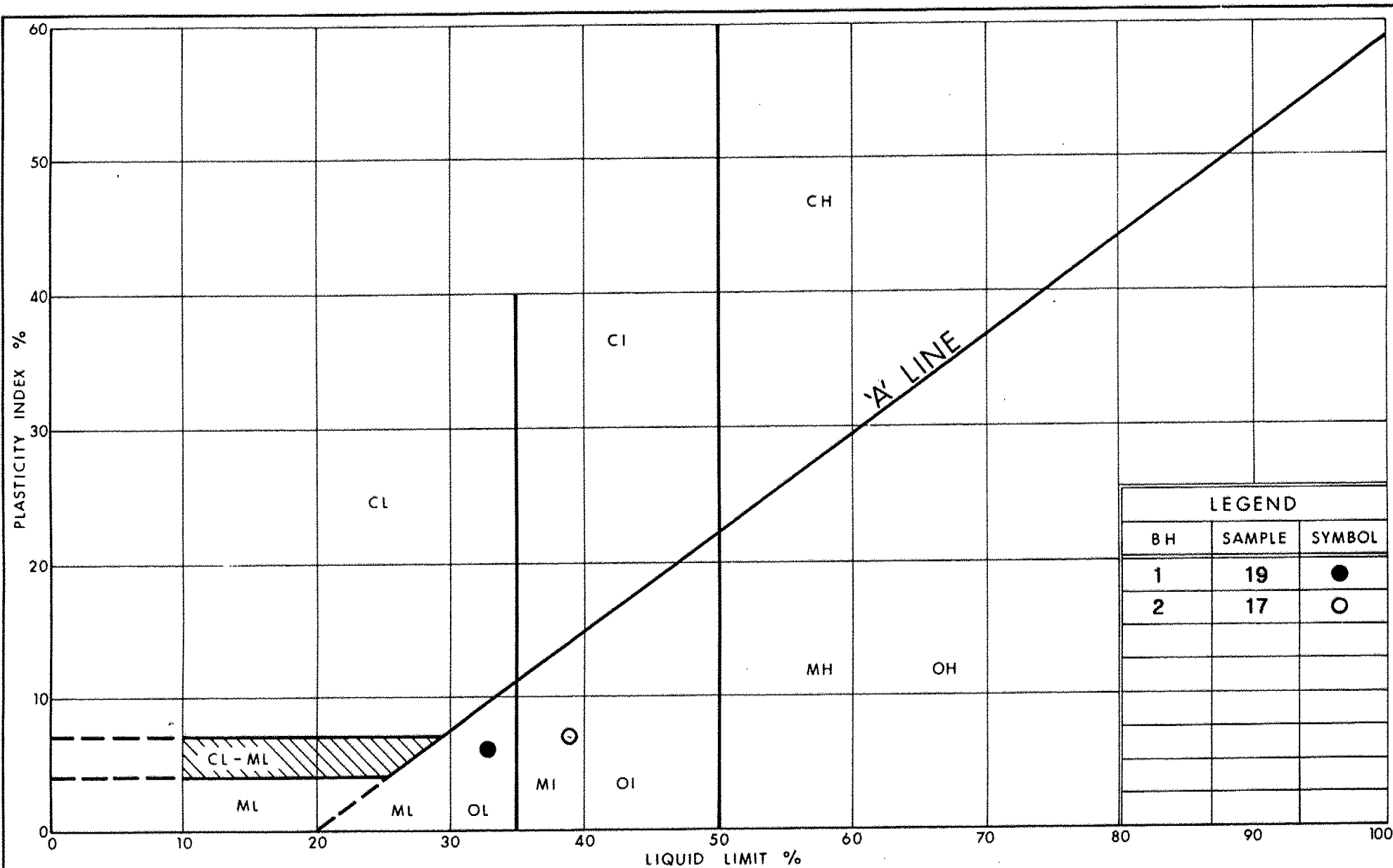
Consolidation Test


 Ministry of
Transportation

Ontario

FIG No 5

W P 439-97-00



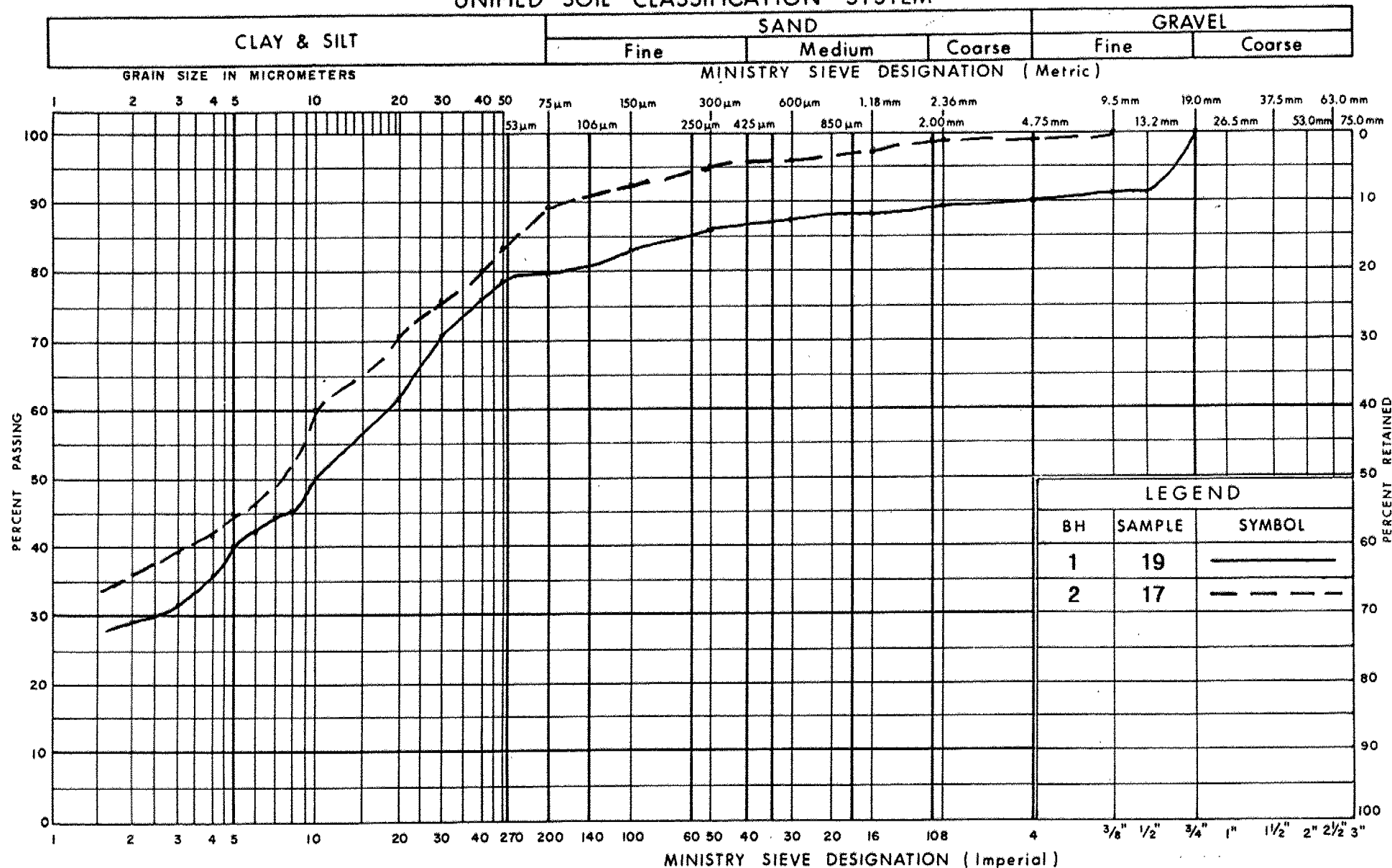
Ministry of
Transportation

PLASTICITY CHART
CLAYEY SILT
TRACE GRAVEL, TRACE SAND

FIG No 6

W P 439-97-00

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
CLAYEY SILT
TRACE GRAVEL, TRACE SAND

FIG No 7

W P. 439-97-00

DESIGN

5. DISCUSSIONS AND DESIGN RECOMMENDATIONS

5.1 Foundations

It is proposed to replace the existing bridge structure by a new structure. The proposed bridge will be a single span structure with a span of about 11.5 m. The existing rigid bridge structure with cross-ties at foundation level will be demolished, except for the half width of the existing footings. A review of the schematic drawing, submitted to our office, indicates that the existing footings are founded at about elevation 171.2 m, about 4.1 m below grade. The foundations for the new structure will be outside the existing foundations. The existing vertical grade will be more or less equal to the existing grade. The surface of the deck will be at about elevation 175.3 m.

