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W.P. No. 95-90-01 ~~96-90-01~~

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W. O. No.

STR. SITE No. 10-44

HWY. No. 401

LOCATION Hwy 401 & Campbellville Rd.

No of PAGES

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

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FOUNDATION INVESTIGATION REPORT

W.P. 95-90-01/96-90-01 Bridge Site 10-44

Proposed Structure Addition

Hwy. 401 and Campbellville Road

District 4, (Burlington)

Ministry of Transportation, Ontario

CONT 93-07

Submission Date: 1991 05 21

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District 4, (Burlington)

Ministry of Transportation, Ontario

1.0 INTRODUCTION

Strata Engineering Corp. has been retained by the Foundation Design Section of the Ministry of Transportation, Ontario, under Consultant Agreement No: 4240-9190-193, to conduct a foundation investigation for a proposed inside widening of Highway 401 at Campbellville Road. The widening is to be accomplished with a deck in the median gap between existing twin structures. The terms of reference were to investigate the subsurface conditions for the support of the deck widening and for any road protection requirements.

This report is submitted in compliance with these terms of reference.

2.0 SITE AND GEOLOGY

The site is located 6.8km west of Highway 25 in the Regional Municipality of Halton.

At this site, Highway 401 crosses Campbellville Road at a 7° skew angle on twin overpasses, one each for the eastbound and westbound lanes. Highway 401 is built up on fill some 7.5m above the prevailing ground level. The clear transverse distance between the twin structures is 4.2m.

The median fill between the twin overpass structures is retained by means of vertical concrete slabs cast in line with the ballast walls of the abutments. Archival drawings show the footings for the twin structures have been taken down a considerable depth below the profile grade of Campbellville Road. The reason for this is not known.

Along the median, a concrete guide rail protects the open gap between the twin structures. Inertia absorbtion barriers are located on either side of the guide rail.

The dominant geological feature of the area is the Niagara Escarpment. The terrain is gently undulating, and a number of gravel pits are evident within a radius of 1km of the site. Outwash gravels are likely present to the east and ice contact kames and eskers to the west.

Drift thickness and bedrock topography maps indicate a bedrock depth in this area of 30±m below prevailing ground surface.

3.0 FIELD AND LABORATORY WORK

Boreholes were drilled between 1991 01 28 and 29 using two bombardier mounted CME 55 drill rigs, each drilling two boreholes. Two boreholes (BH1 and BH3) were accompanied by dynamic cone penetration resistance tests. All boreholes were advanced with hollow stem augers.

Maintenance staff of the MTO Burlington District provided traffic protection assistance when the drill rigs were moved to and from the Highway 401 median.

Four boreholes were drilled along the median of the highway to depths ranging from 11.1m to 17.2m below ground surface, at locations shown on Drawing No: 95/969001-A appended. Boreholes 2 and 3 for the new abutment footings were located as close as practical to the vertical concrete slabs, within the constraints of underground structures and services.

In Boreholes 2 and 4, sealed perforated standpipes were installed to monitor groundwater levels.

Borehole elevations are referenced to geodetic datum.

Recovered soil samples were transported to our Don Mills laboratory where they were visually classified according to the USC system. Index property tests such as moisture contents, grain size analyses and Atterberg limits were performed on selected samples. The results are shown on the Record of Borehole Sheets as well as on Figures 1 to 4 in the Appendix.

4.0 SUBSURFACE CONDITIONS

4.1 General

Fill material was encountered to depths of some 7m below ground surface. The fill is underlain by very dense sandy silt to silty sand on the west side of the gap between the twin structures and a very dense sand and gravel deposit on the west side. The groundwater table was found at depths of about 10m below ground surface. Details are provided below.

4.2 Het. Mixture of Clayey Silt, Sand and Gravel (Road Fill)

Frozen road fill comprising brown sand and gravel with some silt was found to depths ranging from 0.8m to 1.4m.

Below the frozen zone, the fill material consists of a heterogeneous mixture of clayey silt, sand and gravel.

The overall fill material is quite variable in composition, being slightly cohesive in some locations (and depths) and almost a clean non-cohesive sand and gravel at other locations (and depths). Overall, the fill material is classified as being non-cohesive.

The moisture content of the fill material ranged widely depending on its composition and heterogeneity (presence of clayey silt), from lows of about 6 per cent to highs of about 28 per cent. The average moisture content is about 15 percent. Grain size analyses on representative samples are shown on Figures 1A (relatively clean sand and gravel) and Figure 1B (heterogeneous mixture).

N values ranged between 6 and 50 blows/0.3m, being on average about 17 blows/0.3m, indicating the material is loose to dense, being generally compact.

4.3 Clayey Silt (Compressed Topsoil)

At Borehole 3, the fill material was found to be underlain by a 0.9m thick layer of dark brown compressed topsoil consisting of clayey silt (refer Figure 2). The moisture content of this layer (26 per cent) was found to be above the plastic limit of the soil. The consistency of the soil is estimated to be stiff based on tactile examinations in the laboratory.

This material was not encountered in the sampling conducted at the other borehole locations at this site. Nonetheless, its presence should be expected because the highway fill was probably placed without benefit of excavation of the pre-existing topsoil.

4.4 Sand and Gravel

On the east side of the gap between the twin structures (Boreholes 3 and 4), a deposit of sand and gravel was found below either the fill material (Borehole 4) or below the compressed topsoil (Borehole 3). Its thickness ranged from 3.0m (at Borehole 3) to 5.8m+ (at Borehole 4). This deposit was not encountered on the west side of the gap (Boreholes 1 and 2)

In Borehole 3, the stratum was fully penetrated to a lower silty sand to sandy silt deposit. At Borehole 4, the stratum was not fully penetrated.

The average moisture content of the stratum was found to be about 7 per cent.

The gradation of the material found within this stratum was quite variable in gravel content, as shown by the envelope of Figure 3. The silt and clay content was found to be generally less than 5 per cent.

N values of 33 to 70 blows/0.3m indicate the stratum is very dense. One low value of 24 blows/0.3m is attributed to unbalanced hydrostatic uplift causing loosening of the soil within the borehole.

4.5 Silty Sand to Sandy Silt

Below the fill material on the west side of the gap between the twin structures (Boreholes 1 and 2) and below the sand and gravel stratum in Borehole 3, a brown silty sand to sandy silt deposit was encountered. The deposit occurs at elevation 264.5m on the west side and at elevation 259.4m on the east side (in Borehole 3).

The natural moisture content of samples from this deposit ranged from 12 to 26 per cent, being on average about 18 per cent. Gradation curves of the sandy silt portions of the deposit are shown in Figure 4A and of the silty sand portions in Figure 4B.

N values varied from 88 to 122 blows/0.3m in Borehole 1 to 27 to 58 blows/0.3m in Boreholes 2 and 3, indicating a wide variability in relative density. On the basis of these N values the overall deposit is considered to be very dense at the location of Borehole 1 and dense to compact at Boreholes 2 and 3.

4.6 Groundwater Conditions

Groundwater was not encountered in Borehole 1 which was drilled to a final depth of 11.1m below ground surface. Groundwater was observed some five days after completion of drilling at elevation 260.7m to 260.8m in both the sealed standpipes. This corresponds with the observations of water levels made in Borehole 3 (which was backfilled upon completion).

Hence, at the time of this investigation, the groundwater table was located at a depth of about 10m below ground surface.

5.0 DISCUSSIONS AND RECOMMENDATIONS

5.1 General

It is proposed to widen Highway 401 from 4 to 6 lanes between Highway 25 and Guelph Line by the construction of two additional lanes in the existing median. The construction of the additional lanes will require the closing of the gap between twin overpasses carrying Highway 401 across Campbellville Road.

Archival drawings indicate the existing twin structures are supported on almost 2m wide spread footings located at elevation 260.7, some 4m below the present profile grade of Campbellville Road, and presumably at or just below the groundwater table.

The present bridges show some signs of deterioration. Spalling of concrete is present at the base of the abutment on the south bridge, with reinforcing bars visible. Concrete is also spalling at the southeast wing wall and the east abutment of the south structure. The southern end ballast wall of the WBL structure (north bridge) also requires to be examined for possible structural deterioration. Vertical cracking is evident on the southeast and southwest wing walls. There is also some sign of reinforcement corrosion near the base of both deck slabs.

The construction of the additional lanes will entail closing the gap between the twin bridge abutments. This will require the removal of the existing concrete vertical slabs. Road protection will be required if the new abutments are placed on footings to match the existing footings.

The site investigation shows the presence of about 7m of road fill material (heterogeneous mixture of clayey silt, sand and gravel, of quite variable composition) overlying a dense to very dense silty sand to sandy silt deposit on the west side of the gap and a dense to very dense sand gravel stratum on the east side, overlain in one location by a thin compressed topsoil seam. The groundwater table is situated some 10m below the highway median ground surface level.

5.2 Structure Foundations

5.2.1 Spread Footings

Spread footings, 1.5m in width and placed at elevation 260.7±m in the very dense to dense sand and gravel deposit at the east abutment and in the very dense to compact silty sand deposit at the west abutment, may be designed for the following factored bearing capacities:

Factored Capacity at ULS	800 kPa
Capacity at SLS Type II	320 kPa

At the SLS Type II capacity, the total settlement of the new footings is likely to be elastic in nature and not in excess of 6-8mm.

Resistance to sliding may be computed using an unfactored effective angle of internal friction of 35° between concrete and the sand and gravel stratum and of 30° between the silty sand deposit.

Assume the unit weight of the sand and gravel to be $22.0 kN/m^3$.

5.2.2 Deep Foundations

1. Caissons

The preferred alternative to spread footings is a caisson supported foundation scheme, which would eliminate the need for road protection, since the caissons could be augered from the existing median level.

Caissons, with their base located at about elevation 260.7m, may be designed for the following factored load capacities:

Caisson Dia. (mm)	ULS Factored Capacity (kN)	SLS Type II Capacity (kN)	Estimated Diff. Settlement (mm)
508	600	240	15 - 20
508	600	265	18 - 24
600	750	300	18 - 24
600	750	265	16 - 22
750	1200	500	18 - 25
750	1200	265	12 - 18
1200	3000	1200	22 - 28
1200	3000	265	8 - 10

The differential settlement estimates given above are elastic in nature and will occur almost immediately upon application of the design load. They ignore axial compressive strain of the caisson. Natural soil conditions are never the same everywhere and construction practices may cause undesirable disturbance of the soil. Hence, actual differential settlements may differ from these estimated values. Only full scale caisson load tests can provide greater confidence in the calculated settlement estimates.

A heavy steel liner should be used to advance the caissons. The level of unsupported excavation below the liner should not exceed 150mm during installation.

In order to avoid disturbing the foundation soil below the existing footings, the caisson spacing and location in plan should be such that a minimum distance of 1.0m is maintained between the exterior wall of the caisson liner and the existing abutment footing heels. To avoid subsoil overstressing (and increased elastic settlements), the minimum spacing between caissons should be at least 2.0 times the diameter of the larger adjacent caisson.

2. Steel H Piles

Steel H piles may be considered as an alternative to caissons, but are not recommended due to the danger that pile driving, especially close to the existing footing heels, could cause dilation of the dense sand and gravel and consequent settlement of the existing footings.

5.3 Earth Pressures

Earth pressures should be computed as per subsection 6-6.1.2.2 of the OHBD Code. A yielding foundation condition may be assumed. The granular A or B backfill should be in accordance with special provision No.109F03 (latest revision). The following parameters are recommended for granular backfill.

	Gran "A"	Gran "B"
Angle of internal friction ϕ'	35.0°	30.0°
Unit weight (kN/m^3) γ	22.8	21.2

Surcharge effects should be computed as per Clause 6-6.1.2.4 of the OHBD Code.

5.4 Construction Considerations

The spread footing option will require roadway protection by means of a shoring system placed inside the excavation adjacent to the travelled highway. The very dense nature of the sand and gravel and silty sand deposits precludes driven interlocking steel sheet piling as a viable option. Therefore, soldier piles and timber lagging may be the most practical alternative for excavation shoring. Soldier piles would need to be augered down at least 1m into natural soil and concreted in place. The depth of soldier pile toe embedment below the base of the excavation will depend on the shoring design used (whether cantilever, braced or tied back).

