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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

FOUNDATION INVESTIGATION REPORT

CONTRACT NO. 92-95

Ministry of Transportation

Ontario

INDEX

Note: For purposes of the contract, this report supersedes all other Foundation Reports prepared by, or for the Ministry in connection with the above mentioned project.

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/0.3 m)	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

FOUNDATION INVESTIGATION REPORT
For
Proposed Extension of
Kirby Side Road Overpass
W.P. 95-85-01, Site No. 37-95R/1
Hwy. 400, District 6, Toronto

INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the aforementioned site. It is proposed to extend the existing rigid frame structure on the east side only by approximately 3.75 m to facilitate widening of Hwy. 400. The widening is planned in conjunction with proposed rehabilitation work of the existing superstructure. Extension of the existing structure abutments and the replacement of existing retaining walls and wing walls will be required in connection with the proposed widening.

SITE DESCRIPTION AND GEOLOGY

The site is located at the existing Hwy. 400/Kirby Side Road crossing in the Township of Vaughan approximately 4 km north of Major MacKenzie Road. Kirby Road is a two lane roadway that is alternately paved and unpaved along its length on either side of the existing structure. The existing Hwy. 400 overpass is a six (6) lane highway.

The land surrounding the site consists of a gently undulating to rolling terrain. Sloping surface embankments covered with vegetation and grassland are present on either side of the roadway. The area is also populated with forest and low lying shrubs. In general, the land serves as agricultural farmland.

Physiographically, the site is situated in the region known as the "Oak Ridges". The Oak Ridges region is basically covered by an interlobate moraine of the Wisconsinan glacial age and at the site consists of a kame topography that is characterized by a hill terrain with a knob-and-basin relief. The hills are primarily composed of irregularly mixed sandy or gravelly materials, but in some areas, extensive thickness of boulder clay protrude above the outwash. Bedrock at the site consists of shale of the Dundas-Meaford formation.

FIELD INVESTIGATION

The fieldwork for the investigation was carried out between 89 08 26 and 89 08 27 and consisted of three (3) sampled boreholes advanced to depths ranging from 6.9 m to 15.7 m accompanied by two (2) dynamic cone penetration tests advanced to depths ranging from 4.9 m to 5.2 m. Hollow stem auger equipment was used to advance the boreholes in the overburden. In general, subsoil samples were retrieved at 0.7 m intervals for the surficial 6.1 m and at 1.5 m intervals for the remainder of the borehole. All samples were retrieved using a split spoon sampler in accordance with the Standard Penetration Test (ASTM D1586). In situ vane tests were carried out in the surficial cohesive soil (generally at 1.5 m intervals), to determine the undrained shear strength at both the undisturbed and remoulded state. The test was conducted in accordance with ASTM D2573, using the standard MTO 'N' vane. All samples were identified in the field and then returned to the laboratory for applicable testing.

Groundwater levels were obtained in the open boreholes and monitored throughout the duration of the investigation. The boreholes were backfilled at the completion of the investigation. Survey information related to the location and elevation of boreholes was provided by Central Region Surveys and Plans.

LABORATORY ANALYSES

To identify the behaviour, gradation and property of the soil, the following laboratory tests were conducted:

1) Atterberg Limits
2) Grain Size Distributions
3) Natural Moisture Contents
4) Bulk Densities

Laboratory test results have been summarized in subsequent sections of the report and are illustrated graphically in the figures and boreholes attached in the Appendix of this report.

SUBSURFACE CONDITIONS

Subsoil conditions are generally uniform across the site. The native surficial deposit consists of a cohesive clayey silt mixed with some sand and a trace of gravel. The maximum thickness of the deposit encountered in the investigation is 7.8 m (Elevation 243.9 m). The surficial deposit is underlain by a stratum of a plastic silt that also contains some sand and a trace of gravel. In the investigation, this stratum was penetrated to a máximum thickness of 8.1 metres (Elevation 236.2 m). The extent of this deposit was not explored in the investigation.

Fill material consisting of a cohesionless sand with a trace of gravel that serves as a base and sub-base for the existing Kirby Side Road was encountered at BH 2. The fill material was 1.8 m in thickness.

The boundaries between the various soil types, in situ and laboratory test results as well as groundwater levels established at the time of investigation, are shown on the attached Record of Borehole sheets in the Appendix. A plan of the site illustrating the locations and elevations of the boreholes and subsoil stratigraphical sections are provided on Dwg. 958501-A.*

A detailed description of the subsurface conditions encountered is given below.

Sand, trace gravel (Fill Material)

Fill material consisting of a brown sand with a trace of gravel was encountered at BH 2. The surficial fill serves as a base for the existing Kirby Side Road and extends to a thickness of 1.8 metres below ground surface. The 'N' values derived from the Standard Penetration Test varied between 5 and 6 blows/0.3 m.

* SHEET NO 113 OF THE CONTRACT DWG'S

Clayey Silt, some sand, trace gravel (Glacial Till)

Underlying the fill material where it is present and immediately below the natural ground surface elsewhere at the site, exists a deposit of glacial origin consisting of a clayey silt with some sand and a trace of gravel. Occasional seams of sand up to 50 mm in thickness and random zones of plastic silt are also present interbedded throughout the deposit. The maximum thickness of this deposit encountered during the investigation was found to be 7.8 m.

A grain size distribution envelope for the deposit as determined by mechanical sieve and hydrometer analyses in the laboratory is illustrated in Figure 1 in the Appendix. The distribution of the material illustrates significant percentages of silt (39-69%) and clay particle percentages generally ranging from 9-17.5%.

Atterberg Limits were obtained to evaluate the plasticity and behaviour of the fine grained portion of the soil and the results are plotted in Figure 2 and summarized in Table 1 below:

Table 1 - Physical Properties

	Range	# of Tests
Natural Moisture Content (w%)	9.5-13.5	6
Liquid Limit (w_L%)	16.5-23.5	6
Plasticity Index (I_P%)	5-8	6
Unit Weight (kN/m^3)	21.9-23.7	6
Undrained Shear Strength (kPa)		
- Field Vane	40->120	4
Sensitivity	3	4

The test results reveal that the deposit consists of an inorganic, low plasticity cohesive soil with random zones of plastic silt. The natural moisture content generally exceeds the plastic limit of the soil but is significantly lower (approximately 7 to 13.5%) than the liquid limit of the soil.

The consistency of the stratum was derived from the results of the Standard Penetration Test and field vane tests conducted. The 'N' values obtained from the Standard Penetration Test range from 8 blows/ 0.3 m to 50 blows/0.3 m. This indicates a range of consistency of firm to hard. In general, however, 'N' values are in the 12 blows/0.3 m to 25 blows/0.3 m range and hence, the soil can be categorized as stiff to very stiff.

Undrained shear strength measurements (c_u) were obtained in situ within the top 3 m of the deposit by conducting field vane tests and the results are plotted in the Record of Boreholes in the Appendix and summarized in Table 1 above. The undrained shear strength values determined ranged from 40 to >120 kPa, confirm that the soil, although ranging from firm to hard, generally has a stiff to very stiff consistency.

The sensitivity of the soil as defined by the ratio of the undrained strength in the undisturbed state to the undrained strength, at the same water content, in the remoulded state, was determined by the in situ vane test to have a value of 3 or equivalently, a low sensitivity.

<u>SILT</u>

Underlying the clayey silt deposit and penetrated to a maximum thickness of 8.1 m (depth equivalent to 236.2 m), exists a stratum consisting of a plastic silt with some sand and a trace of gravel. The upper boundary of the stratum was found at an elevation ranging from 243.9 m to 244.3 m or depths equivalent to 7.8-7.6 m below the ground surface. The extent of this stratum was not determined in the investigation.

Observation of the 'N' values obtained from the Standard Penetration Test reveal values ranging from 4 blows/0.3 m to 55 blows/0.3 m. The lower values, however, are not necessarily indicative of the denseness of the soil because the values may have been disturbed as a result of unbalanced hydrostatic head present during the sampling process. In fact, in some cases during the sampling in this deposit, up to 2 metres of the silt material sloughed into the open borehole. Consequently, in consideration of the induced disturbance, the denseness of the soil can be categorized as compact to very dense.

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water level in the open boreholes. These observations are recorded on the Record of Borehole sheets as well as on Drawing 958501-A*. Measurements obtained at the time of investigation revealed levels at an elevation ranging from 251.8 m to 251.4 m corresponding to depths below ground surface of approximately 6 metres at BH 3 (top of approach embankment) and 0.3 metres at the abutment extension locations. Groundwater levels, however, are subject to seasonal fluctuations and hence can vary from the values given in this report.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer and A. Lako, Student Engineer, utilizing equipment owned and operated by Marathon Drilling.

The project was carried out by T. Sangiuliano under the general supervision of Dr. B. Iyer, Senior Foundation Engineer. The report was written by T. Sangiuliano, reviewed by Dr. B. Iyer and approved by Mr. M.S. Devata, Chief Foundation Engineer.