The soil conditions indicate the presence of a desiccated crust, below the fill, in the upper 4.0 to 5.0 m depths. At the abutment locations, below elevation 170.0 m, the deposit becomes soft to stiff with shear strength varying from 17 to 55 kPa. The findings also indicate that the soil, below the crust, at north abutment (borehole 2) is significantly weaker than the soil at south abutment (borehole 1).

The proposed structure can be supported on deep foundations, such as steel pipe piles or by cellular abutments supported on floating foundations, i.e. the bearing pressure at S.L.S. marginally exceeds the weight of the soil removed.

Steel Pipe Piles

Foundation support for the structure can utilize driven steel pipe piles. The steel pipe piles will be friction type piles and as such should be terminated at the depth indicated. The pile capacities are as follows:

TABLE 1
PILE CAPACITIES

Pile Type	Embedded Length (m)	Capacity at S.L.S. (KN)	Factored Capacity at U.L.S. (KN)
Steel Pipe Pile 300 mm diameter by 6.5 mm W.T.	25.0	190	190
Steel Pipe Pile 300 mm diameter by 6.5 mm W.T.	31.0	200	250

The settlement of the pile group is estimated to be in the order of 25 mm.

The center to center of piles at the underside of the pile cap shall not be smaller than the least of either $2.5 b + 0.02D'$ or $5 b$ (where b is the diameter of the pile and D' is the embedment depth of deep foundation below the underside of the pile cap).

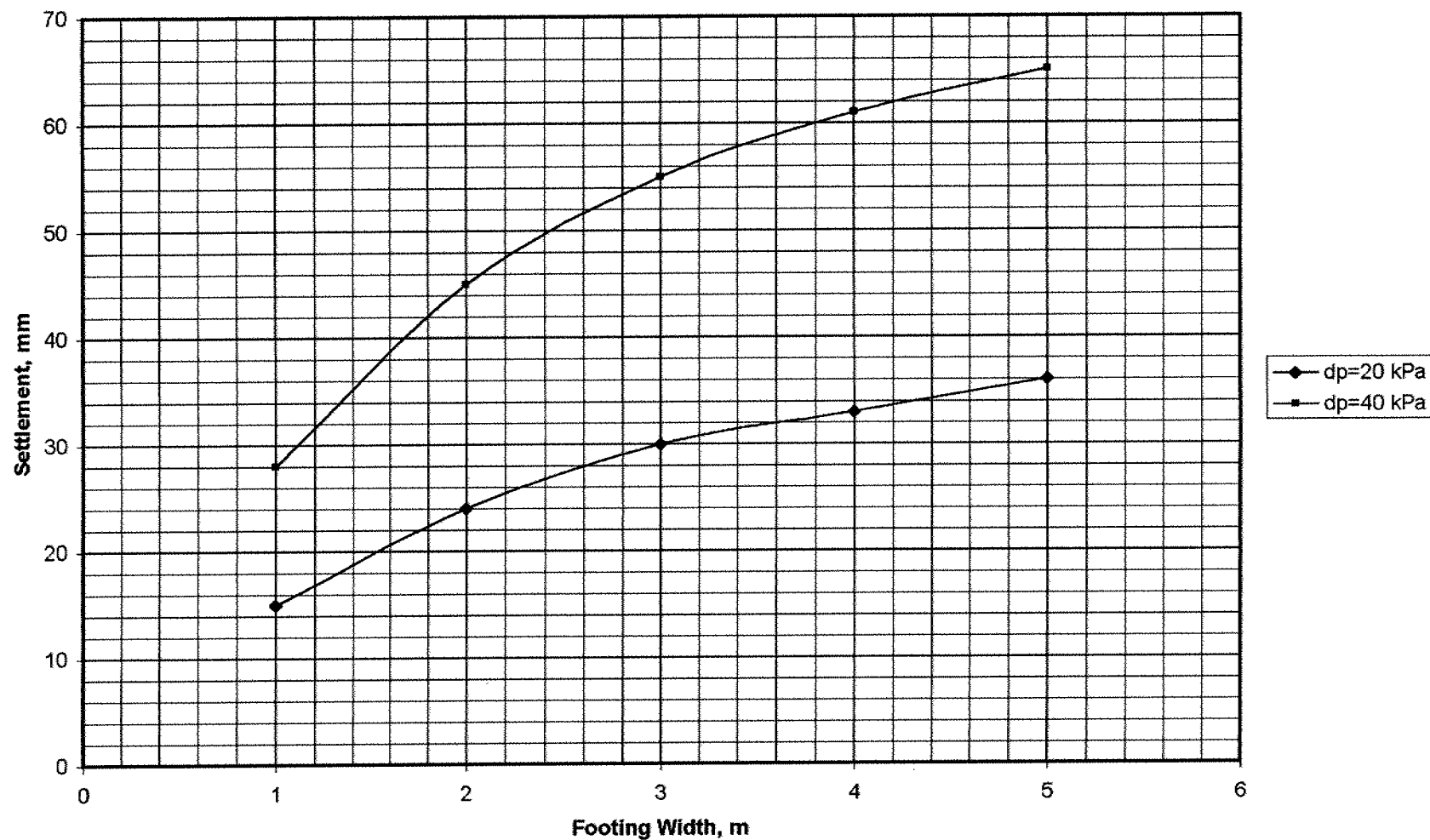
Cellular abutments

It is proposed to found the cellular abutments at the existing footing elevation of 171.2 m, about 4.1 m below existing grade.

The existing soils are normally consolidated at or near the founding level and therefore the stress on the founding soils must not exceed significantly above the existing overburden pressure at the founding level. The settlement of the soil is a function of the applied stress and the width of the proposed footing. A graph of settlement versus the footing width for geotechnical resistance values of 80 and 100 kPa is given in Drawing A. For a 5.0 m wide footing, the geotechnical resistance of 80 kPa at SLS will result in a 36 mm total settlement and about 25 mm differential settlement. Therefore, the geotechnical resistance at SLS of 70 kPa can be used for design of 5.0 m wide footing, as it will result in total settlement of about 25 mm. The resistance value includes the overburden pressure of 60 kPa. The geotechnical resistance at ULS is 95 kPa.

Drawing No. A

Settlements for Various Footing Widths and Pressures Greater than Overburden Value
Project: J461/SP2531 Project Name: Mill Race Bridge Replacement



Note: The overburden pressure is 60kPa for a footing founded at 4.1m below existing grade.

Precast Concrete Culverts

Alternatively, precast concrete culverts may be considered to span the crossing. This will be very economical but it is our understanding that it is not acceptable by the Ministry of the Environment. If this option is pursued, the precast concrete culverts can be placed on a layer of OPSS Granular A to ensure that the load is distributed evenly to the founding soil. The Granular A layer should be 300 mm thick and it should be compacted to at least 100 percent of the Standard Proctor Maximum Dry Density. The water in the creek must be diverted, and all loose and disturbed materials must be sub-excavated prior to placing Granular A material. The thickness of the fill above the culvert must be limited to the thickness of the pavement structure. Adequate erosion protection must be provided.

5.2 Frost Protection

The pile cap footing must have at least 1.2 metres of soil cover for frost protection. The cellular abutment must also be protected from frost by equivalent insulation equal to 1.2 m of soil cover (about 25 mm thick insulation is equivalent to 0.3 m of soil cover).

5.3 Lateral Earth Pressures

Free draining granular materials such as Granular A or Granular B is recommended as appropriate backfill to abutment and wing walls. To prevent hydrostatic pressure build-up, perimeter drainage or weep holes will be required to drain the accumulation of the water in the backfill. The lateral earth pressures should be calculated in accordance with the Ontario Highway Bridge Design Code.

The design parameters are as follows:

	Granular A	Granular B
Angle of Internal Friction (degrees)	35	30
Unit Weight (KN/cu.m.)	21.8	21.0
Active earth pressure co-efficient (Ka)	0.27	0.33
Active earth pressure including backfill pressure (Kb)	0.35	0.41
At-rest earth pressure co-efficient (Ko)	0.43	0.50
At-rest earth pressure including compaction effects (K*)	0.45	0.57

An active condition (Ka) may be assumed to apply for an unrestrained wall and the structural engineer needs to consider the compaction pressure to design the wall. For restrained walls, backfill and at-rest pressure must be considered for both geotechnical and structural design.

5.4 Lateral Resistance

The lateral resistance will be provided by the horizontal load component of the battered piles. The piles must be battered to 4 vertical to 1 horizontal. The passive resistance of the piles can be calculated using unfactored shear strength value of 60 kPa from elevation 171.2 to 170.2 m, and reduce to 20 kPa below elevation 170.2 m.