For the design of an internally braced system, use a rectangular distribution of earth pressure with a base width of $0.65\gamma Hk_a$ where H is the internal braced height. The granular B earth pressure and unit weight values given in section 5.3 above may be used in design.

Roadway protection, if required, should be of such length parallel to the highway that the angle, measured with the horizontal, from the end of the protection scheme to the new footings is 30° or less.

Excavated material may be re-used as general backfill to the new abutments.

6.0 CLOSURE

The field work for this investigation was carried out by Ms. Andrea C. Abel and Mr. Zareh Dervichian.

Drilling equipment and crew was provided by Master Soil Investigation Ltd. of Weston, Ontario.

Mr. Jim McLean of the MTO Burlington District kindly provided traffic protection services for this investigation.

Respectfully Submitted:
STRATA ENGINEERING CORP.

A.C. Abel, M.Sc.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
C. MIRZA
PROVINCE OF ONTARIO

for C. Mirza, P.Eng.
Senior Principal

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S-91-311

A P P E N D I X

Explanation of Terms Used in Report

Record of Boreholes 1 to 4

Figures 1 to 4

Drawing 95/969001-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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

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_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_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = $w_L - w_P$
I_L	1	LIQUIDITY INDEX = $\frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_P}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE

e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 1 METRIC

W P 95-90-01, 96-90-01 LOCATION N: 4 817 192.4 ; E: 266 746.0 ORIGINATED BY A.A.

DIST 4 HWY 401 BOREHOLE TYPE Hollow Stem Auger, Dynamic Cone Test COMPILED BY A.K.

DATUM Geodetic DATE 1991 01 28 & 29 CHECKED BY C.M.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
271.3	Ground Surface					
0.0	Frozen Zone				271	
	Het. Mixture of Clayey Silt, Sand and Gravel (Road Fill)	1	SS	27	270	
	Compact to Loose	2	SS	14	269, 268	
	Brown	3	SS	6	267, 266	
264.5		4	SS	9	265	0 10 72 18
6.8	Sandy Silt to Silty Sand	5	SS	88	264	
	Very Dense	6	SS	103/20cm	263, 262	
260.2	Brown	7	SS	122	261	0 19 76 5
11.1	End of Borehole * Borehole dry upon completion					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ

GROUND WATER CONDITIONS *

$+^3, \times^5$: Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 2

METRIC

W P 95-90-01 & 96-90-01 LOCATION N: 4 817 193.2 ; E: 266 753.8 ORIGINATED BY A.A.

DIST 4 HWY 401 BOREHOLE TYPE Hollw Stem Auger, COMPILED BY A.K.

DATUM Geodetic DATE 1991 01 28 CHECKED BY C.M.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
271.3	Ground Surface						
0.0	Frozen Zone	1	SS	–		271	
	Sand and Gravel (Road Fill) Dense to Compact Brown	2	SS	39		270	50 47 (3)
		3	SS	17		269, 268	
	Heterogeneous mixture of Clayey Silt,Sand and Gravel (Road Fill) Compact to Dense Brown	4	SS	17		267, 266	
		5	SS	50		265	
264.5 6.8	Sandy Silt to Silty Sand	6	SS	58		264	NP
	V. Dense to Compact	7	SS	44		263, 262	0 31 55 14
		8	SS	35	Seal	261, 260	NP; W.L. on 1991 02 04
	Brown	9	SS	28		259	
		10	SS	36	Standpipe	258, 257	14 15 (71)
256.3 15.0	Cont. on Sheet 2						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ

$+^3$, $\times^5$: Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 2 cont'd

METRIC

W P 95-90-01 & 96-90-01 LOCATION N: 4 817 193.2 ; E: 266 753.8 ORIGINATED BY A.A.

DIST 4 HWY 401 BOREHOLE TYPE Hollow Stem Auger, COMPILED BY A.K.

DATUM Geodetic DATE 1991 01 28 CHECKED BY C.M.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
256.3	Cont. from Sheet 1										
15.0	Sandy Silt to Silty Sand Compact to Dense		11	SS	27		256		○		0 91 (9)
254.7	Brown		12	SS	36		255		○		
16.6	End of Borehole										

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3

METRIC

W P 95-90-01 & 96-90-01 LOCATION N: 4 817 194.0 ; E: 266 778.0 ORIGINATED BY Z.D.

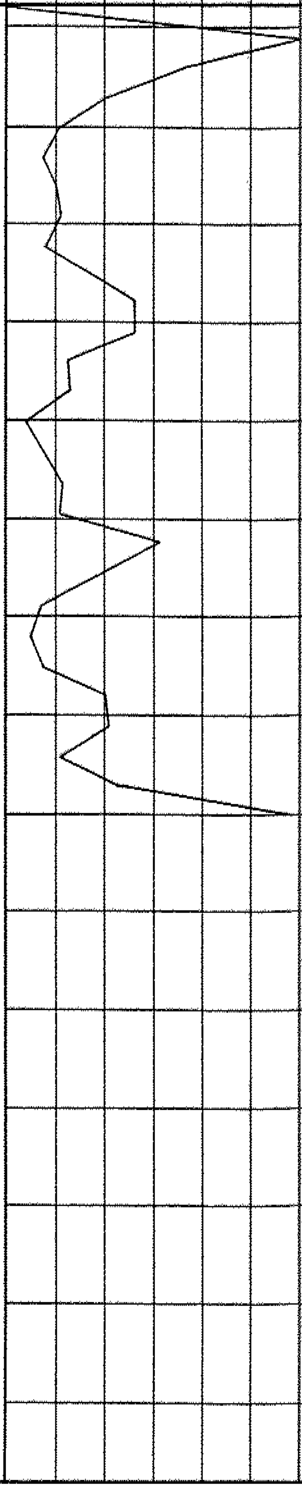
DIST 4 HWY 401 BOREHOLE TYPE Hollow Stem Auger, Dynamic Cone Penetration Test COMPILED BY A.K.

DATUM Geodetic DATE 1991 01 28 CHECKED BY C.M.

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
271.2	Ground Surface						
0.0	Frozen Zone	1	SS	–		271	
	Sand and Gravel (Road Fill)	2	SS	17		270	
	Compact to Dense	3	SS	19		269, 268	44 53 (3)
		4	SS	27		267	
	Reddish Brown	5	SS	32		266, 265	51 42 (7)
263.7 / 7.5	Clayey Silt (Compressed Topsoil)	6	SS	11		264, 263	
262.8 / 8.4	Stiff Brown Sand and Gravel						
	Very Dense to Dense	7	SS	55		262	51 45 (4)
					W.L.	261	W.L. on 1991 01 28
	Brown	8	SS	45		260	81 17 (2)
259.4 / 11.8	Sandy Silt to Silty Sand	9	SS	34		259	
	Dense					258	
	Brown	10	SS	45		257	0 96 (4)
256.2 / 15.0	Cont. on Sheet 2						

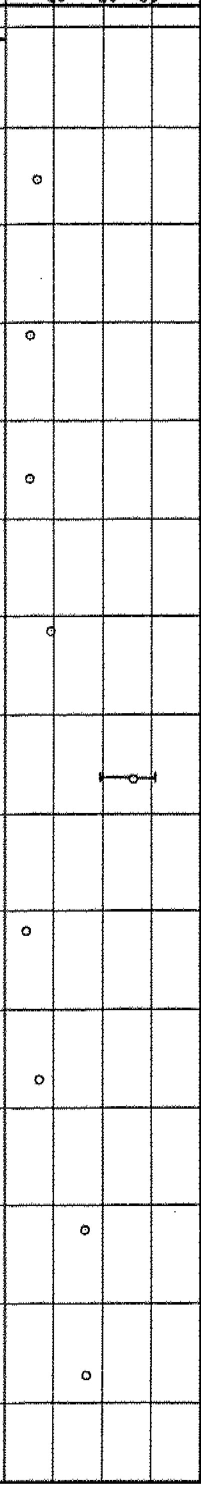
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE



PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ



$+^3$, $\times^5$: Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 3 cont'd

METRIC

W P 95-90-01 & 96-90-01 LOCATION N: 4 817 194.0 ; E: 266 778.0 ORIGINATED BY Z.D.

DIST 4 HWY 401 BOREHOLE TYPE Hollow Stem Auger, Dynamic Cone Penetration Test COMPILED BY A.K.

DATUM Geodetic DATE 1991 01 28 CHECKED BY C.M.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
256.2	Cont. from Sheet 1										
15.0	Sandy Silt to Silty Sand						256				
			11	SS	29				o		0 19 (81)
	Compact to Dense						255				
254.0			12	SS	46				o		
17.2	End of Borehole										

$+^3$, $\times^5$: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 4

METRIC

W P 95-90-01 & 96-90-01 LOCATION N: 4 817 196.8 ; E: 266 787.8 ORIGINATED BY Z.D.

DIST 4 HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY A.K.

DATUM Geodetic DATE 1991 01 29 CHECKED BY C.M.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
271.0	Ground Surface						
0.0	Frozen Zone	1	SS	-			
	Het. mixture of Clayey Silt, Sand and Gravel (Road Fill)					270	
		2	SS	9		269	12 73 (15)
	Loose to Dense	3	SS	20		268	
						267	
		4	SS	12		266	
	Brown					265	
		5	SS	34			
264.2 6.8	Sand and Gravel				Seal	264	
		6	SS	70		263	
	V. Dense to Compact					262	
		7	SS	47			37 59 (4)
						261	W.L. on 1991 02 04
		8	SS	33		260	10 88 (2)
	Brown				Standpipe	259	
258.4		9	SS	24			40 58 (2)
12.6	End of Borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

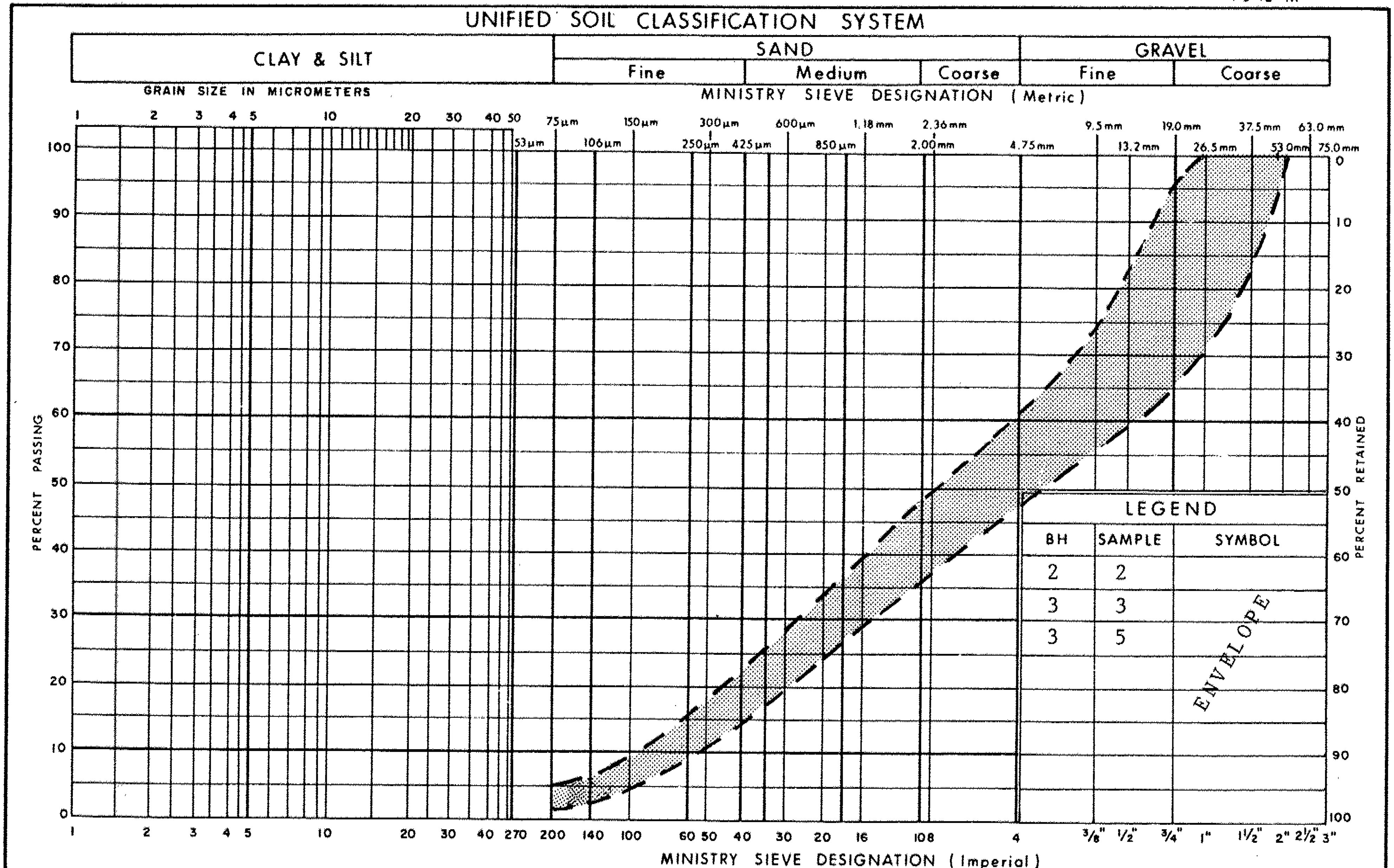
UNIT WEIGHT γ

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20, 10)