* SHEET NO 113 OF THE CONTRACT DWG'S

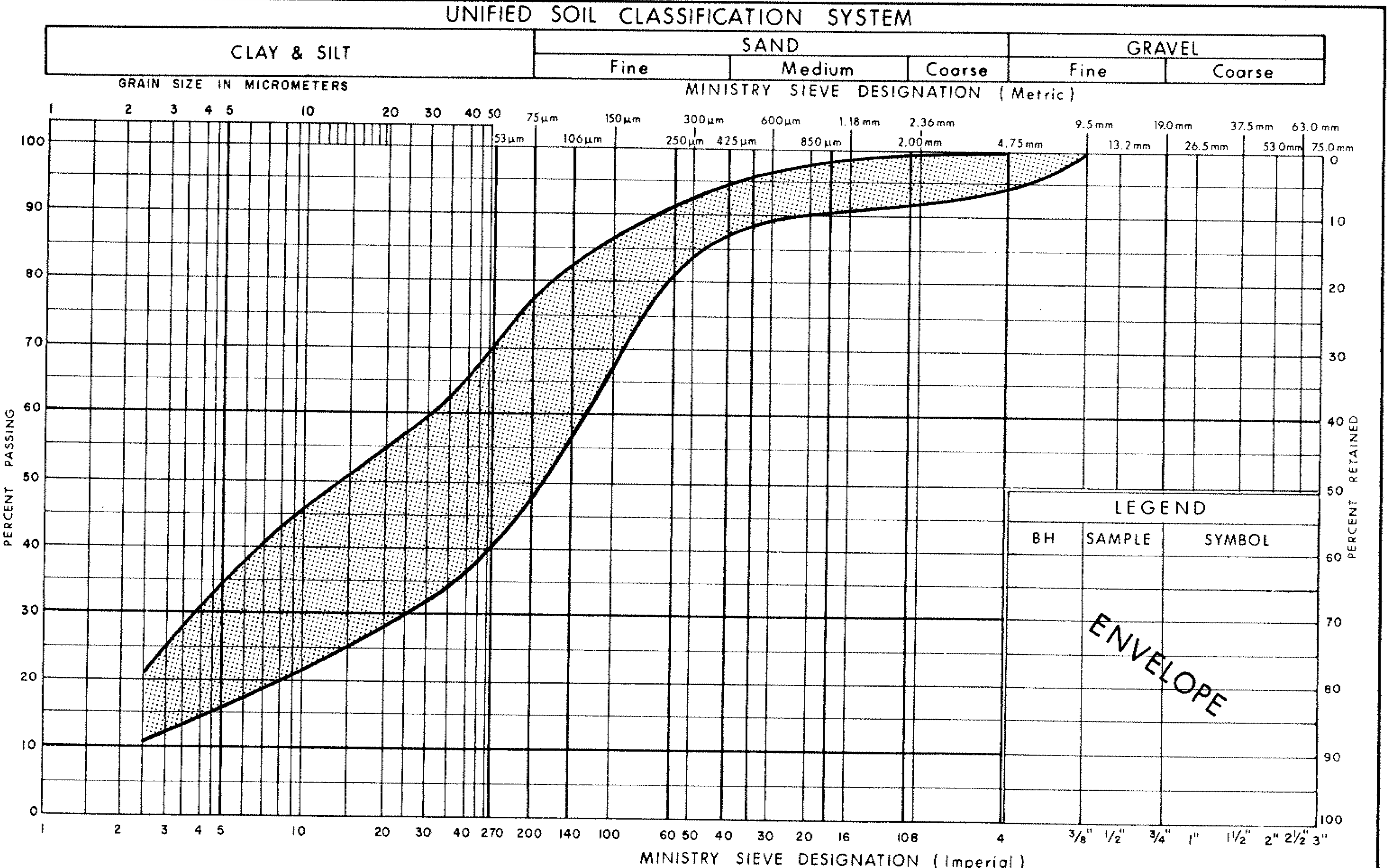
LICENSED PROFESSIONAL ENGINEER
B. IYER
PROVINCE OF ONTARIO

B. Iyer, P. Eng.
Sr. Foundation Engineer

M. Devata, P. Eng.
Chief Foundation Engineer

APPENDIX

78 12 M



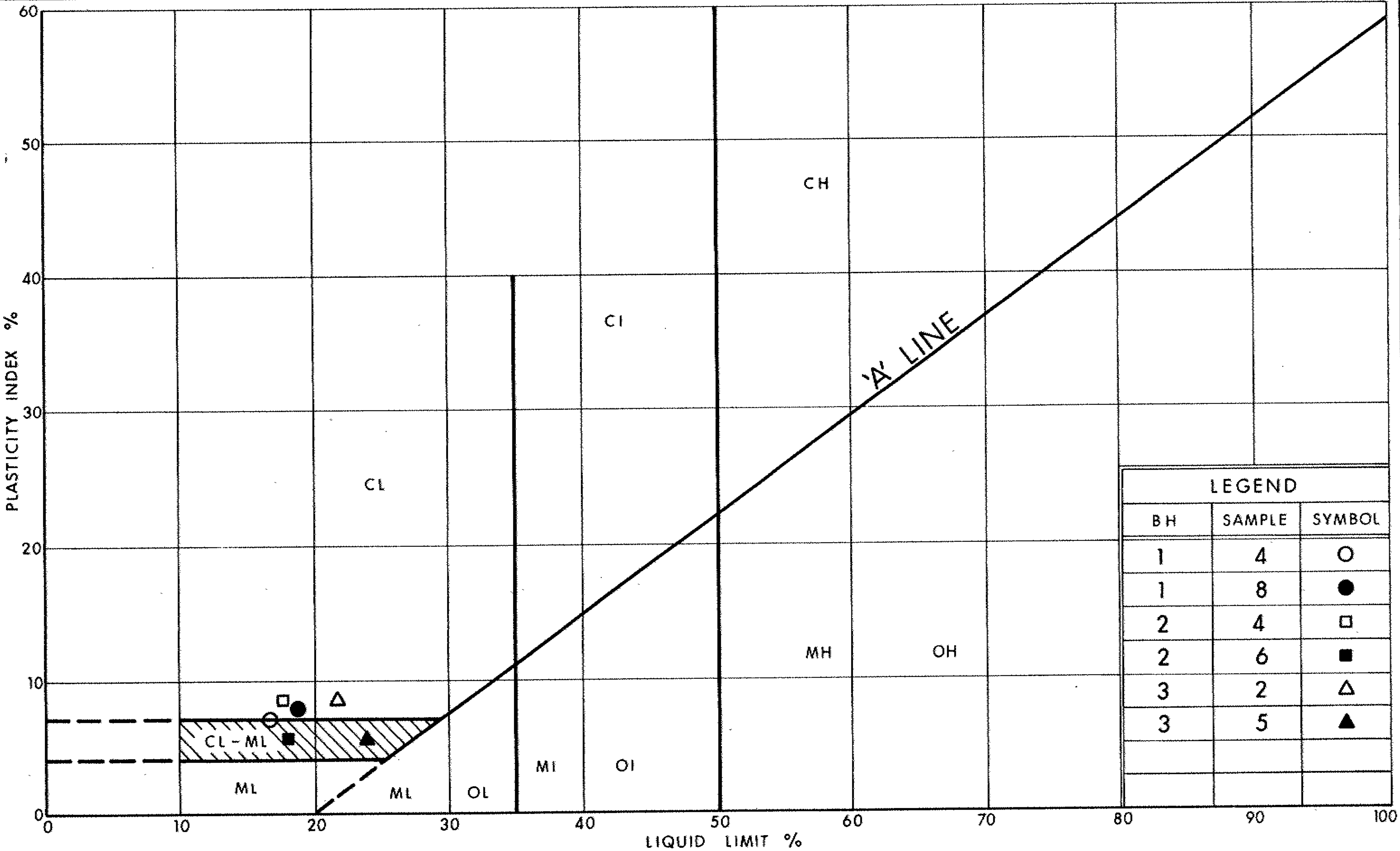
Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
CLAYEY SILT, SOME SAND, TRACE OF GRAVEL
(Glacial Till)

FIG No 1

W P 95-85-01

Oct 75, FF-S-21



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PLASTICITY CHART
CLAYEY SILT, SOME SAND, TRACE OF GRAVEL
(Glacial Till)

FIG No 2
WP 95-85-01

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 1

METRIC

W P 95-85-01 LOCATION Co-ords: N 4 860 219.7; E 300 191.5 ORIGINATED BY AL

DIST 6 HWY 400 BOREHOLE TYPE H.S. Auger & Cone Test COMPILED BY AL

DATUM Geodetic DATE 89 08 27 CHECKED BY TS

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
251.7 / 0.0	Ground Surface				
	Clayey Silt, Some Sand, Trace Gravel (Glacial Till), Grey, Stiff - V. Stiff	1	SS	9	
		2	SS	14	
		3	SS	12	
		4	SS	12	8 29 48 15
		5	SS	12	
		6	SS	25	
		7	SS	24	
		8	SS	24	2 26 69 3
243.9 / 7.8	Silt, Some Sand, Trace Gravel, Grey, Compat to Very Dense	9	SS	7	
		10	SS	15	
		11	SS	20	
239.1 / 12.6		12	SS	55	0 15 76 9
	End of Borehole				

Ground water level at ground surface. Elevation scale: 250, 248, 246, 244, 242, 240.

Dynamic Cone Penetration Resistance Plot (20 40 60 80 100); Shear Strength kPa: ○ Unconfined, + Field Vane, ● Quick Triaxial, × Lab Vane (20 40 60 80 100). Field vane readings with sensitivity 3 near elev. 249.8 and 249.

Plastic Limit W_p, Natural Moisture Content W, Liquid Limit W_L — Water Content (%) 10 20 30.

Unit Weight γ (kN/m³): 23.7 (at elev. ~247.8), 22.6 (at elev. ~246).

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, x^5 : Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 2

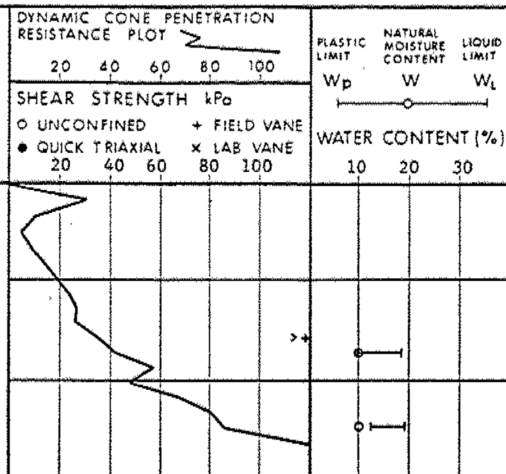
METRIC

W P 95-85-01 LOCATION Co-ords: N 4 860 211.4; E 300 193.0 ORIGINATED BY AL

DIST 6 HWY 400 BOREHOLE TYPE H.S. Auger & Cone Test COMPILED BY AL

DATUM Geodetic DATE 89 08 26 CHECKED BY TS

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
251.9	Ground Surface							
0.0	Sand, Trace Gravel (Fill), Brown, Loose	1	SS	5	▽			
		2	SS	6				
250.1 / 1.8	Clayey Silt, Some Sand, Trace Gravel (Glacial Till), Grey, Firm to V. Stiff, Occ. Sand Seams					250		
		3	SS	8				
		4	SS	12				3 30 50 17
		5	SS	13		248	22.7	
		6	SS	22				3 18 68 11
		7	SS	16			22.8	
		8	SS	22		246		
244.3 / 7.6	Silt, Trace Sand, Trace Gravel, Grey, Compact to Dense	9	SS	24		244		
		10	SS	34		242		
		11	SS	28		240		
		12	SS	18				
		13	SS	19		238		
		14	SS	4				
236.2 / 15.7	End of Borehole							



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

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

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 3 METRIC

W P 95-85-01 LOCATION Co-ords: N 4 860 236.0; E 300 191.8 ORIGINATED BY AL

DIST 6 HWY 400 BOREHOLE TYPE H. S. Auger COMPILED BY AL

DATUM Geodetic DATE 89 08 27 CHECKED BY TS

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR SA SI CL (%)
257.8	Ground Surface							
0.0	Clayey Silt, Some Sand, Trace Gravel (Glacial Till)	1	SS	10		256	22.5	6 28 56 10
	Brown / Grey	2	SS	50				
		3	SS	20		254		
	Stiff to Hard	4	SS	16			21.9	
	Occ. Sand Seams	5	SS	17	▼	252		0 52 38 10
250.9		6	SS	13				
6.9	End of Borehole							

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

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

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

OFFICE REPORT ON SOIL EXPLORATION

FOUNDATION INVESTIGATION REPORT
For
Proposed Extension of
King Township Road 15/16 Overpass
W.P. 95-85-02, Site No. 37-59/R2
Hwy. 400, District 6, Toronto

INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the aforementioned site. It is proposed to extend the existing rigid frame structure on the east side only by approximately 3.75 m to facilitate widening of Hwy. 400. The widening is planned in conjunction with proposed rehabilitation work of the existing superstructure. Extension of the existing structure abutments and the replacement of existing retaining walls and wing walls will be required in connection with the proposed widening.

SITE DESCRIPTION AND GEOLOGY

The site is located at the existing Hwy. 400/King Sideroad 16 crossing in the Township of King approximately 4 km north of King Road. The closest interchange to the site along Hwy. 400 is situated at King Road. The King Sideroad 16 is a two lane roadway that is only paved along limited segments of the road on either side of the structure. The existing Hwy. 400 overpass is a six (6) lane highway.

The land surrounding the site consists of a gently undulating to rolling terrain. In general, the area is used for agricultural and dairy farming. A low lying wetland comprised of cattails and shrubs is located approximately 30 m east of the existing structure.

Physiographically the site is situated in the region known as the "Oak Ridges". The Oak Ridges region is basically covered by an interlobate moraine of the Wisconsinan glacial age and at the site consists of a kame topography that is characterized by a hilly terrain with a knob-and-basin relief. The hills are primarily composed of irregularly mixed sandy or gravelly materials, but in some areas, extensive thickness of boulder clay protrude above the outwash. Bedrock at the site consists of shale of the Dundas-Meaford Formation.

FIELD INVESTIGATION

The fieldwork for the investigation was carried out between 89 08 25 and 89 08 26 and consisted of two sampled boreholes advanced to depths of 14.2 metres accompanied by two dynamic cone penetration tests advanced to depths ranging from 6.4 metres to 8.4 metres. Additional sampled boreholes (BH's 1 and 2) advanced at the site in conjunction with a previous abutment widening (W.P. 105-70-10) have also been included in this report. Hollow stem auger equipment was used to advance the boreholes in the overburden. In general, subsoil samples were retrieved at 0.7 m intervals for the surficial 6.1 metres and at 1.5 m intervals for the remainder of the borehole. All samples were retrieved in accordance with the Standard Penetration Test (ASTM D1586) using a split spoon sampler. In situ vane tests were carried out at 1.5 m intervals in the surfical cohesive soil to determine the undrained shear strength at both the undisturbed and remoulded state. The test was conducted in accordance with ASTM D2573, using the standard MTO 'N' vane. All samples were identified in the field and then returned to the laboratory for applicable testing.

Groundwater levels were obtained in the open boreholes and monitored throughout the duration of the investigation. The boreholes were backfilled at the completion of the investigation. Survey information related to the location and elevation of boreholes was provided by Central Region Surveys and Plans.

LABORATORY ANALYSES

To identify the behaviour, gradation and property of the soil, the following laboratory tests were conducted:

1) Atterberg Limits
2) Grain Size Distributions
3) Natural Moisture Contents
4) Bulk Densities

Laboratory test results have been summarized in subsequent sections of this report and are illustrated graphically in the figures and boreholes attached in the Appendix of this report.

SUBSURFACE CONDITIONS

Subsoil conditions are generally uniform across the site. The predominant deposit consists of a clayey silt with some sand and a trace of gravel and extends for a minimum thickness of 10.9 m. The maximum thickness of the deposit was not ascertained in the scope of this investigation. The stratum is a surficial deposit at certain locations of the site but is overlain by a sand with some silt and gravel layer at various locations (BH 's 2, 3 and 4). The maximum thickness of the sand layer was approximately 2 m.

The boundaries between the various soil types, in situ and laboratory test results as well as groundwater levels established at the time of investigation, are shown on the attached Record of Borehole sheets in the Appendix. A plan of the site illustrating the locations and elevations of the boreholes and subsoil stratigraphical section are provided on Dwg. 958502-A.*

A detailed description of the subsurface conditions encountered is given below.

Sand, some Silt, trace Gravel

A thin, cohesionless deposit of sand with some silt and a trace of gravel overlies the main clayey silt deposit at certain locations at the site. The deposit is unoxidized and ranges in thickness from 0.8 m to 2.0 m.

Standard Penetration Tests carried out in this layer revealed 'N' values ranging from 5 blows/0.3 m to 27 blows/0.3 m indicating that the deposit ranges in denseness from loose to compact.

* SHEET NO 129 OF THE CONTRACT DWG'S

Clayey Silt, some Sand, trace of Gravel (Glacial Till)

Underlying the surficial cover of sand, where it exists and immediately below the natural ground surface elsewhere, exists the main deposit at the site consisting of clayey silt with some sand and a trace of gravel. Occasional seams of silty sand up to 50 mm in thickness are also present interbedded throughout the deposit. The clayey silt deposit was not fully penetrated at any of the boring locations but was found to have a minimum thickness of 10.9 m.

A grain size distribution envelope as determined by mechanical sieve and hydrometer analyses in the laboratory is illustrated in Figure 1 in the Appendix. The distribution of the material illustrates significant percentages of silt (57-69%) and clay sizes generally ranging from 19-28%. This breakdown of fine grained material is inherent of a material that exhibits a clayey silt behaviour.

Atterberg Limits were obtained to evaluate the plasticity and behaviour of the fine grained portion of the soil and the results are plotted in Figure 2 and summarized in Table 1 below. Additional physical properties of the soil as determined by field and laboratory testing, including bulk densities and undrained shear strengths also comprise Table 1.