For cellular abutments founded on large spread footings, the resistance to sliding can be calculated assuming an equivalent unfactored friction angle of 20 degrees to apply between the underside of the footings and the founding soils.

5.5 Approach Fills

Present planning indicates that the vertical alignment will be close to the existing grade and the width of the proposed bridge will be wider than the existing bridge by about 2.5 m. Based on the soil conditions, the vertical alignment should not be raised as the clay materials encountered are interpreted to be

normally consolidated, and any increase in load due to the addition of the load on the approaches, will result in significant settlement of the fill.

Increasing the width of the bridge may result in placing greater amount of fill, above the virgin grade, outside the limits of the existing embankments. This will result in some settlement of the approaches, outside the limits of the existing embankments, depending on the geometry and depth of fill placed. It is probable that the approach slab will span most of the deep fill. About $\frac{3}{4}$ of the total settlement will occur within one year and therefore considerations should be made to delay the placement of wearing course.

It is also recommended that the widening of the bridge must be spread at to both sides of the existing pavement rather than one side, as it will result in less differential settlement.

All topsoil and buried peat and organic material should be removed within the plan limits of the approach embankments. No stability problems are anticipated for the approaches. The existing and new slopes must be dressed to 2 horizontal to 1 vertical.

5.6 Excavations and Dewatering

All temporary excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the fill is classified as Type 3 and very stiff clay to silty clay as Type 2. Below elevation 170.5 m, the soft to firm clay is classified as Type 4 to 3.

No major excavation problems are anticipated for excavations due to the relatively low permeability of the cohesive foundation soils. The footings of the existing bridge will act as a coffer dam, which will help to carry out the excavations in relatively dry conditions. However, the localized seepage and surface run-off in the excavations can be controlled by perimeter ditches and pumping from sumps.

It is understood that half the width of the existing footing will be removed. In order to minimize any disturbance to the founding soil, we recommend that the base must be hand cleaned of loose soils and covered with a 75 mm thick mud slab.

5.7 Construction Staging

It is understood that the construction will be carried out in three stages to maintain the flow of the traffic. This may result in additional loads on the existing footings due to the additional traffic in restricted lanes. If this occurs there will be some additional settlement of the existing footing. Since the structure will be demolished, this is not a concern.


Considerations can be given to piling in two stages rather than three stages for continuity and economical reasons. This may require piling half the abutments, backfill the half side to grade and then move to the other half. Construction of the pile caps, abutment walls and slab can proceed later in three stages.

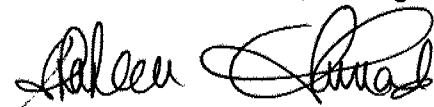
5.8 Miscellaneous

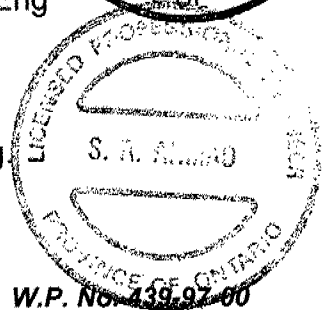
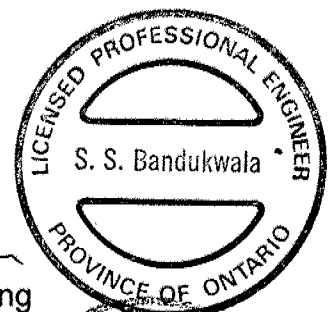
The fieldwork for this investigation was carried out under the supervision of Mr. G. Itchenko. The drilling equipment was owned and operated by Groundwork Drilling Inc. The project was carried out under the supervision of S. Bandukwala, Principal Project Manager. The report was written by S. Bandukwala and reviewed by S. Ahmad, Principal Engineer. We trust that the information contained in this report is satisfactory.

Should you have any questions, please do not hesitate to contact this office.

Yours very truly
SHAHEEN & PEAKER LIMITED


Shabbir Bandukwala, M.Eng., P.Eng


Shaheen Ahmad, M.A.Sc., P.Eng



FAX MESSAGE

Date: 1999-03-23
To: Mr. M. Kobiela, P. Eng.
Paul Theil Associates
Fax: 905-792-8110
Re: **Hwy 3/Millrace Bridge Replacement**
GWP 439-97-00
Foundation Bearing Pressure & Settlement
From: Dennis Wong, MTO
Ph # 235-5512 Fax #: 416-235-4008
E-mail: WongD@MTO.GOV.ON.CA
cc: G. Dales, Hwy Engineering (memo only)
B. Bennet, FDS
R. Yu, Structural Section
pages: 7



Mariusz:

I still want to pursue the concept of Cellular Abutment in relation to the geotechnical resistance and settlement etc. I like to draw your attention to the following points:

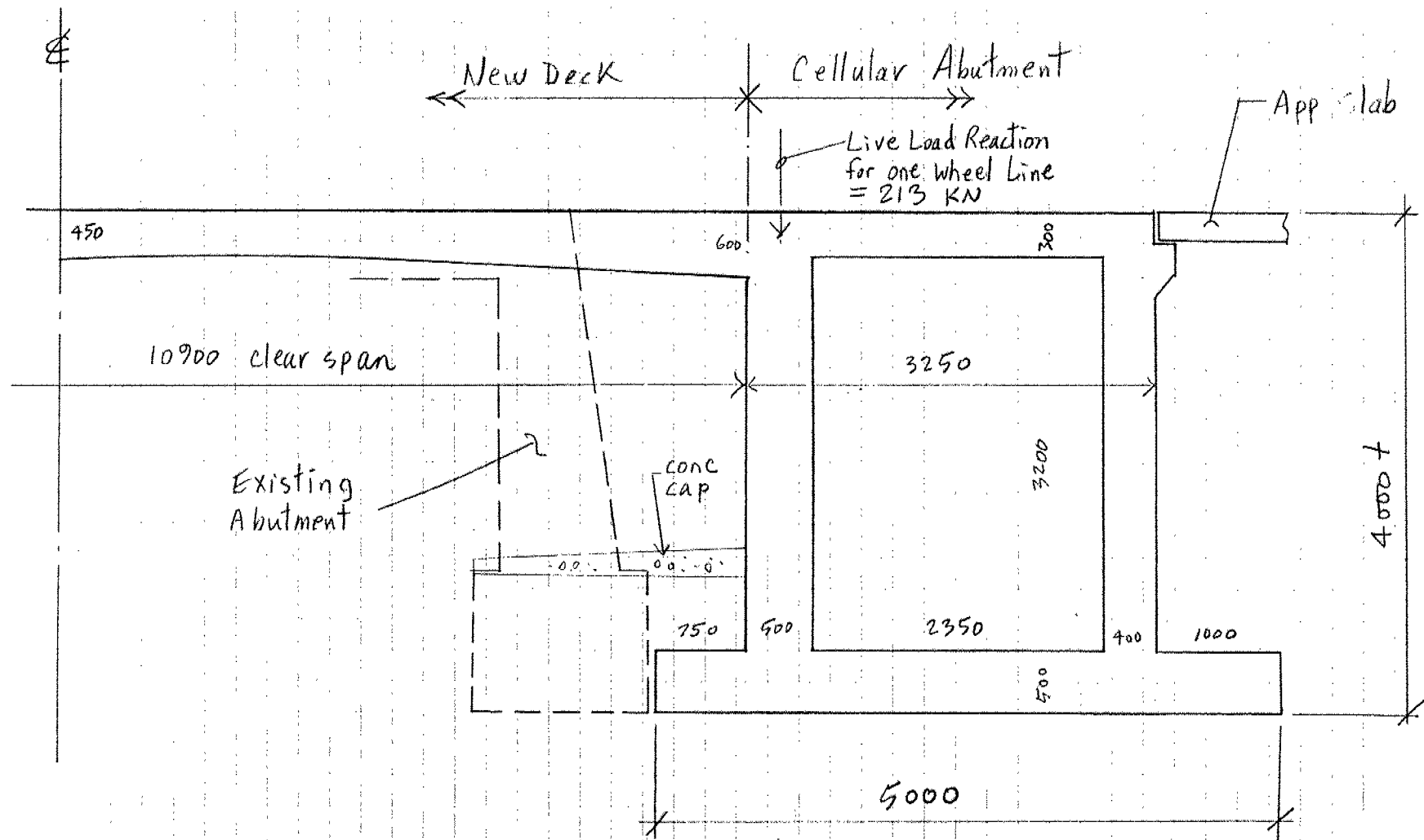
- I had a close look at the existing structure and worked out the soil pressures, which are 190 KPa at SLS and 295 KPa at ULS. Recognising the weak soil at this location, there is no sign of any settlement or problems associated with foundation failure of the existing structure for the last 60 years. We would have to justify why the soil capacity for the new structure constructed immediately behind the existing structure is limited to 70 KPa at SLS?
- In my proposal of a Cellular Abutment, the permanent load at SLS of 74 KPa is lower than the existing 4 m of soil overburden of 80 KPa. Since there is no increase of soil pressure, there should be no settlement as a result of the construction of this bridge. I have made up a hypothetical case of a concrete box being constructed in a location susceptible to settlement very similar to the construction of the proposed Cellular Abutment and demonstrated that there is no increase of effective pressure whether the box is COLOSED or DRAINED. If there is no net increase of effective pressure, there will be no agent to cause the settlement.
- If there is a general lowering of water table in this area in the future after the construction of this bridge, due to the increase of effective pressure, there will be some settlement. But this settlement will happen in the whole region and not limited to the area of newly constructed bridge and surely is not caused by the construction of this bridge. On the other hand, if for some reason, the water table rises, the pore water pressure will help to support the imposed load due

- to overburden and there will be no settlement. To my mind, regardless of the situation, the construction of this bridge with the Cellular Abutment will not cause settlement at this location.
- If settlement does not occur as a result of the construction of the Cellular Abutment, the determination of the geotechnical resistance (soil bearing pressure) should be based on factors other than on 25 mm settlement criterion.