78 12 M



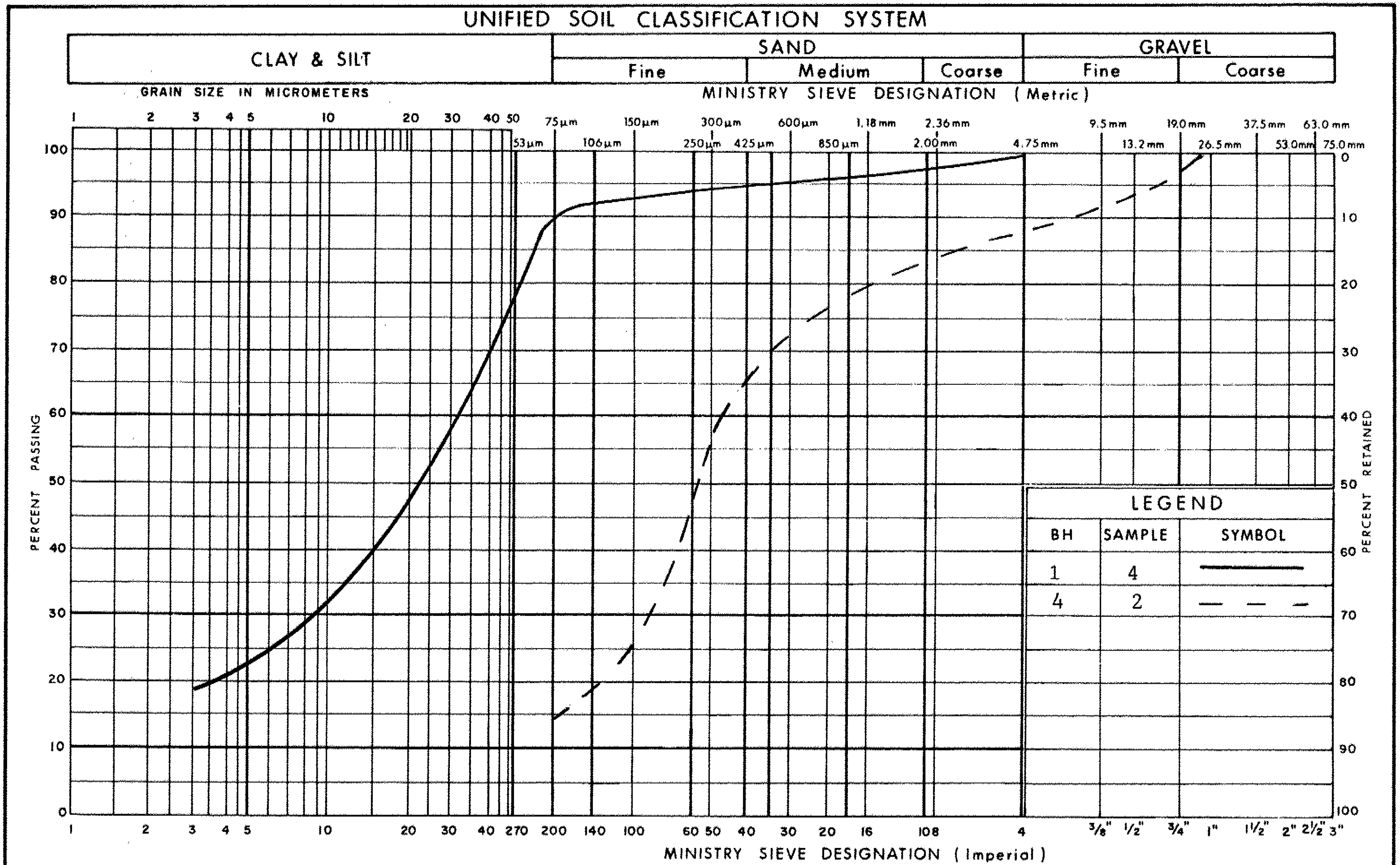
Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Sand and Gravel
(Road Fill)

FIG No	1A
W P	95-90-01 & 96-90-01
Hwy.401/Campbellville Rd.	

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Clayey Silt, Sand and Gravel

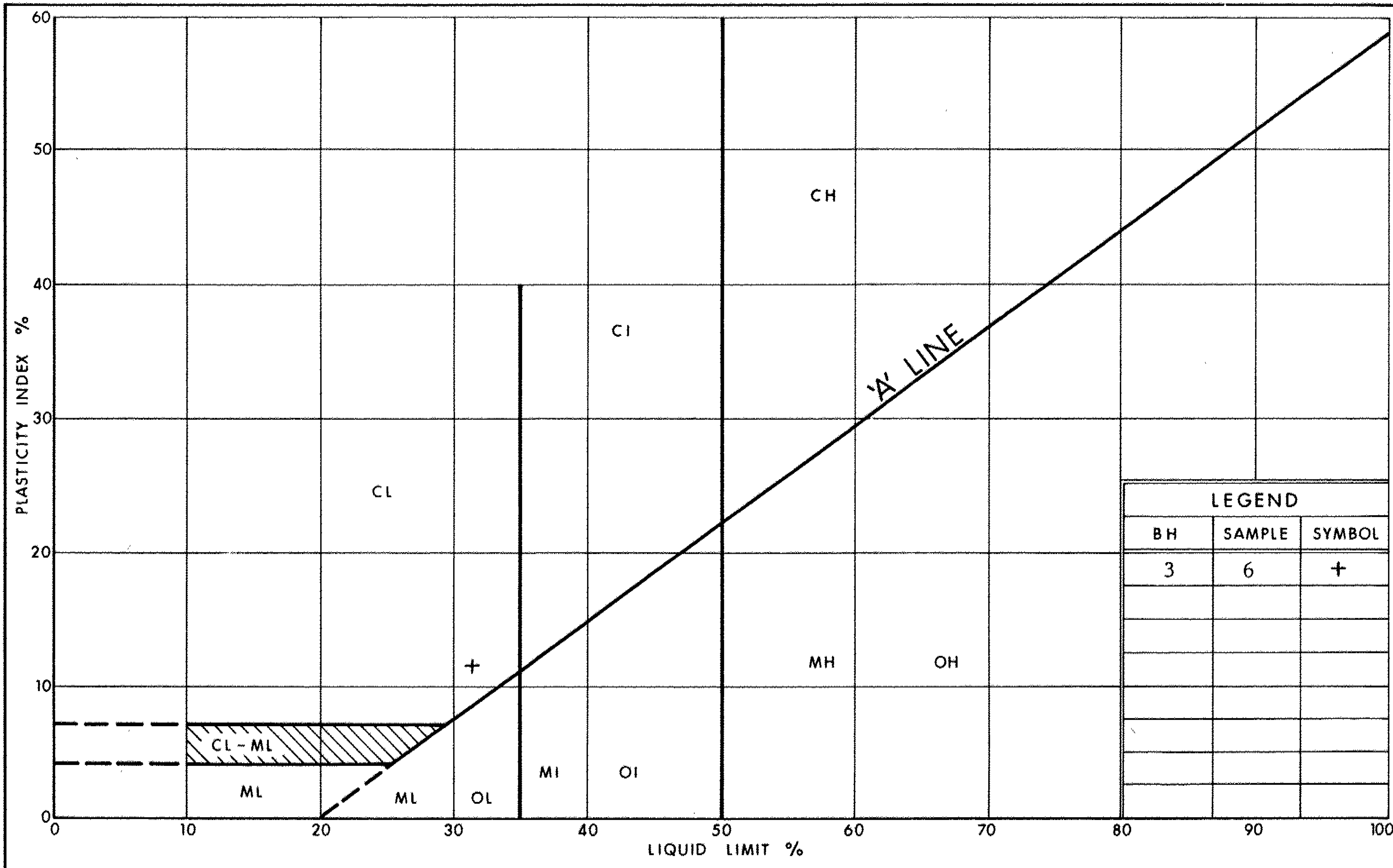
(Road Fill)

FIG No 1B

W P 95-90-01 & 96-90-01

Hwy. 401/Campbellville Rd.

Oct 75, FF-S-21



Ministry of Transportation
Ontario

PLASTICITY CHART

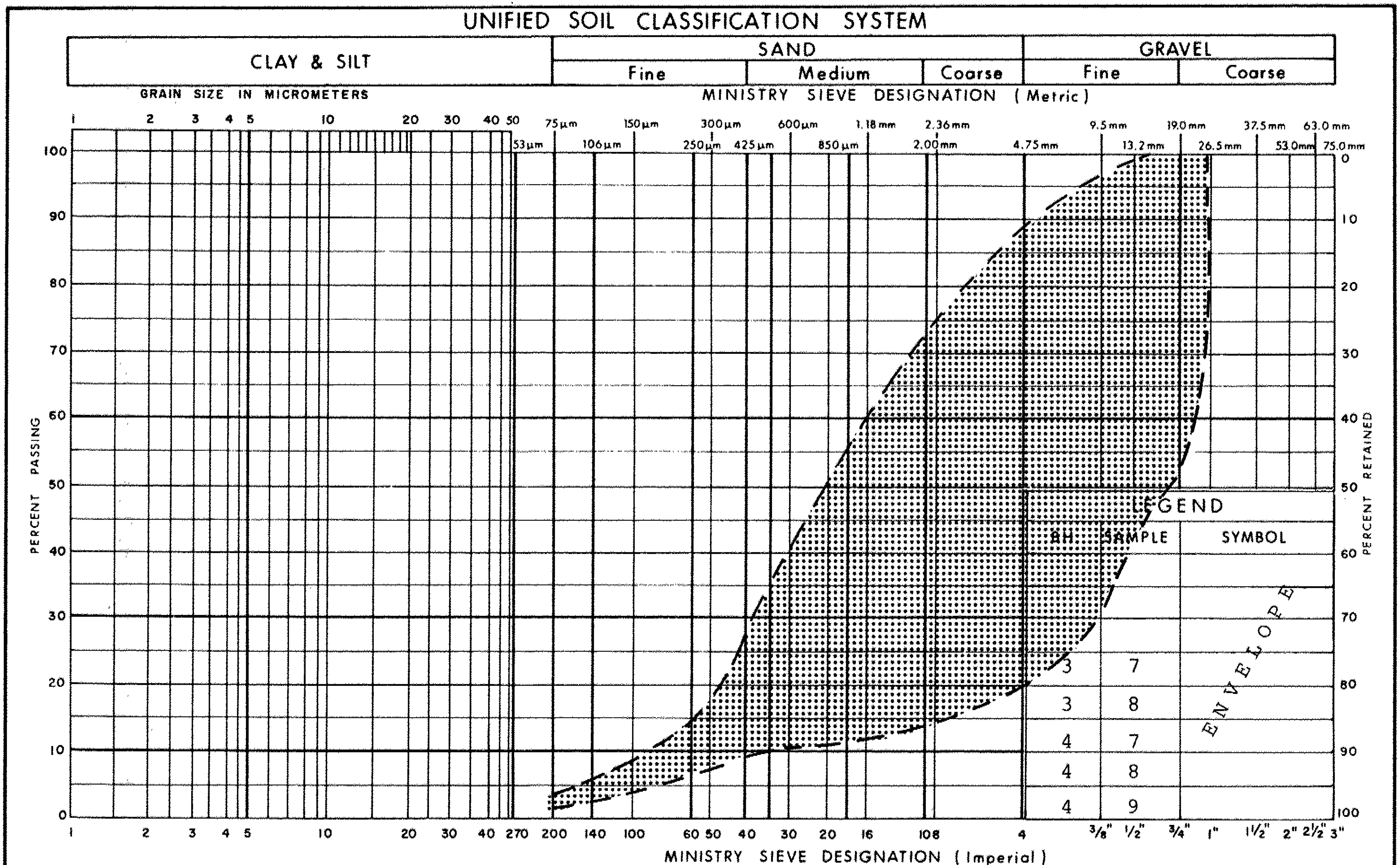
Clayey Silt
Compressed Topsoil

FIG No 2

W P 95-90-01 & 96-90-01

Hwy. 401/Campbellville Rd.