Table 1 - Physical Properties

	Range	# of Tests
Natural Moisture (w%)	15-17	5
Liquid Limit (w_L%)	20-24	4
Plasticity Index (Ip%)	6.5-7.5	4
Unit Weight (kN/m^3)	20.5-21.4	4
Undrained Shear Strength (kPa)		
- Field Vane	60->120	4
Sensitivity	2-4	3

The test results reveal that the cohesive deposit is inorganic with a low plasticity. The natural moisture content varies randomly throughout the deposit but generally is approximately equivalent to the plastic limit of the soil.

The consistency of the stratum was derived from the results of the Standard Penetration tests and field vane tests. Observation of the 'N' values obtained from the Standard Penetration Test reveal that the values from this field investigation are noticeably lower than 'N' values obtained from the previous investigation. At the proposed abutment extension locations, N-values ranged from 5 blows/0.3 m to 28 blows/0.3 m, averaging 16 blows/0.3 m for the upper 12-13 m of the deposit for the field investigation that was recently conducted. For the field investigation conducted previously for the original extension, N-values ranged from 22 blows/0.3 m to 37 blows/0.3 m, averaging 29 blows/0.3 m over the same depth and elevation. Based on these values, the soil appears to have a consistency ranging from firm to hard but in general can be categorized as stiff to very stiff.

Somewhat weaker subsoil conditions were encountered at BH 4, the location of the proposed replacement retaining walls at the toe of the south approach fills. At this location, N-values obtained from the Standard Penetration Test ranged from 5 blows/0.3 m to 14 blows/0.3 m for the upper 8 m of the deposit and from 28 blows/0.3 m to 64 blows/0.3 m for the remainder 3 m depth investigated. Based on these values, the consistency of the soil can be classified as firm to stiff for the upper zone of the deposit and very stiff to hard for the lower zone.

Undrained shear strength measurements (c_u) were also obtained in situ by conducting field vane tests and the results are plotted on the Record of Boreholes in the Appendix and summarized in Table 1 above. At the location of the weaker subsoil (BH 4) as mentioned above, undrained shear strength values ranged from 60 to 70 kPa indicating a stiff consistency. At the proposed abutment extension location (BH 3), the subsoil conditions were such that the vane could not be torqued within the limitations of the test. This again illustrates the very stiff consistency predominant at this location.

The sensitivity of the soil as defined by the ratio of the undrained strength in the undisturbed state to the undrained strength, at the same water content, in the remoulded state, as determined by the in situ vane test, ranged in value from 2 to 4 indicating a low sensitivity.

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water level in open boreholes. These observations are recorded on the Record of Borehole sheets as well as on Dwg. 958502-A.* Measurements obtained at the time of investigation revealed levels at an elevation ranging from 328.3 m to 327.6 m which corresponds to a depths of approximately 0.6 m below natural ground surface. Groundwater levels, however, are subject to seasonal fluctuations and hence can vary from the values given in this report.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer and A. Lako, Student Engineer, utilizing equipment owned and operated by Marathon Drilling.

The project was carried out by T. Sangiuliano under the general supervision of Dr. B. Iyer, Senior Foundation Engineer. The report was written by T. Sangiuliano, reviewed by Dr. B. Iyer and approved by Mr. M.S. Devata, Chief Foundation Engineer.

* SHEET NO 129 OF THE CONTRACT DWG'S

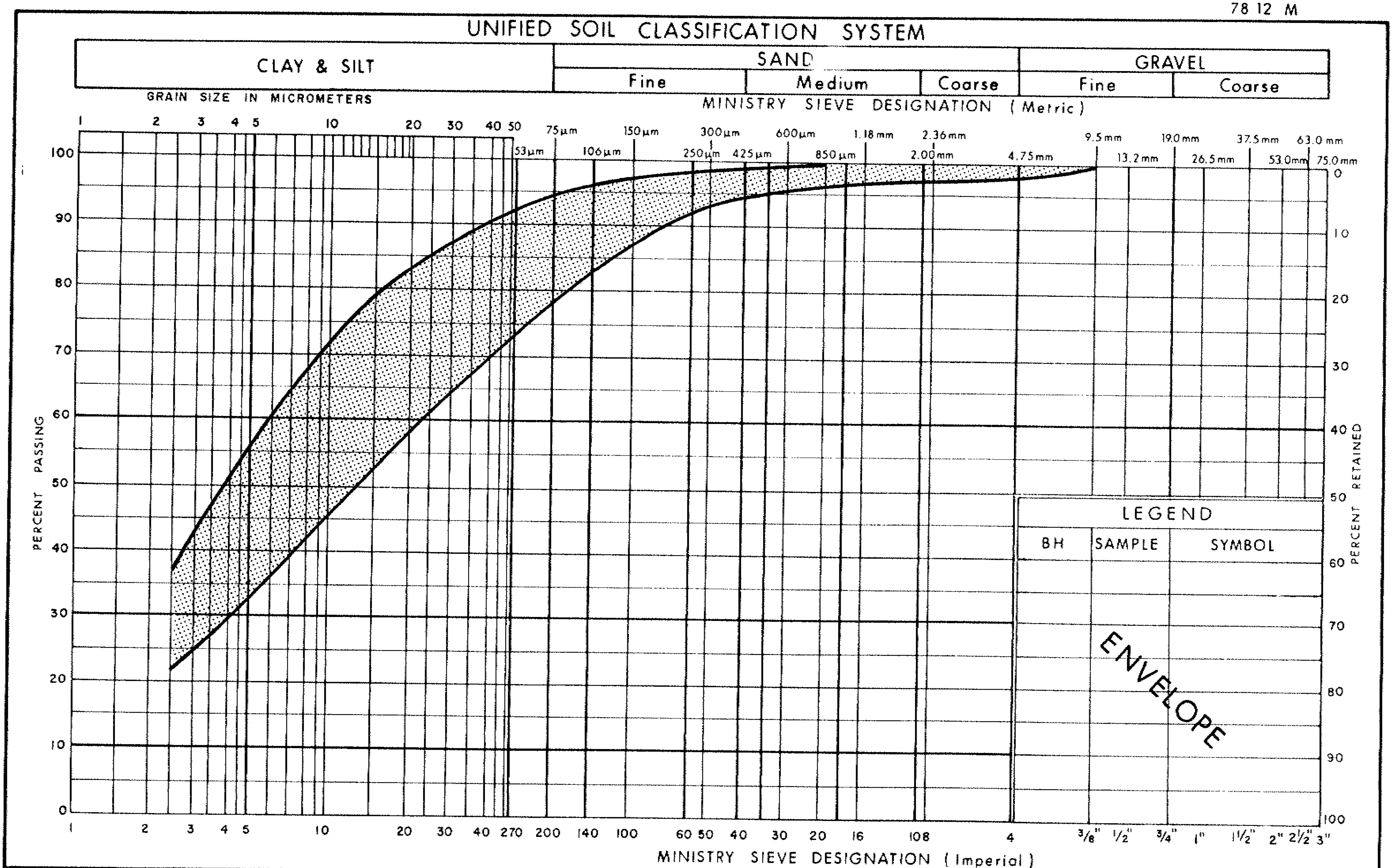
LICENSED PROFESSIONAL ENGINEER
B. IYER
PROVINCE OF ONTARIO

B. Iyer, P. Eng.
Sr. Foundation Engineer

for M. Devata, P. Eng.
Chief Foundation Engineer

APPENDIX

78 12 M



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GRAIN SIZE DISTRIBUTION

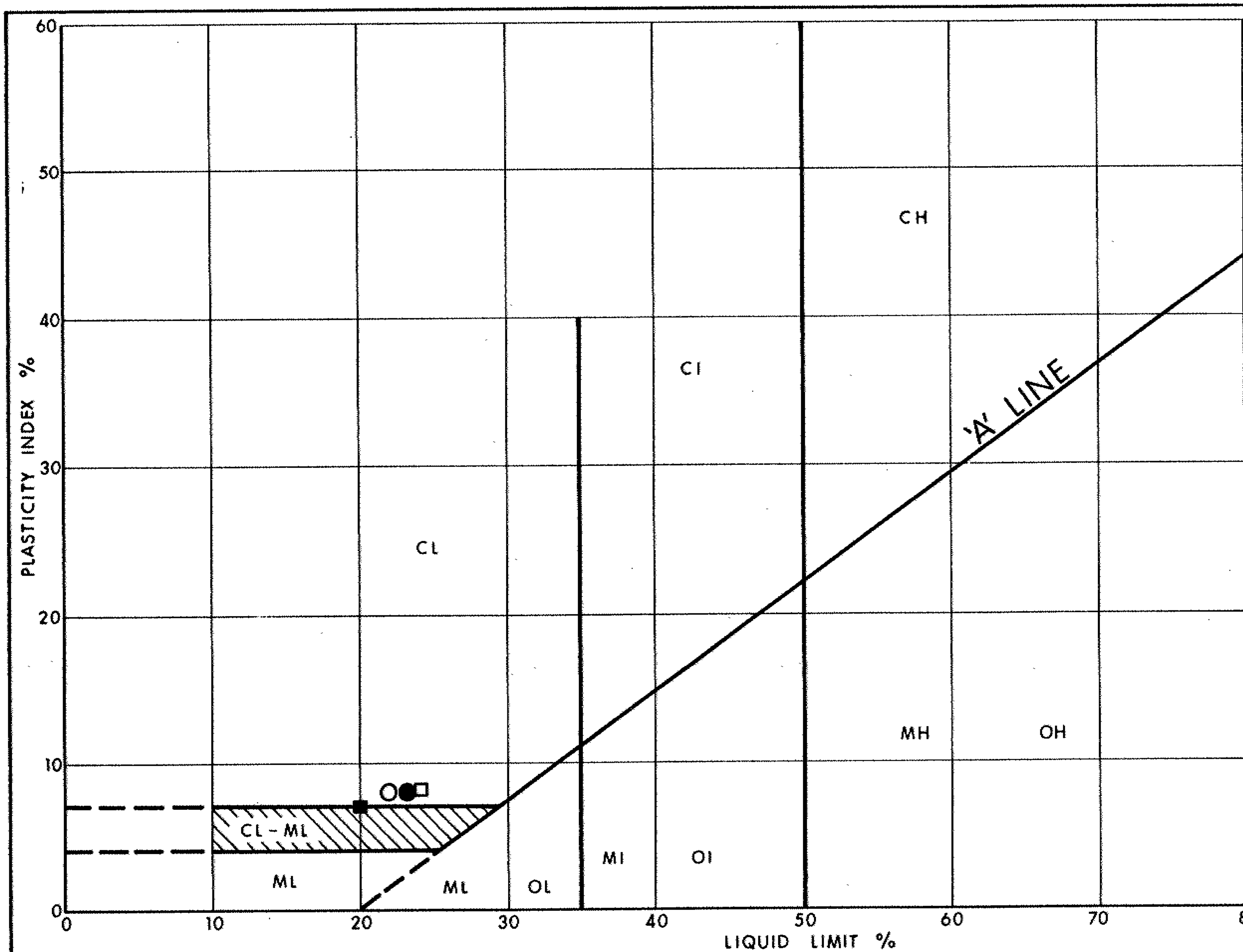
CLAYEY SILT, SOME SAND, TRACE OF GRAVEL (Glacial Till)

FIG No 1

W P 95-85-02

22

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
3	4	○
3	6	●
3	10	□
4	4	■

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Ontario

PLASTICITY CHART
CLAYEY SILT, SOME SAND, TRACE OF GRAVEL
(Glacial Till)

FIG No 2

WP 95-85-02

RECORD OF BOREHOLE No 1 METRIC

W P 95-85-02 LOCATION Co-ords: N 4 868 340.6; E 298 765.1 ORIGINATED BY VK

DIST 6 HWY 400 BOREHOLE TYPE Continuous Flight Auger & Cone Test COMPILED BY WH

DATUM Geodetic DATE 1970 10 16 CHECKED BY TS

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
329.6	Ground Surface						
0.0	Clayey Silt, Trace of Sand and Gravel, Occasional Silty Sand Seams, Stiff to Hard (Glacial Till), Brown to Grey	1	SS	18			
		2	SS	7	70 11 18	328	
		3	SS	30			
		4	SS	27		326	
		5	SS	41			
						324	
		6	SS	30			
	Sand Seam	7	SS	14		322	0 84 (16)
		8	SS	25			
						320	
		9	SS	29			
						318	
317.3		10	SS	100/15cm			
12.3	End of Borehole						

Dynamic Cone Penetration Resistance Plot (20 40 60 80 100): 100/20cm

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

OFFICE REPORT ON SOIL EXPLORATION

$+^3$, $\times^5$: Numbers refer to Sensitivity   15 ◇ 5 (20, 10) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

W P 95-85-02 | LOCATION Co-ords: N 4 868 359.3; E 298 797.9 | ORIGINATED BY VK

DIST 6 HWY 400 | BOREHOLE TYPE Continuous Flight Auger & Cone Test | COMPILED BY WH

DATUM Geodetic | DATE 1970 10 16 | CHECKED BY TS

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
328.8	Ground Surface						
0.0	Sand, Some Silt and Gravel Compact, Grey				70 11 18		
327.6 / 1.2		1	SS	27			16 60 (24)
	Clayey Silt, Trace of Sand and Gravel, Occasional Silty Sand Seams, Very Stiff to Hard (Glacial Till), Brown to Grey	2	SS	22			
		3	SS	24			
		4	SS	31			
		5	SS	30			
		6	SS	28			
		7	SS	37			
	Sand & Gravel Seam	8	SS	36			29 56 (15)
316.7		9	SS	136/25cm			
12.1	End of Borehole						