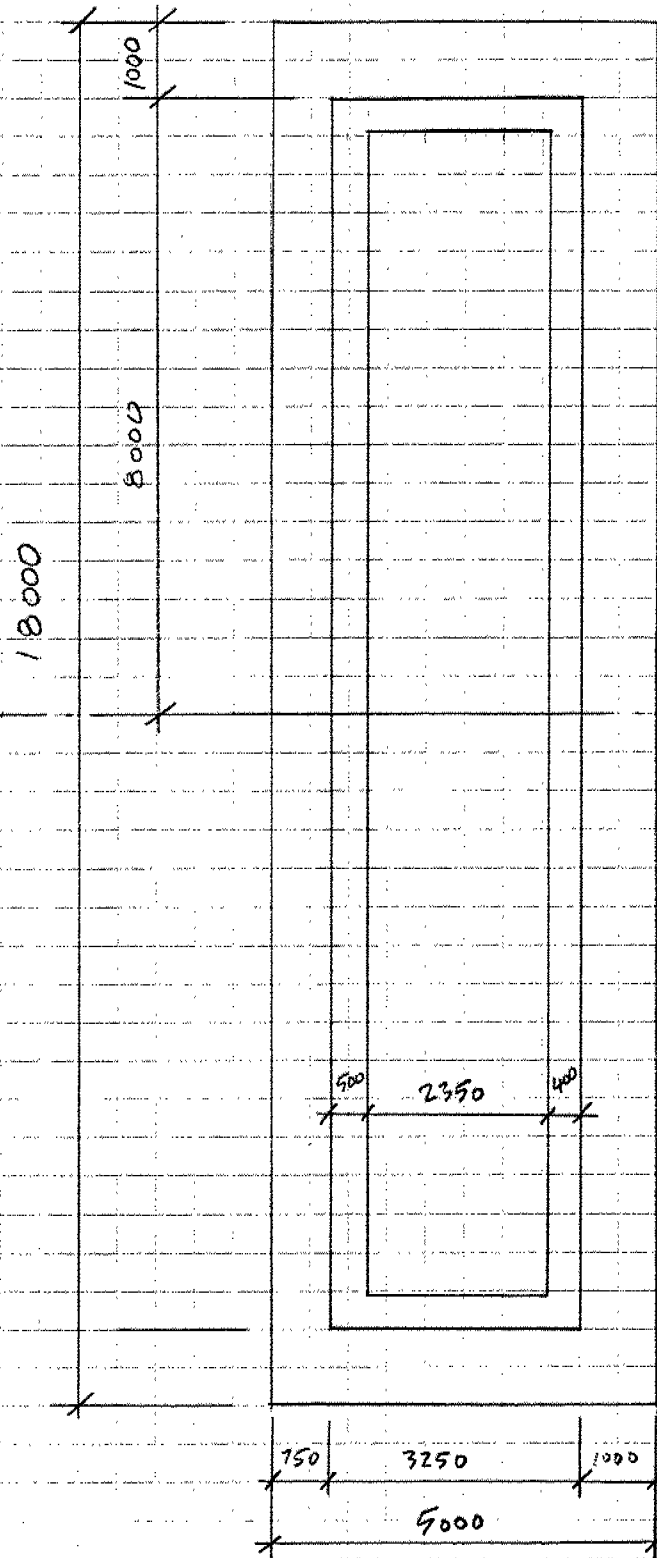
I like to see this issue resolved so that we can get on with the detail design.

I attach all the relevant sketches for your information and consideration. Please discuss.

Thanks.....Dennis



Abutment Cross-section



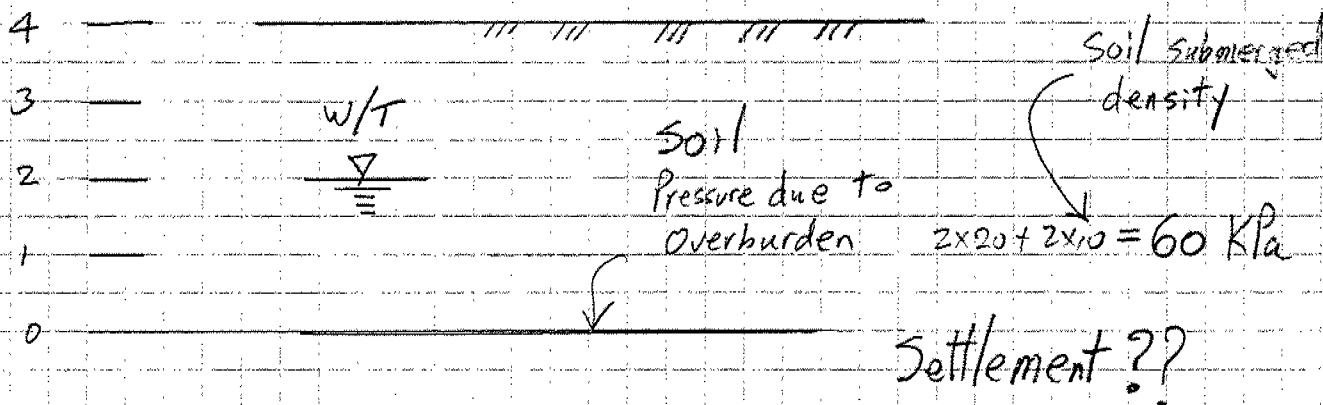
HW/3

Footing Dimensions

CASE I Ground with no structure

4 m below grade ; Water table at 2 m

Soil density 20 kN/m^3 , Water 10 kN/m^3

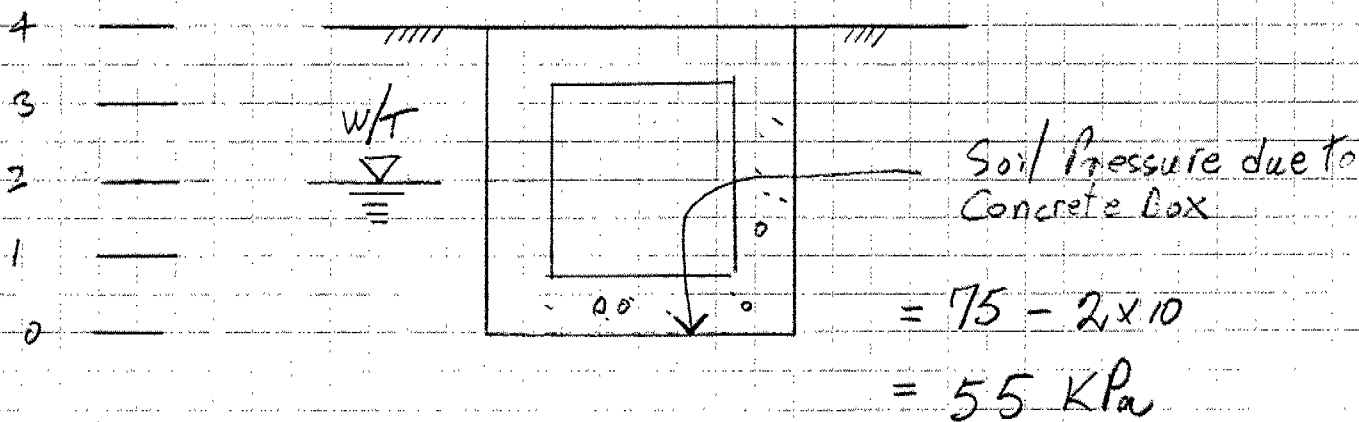


CASE II A CLOSED Concrete Box constructed at this location

Size $4\text{m} \times 4\text{m}$, $WT = 1200 \text{ kN}$ (78% solid)