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

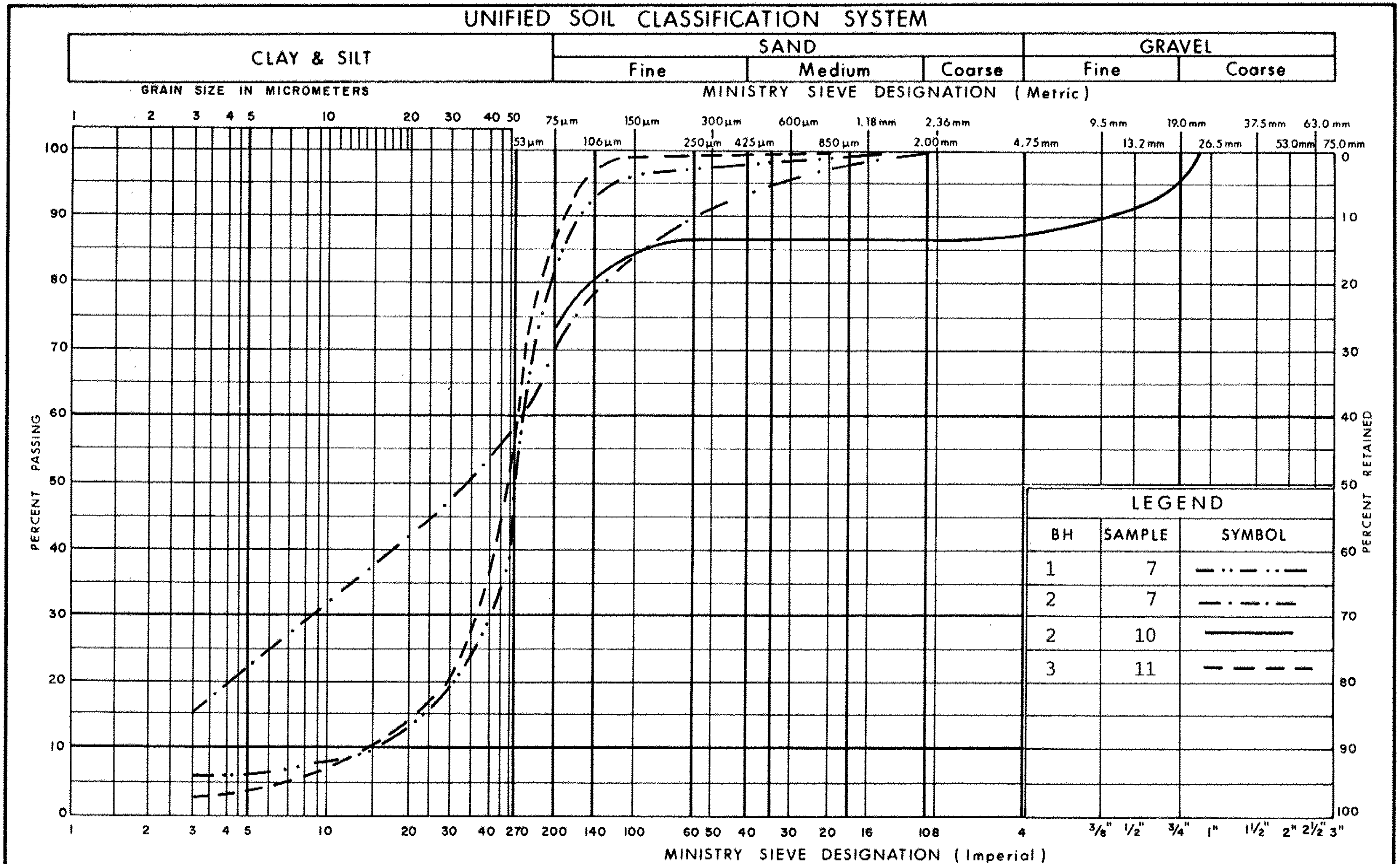
Sand and Gravel

FIG No 3

W P 95-90-01 & 96-90-01

Hwy. 401/campbellville Rd.

78 12 M



Ministry of Transportation
Ontario

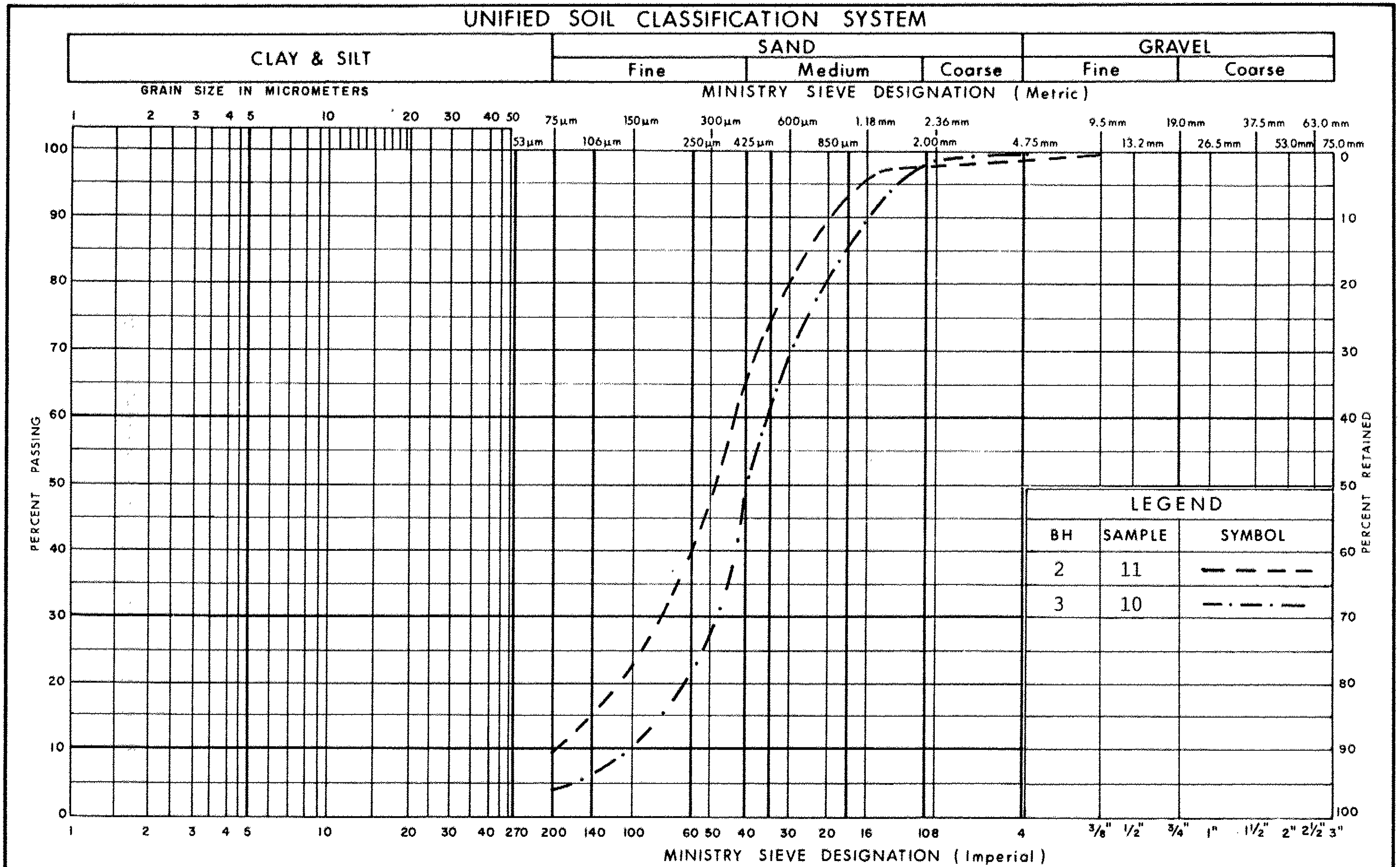
GRAIN SIZE DISTRIBUTION

Sandy Silt

FIG No 4A

W P 95-90-01 & 96-90-01

Hwy. 401/Campbellville Rd.



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION Silty Sand	FIG No 4 B W P 95-90-01 & 96-90-01 Hwy.401/Campbellville Rd.