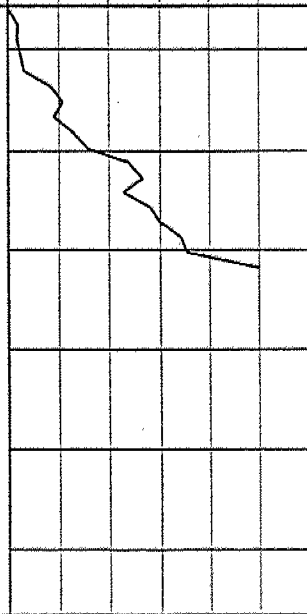
ELEVATION SCALE: 328, 326, 324, 322, 320, 318

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 (20, 10) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3 — METRIC

W P 95-85-02 — LOCATION Co-ords: N 4 868 350.6; E 298 799.6 — ORIGINATED BY AL

DIST 6 HWY 400 — BOREHOLE TYPE H.S. Auger & Cone Test — COMPILED BY AL

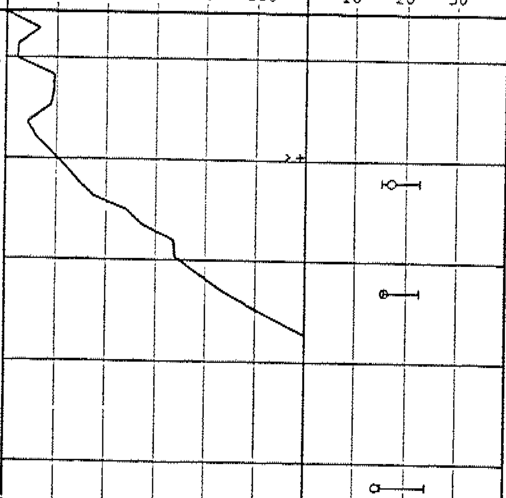
DATUM Geodetic — DATE 89 08 26 — CHECKED BY TS

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
328.9	Ground Surface							
0.0 / 328.1	Sand, Some Silt, Trace Gravel				▽			
0.8	Clayey Silt, Some Sand, Trace Gravel (Glacial Till), Occasional Silty Sand Seams	1	SS	12		328		
		2	SS	12				
		3	SS	5				
		4	SS	14		326		2 22 57 19
	Brown	5	SS	21			20.7	
	Grey	6	SS	18		324		3 11 62 24
		7	SS	16			21.4	
		8	SS	14				
						322		
		9	SS	17				
						320		
		10	SS	20				2 8 69 21
		11	SS	16		318		
	Firm to V. Stiff	12	SS	28				
	Hard					316		
314.7 / 14.2	End of Borehole	13	SS	103				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

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



OFFICE REPORT ON SOIL EXPLORATION

$+^3$, x^5 : Numbers refer to Sensitivity — 15 ⌖ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

METRIC

W P 95-85-02 LOCATION Co-ords: N 4 868 352.2; E 298 810.7 ORIGINATED BY AL

DIST 6 HWY 400 BOREHOLE TYPE H.S. Auger & Cone Test COMPILED BY AL

DATUM Geodetic DATE 89 08 25 CHECKED BY TS

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (+ field vane)	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
328.3	Ground Surface								
0.0	Sand, Some Silt, Trace Gravel, Grey, Loose to Compact	1	SS	10	▽	328			
326.3		2	SS	5					
2.0	Clayey Silt, Some Sand, Trace Gravel (Glacial Till) Grey, Occasional Silty Sand Seams	3	SS	9		326	+3		0 14 67 19
		4	SS	5				20.2	
		5	SS	6		324	+2		
		6	SS	8				20.2	
		7	SS	8			+4		
		8	SS	9		322			
		9	SS	14		320			1 6 65 28
	Firm to Stiff	10	SS	11		318			
	V. Stiff to Hard	11	SS	28					
		12	SS	29		316			
314.1		13	SS	64					
14.2	End of Borehole								

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; REMARKS & GRAIN SIZE DISTRIBUTION (%).

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

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

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Ministry of Transportation

Ontario

FOUNDATION DESIGN SECTION

foundation investigation and design report

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 95-85-02 DIST 6

HWY 400 STR SITE 37-59

Proposed Extension of
King Township Road 15/16 Overpass

CONT 92-95

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GEOCRES #30M13-91 DATE DEC 07 1989

FOUNDATION INVESTIGATION REPORT

For

Proposed Extension of

King Township Road 15/16 Overpass

W.P. 95-85-02, Site No. 37-59

Hwy. 400, District 6, Toronto

INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the aforementioned site. It is proposed to extend the existing rigid frame structure on the east side only by approximately 3.75 m to facilitate widening of Hwy. 400. The widening is planned in conjunction with proposed rehabilitation work of the existing superstructure. Extension of the existing structure abutments and the replacement of existing retaining walls and wing walls will be required in connection with the proposed widening.

The subsurface conditions at the site, together with discussion and recommendations pertaining to the (1) proposed foundations for the abutment structure extension and associated retaining walls, (2) stability and settlement considerations of the approach embankments, and (3) related earthworks including temporary shoring requirements, are included in the scope of this report.

SITE DESCRIPTION AND GEOLOGY

The site is located at the existing Hwy. 400/King Sideroad 16 crossing in the Township of King approximately 4 km north of King Road. The closest interchange to the site along Hwy. 400 is situated at King Road. The King Sideroad 16 is a two lane roadway that is only paved along limited segments of the road on either side of the structure. The existing Hwy. 400 overpass is a six (6) lane highway.

The land surrounding the site consists of a gently undulating to rolling terrain. In general, the area is used for agricultural and dairy farming. A low lying wetland comprised of cattails and shrubs is located approximately 30 m east of the existing structure.

Physiographically the site is situated in the region known as the "Oak Ridges". The Oak Ridges region is basically covered by an interlobate moraine of the Wisconsinan glacial age and at the site consists of a kame topography that is characterized by a hilly terrain with a knob-and-basin relief. The hills are primarily composed of irregularly mixed sandy or gravelly materials, but in some areas, extensive thickness of boulder clay protrude above the outwash. Bedrock at the site consists of shale of the Dundas-Meaford Formation.

FIELD INVESTIGATION

The fieldwork for the investigation was carried out between 89 08 25 and 89 08 26 and consisted of two sampled boreholes advanced to depths of 14.2 metres accompanied by two dynamic cone penetration tests advanced to depths ranging from 6.4 metres to 8.4 metres. Additional sampled boreholes (BH's 1 and 2) advanced at the site in conjunction with a previous abutment widening (W.P. 105-70-10) have also been included in this report. Hollow stem auger equipment was used to advance the boreholes in the overburden. In general, subsoil samples were retrieved at 0.7 m intervals for the surficial 6.1 metres and at 1.5 m intervals for the remainder of the borehole. All samples were retrieved in accordance with the Standard Penetration Test (ASTM D1586) using a split spoon sampler. In situ vane tests were carried out at 1.5 m intervals in the surfical cohesive soil to determine the undrained shear strength at both the undisturbed and remoulded state. The test was conducted in accordance with ASTM D2573, using the standard MTO 'N' vane. All samples were identified in the field and then returned to the laboratory for applicable testing.

Groundwater levels were obtained in the open boreholes and monitored throughout the duration of the investigation. The boreholes were backfilled at the completion of the investigation. Survey information related to the location and elevation of boreholes was provided by Central Region Surveys and Plans.

LABORATORY ANALYSES

To identify the behaviour, gradation and property of the soil, the following laboratory tests were conducted:

1) Atterberg Limits
2) Grain Size Distributions
3) Natural Moisture Contents
4) Bulk Densities

Laboratory test results have been summarized in subsequent sections of this report and are illustrated graphically in the figures and boreholes attached in the Appendix of this report.

SUBSURFACE CONDITIONS

Subsoil conditions are generally uniform across the site. The predominant deposit consists of a clayey silt with some sand and a trace of gravel and extends for a minimum thickness of 10.9 m. The maximum thickness of the deposit was not ascertained in the scope of this investigation. The stratum is a surficial deposit at certain locations of the site but is overlain by a sand with some silt and gravel layer at various locations (BH 's 2, 3 and 4). The maximum thickness of the sand layer was approximately 2 m.

The boundaries between the various soil types, in situ and laboratory test results as well as groundwater levels established at the time of investigation, are shown on the attached Record of Borehole sheets in the Appendix. A plan of the site illustrating the locations and elevations of the boreholes and subsoil stratigraphical section are provided on Dwg. 958502-A.

A detailed description of the subsurface conditions encountered is given below.

Sand, some Silt, trace Gravel

A thin, cohesionless deposit of sand with some silt and a trace of gravel overlies the main clayey silt deposit at certain locations at the site. The deposit is unoxidized and ranges in thickness from 0.8 m to 2.0 m.

Standard Penetration Tests carried out in this layer revealed 'N' values ranging from 5 blows/0.3 m to 27 blows/0.3 m indicating that the deposit ranges in denseness from loose to compact.

Clayey Silt, some Sand, trace of Gravel (Glacial Till)

Underlying the surficial cover of sand, where it exists and immediately below the natural ground surface elsewhere, exists the main deposit at the site consisting of clayey silt with some sand and a trace of gravel. Occasional seams of silty sand up to 50 mm in thickness are also present interbedded throughout the deposit. The clayey silt deposit was not fully penetrated at any of the boring locations but was found to have a minimum thickness of 10.9 m.

A grain size distribution envelope as determined by mechanical sieve and hydrometer analyses in the laboratory is illustrated in Figure 1 in the Appendix. The distribution of the material illustrates significant percentages of silt (57-69%) and clay sizes generally ranging from 19-28%. This breakdown of fine grained material is inherent of a material that exhibits a clayey silt behaviour.

Atterberg Limits were obtained to evaluate the plasticity and behaviour of the fine grained portion of the soil and the results are plotted in Figure 2 and summarized in Table 1 below. Additional physical properties of the soil as determined by field and laboratory testing, including bulk densities and undrained shear strengths also comprise Table 1.

Table 1 - Physical Properties

	Range	# of Tests
Natural Moisture (w%)	15-17	5
Liquid Limit (w_L%)	20-24	4
Plasticity Index (I_P%)	6.5-7.5	4
Unit Weight (kN/m^3)	20.5-21.4	4
Undrained Shear Strength (kPa)		
- Field Vane	60->120	4
Sensitivity	2-4	3

The test results reveal that the cohesive deposit is inorganic with a low plasticity. The natural moisture content varies randomly throughout the deposit but generally is approximately equivalent to the plastic limit of the soil.

The consistency of the stratum was derived from the results of the Standard Penetration tests and field vane tests. Observation of the 'N' values obtained from the Standard Penetration Test reveal that the values from this field investigation are noticeably lower than 'N' values obtained from the previous investigation. At the proposed abutment extension locations, N-values ranged from 5 blows/0.3 m to 28 blows/0.3 m, averaging 16 blows/0.3 m for the upper 12-13 m of the deposit for the field investigation that was recently conducted. For the field investigation conducted previously for the original extension, N-values ranged from 22 blows/0.3 m to 37 blows/0.3 m, averaging 29 blows/0.3 m over the same depth and elevation. Based on these values, the soil appears to have a consistency ranging from firm to hard but in general can be categorized as stiff to very stiff.

Somewhat weaker subsoil conditions were encountered at BH 4, the location of the proposed replacement retaining walls at the toe of the south approach fills. At this location, N-values obtained from the Standard Penetration Test ranged from 5 blows/0.3 m to 14 blows/0.3 m for the upper 8 m of the deposit and from 28 blows/0.3 m to 64 blows/0.3 m for the remainder 3 m depth investigated. Based on these values, the consistency of the soil can be classified as firm to stiff for the upper zone of the deposit and very stiff to hard for the lower zone.

Undrained shear strength measurements (c_u) were also obtained in situ by conducting field vane tests and the results are plotted on the Record of Boreholes in the Appendix and summarized in Table 1 above. At the location of the weaker subsoil (BH 4) as mentioned above, undrained shear strength values ranged from 60 to 70 kPa indicating a stiff consistency. At the proposed abutment extension location (BH 3), the subsoil conditions were such that the vane could not be torqued within the limitations of the test. This again illustrates the very stiff consistency predominant at this location.

The sensitivity of the soil as defined by the ratio of the undrained strength in the undisturbed state to the undrained strength, at the same water content, in the remoulded state, as determined by the in situ vane test, ranged in value from 2 to 4 indicating a low sensitivity.

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water level in open boreholes. These observations are recorded on the Record of Borehole sheets as well as on Dwg. 958502-A in the Appendix. Measurements obtained at the time of investigation revealed levels at an elevation ranging from 328.3 m to 327.6 m which corresponds to a depths of approximately 0.6 m below natural ground surface. Groundwater levels, however, are subject to seasonal fluctuations and hence can vary from the values given in this report.

DISCUSSION AND RECOMMENDATIONS

It is proposed to widen the existing rigid frame structure at the King Twp. Road 15/16-Hwy. 400 overpass on the east side only by approximately 3.75 m. The extension will be coordinated with proposed structure rehabilitation. In addition, the existing forward slope toe retaining walls parallel to King Twp. Road 15/16 will either be replaced with new toe slope retaining walls or eliminated by increasing the length of the new replacement east wingwalls. In view of the considerable length of wingwall required to enable elimination of the toe retaining walls, consideration has been given to a combination east wing wall and retaining wall parallel to Hwy. 400.

The existing rigid frame concrete bridge is approximately 29 m in width and 11 m in length. The bridge had been previously widened in 1971 by approximately 1.2 m on each side by cantilevering off the existing structure. New footings were not required for this extension.