i.e. Contact Pressure = 75 kPa

w/t @ 2 m as Case I



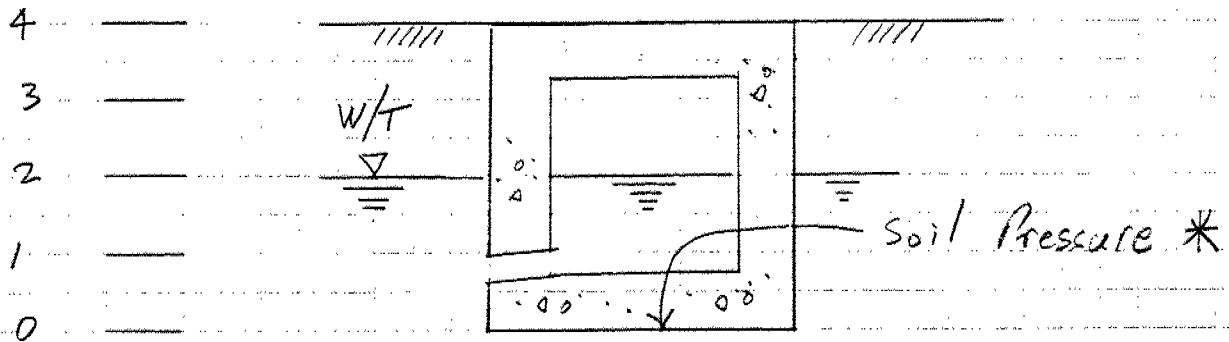
Settlement??

CASE III

A DRAINED Concrete Box Constructed at this Location

Size $4m \times 4m$, $Wt = 1200 \text{ kN}$

Contact Pressure = 75 kPa



$$\begin{aligned}
 * \quad Wt \text{ of Conc. Box} &= 1200 \text{ kN} \\
 \text{Buoyance } \frac{1}{2}(4 \times 4 \times 4 \times 10) \times 78\% &= - 250 \\
 &= 950 \text{ kN}
 \end{aligned}$$

$$\text{Soil Pressure} = \frac{950}{4 \times 4} = 59 \text{ kPa}$$

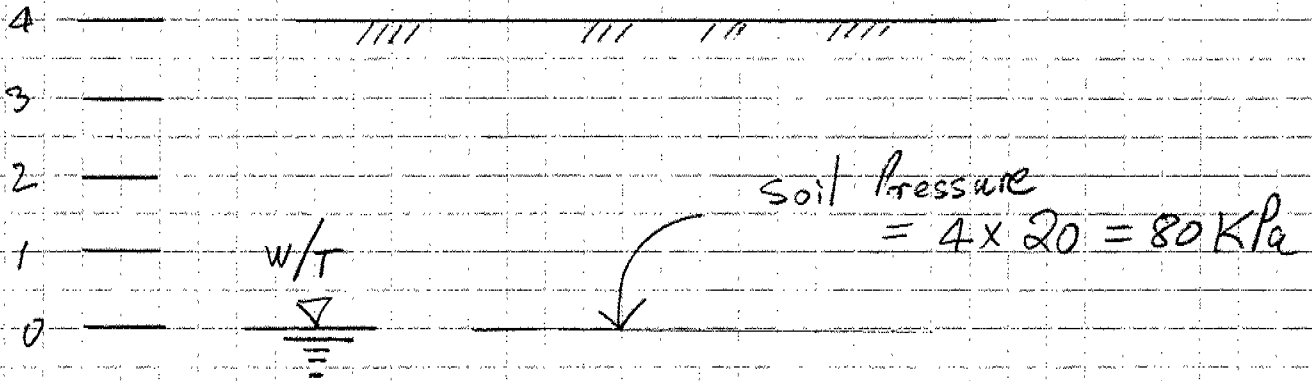
Increase of Pore Water Pressure

$$\begin{aligned}
 \text{i.e. decrease of effective Pressure} &= 2 \times 10 \\
 &= 20 \text{ kPa}
 \end{aligned}$$

Settlement ??

CASE IV

General Lowering of Water Table
by 2 m



If the general water table is lowered by 2 m, there will be an increase of effective soil pressure by 20 KPa. Under this scenario, there will be a settlement in the whole region whether the concrete box is constructed there or not.

paul theil associates limited
consulting engineers131 DELTA PARK BOULEVARD, BRAMPTON, ONT. L6T 5M8
TEL: (905) 792-2215 FAX: (905) 792-8110

facsimile transmittal

Date: March 17/99 Project No.: 3843Person: Mr. DENNIS WONG, P. Eng.Company: ITIO Central Region, Struct. Engin. Fax. No.: 416-235-4008 Sent: ☐Also CC: _____ Fax. No.: _____ Sent: ☐Also CC: _____ Fax. No.: _____ Sent: ☐Also CC: _____ Fax. No.: _____ Sent: ☐Also CC: _____ Fax. No.: _____ Sent: ☐From: MARIUSZ KOBIELANo. of Pages: 4 (Including Cover Sheet)Original will be forwarded: ☐

Message: Enclosed please find an updated letter from
our Foundation Sub-consultant, for your information.

H.K.

SHAHEEN & PEAKER LIMITED**CONSULTING GEO-ENVIRONMENTAL AND CONSTRUCTION MATERIALS ENGINEERS**250 Galaxy Boulevard
Etobicoke, Ontario, M9W 5R8Tel. (416) 213-1255
Fax. (416) 213-1260

Project No.: SP 2531**March 15, 1999****Paul Theil Associates Limited
131 Delta Park Boulevard
Brampton, Ontario
L6T 5M8****Attention: Mr. M. Kobiela, P.Eng.**

Dear Sirs:

**Re: Geotechnical Comments
Highway 3, Wainfleet**

Further to our draft report, the additional geotechnical comments are as follows:

The options for conventional foundations were ruled out in the draft report, as it will result in large unacceptable settlements from the compression of the underlying weak soils. However, you have indicated that you need the geotechnical resistance values for the conventional foundations at the existing footing elevation of 171.2 m, about 4.1 m below existing grade.

The geotechnical resistance at SLS (25 mm settlement value) is 80 kPa for a 2.2 m wide footing. The resistance value includes the overburden pressure of 60 kPa. A graph of settlement versus the footing width for geotechnical resistance values of 80 and 100 kPa is given in Drawing 1. The geotechnical resistance at ULS is 95 kPa. Based on these values, it is unlikely that the bridge can be supported on conventional foundations at the existing footing elevation of 171.2 m.

You have indicated cellular abutment with 5.0 m wide footing as an alternative. The geotechnical resistance of 80 kPa at SLS will result in a 36 mm total settlement and about 25 mm differential settlement. If the total settlement acceptable is 25 mm, then the geotechnical resistance can be reduced to 70 kPa at SLS. The geotechnical resistance at ULS is 95 kPa.

Project: SP2531

2

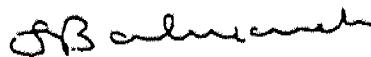
As mentioned in our earlier correspondence, the bridge could be supported on friction piles. You have indicated that the Ministry's concerns are vibration and noise. Based on our experience of driving piles in the last 35 years, we do not consider vibration as a concern for structural damage to the house located 25 m away from the closest pile in cohesive soils. It is imperative a pre-construction survey of the houses are carried out prior to pile driving and vibration monitoring during pile driving in order to avoid any frivolous claims. In the unlikely event, if vibration does becomes a concern, then the piles can be auger drilled of a smaller diameter (200 mm diameter) in the upper 6.0 m, prior to driving 300 mm diameter pipe piles

Noise is always a problem at any construction site. It is normally dealt with restricted working hours.

Should you have any questions, please do not hesitate to contact this office.

Yours very truly,

SHAHEEN & PEAKER LIMITED



Shabbir Bandukwala, M.Eng., P.Eng.