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10

METRIC

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

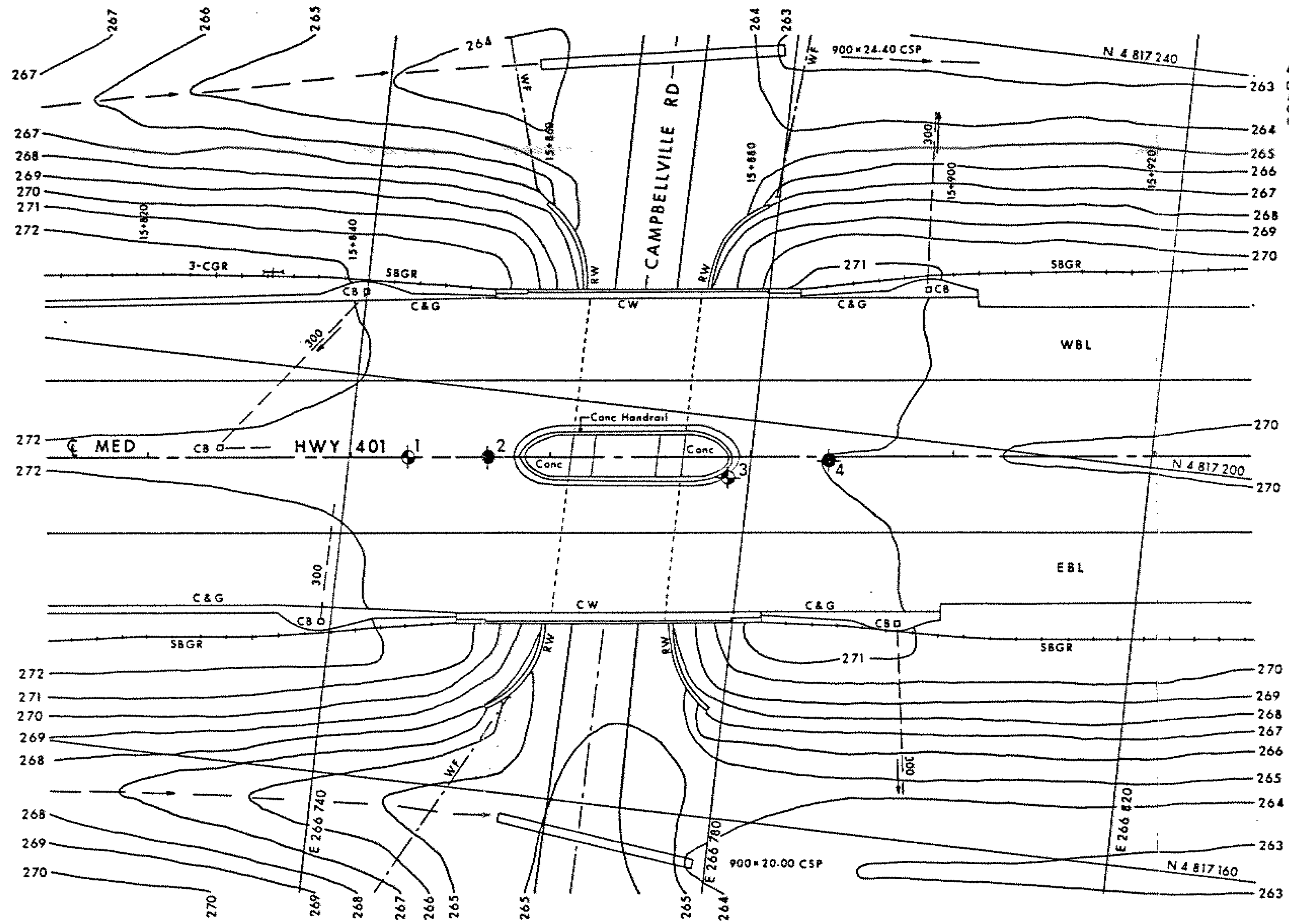
CONT No
WP No95&96-90-01

SHEET

CAMPBELLVILLE RD. OVERPASS

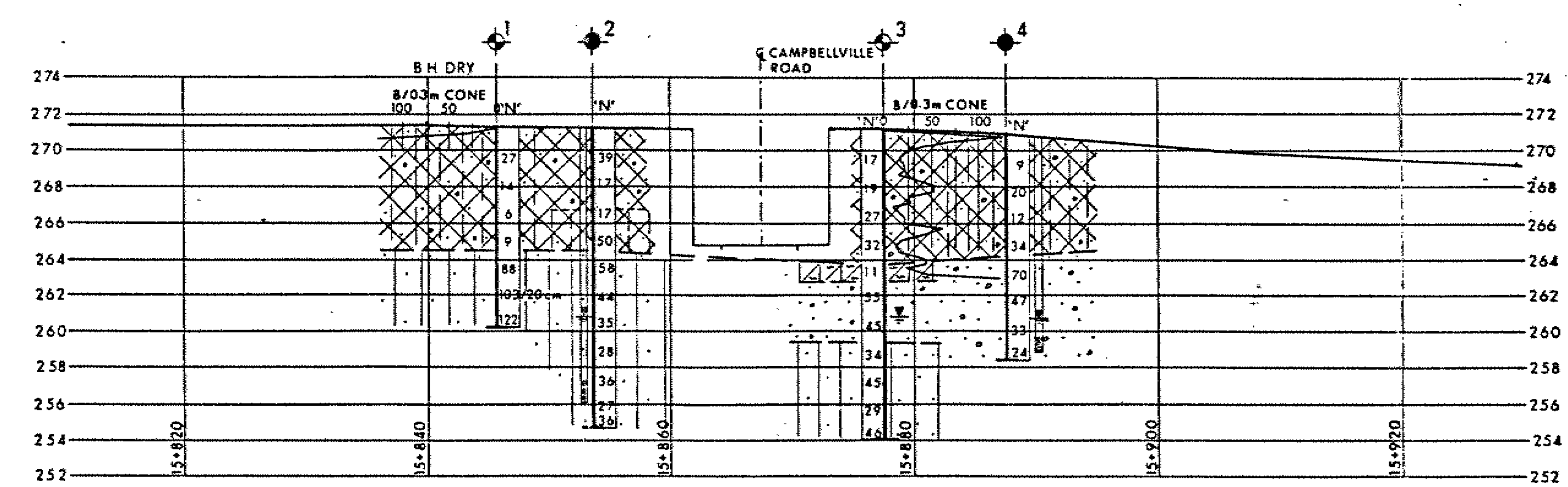
BORE HOLE LOCATIONS & SOIL STRATA

STRATA ENGINEERING CORP.



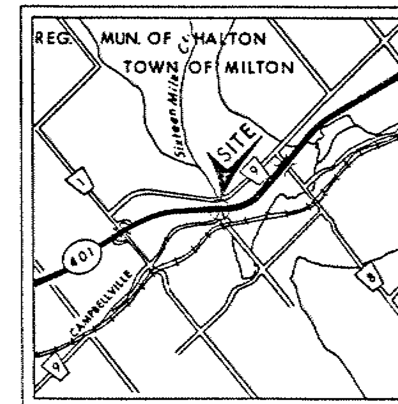
PLAN

SCALE 5m 0 5m



℄ PROFILE MED HWY 401

SCALE
HOR 5m 0 5m
VERT 5m 0 5m



KEY PLAN

SCALE 1Km 0 1 2Km

SOIL STRATIGRAPHY-LEGEND

- SAND & GRAVEL (ROAD FILL) Compact to Dense
- HET. MIXTURE OF CLAYEY SILT, SAND & GRAVEL (ROAD FILL) Loose to Dense
- CLAYEY SILT (COMPRESSED TOPSOIL) Stiff
- SANDY SILT TO SILTY SAND Compact to V.Dense
- SAND & GRAVEL Compact to V.Dense

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation Jan. & Feb. 1991
- Stand Pipe

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	271.3	4 817 192.4	266 746.0
2	271.3	4 817 193.2	266 753.8
3	271.2	4 817 194.0	266 778.0
4	271.0	4 817 196.8	266 787.8

NOTE

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

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

REV.	DATE	BY	DESCRIPTION

Geocres No 30M5-182

HWY No 401			DIST 4
SUBM'D A A	CHECKED AA	DATE Mar 26 1991	SITE 10-44
DRAWN A K	CHECKED AA	APPROVED	DWG 95&96900LA

REF No E-162-401-1; 90 11

Construction problem {Cont 93-07 / WP 452-89-00}

1.1 Caisson can not penetrate to design depth
about 6' above.

2. Location East of Guelph Line.

3. Steve Rideout - Contract administrator
519-824-9730

Site visit

1) Auger to penetrate with smaller size → to design size

2) Or chiseling

3) Time 12:00 - 15:30.
met Steve Rideout and Mike
- hit Boulders.

4) Recommendation.
a) drill pilot hole
b) use larger auger gradually
c) To Final Diameter
d) OK now.

65 Km x 2 + 30 Km local ~

B. M. ROSS AND ASSOCIATES LIMITED

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FAX (519) 524-4403

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K. G. DUNN, P.ENG.
S. D. BURNS, P.ENG.
B. W. POTTER, P.ENG.
R. R. ANDERSON, P.ENG.

FAXED COPY

OUR FILE NO.

BR-567

June 19, 1991

Mr. Allan Ma
Ministry of Transportation
Planning and Design Section
659 Exeter Road
P.O. Box 5338
London, Ontario
N6A 5H2

Dear Sir:

Re: Campbellville Road Overpass
Hwy. #401, WP95/96-90-01

As discussed in our recent telephone conversation, we have revised our concrete filled end bearing support caisson layout to match the minimum clear distance of 1.5 times the dia. of the caisson as required by your M.T.O. Foundation Office. We understand that they will not accept the use of any piling as outlined in our proposal of June 14, 1991.

We are enclosing sketches showing three different views. The first is a plan view taken at the bottom of the deck haunch or the approximate top of the caisson. The second is a plan view at the bottom of the caisson which is also the bottom level of the existing footings. The third view is a longitudinal half section taken along the centreline of the new central structure.

We have attempted to try to find a support solution which results in the bottom of the caissons at a minimum clear distance of 1.5 times the dia. of the caisson. We have used two rows of caissons with a greater total number of support units, in order to lower the individual loading and the anticipated settlement.

We are proposing the use of five-508mm dia. concrete filled caissons with three in the front row and two in the rear row. If we were to install both rows of caissons at a 1 in 3 batter, this would mean that the top of the caissons would be approximately 1,280 mm between the rows as measured along the centreline of the widening. This adds greatly to the span of the structure, and presents design problems with the distribution of the negative moment from the caissons and the distribution of the loading equally between the front and the rear row caissons.

To solve our design concerns at the top of the caisson, we have altered the batter on the front row to 1 in 4, while maintaining the batter on the rear row at 1 in 3. This results in the distance between the front and rear rows of caissons at the top being reduced to 500 mm while providing the distance between the bottom of the caisson rows at 1,280 mm.

This layout matches foundations individual spacing of the caissons at the bottom of caisson level where in the front row the distance between caissons is 1,512 mm centres, or 1.98 times the dia. clearance. The closest distance of the caissons in the second row to the caisson in the first row is 1,367 mm centres or a minimum clear distance of 1.69 times the dia.

The use of the front row of caissons at the 1 in 4 batter does result in the closest outside caisson distance being slightly reduced to 1.924 m from the centreline of the caisson to the corner of the existing footing projection at the northeast corner of the new structure which is the closest point. With the original 1 in 3 batter, this distance was 2.689 m.

With the slight increase in loadings using the increased half span length of 6,609 mm to the mid-point between the first and second row or caissons, we have a total dead load and live load force of ULS 1,586 kN and SLS 1,154 kN. Distributing the SLS load evenly among the 5 caissons would result in a load per caisson of 231 kN. From the soil investigation report we anticipate that this would be an estimated settlement of 14 to 19 mm. If this is correct, we would propose that we dowel the new central bridge structure to the sides of the existing structures.

May we please have your comments on the above.

If you have any questions of the enclosed, please contact us.

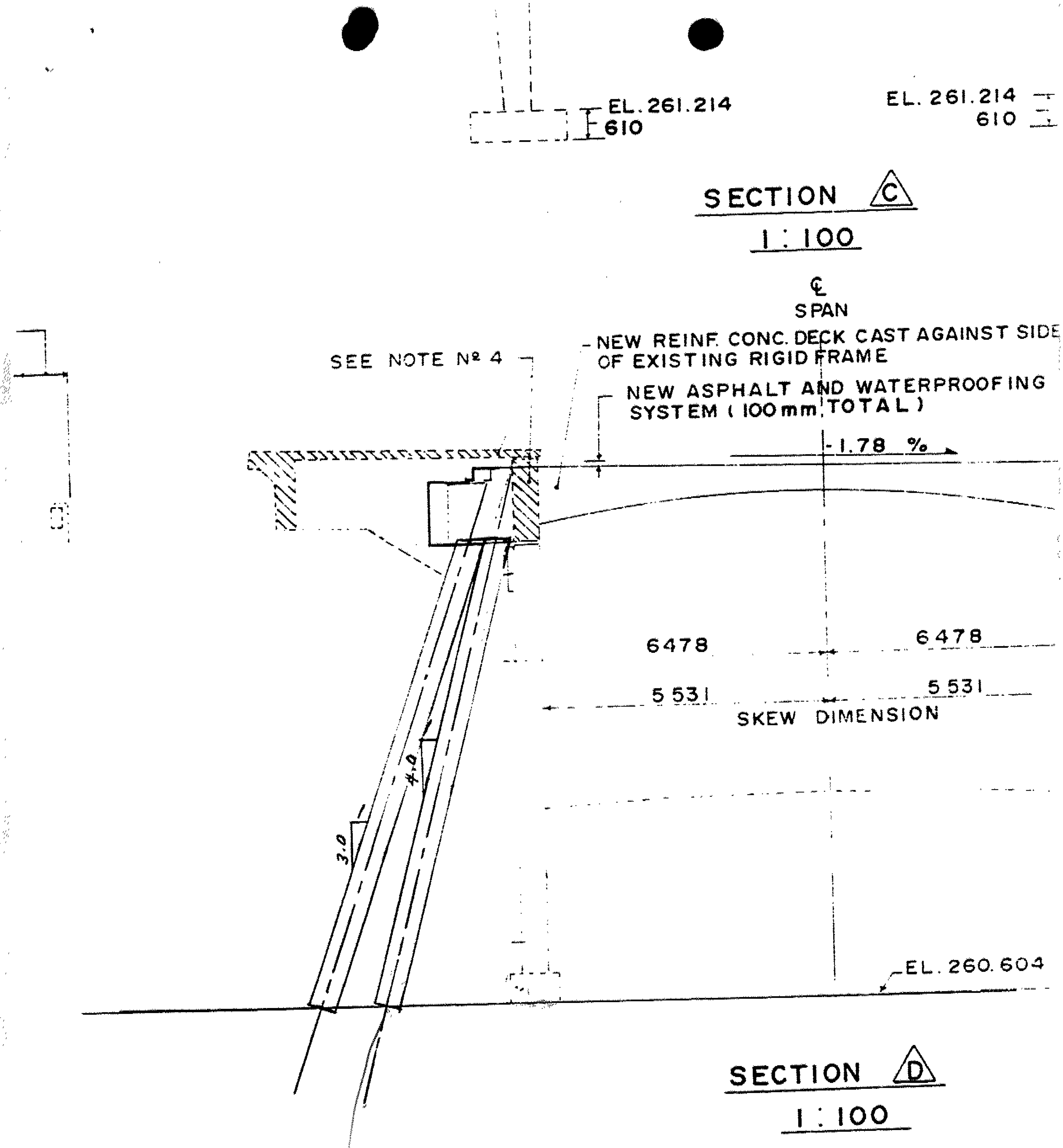
Yours very truly,

B. M. ROSS AND ASSOCIATES LIMITED

Per ______________________
K. G. Dunn, P. Eng.