The structures' closed-type abutments are founded on spread footings within the upper portion of the clayey silt stratum at an elevation of approximately 327.4 m. This corresponds to approximately 1.5 m below ground surface. The width of these existing footings is approximately 3.7 m. In addition, forward slope retaining walls located at either end of each abutment are also founded on spread footings at the same elevation within the upper clayey silt stratum. The retaining walls have an approximate arc length of 8.5 m.

The existing alignments and profile grades of Hwy. 400 and King Twp. Road 15/16 will be maintained without any changes. The profile grade of Hwy. 400 at the structure is approximately 336 metres whilst the profile grade of King Twp. Road is approximately 329 m. Consequently, existing approach fills are in the order of magnitude of 7 m and will also be required for the extended approaches.

Although the superstructure reveals evidence of concrete spalling and delamination, concrete shrinkage cracking and exposed rebar appears to be somewhat corroded, there appears to be no visible signs of distress caused by an unsound substructure. The approach fills also seem to be performing satisfactorily.

Recommendations pertaining to the following geotechnical considerations are provided in the scope of this report.

1. Structure Foundations
2. Lateral Earth Pressures on Structure
3. Approach Embankments
4. Temporary Shoring

1. Structure Foundations

The proposed abutment structure extension, and associated retaining walls, including the forward slope toe retaining walls can be supported on conventional spread footings founded in the surficial clayey silt deposit. The design values for purposes of the O.H.B.D.C. and the founding elevations are provided in Table 2. The underside of all footings should be provided with a minimum 1.2 m of earth cover for frost protection.

The bearing capacity values recommended reveal lower values at the proposed abutment extension location and south retaining wall as well as the proposed forward slope retaining wall locations. The lower values reflect the sowewhat weaker subsoil conditions encountered at these locations.

Table 2 - Structure Foundations

Structure	Founding Elevation (m)	Factored Capacity at U.L.S. (kPa)	Bearing Capacity at S.L.S. Type II (kPa)
North Abutment	<327.4	400	275
South Abutment	<327.4	375	250
North Retaining Wall*	<327.4	400	275
South Retaining Wall*	<327.4	375	250
Forward Slope Toe Retaining Walls (North and South)	<326.0	150	225

*parallel to Hwy. 400 to replace existing wing walls.

Settlements induced on the founding soil as a result of the applied bearing pressures will be recompression in nature and hence will take place almost immediately. It is anticipated that differential and total settlements induced within the proposed footing, would not exceed 25 mm. However, to avoid any undue stress on the existing abutment structure caused by differential settlement, it is recommended that the abutment extensions be constructed by employing a construction joint between the existing and widened portion of the abutment.

Sliding resistance between the concrete footing and the foundation soil should be calculated in accordance with Section 6-7.3.3.2 of the O.H.B.D.C. assuming an unfactored adhesion value of 75 kPa. Sliding resistance can be supplemented by constructing shear keys in the founding soil below the base of the footing.

Construction of shallow foundations shall be carefully controlled and monitored to avoid any potential undermining of existing footings. Any loss of soil beneath existing footings shall be immediately replaced with mass concrete or granular material.

The soil at the founding elevation shall be inspected to ensure that any softened or organic material is removed and replaced with mass concrete. Any fill that may have been placed in the construction of the existing footing shall be inspected to determine its acceptability as a footing base. Should the fill NOT be granular in nature and consist of organics and softened material, the material should be subexcavated and replaced with a suitable granular material or mass concrete. Again, caution must be exercised to avoid undermining existing footings during any subexcavation.

In addition, a concrete working slab shall be placed immediately following excavation to protect the entire bearing surface at the proposed footing from the effects of weathering and other disturbances.

No major dewatering difficulties are anticipated for footing excavations in consideration of the relatively low permeability of the clayey silt deposit. Conventional sump pumping techniques will suffice in discharging any localized seepage.

2. Lateral Earth Pressures on Structure

Free draining material such as Granular 'A' or Granular 'B' is recommended as appropriate backfill to the abutments to prevent hydrostatic pressure build-up. Design parameters of the soil are given below:

	Granular 'A'	Granular 'B'
Angle of Internal Friction (Ø)	35°	30°
Unit Weight (kN/m^3)	22.8	21.2
Coefficient of Active Earth Pressure (K_a)*	0.27	0.33
Coefficient of Earth Pressure at Rest (K_o)*	0.43	0.5

*for horizontal backfill

The earth pressure coefficient at rest is to be used in design if the abutment walls are rigid and unyielding. Weep holes in the abutment walls should be designed to drain any accumulation of water in the backfill.

The backfill should be constructed in 300 mm lifts on alternating sides of the rigid frame structure so that the maximum differential in backfill heights at no time exceeds 300 mm. O.P.S.D. 803 series illustrates the applicable backfill standards and specifications.

3. Approach Embankments

No deep-seated stability problems are anticipated for the proposed widening of the approach embankments constructed at 2H:1V slopes in view of the competent nature of the subsoil and the established fill height magnitudes in the order of 7 m. In addition, internal stability of the new fill sections can be effectively controlled by (a) "benching" the new approach fills to the existing fills in accordance with MTO standards (OPSD-208.01) and (b) providing an adequate surface erosion protection scheme, such as sodding, on the exposed slopes.

Settlements in the order of magnitude of 25 mm attributable to the elastic compression of the native subsoil and settlement within the fills under its own weight are anticipated. It is predicted that the majority of the settlements will be realized during or immediately after the construction of the additional fill sections. Differential settlement problems between the existing and new fills are not anticipated provided the new fills are "benched" as previously discussed.

In the construction of the embankment fills, all softened and/or organic material should be excavated for their full depth within the plan limits prior to fill placment.

4. Temporary Shoring

To facilitate construction of the proposed abutment extension and associated retaining walls, and simultaneously maintaining traffic protection on Hwy. 400 at the site, a temporary shoring system will be required. The temporary shoring systems installed both parallel and perpendicular to the extension will retain the existing approach embankments during construction.

A cantilever soldier pile-lagging system is one method of shoring recommended. For purposes of shoring design, the following soil parameters are provided:

Table 3 - Shoring Design Parameters

Soil	Elevation (m)	Saturated Unit Weight (kN/m^3)	Effective Shear Strength Parameters
Embankment Material	336-329	21	30*
Clayey Silt	<329	21	30

*assumed values. Embankment material was not investigated because of impractical access.

Earth pressures shall be computed in accordance with section 6.6.1 of the O.H.B.D.C.. Appropriate consideration for sloping soil surfaces and surcharge loading at the road surface shall be included in the design. Buoyant unit weight of soil shall be employed below the prevailing groundwater table.

The soldier piles can be installed in preaugered holes or driven using conventional pile driving methods. Soil anchors may be installed to augment the lateral resistance of the shoring wall. Pertinent pull-out capacities can be provided by this office if anchors are selected in the design.

The shoring installation shall comply with Construction Specification OPSS 538-539.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer and A. Lako, Student Engineer, utilizing equipment owned and operated by Marathon Drilling.

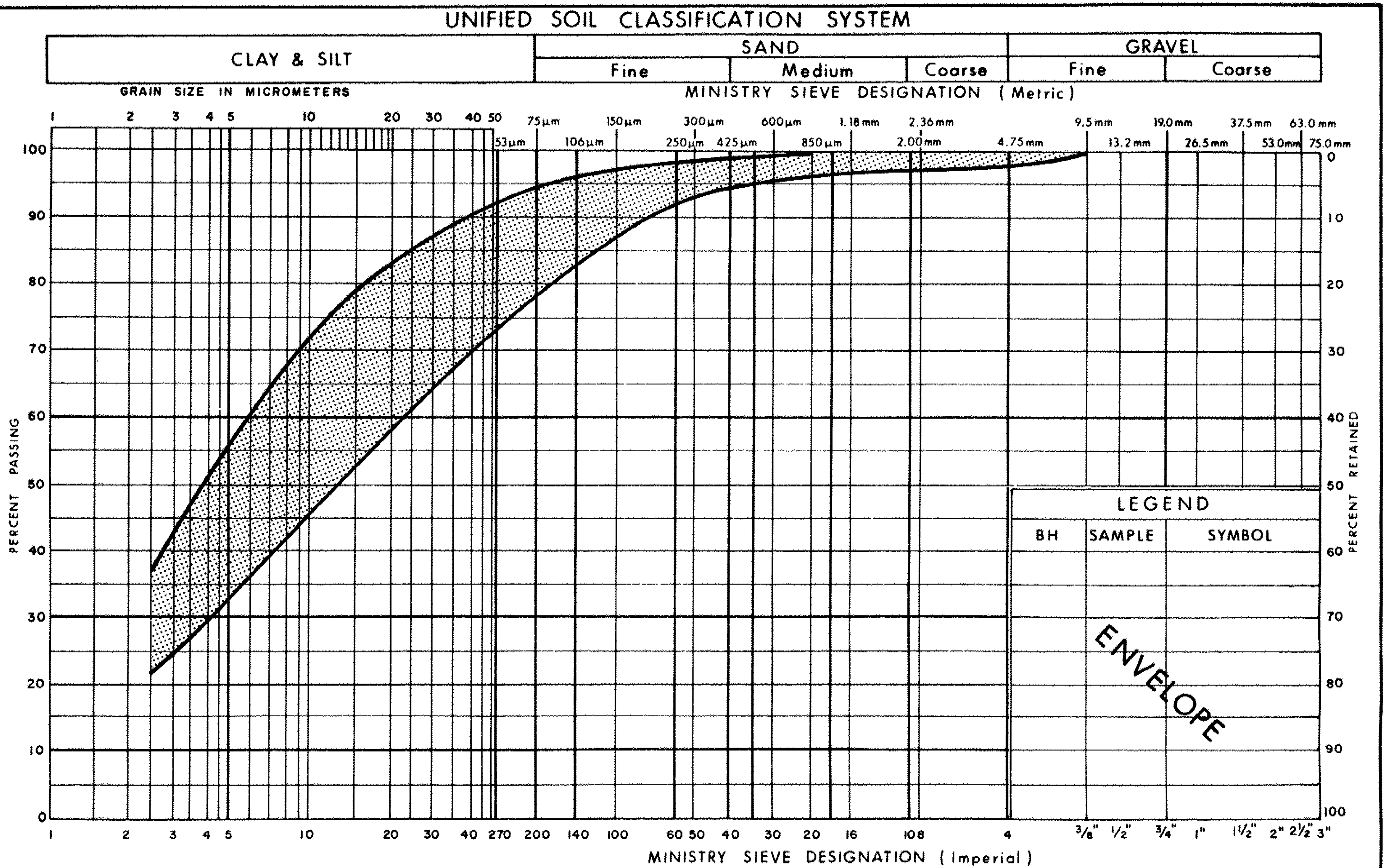
The project was carried out by T. Sangiuliano under the general supervision of Dr. B. Iyer, Senior Foundation Engineer. The report was written by T. Sangiuliano, reviewed by Dr. B. Iyer and approved by Mr. M.S. Devata, Chief Foundation Engineer.

T. Sangiuliano, P.Eng.
Foundation Engineer

M.S. Devata, P.Eng.
Chief Foundation Engineer

APPENDIX

78 12 M



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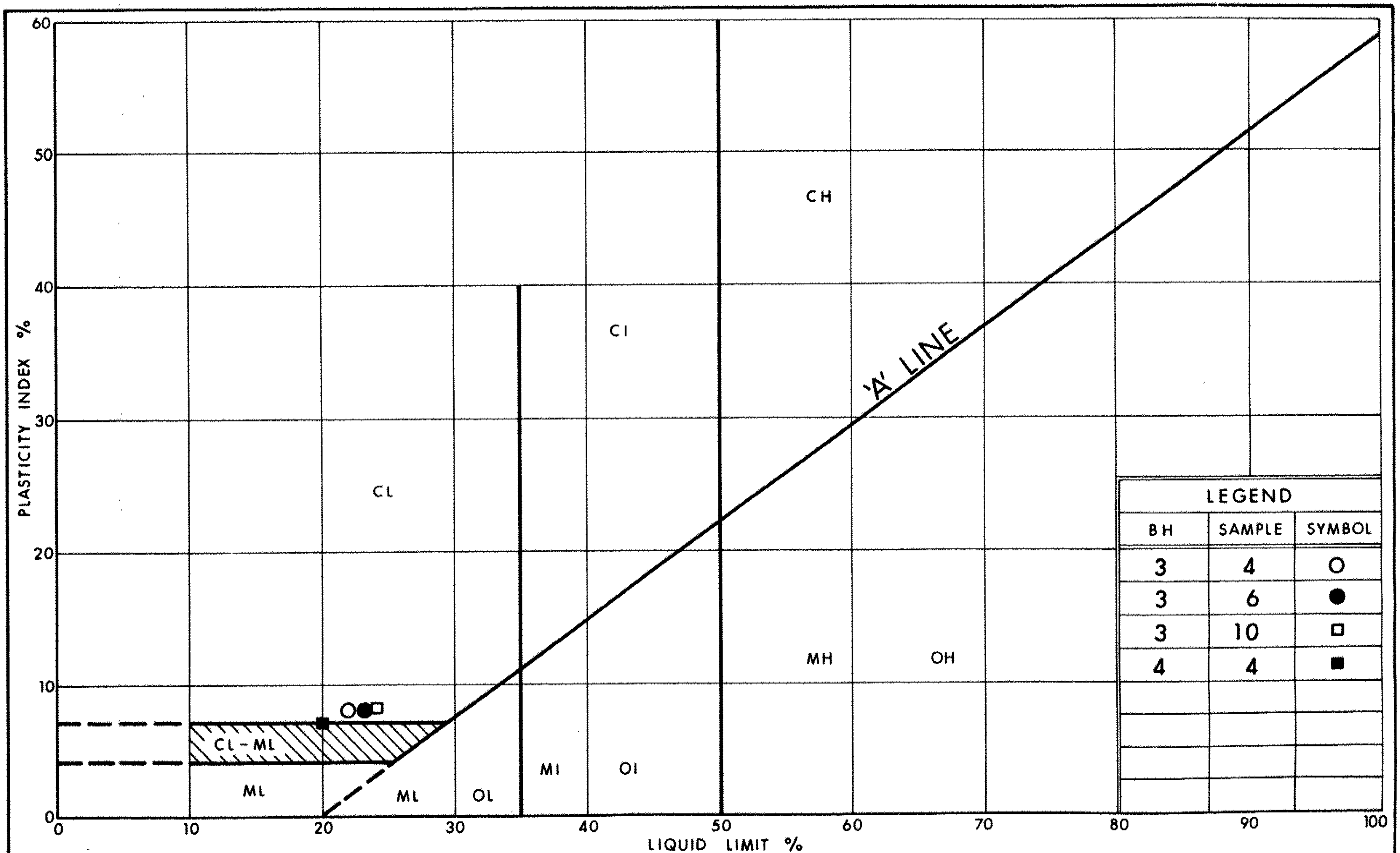
GRAIN SIZE DISTRIBUTION
CLAYEY SILT, SOME SAND, TRACE OF GRAVEL (Glacial Till)