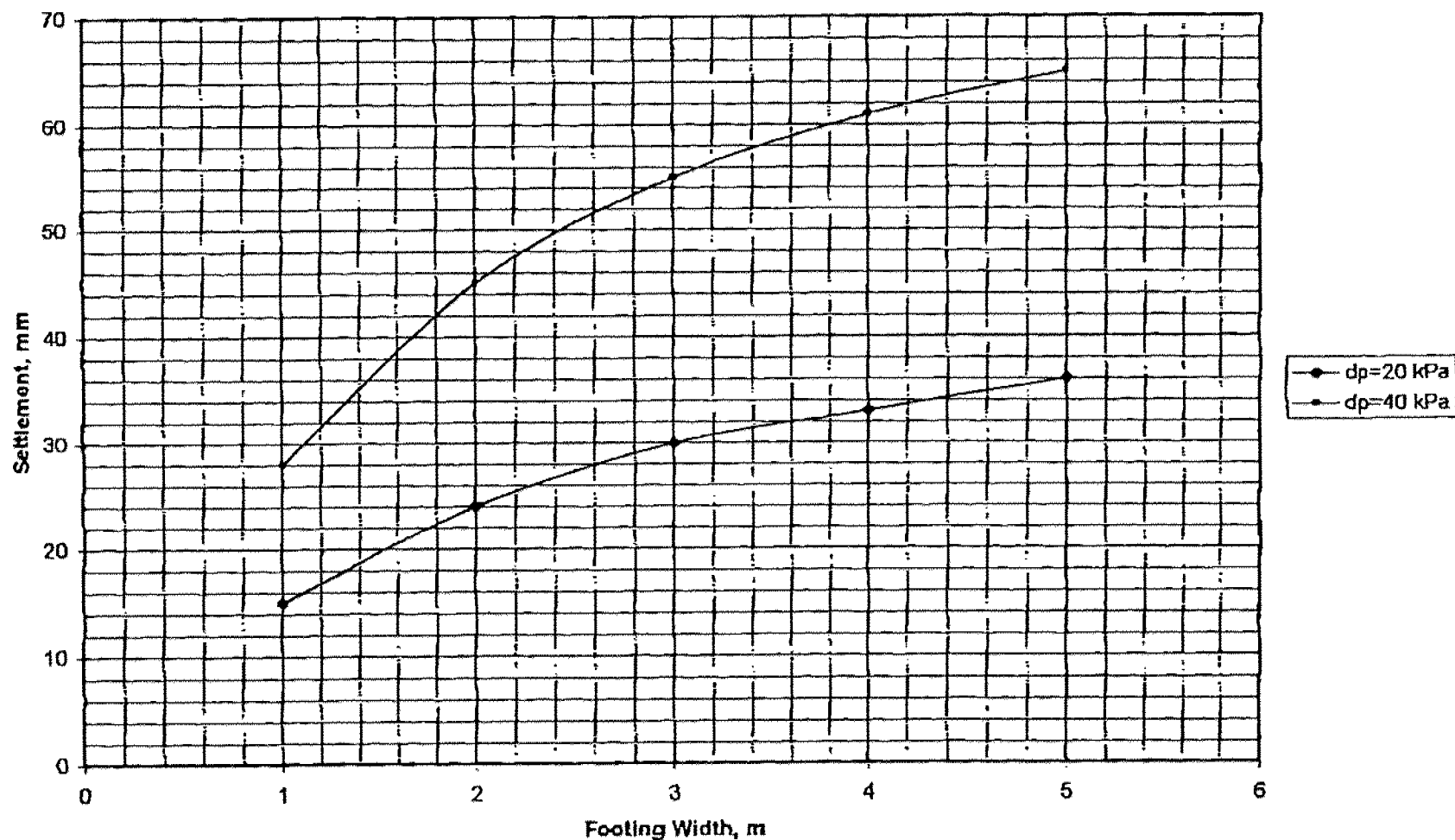
Shaheen Ahmad, M.A.Sc., P.Eng.

P.004
11/04/04

TEL: 7928110

MAR. -17' 99 (WED) 09:45 PAUL THEIL ASSOC. INC.

Settlements for Various Footing Widths and Pressures Greater than Overburden Value
Project: J461/SP2531 Project Name: Mill Race Bridge Replacement



Note: The overburden pressure is 60kPa for a footing founded at 4.1m below existing grade

SHAHEEN & PEAKER LIMITED**CONSULTING GEO-ENVIRONMENTAL AND CONSTRUCTION MATERIALS ENGINEERS**250 Galaxy Boulevard
Etobicoke, Ontario, M9W 5R8Tel. (416) 213-1255
Fax. (416) 213-1260
E-mail: info@shahcenpeaker.ca**Project: SP2531****February 10, 1999****Paul Thiel Associates Limited
131 Delta Park Boulevard
Brampton, Ontario
L6T 5M8****Attention: Mr. M. Kobiela**

Dear Sirs:

**Re: Millrace Bridge Replacement
Wainfleet
Highway 3, Ontario**

Further to our letter dated January 21, 1999 and with reference to the comments from the Pavement and Foundation Section of Ministry of Transportation, the timber piles are omitted due to low capacities and the steel pipe pile capacities are revised as follows:

**Table 1
Pile Capacities**

Pile Type	Embedded Length (m)	Factored Capacity at U.L.S. (kN)	Capacity at S.L.S. (kN)
Steel Pipe Pile 300 mm diameter x 6.5 mm W.T.	25.0	190	190
Steel Pipe Pile 300 mm diameter x 6.5 mm W.T.	31.0	250	200

For the above values, the load test is not required to confirm the capacity.

Project: SP2531

2

Should you have any questions, please do not hesitate to contact this office.

Yours very truly,

SHAHEEN & PEAKER LIMITED



Shabbir Bandukwala, P.Eng.

SB:tr/d#z2

Copy: Ministry of Foundation
Pavement & Foundation, Attention: Ms. Betty Bennett, Fax: 416-235-5240

From: Betty Bennett
To: MTOCR.DOWNSVCR.WongD
Date: 1999/02/03 10:55 am
Subject: Hwy 3 - Mill Race Bridge - Report Review

Dennis

As requested, the draft structural report and the additional foundation recommendations for the Mill Race Creek bridge have been reviewed.

Our only comment regards the pile resistances provided in the letter of foundation recommendations dated January 21, 1999. We are concerned that the resistances for the various pile types are somewhat high, based on pile load tests carried out in similar subsurface conditions in the past.

Generally, for friction piles in stiff cohesive deposits, failure of the pile has occurred prior to the SLS criteria of 25 mm of movement being reached. The factored ULS resistance has governed design.

It is our feeling that the factored ULS resistances provided at this site would govern the design and the magnitude of the resistances should be closer to those provided for SLS.

If there are any questions, please advise.

Betty

Feb. 10/99

Call from Shabbir re: pile capacity

→ he felt that the loads provided in his memo were accurate.

→ discussions with Dave resulted in Shaheen & Peaker's agreement to consider pile lengths of 30m in order to accommodate the regional structural loads.

RB

From: Betty Bennett
To: MTOCR.DOWNSVCR.WongD
Subject: Highway 3 - Mill Race Bridge Foundations

Dennis

I have looked over the draft foundations report together with the correspondence from the structural and foundations consultants.

The foundation recommendations provided in the draft report are acceptable given the findings of the subsurface investigation.

Although the existing structure is on spread footings, the footings span the length of the bridge in a manner similar to a box culvert. The replacement structure will be wider than the existing with a configuration different from existing structure. The values of settlement and bearing resistance provided by the foundations consultant are reasonable for the soil conditions encountered. There are numerous risks associated with the use of shallow foundations at this site as explained by both the consultants.

Had this site been one for which there was no existing structure, the recommendation would have been either to construct a box culvert or to found the bridge on deep foundations.

Betty

SHAHEEN & PEAKER LIMITED

CONSULTING GEO-ENVIRONMENTAL AND CONSTRUCTION MATERIALS ENGINEERS

250 Galaxy Boulevard
Etobicoke, Ontario, M9W 5R8

Tel. (416) 213-1255

Fax. (416) 213-1280

Email: info@shaheenpeaker.ca

Project: SP2531

December 21, 1998

Paul Thiel Associates Ltd.
131 Delta Park Blvd.
Brampton, Ontario
L6T 5M8Attention: Mr. M. Kobiela

Dear Sirs:

Re: Settlement of Footings for Proposed Bridge
Wainfleet
Highway 3, Ontario

Further to our draft report, you have requested settlement values for the proposed structure supported on conventional foundation.

The idealized profile used in settlement analysis is attached. The calculated settlements for the bearing pressures at serviceability state are as follows:

Bearing Pressure at S.L.S. (kPa)	Footing Elevation (m)	Calculated Settlement (mm)	Estimated* Settlement (mm)
120	171.2	112	75
80	171.2	96	64
100	172.5	61	41

*0.67 times the calculated settlement, based on experience and the fact that the field condition cannot be idealized in the laboratory.

The above data indicates that the estimated settlement of the proposed footings at different founding elevation and bearing pressures. If the footings are founded at the same level as those existing, Elevation 171.2 m, and designed for a low bearing pressure of 80 kPa estimated settlement is 64 mm. Since the main contribution to settlement originates in the normally consolidated clay between Elevation 170 m and 167 m, it could be reduced by placing as high as practically possible, thereby reducing the stress at the weaker clay level. For example, if the footings are founded at Elevation 172.5 m and designed for a bearing pressure of 100 kPa the estimated

Project: SP2531

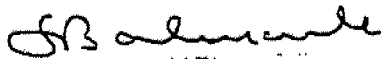
settlement is 41 mm. The footings should then be founded sufficiently back from the existing footings so that using a load spread of 1 horizontal to 2 vertical no load is transferred to the existing fill behind the existing abutments. This effectively increases the span of the bridge.