KGD:kf
Encl.

cc Ken Mossop, M.T.O. London
Paul Payer, M.T.O. Downsview

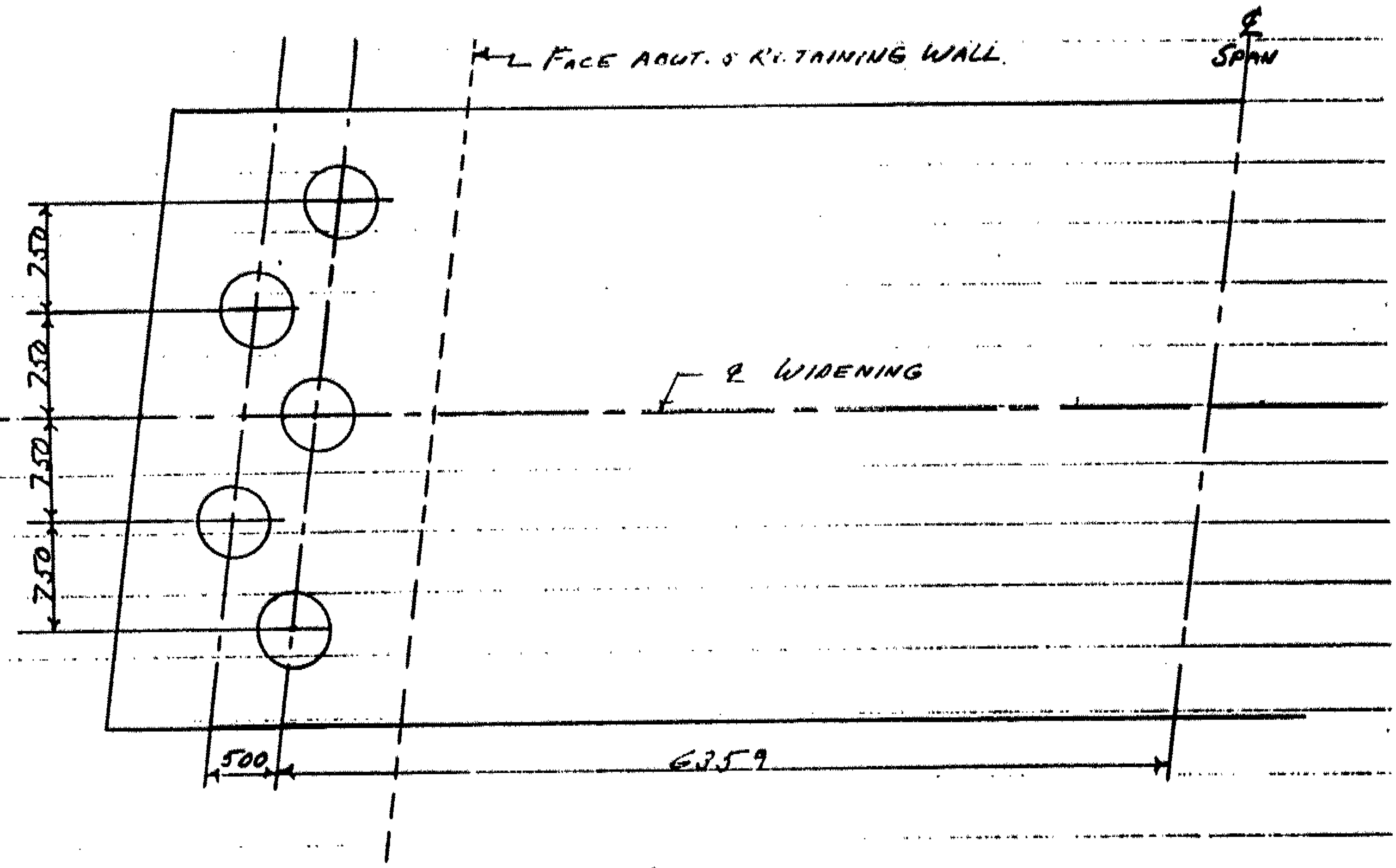


CAMPBELLVILLE ROAD OVERPASS

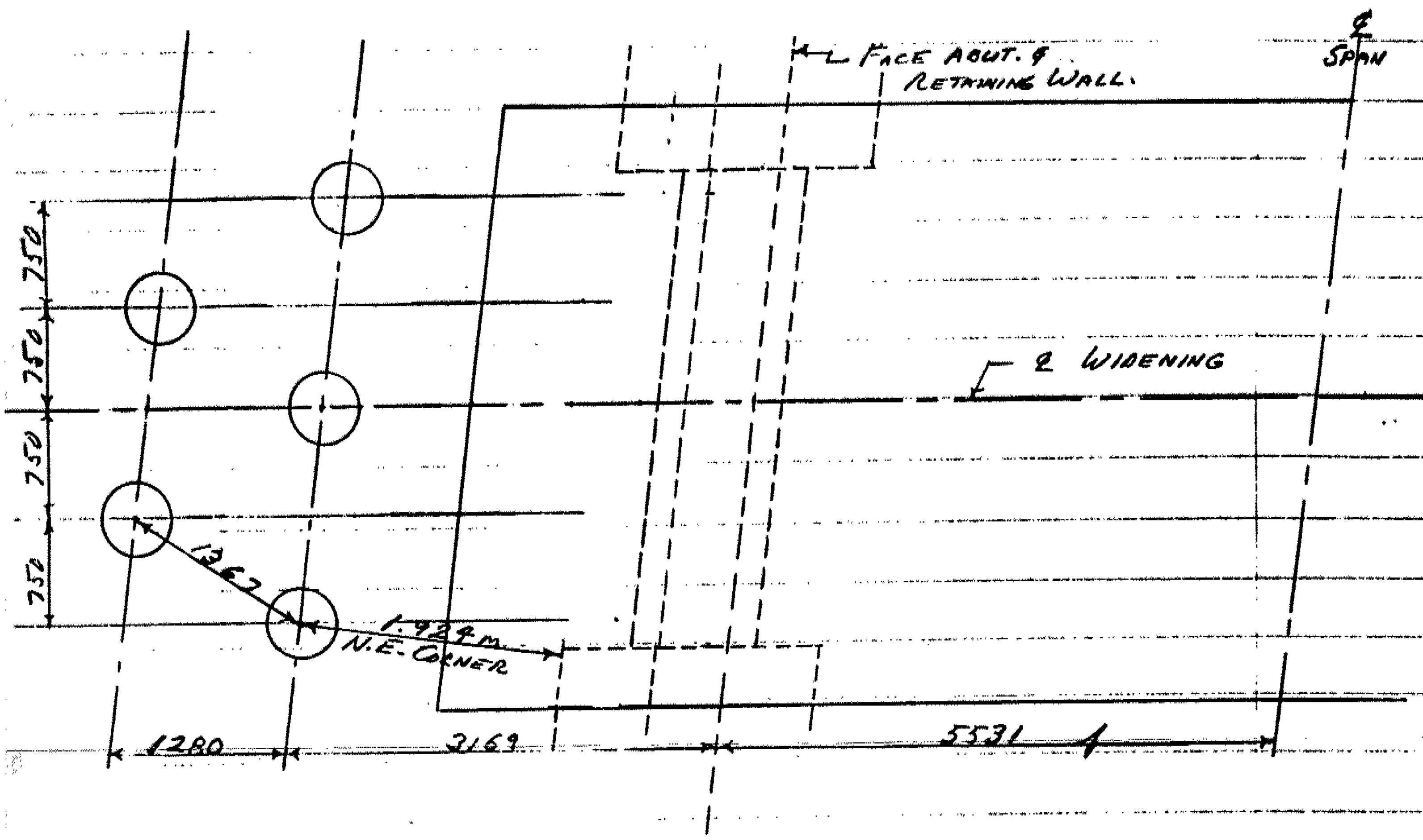
DOUBLE ROW 508 mm ø CAISSONS

JUNE 18/91

Nr 562 Campbellville Rd. Overpass - Hwy 401 June 18/91



Plan At Top of Caissons



Plan At Bottom of Caissons.

Notes.

- Front Row On 1:4 Batter & 3 - 508 mm O.D. Caissons. Distance In Row 1512 mm ctr. or 1.98 x Dia. Clear.
- Second Row On 1:3 Batter & 2 - 508 mm O.D. Caissons. Closest Distance To Front Row Caisson 1367 mm ctr. or 1.69 x Dia. Clear.

memorandum

Ontario

Tel: (416) 235-5654

To: A. Ho
Head, Structural Section
Southwestern Region, London

Date: 91 06 18

Attn: A. Ma

Re: Campbellville Road O'pass
W.P. 95/96-90-01, Site 10-44/R1

The review of structure drawings shows the footings for the existing structure to be approximately 3.5m below existing ground level. There appears to be no apparent reason to justify this provision in the design. The foundation design recommendations are based on the assumption that the structure was built as shown on drawing and can lead to serious consequences, if the footings for some reason did not extend to those elevations, due to disturbance of the very sensitive soil conditions. It is, therefore, strongly recommended that the elevation of the existing footing be located in the field.

I. Husain
Design Engineer
Structural Office

IH/sl

c.c. M. Devata
K. Bassi

RECEIVED
JUN 19 1991
FOUNDATION DESIGN SECTION

P. P

7540-1478 (Rev. 10/89)

memorandum

Ontario

To: A. Ho
Head, Structural Section
Southwestern Region

Date: 1991 06 18

Atten: A. Ma

From: Foundation Design Office
Room 315, Central Bldg.

Re: Elevation of Existing Footings
Hwy. #401 and Campbellville Road
W.P. 95-90-01/96-90-01
District #4 (Burlington)

As indicated, the base of the above structure footings is located some 4 m below Campbelville Road. profile grade.

It is our opinion, that the footing elevations should be determined by the Southwestern Region. The foundation recommendations can not be finalized until this information is available. This problem was discussed with Mr. K.G. Bassi and with Mr. I. Hussain of the Structural Office.

P. Payer

P. Payer, P. Eng.
Sr. Foundation Engineer

for

M. Devata, P.Eng.
Chief Foundation

PP/mmj

c.c. - I. Hussain

B. M. ROSS AND ASSOCIATES LIMITED

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B. W. POTTER, P.ENG.
R. R. ANDERSON, P.ENG.