FIG No 1

W P 95-85-02

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
3	4	○
3	6	●
3	10	□
4	4	■

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Ontario

PLASTICITY CHART
CLAYEY SILT, SOME SAND, TRACE OF GRAVEL
(Glacial Till)

FIG No 2

W P 95-85-02

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

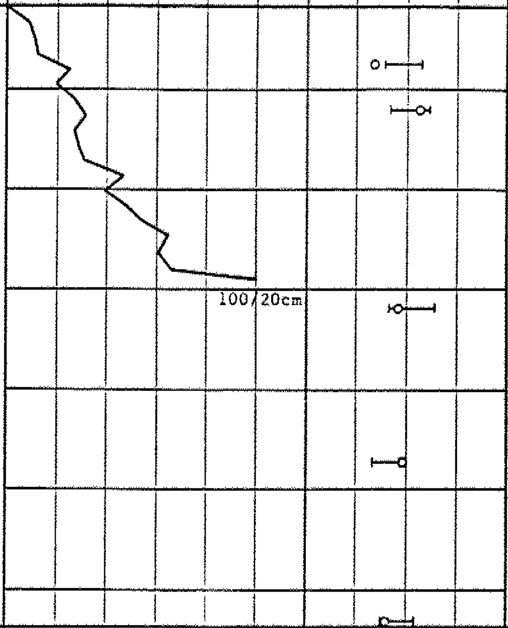
W P 95-85-02 LOCATION Co-ords: N 4 868 340.6; E 298 765.1 ORIGINATED BY VK

DIST 6 HWY 400 BOREHOLE TYPE Continuous Flight Auger & Cone Test COMPILED BY WH

DATUM Geodetic DATE 1970 10 16 CHECKED BY TS

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
329.6	Ground Surface						
0.0	Clayey Silt, Trace of Sand and Gravel, Occasional Silty Sand Seams, Stiff to Hard (Glacial Till), Brown to Grey	1	SS	18	▽ 70 11 18		
		2	SS	7		328	
		3	SS	30			
		4	SS	27		326	
		5	SS	41			
		6	SS	30		324	
	Sand Seam	7	SS	14		322	0 84 (16)
		8	SS	25		320	
		9	SS	29		318	
317.3		10	SS	100/15cm			
12.3	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

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

15 ⌖ 5 (20, 10) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 2

METRIC

W P 95-85-02 LOCATION Co-ords: N 4 868 359.3; E 298 797.9 ORIGINATED BY VK

DIST 6 HWY 400 BOREHOLE TYPE Continuous Flight Auger & Cone Test COMPILED BY WH

DATUM Geodetic DATE 1970 10 16 CHECKED BY TS

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
328.8	Ground Surface						
0.0	Sand, Some Silt and Gravel				70 11 18	328	
327.6	Compact, Grey	1	SS	27			16 60 (24)
1.2	Clayey Silt, Trace of Sand and Gravel	2	SS	22			
	Occasional Silty Sand Seams	3	SS	24		326	
	Very Stiff to Hard	4	SS	31			
	(Glacial Till)	5	SS	30		324	
	Brown to Grey	6	SS	28		322	
		7	SS	37			
	Sand & Gravel Seam	8	SS	36		320	29 56 (15)
						318	
316.7		9	SS	136/25cm			
12.1	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 (20 / 10) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 3

METRIC

W P 95-85-02 LOCATION Co-ords: N 4 868 350.6; E 298 799.6 ORIGINATED BY AL

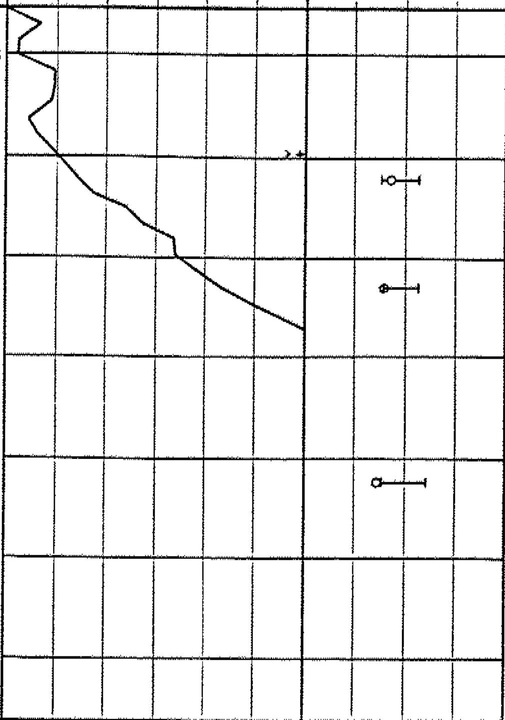
DIST 6 HWY 400 BOREHOLE TYPE H.S. Auger & Cone Test COMPILED BY AL

DATUM Geodetic DATE 89 08 26 CHECKED BY TS

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
328.9	Ground Surface							
0.0 / 328.1 / 0.8	Sand, Some Silt, Trace Gravel							
	Clayey Silt, Some Sand, Trace Gravel (Glacial Till), Occasional Silty Sand Seams; Brown / Grey; Firm to V. Stiff / Hard	1	SS	12	328			
		2	SS	12				
		3	SS	5				
		4	SS	14	326			2 22 57 19
		5	SS	21			20.7	
		6	SS	18	324			3 11 62 24
		7	SS	16			21.4	
		8	SS	14	322			
		9	SS	17				
		10	SS	20	320			2 8 69 21
		11	SS	16	318			
		12	SS	28	316			
314.7 / 14.2	End of Borehole	13	SS	103				

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100



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

GROUND WATER CONDITIONS

OFFICE REPORT ON SOIL EXPLORATION

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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 4

METRIC

W P 95-85-02 LOCATION Co-ords: N 4 868 352.2; E 298 810.7 ORIGINATED BY AL

DIST 6 HWY 400 BOREHOLE TYPE H.S. Auger & Cone Test COMPILED BY AL

DATUM Geodetic DATE 89 08 25 CHECKED BY TS

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
328.3	Ground Surface						
0.0	Sand, Some Silt, Trace Gravel, Grey, Loose to Compact	1	SS	10	328		
326.3		2	SS	5			
2.0	Clayey Silt, Some Sand, Trace Gravel (Glacial Till) Grey, Occasional Silty Sand Seams; Firm to Stiff / V. Stiff to Hard	3	SS	9	326		
		4	SS	5			0 14 67 19
		5	SS	6		20.2	
		6	SS	8	324		
		7	SS	8		20.2	
		8	SS	9	322		
		9	SS	14			1 6 65 28
					320		
		10	SS	11			
					318		
		11	SS	28			
		12	SS	29	316		
314.1		13	SS	64			
14.2	End of Borehole						

Column headings: GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30).

Field vane sensitivity values plotted: 3, 2, 4.

+3, ×5 : Numbers refer to Sensitivity

20 / 15 -o- 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

METRIC

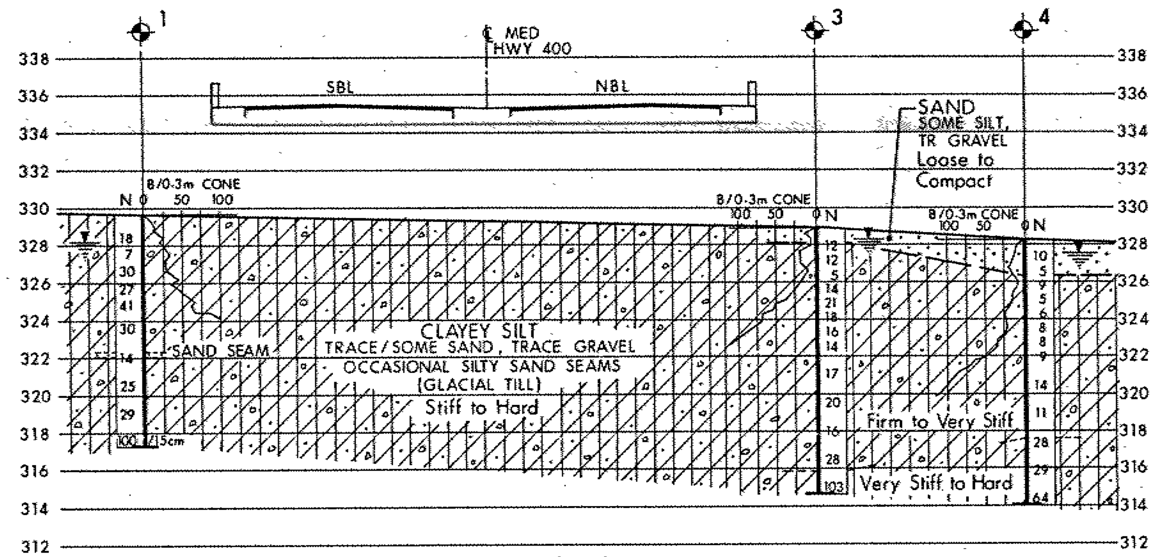
DIMENSIONS ARE IN METRES AND / OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No
WP No 95-85-02