For most structures, the bearing capacity at Serviceability Limit States, is defined for total settlement of 25 mm. If this definition is applied to this structure then spread footing for the abutment is not a viable option and deep foundation should be used. However, past experience with bridge structures suggests that such structures can accommodate relatively large settlements without impairing their performance. If this is the case, then the footing founded at Elevation 172.5 m would be a viable option.

Should you have any questions, please do not hesitate to contact this office.

Yours very truly

SHAHEEN & PEAKER LIMITED



Shabbir Bandukwala, P.Eng.



Shaheen A. Ahmad, M.A.Sc., P.Eng.

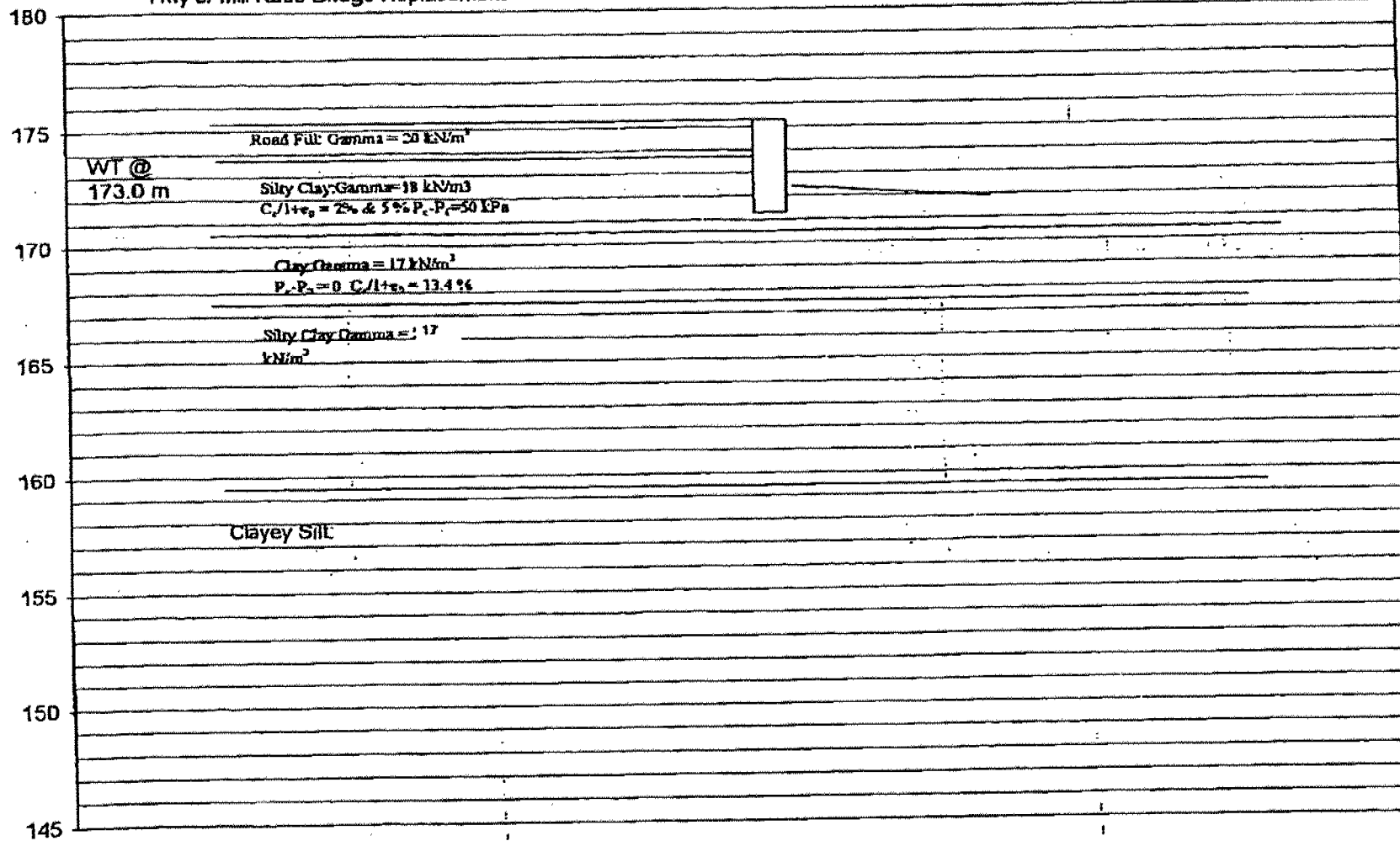
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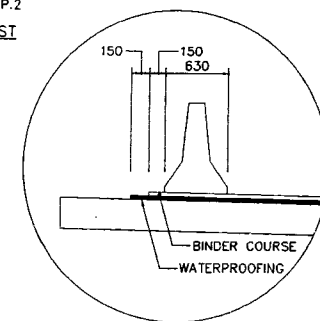
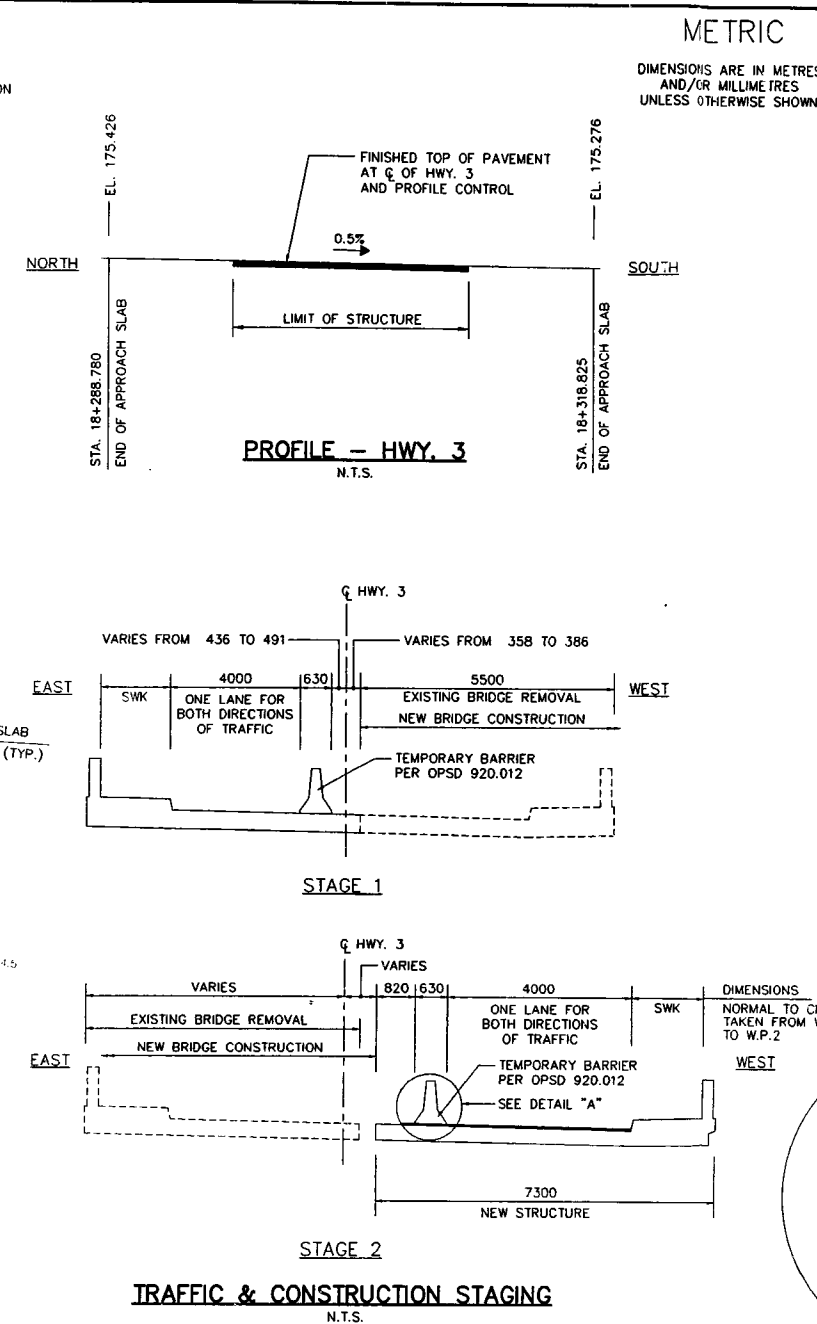
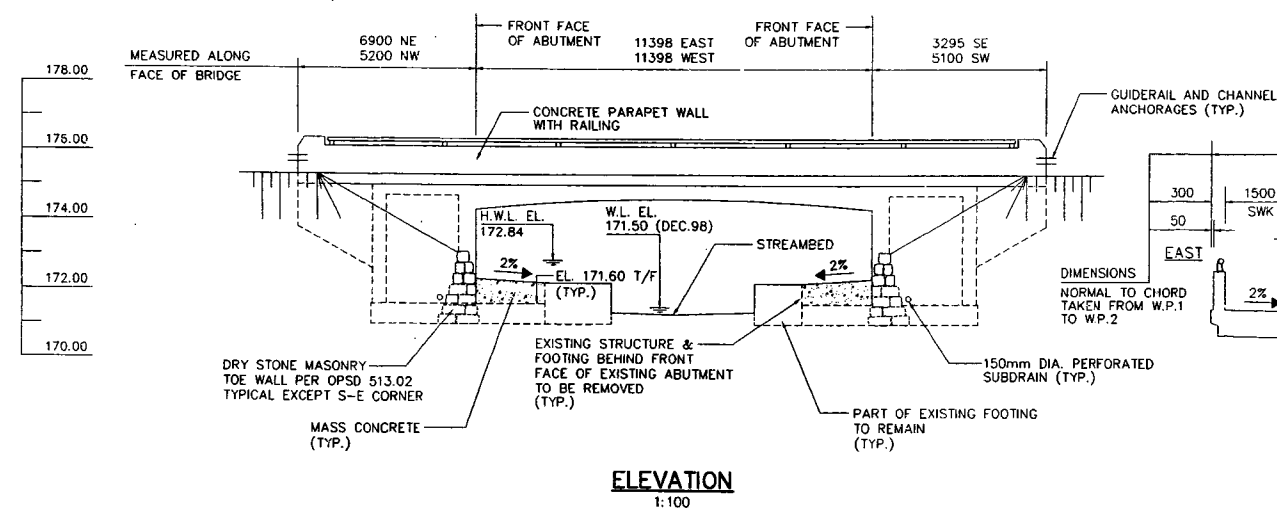
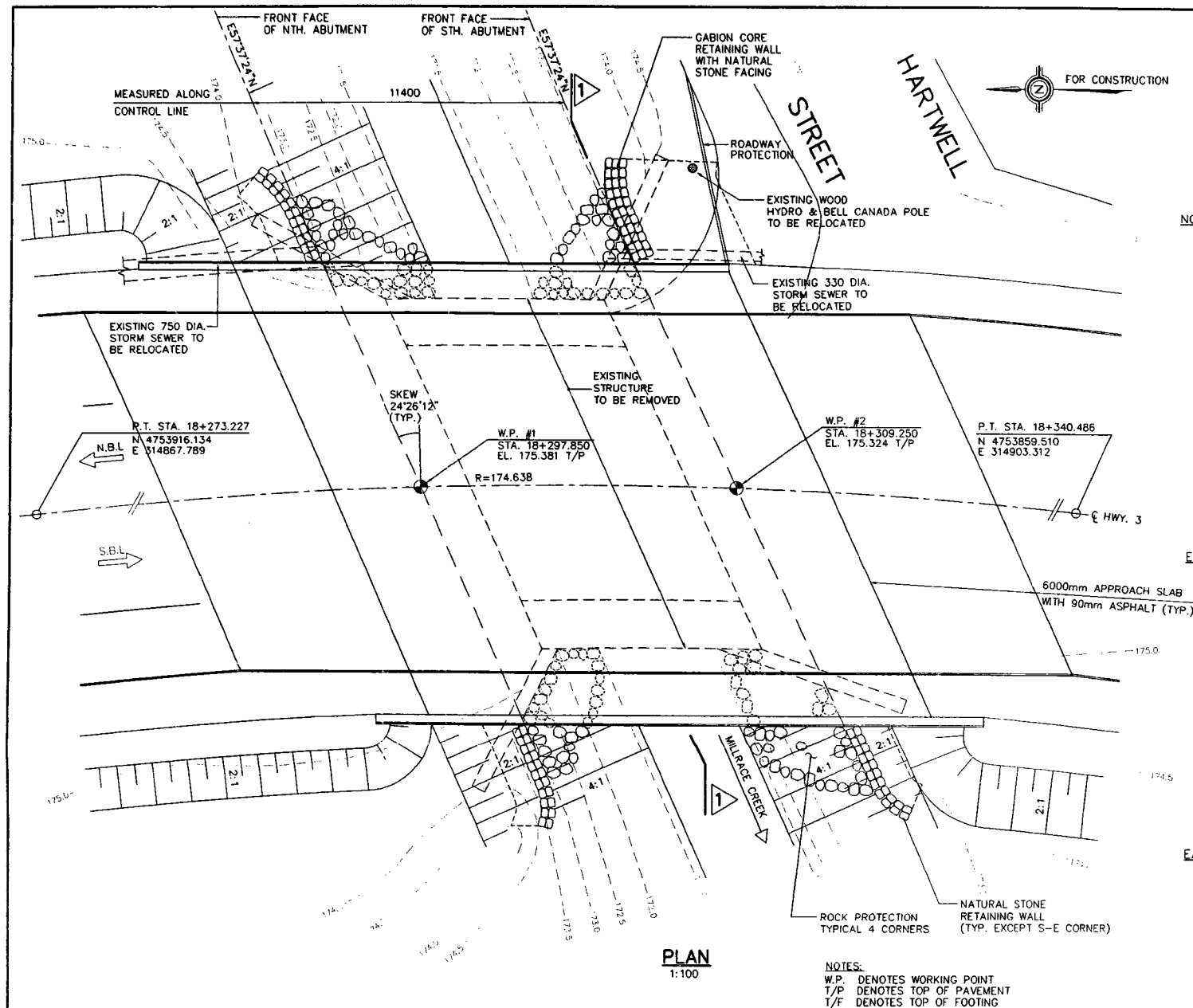
Idealized Profile Used In Settlement Analysis