COPY

RECEIVED
JUN 26 1991
FOUNDATION DESIGN SECTION

OUR FILE NO.
BR-567

June 14, 1991

FAXED

Allan Ma
Ministry of Transportation
659 Exeter Road, P.O. Box 5338
London, Ontario
N6A 5H2

Dear Sir:

RE: Campbellville Road Overpass
Highway #401 - WP 95/96-90-01

As discussed in our recent telephone conversation, we understand that your M.T.O. Foundation Office would like our proposed end bearing concrete filled steel pipe caissons at a minimum clear distance of 1.5 times the diameter of the caisson. This is to be measured between the faces of the outside of the pipe. This would mean that our four 762 mm diameter caissons per support unit, as proposed in our general arrangement drawings, would not meet this requirement.

If we are to use concrete filled end bearing caissons, the only possible alternative would be to use three 610 mm diameter caissons per support unit. We would estimate the dead load and live load SLS per caisson at approximately 325 kN. We note in the soil investigation report that the SLS capacity is given as 300 kN for a 600 mm diameter. We also note where the settlement for the same 300 kN loading is estimated at 18-24 mm. This becomes a very difficult judgement call on our part. We would prefer to see the new central bridge section dowelled to the existing bridges, which provides a much more conservative approach with extra capacity to resist the horizontal forces and also the prospect of additional live load sharing capabilities.

Assuming that the 325 kN capacity is suitable with the estimated differential settlement dowelling of the structure could lead to future cracking. On the other hand, if we do not dowel the new central deck into the existing decks, there could be considerable future maintenance of the pavement along the joint line.

Because of the various complex and difficult conditions at the Campbellville Overpass site, we would like your Foundations section to reconsider the use of driven bearing piles inside the augered caisson unit on the 1:3 batter. We would propose the use of four 508 mm outside diameter tube caissons augered to the bottom of the footing level with 310 x 110 bearing piles driven inside the caisson units to suitable low capacity using the Hiley pile driving formula.

With the height of the existing abutment leg, and the 1:3 pile batter, this allows us to be at a greater distance from the bottom of the existing footing. We would be prepared to increase the half span length by 465 mm in order to have the steel "H" piling a minimum 3.0 metres clear of the closest outside corner of the existing footing projection. We would also propose that specializing rock points be installed on the piling to reduce the amount of disturbance, and that the piling be driven with a specified lighter energy level hammer to match the relatively low dead load and live load requirement for the support.

We are enclosing a revised sketch showing the combination steel tube casing piling installation and the increased half span to 6 815 mm to allow for the 3.0 metre minimum clearance. With this proposal, we are assuming that settlement would be very low and that the new central structure would be dowelled into the existing structures. We estimate the total dead load and live load forces per support end at ULS 1425 and SLS 1033 kN. This would be shared amongst the 4 units per support end.

May we please have your comments on the above.

If you have any questions on the enclosed, please contact us.

Yours very truly,

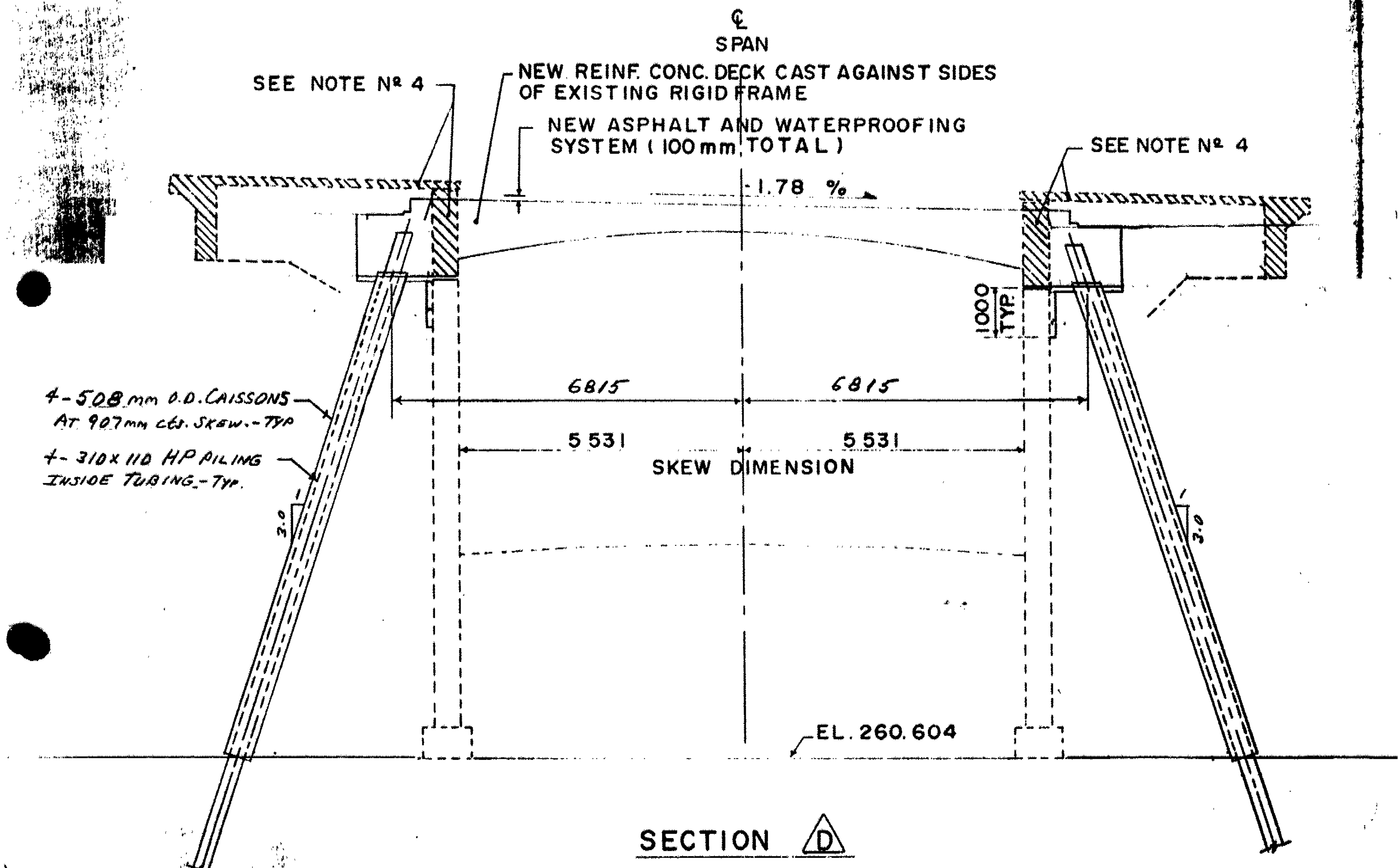
B. M. ROSS AND ASSOCIATES LIMITED

Per [signature]
K. G. Dunn, P. Eng.

KGD:bf
Encl.

c.c. Ken Mossop -- M.T.O., London
Paul Payer -- M.T.O., Downsview

1 : 100



SECTION D

1 : 100

Hwy 401 - Campbellville
Piling Proposal - June 14/91

B. M. ROSS AND ASSOCIATES LIMITED

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S. D. BURNS, P.ENG.
B. W. POTTER, P.ENG.
R. R. ANDERSON, P.ENG.

OUR FILE NO.

BR-567

May 8, 1991

Mr. Alfred S. Ho
Ministry of Transportation
659 Exeter Road
P.O. Box 5338
London, Ontario
N6A 5H2

ATTENTION: Ken Mossop

Dear Sir:

RE: Campbellville Road Overpass
WP 95/96-90-01

Confirming our recent telephone conversations, we find we have very serious design problems with our caisson/piling design alternative for the central bridge of your Campbellville Road Overpass structure.

Strata Engineering Corporation soils information indicated that steel "H" piling was not recommended at this site. In the same April 19th faxgram, they also indicated that the serviceability limit state capacity of a 508 mm dia. caisson augered to the bottom of the footing and filled with concrete, was 240 kN, with an estimated settlement of 20 mm. As outlined in our letter of April 22, 1991, this serviceability limit state capacity of the 508 mm dia. caissons is below the 300 kN/unit design requirements, based on four support units at each end of the structure. Strata subsequently provided recommendations for the capacity of 610 and 762 mm dia. size caissons, using a serviceability limit state of 320 kN and a settlement of 25 mm for the 610 mm caisson and 500 kN and 25 mm settlement for the 762 mm dia. caisson. The same 762 mm dia. caisson loaded at 300 kN would have an anticipated settlement of 17 mm.

In investigating the various alternative foundation solutions to solve the problems at this site, we considered using five 762 mm dia. caisson units at each end of the structure. With the additional superimposed dead load of the larger caisson unit, the loading per caisson is 312 kN SLS. This is based on using the centre to centre span for the 508 mm dia. caissons and the originally proposed dead load of 75 mm of concrete overlay and 90 mm of asphalt and waterproofing. With the additional centre to centre support to match the larger 762 mm caissons and the additional asphalt loading required to match Planning and Design's proposed road grades, the loading per unit would be in excess of this 312 kN and we anticipate that settlement would be in excess of 20 mm. In addition, the spacing of five 762 mm dia. units is very close with only 145 mm between the caissons. We are very concerned with the construction of a central deck section supported on the augered caissons with settlements in excess of 20 mm and dowelling of the sides of the new deck with the existing structure. This could create large shearing forces on the dowelling and could crack the concrete.

A second alternative would be to use the augered caisson units and not dowel the new deck into the existing deck soffits. With this alternative, we would be concerned with possible settlements in excess of 20 mm located longitudinally in the asphalt median shoulder. This would likely lead to cracking of the pavement and high future maintenance costs. In addition, our original concept was based on driven steel "H" piling and we have concerns of the ability of augered caissons to handle the longitudinal forces in the deck in accordance with the OHBDC. We do not consider that this is a problem in the case of the Halton Road No. 8 Overpass and the Oakville Creek bridge, with the new deck being dowelled to the sides of the existing decks.

For this Campbellville site, it may well be that we are forced to revert to a third alternative scheme which would be your originally proposed construction of a complete rigid frame deck and abutments supported on spread footings founded at the same elevation as the existing structures. As you are aware, this will involve a very extensive braced steel sheet piling roadway protection to permit the excavation adjacent to the Highway 401 traffic. It will be necessary to drive this sheeting right up to the original footings and the back of the abutment. The leg height of the existing structure from the top of the deck to the bottom of the footing is some 10.9 metres. It will also be necessary to provide roadway protection for Campbellville Road at the front of the abutment where the depth to the bottom of the footing is some 4.2 metres. In addition, once both of these enclosures are properly designed, there is also the serious dewatering problems to solve and the very great possibility of disturbance of the sandy foundation material under the adjacent existing footings.

We would ask that you reconsider Strata Engineering's decision not to allow steel "H" piles driven inside the preaugered caissons on our proposed 1 in 3 batter where the closest centreline of pile is 2.69 metres from the corner of the footing projection beyond the end of the existing abutment. The loadings on these pilings is very low and if we were to use specialized driving points, the vibration and disturbance could be kept to a minimum. In our opinion, the "H" piling at 2.69 metres distance would be a lot less disturbance than the driving of steel sheet piling enclosures up to the edge of the footing and the back of the abutment in addition to the disturbance possible with dewatering of the footing excavation.

If you wish, we would be willing to have a design meeting in Toronto to try to resolve this problem.

Yours very truly,

B. M. ROSS AND ASSOCIATES LIMITED

Per [signature]
K. G. Dunn, P. Eng.

KGD:bf

B. M. ROSS AND ASSOCIATES LIMITED

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OUR FILE NO.

BR-567

April 10, 1991

Mr. A. S. Ho
Head, Structural Section
Ministry of Transportation
659 Exeter Road
P.O. Box 5338
London, Ontario
N6A 5H2

Attention: Ken Mossop, P. Eng.

Dear Sir:

Re - Highway 401 - Campbellville Road and Halton Road No. 8 Overpasses and Oakville Creek Bridge

We received your faxed copy of the Strata Engineering Corporation summary of caisson capacities for the above Highway 401 bridges. You wished our comments on this information.

For all three structure sites, our preliminary plans show the use of 508 mm diameter caisson tubes; whereas, we notice the soils consultant has used 0.6 metre as their smallest size.