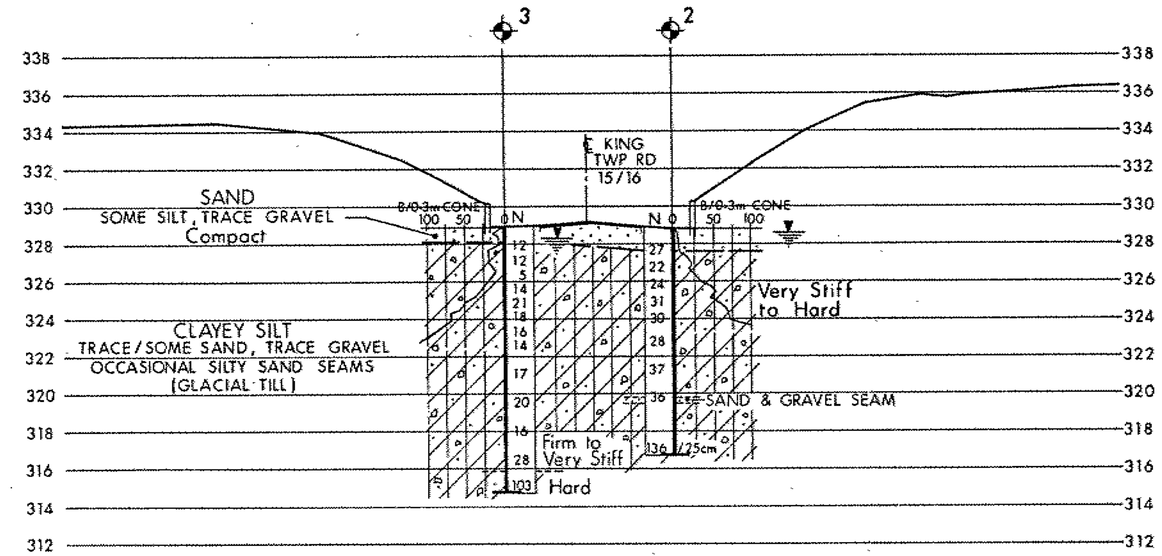
KING TWP RD 15/16 OVERPASS

BORE HOLE LOCATIONS & SOIL STRATA

SHEET

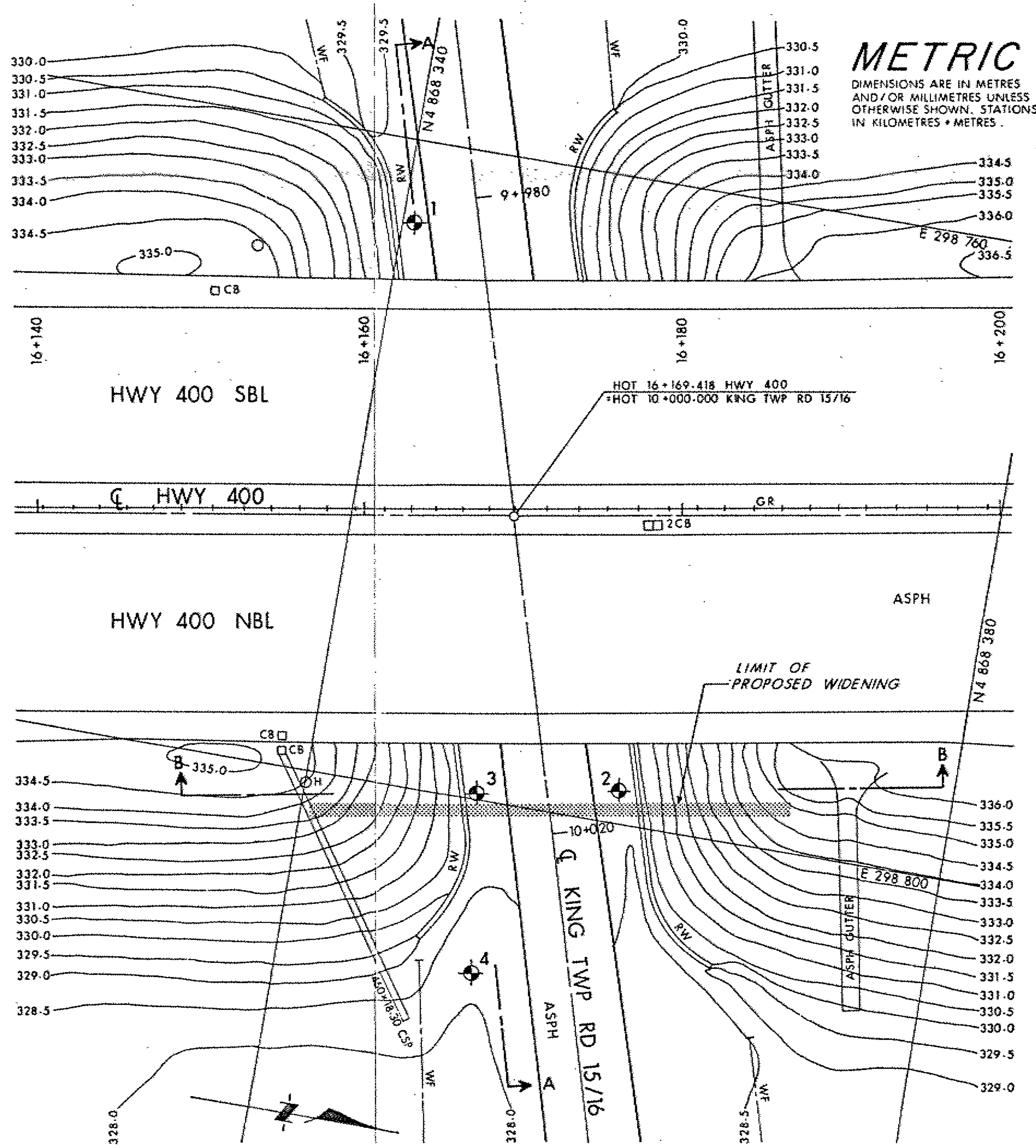


A - A

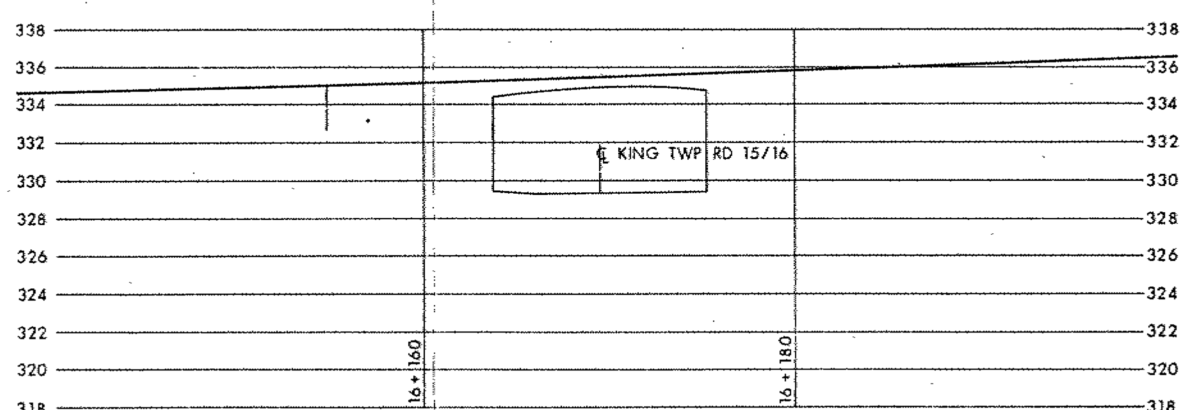


B - B

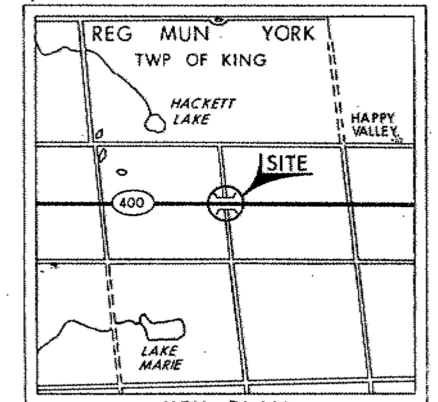
SECTIONS
SCALE
4m 2 0 4m



PLAN
SCALE
4m 2 0 4m



℄ PROFILE HWY 400
SCALE
4m 2 0 4m



KEY PLAN
SCALE
1.0km 0 1.0km

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 70 11 and 89 08

	No	ELEVATION	CO-ORDINATES NORTH	EAST
70 10	1	329.6	4 868 340.6	298 765.1
	2	328.8	4 868 359.3	298 797.9
89 08	3	328.9	4 868 350.6	298 799.6
	4	328.3	4 868 352.2	298 810.7

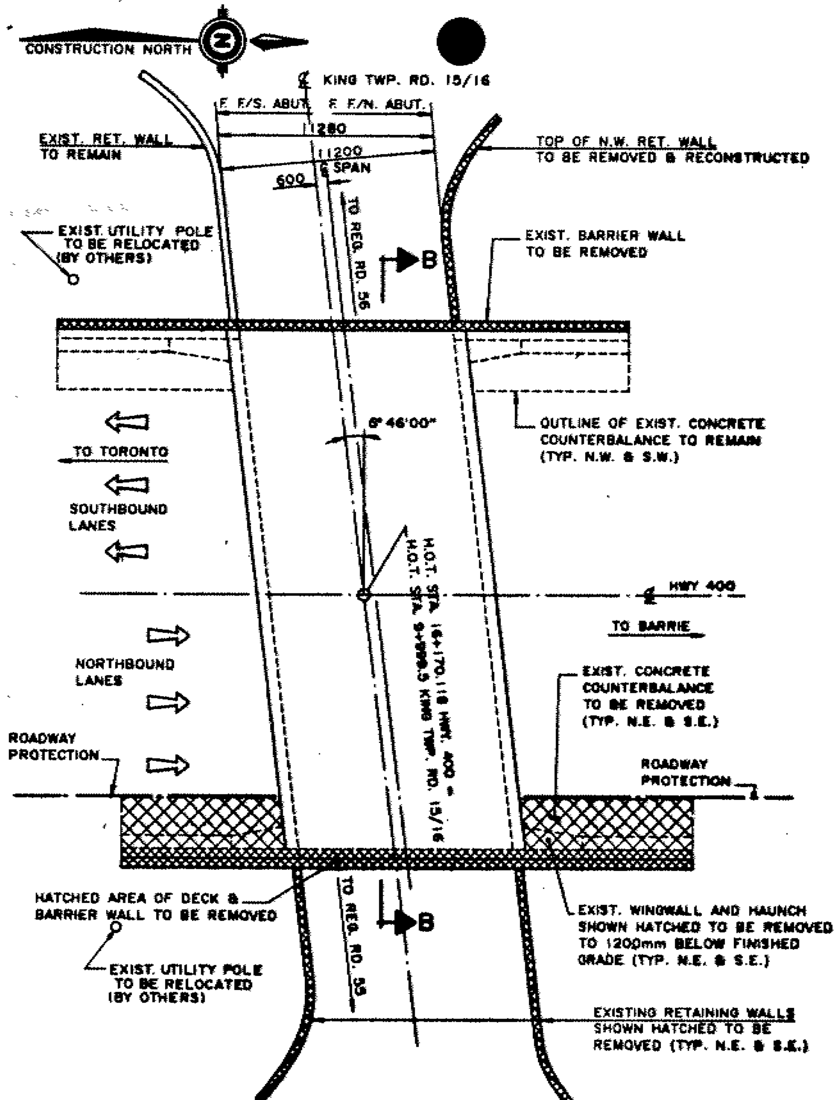
=NOTE=
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

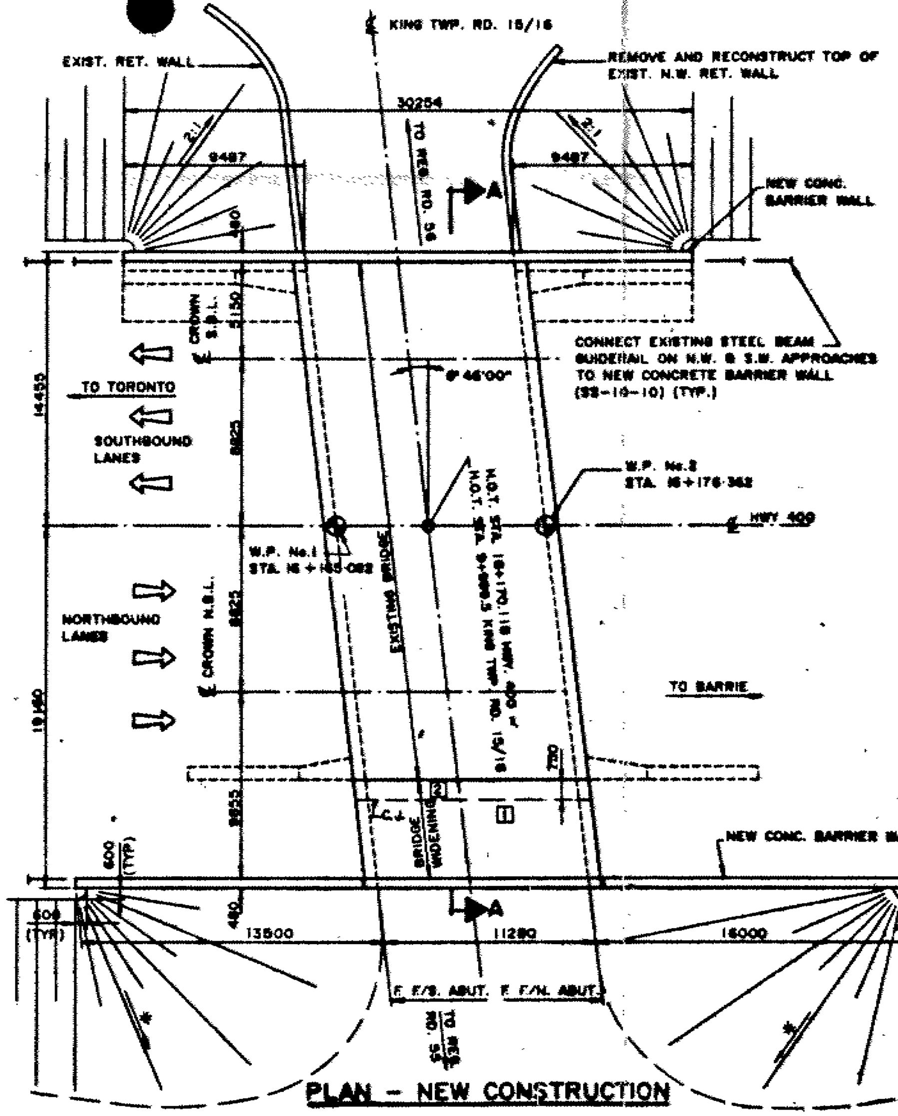
Geocres No 30M13-91

HWY No 400			DIST 6
SUBM'D TS	CHECKED	DATE 89 11 29	SITE 37-59
DRAWN DT	CHECKED	APPROVED	DWG 958502-A



PLAN - EXISTING BRIDGE

SCALE 1:200



PLAN - NEW CONSTRUCTION

SCALE 1:200

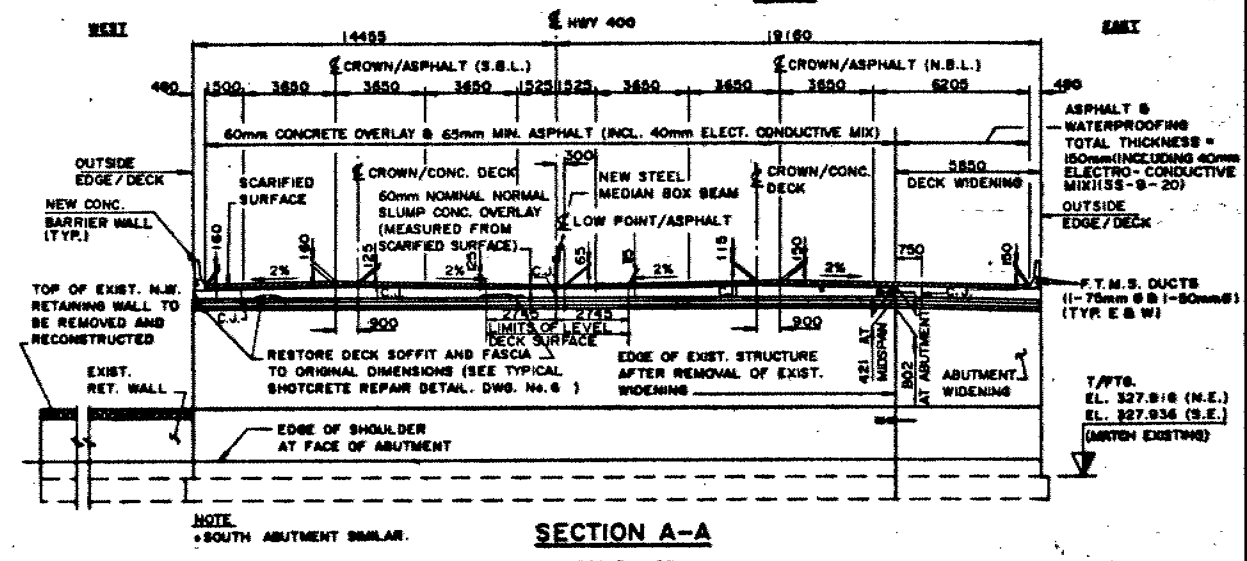
DECK POURING SEQUENCE
* SEE PLAN - NEW CONSTRUCTION.
☐ DENOTES DECK POURING SEQUENCE.
* STAGE 2 CONCRETE SHALL NOT BE PLACED UNTIL FALSEWORK FROM STAGE 1 POUR HAS BEEN REMOVED.