Project: 461/SP2431

Hwy 3/ Mill Race Bridge Replacement

December 2, 1998





METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST No 6

CONT No

WP No 439-97-01

MILLRACE BRIDGE REPLACEMENT
HWY. 3, WAINFLEET

GENERAL ARRANGEMENT



paul theil associates limited
consulting engineers
131 Delta Park Blvd. , Brampton, Ontario

GENERAL NOTES:

1. CLASS OF CONCRETE
 * ALL 30 MPa
2. CLEAR COVER TO REINFORCING STEEL
 * FOOTINGS 100 ± 25
 * DECK:
 TOP. 70 ± 20
 BOT. 40 ± 10
 REMAINDER 70 ± 20 UNLESS
 OTHERWISE NOTED
3. REINFORCING STEEL
 * REINFORCING STEEL SHALL BE GRADE 400
 UNLESS OTHERWISE SPECIFIED. BAR MARKS
 WITH PREFIX "C" DENOTE COATED BARS.
 * UNLESS SHOWN OTHERWISE, TENSION LAP LENGTHS NOT
 INDICATED ON THE CONTRACT DRAWINGS SHALL BE CLASS B.
 * BAR HOOKS SHALL BE MINIMUM LENGTH
 UNLESS INDICATED OTHERWISE.
- LIST OF DRAWINGS:
1. GENERAL ARRANGEMENT
 2. BORE HOLE LOCATIONS AND SOIL STRATA
 3. FOUNDATION LAYOUT AND FOOTING REINFORCING
 4. ABUTMENTS AND WINGWALLS
 5. DECK DETAILS AND REINFORCING
 6. PARAPET WALL WITH S/W & RAILING—PERFORMANCE LEVEL 2
 7. RAILING FOR BARRIER/PARAPET WALL
 8. RETAINING WALLS
 9. 6000 mm APPROACH SLAB
 10. QUANTITIES — STRUCTURES

BM 176.226
GEODETIC DATUM
MTO BM 372-69

PRINTED

SEP 10 1999

PAUL THEIL
ASSOCIATES LIMITED

APPLICABLE STANDARD DRAWINGS:

OPSD - 3501.00 GRANULAR BACKFILL REQUIREMENTS - ABUTMENTS
OPSD - 4010.00 GUIDE RAIL & CHANNEL ANCHORAGE

REVISIONS									
	DATE	BY	DESCRIPTION						
DESIGN	CHK	MW	CODE	OHBC-91	LOAD	CL-A	DATE	SEPT. 99	
DRAWN	KS	CHK	MW	SITE	34-102	STRUCT	SCHUF	OWC	