In the case of the Regional Road No. 8 structure, we notice the allowable bearing capacity of the caisson as given at Elev. 256.2. While it is a small difference, the elevation of the bottom of the footing is 256.032.

For the Campbellville Road Overpass, the bottom of the existing footings is at Elev. 260.604. In our preliminary plan design, we intended to augur the .508 metre diameter tube casings to the bottom of the footing elevation and then drive steel "H" piling to a suitable tip elevation. The steel "H" piling was being used because of the higher water table.

Mr. A. S. Ho

At the Oakville Creek Bridge site, the bottom of the east footing is at Elev. 259.08 and the bottom of the west footing is at Elev. 259.38. We intended to pre-augur the .508 metre diameter steel tube casing to the bottom of footing elevation and then drive steel "H" piling to a suitable tip elevation. Again, similar to the Campbellville Road Overpass, the support of the central structure would be on the steel "H" piling.

We will also be relying on foundation section to provide estimates of settlement under the loadings as previously provided by us at the time of submission of the preliminary plans. As was mentioned, we consider the loadings to be spread evenly amongst the four units at each support point. We also pointed out that we had not included the deadload of the support unit since we do not know the tip elevations.

We trust the above provides further background information.

Should there be any questions, please contact us.

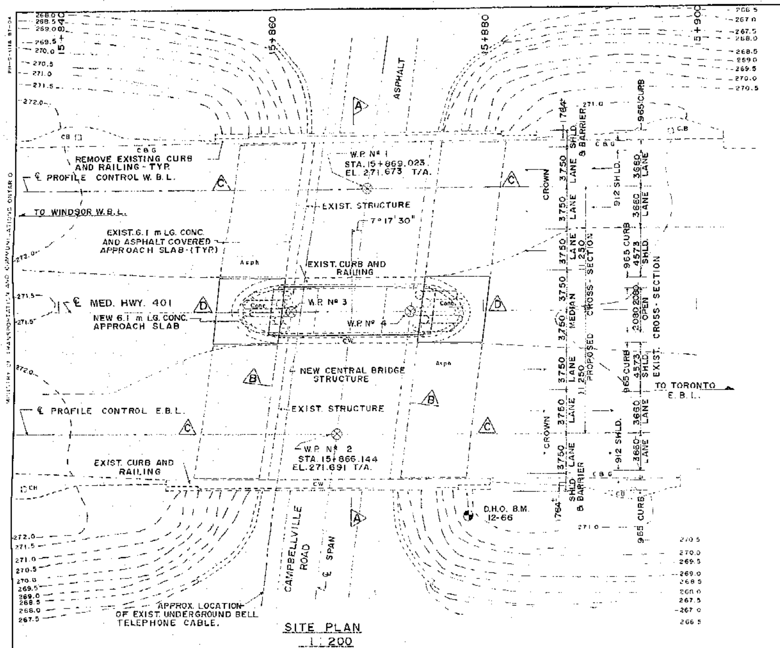
Yours very truly,

B. M. ROSS AND ASSOCIATES LIMITED

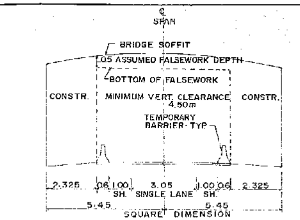
Per

K. G. Dunn, P. Eng.

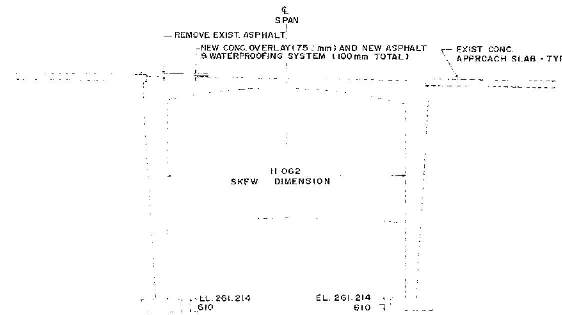
KGD:jj



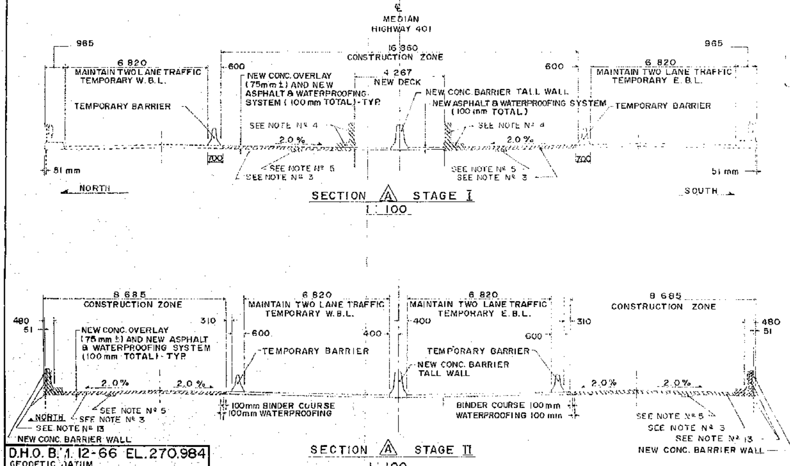
SITE PLAN
1 : 200



SECTION (B) STAGE I & II
1 : 100
CAMPBELLVILLE ROAD
TRAFFIC STAGING DURING CONSTRUCTION



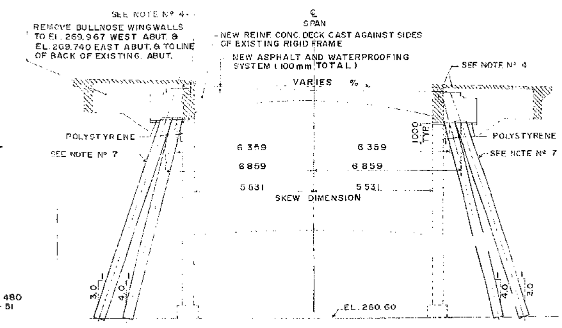
SECTION (C)
1 : 100



SECTION (A) STAGE I
1 : 100

SECTION (A) STAGE II
1 : 100

HIGHWAY 401 TRAFFIC STAGING DURING CONSTRUCTION



SECTION (D)
1 : 100

D.H.O. B.M. 12-66 EL. 270.984
GEODETIC DATUM
Tablet in S Face of E Abut., 2.1 m W of S.E. Cor. Conc. Bridge 16.8 m Rt. 15+878.6
ROUTE 14 CAMPBELLVILLE

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

NOTE: MINISTRY DENOTES MINISTRY OF TRANSPORTATION, ONTARIO

DIST. Nº 4 - HWY. Nº 401

CONT No 95-90-01 (E.B.L.)
WP No 96-90-01 (W.B.L.)

CAMPBELLVILLE ROAD OVERPASS
WIDENING & REHABILITATION
6.8 km WEST OF HWY. Nº 25
GENERAL ARRANGEMENT

SHEET

S. M. ROSS AND ASSOCIATES LIMITED
Consulting Civil Engineers
GODERICH ONTARIO
8R-567

GENERAL NOTES:

CLASS OF CONCRETE

. ALL CONCRETE — 30 MPa.

CLEAR COVER TO REINFORCING STEEL

. HAUNCH (BOTTOM AND BACK) — 80± 20mm
. DECK (TOP) — 70± 20mm
. DECK (BOTTOM) — 50± 10mm
. REMAINDER — 70± 20mm UNLESS OTHERWISE NOTED

REINFORCING STEEL

. REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BARS MARKED WITH SUFFIX 'C' DENOTE COATED BARS.

CONSTRUCTION NOTES

. THE CONTRACTOR SHALL CHECK AND ENSURE THAT ALL RELEVENT EXISTING STRUCTURE DIMENSIONS AND ELEVATIONS SHOWN ON THE DRAWINGS ARE CORRECT AND ADJUST THEM AS REQUIRED TO MATCH THOSE OF THE EXISTING STRUCTURE AND APPROVED BY THE ENGINEER.
. IT IS NOT NECESSARY TO REMOVE THE FALSEWORK SUPPORTING THE DECK PRIOR TO CONSTRUCTING THE MEDIAN BARRIER TALL WALL.
. AREAS OF REMOVAL SHALL BE DELINEATED BY A SAWCUT 25mm DEEP OR TO FIRST LAYER OF REINFORCING STEEL, WHICHEVER IS LESS.

CONSTRUCTION STAGING.

1. CONTRACTOR TO SUPPLY, ERECT, AND REMOVE TEMPORARY TRAFFIC SIGNALS FOR CAMPBELLVILLE ROAD.
2. CLOSE OFF TRAFFIC FROM STAGE I CONSTRUCTION AREA AND PLACE TEMPORARY CONCRETE BARRIERS FOR STAGE I CONSTRUCTION.
3. REMOVE EXISTING ASPHALT AND WATERPROOFING FROM CLOSED PORTION OF BRIDGE INCLUDING 300mm ONTO ABUTMENT END AND BOTH SIDES OF APPROACH SLABS.
4. REMOVE EXISTING CONCRETE CURB AND RAIL SYSTEM AND PORTIONS OF WINGWALLS AND RETAINING WALLS FROM CLOSED PORTION OF BRIDGE.
5. REMOVE CONCRETE FROM AREAS OF BRIDGE DECK, AND DECK FACIA REQUIRING RESTORATION. AREAS TO BE IDENTIFIED BY MINISTRY. SCARIFY EXISTING DECK SURFACE 6–10mm DEPTH.
6. REPAIR THE DECK SOFFIT AND OTHER AREAS OF THE SUBSTRUCTURE NEEDING RESTORATION AS IDENTIFIED BY THE MINISTRY.
7. CONSTRUCT PRE-AUGERED 508 mm O.D. STEEL TUBE CAISSONS TO EL. 260.60. AUGERING TO NOT PRECEDE TUBING BY MORE THAN 150mm DEPTH. FIVE CAISSONS REQUIRED AT EACH SUPPORT END. STEEL TUBE CAISSON TO BE FILLED WITH TREMIE CONCRETE BELOW EL. 261.50 AND 30 MPa. CONCRETE ABOVE EL. 261.50
8. CONSTRUCT CENTRAL BRIDGE DECK CAST AGAINST SIDES OF EXISTING RIGID FRAME.
9. CONSTRUCT NEW 30 MPa. CONCRETE DECK OVERLAY.
10. CONSTRUCT NEW MEDIAN CONCRETE BARRIER TALL WALL.
11. WATERPROOF AND PAVE AS SHOWN ON DWG. STAGE I.
12. RELOCATE TRAFFIC TO COMPLETED PORTION OF THE BRIDGE AND RELOCATE TEMPORARY CONCRETE BARRIERS FOR STAGE II CONSTRUCTION.
13. REMOVE EXISTING CONCRETE CURB AND RAIL SYSTEM AND TOP OF WINGWALLS FROM CLOSED PORTION OF BRIDGE.
14. REPEAT STEPS 3, 5 AND 9.
15. CONSTRUCT NEW CONCRETE BARRIER WALL.
16. WATERPROOF AND PAVE AS SHOWN ON DWG. STAGE II.
17. REMOVE TEMPORARY CONCRETE BARRIERS AND OPEN TOTAL BRIDGE TO TRAFFIC.

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATION AND SOIL STRATA
3. PLAN AND FINISHED GRADES
4. DECK WIDENING
5. BARRIER WALL
6. 6100mm APPROACH SLAB
7. AS CONSTRUCTED ELEV. & DIM.
8. STANDARD DETAILS

APPLICABLE STANDARD DRAWINGS

LICENSED PROFESSIONAL ENGINEER — A. J. ROSS — PROVINCE OF ONTARIO
REGISTERED PROFESSIONAL ENGINEER — CIVIL — K. G. DUNN — PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN K.G.D.	CHK A.J.R.	CODE O.H.B.D.C.-83	LOAD CLASS A	DATE 91-03
DRAWN J.O.	CHK	SITE 10-44/R1	STRUCT	SCHEME	DWG 1