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHWISE SHOWN

DISTRICT No. 6 CONT No WP No 95-85-02	
KING TWP. RD. 15/16 OVERPASS BRIDGE REPAIR & WIDENING HWY. 400 GENERAL ARRANGEMENT	SHEET

totten sims hubicki associates
ENGINEERS ARCHITECTS AND PLANNERS



SECTION A-A

SCALE 1:150

GENERAL NOTES

CLASS OF CONCRETE

- ALL CONCRETE 30MPa

REINFORCING STEEL

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

CLEAR COVER TO REINFORCING STEEL

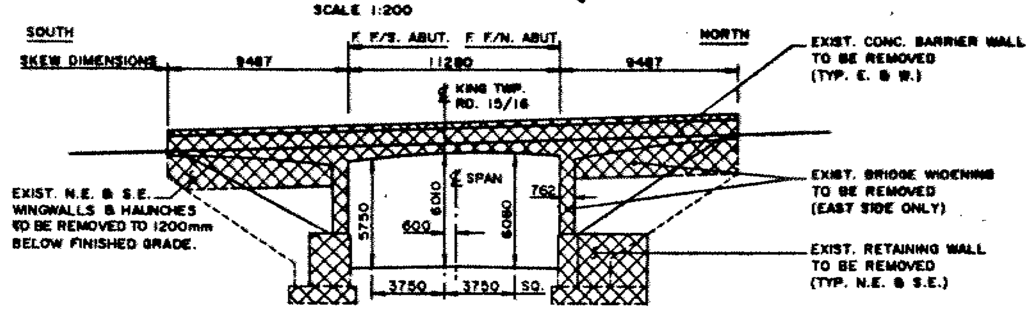
- FOOTINGS . . . . 100 ±25mm
- ABUTMENTS, WINGWALLS
  - FRONT FACE . . . . 80 ±20mm
  - BACK FACE . . . . 70 ±20mm
- DECK
  - TOP . . . . 70 ±20mm
  - BOTTOM . . . . 50 ±10mm
- REMAINDER (UNLESS NOTED) . . . . 70 ±20mm

CONSTRUCTION

- BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATIONS BE GREATER THAN 500mm.
- SAWCUTS IN CONCRETE, WHERE DESIGNATED, SHALL BE 25mm DEEP OR TO FIRST LAYER OF REINFORCING STEEL, WHICHEVER IS LESS.
- FOR CONSTRUCTION STAGING SEE DWG. No.3

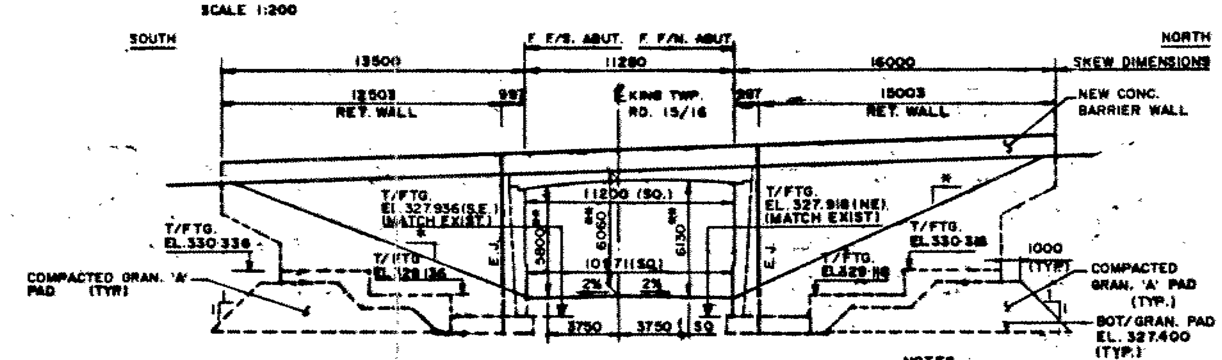
NOTE TO CONTRACTOR

- CONTRACTOR TO CHECK ALL RELEVANT STRUCTURE DIMENSIONS & ELEVATIONS SHOWN ON THE DRAWINGS AND ADJUST DIMENSIONS & ELEVATIONS AS REQUIRED TO MATCH EXISTING STRUCTURE AS APPROVED BY THE ENGINEER.



EAST ELEVATION - EXISTING BRIDGE

SCALE 1:200



EAST ELEVATION NEW CONSTRUCTION

SCALE 1:200

NOTES

- SOFTENED OR ORGANIC MATERIAL BELOW THEORETICAL BOTTOM OF GRANULAR PAD SHALL BE EXCAVATED & BACKFILLED WITH COMPACTED GRANULAR 'A' AS DIRECTED BY THE ENGINEER.
- # DENOTES SLOPE VARIES TO SUIT WALL LENGTH (2 MIN. HORIZ. : 1 VERT.)
- ## DENOTES VERTICAL CLEARANCES AT EAST EDGE OF NEW BRIDGE WIDENING

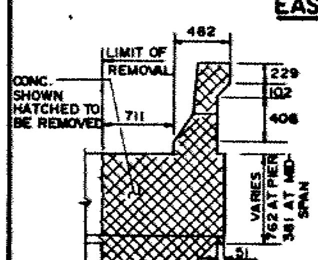
LIST OF DRAWINGS

APPLICABLE STANDARD DRAWINGS

DD-3503
DD-3504
DD-3516
DD-3922
DD-3952
DD-4670

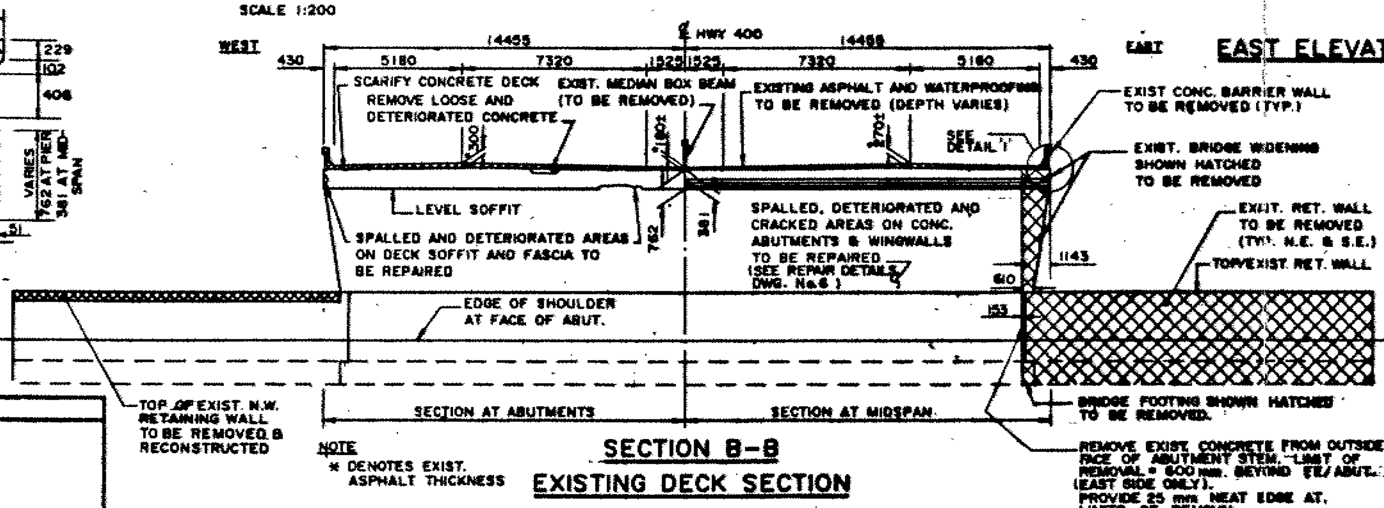
REFERENCE DRAWINGS

- DETAILS OF THE EXISTING STRUCTURE HAVE BEEN OBTAINED FROM DRAWINGS OF THE KING TOWNSHIP OVERPASS, TORONTO - BARRIE HIGHWAY, PREPARED BY THE DEPARTMENT OF HIGHWAYS - ONTARIO, DATED MAY, 1948, AND THE DRAWINGS OF THE HIGHWAY 400 WIDENING - KING TOWNSHIP OVERPASS, PREPARED BY M.M. DILLON, LIMITED, CONSULTING ENGINEERS, JANUARY 1971.



DETAIL 'I'
REMOVALS FROM EXISTING

N.T.S.



NOTE
* DENOTES EXIST. ASPHALT THICKNESS

SECTION B-B
EXISTING DECK SECTION

SCALE 1:150

B.M. 337.066
GEODETIC DATUM
CC ON N.W. CONC. RAILING
14.3 LT., 16+186.0

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		
DATE	BY	DESCRIPTION

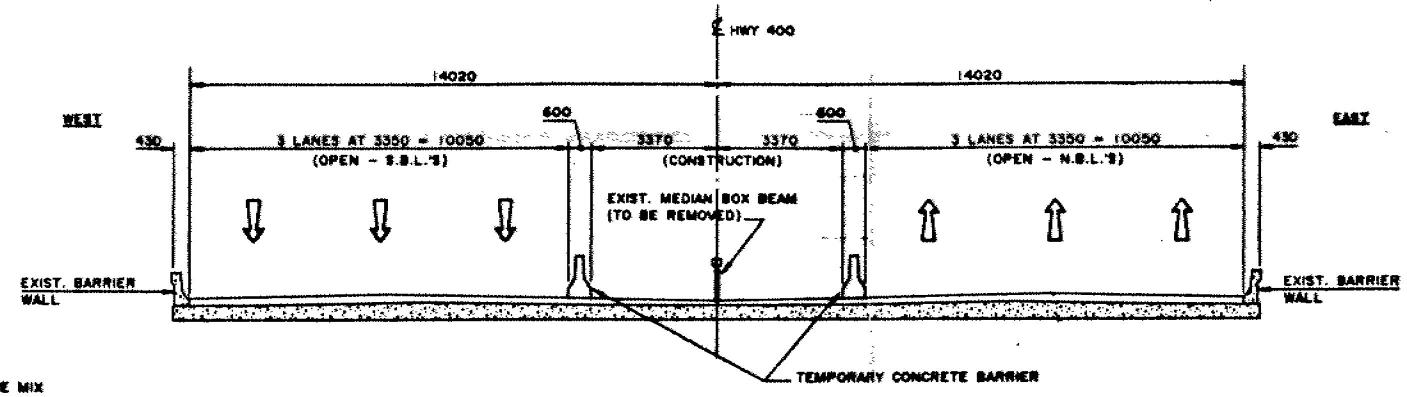
DESIGN G.O.L.	CHK M.G.S.	CODE OHBDC-83	LOAD CLASS 'A'	DATE JAN. '91	
DRAWN C.O.T.	CHK G.L.A.	SITE 37-59/R2	STRUCT	SCHEME	DWG 1

METRIC

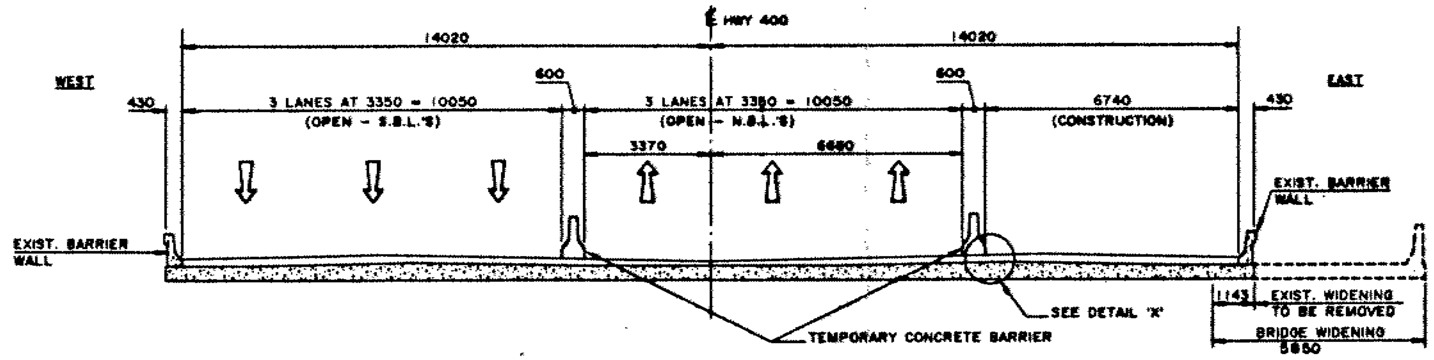
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No	
WP No	95-85-02
KING TWP. RD. 15/16 OVERPASS BRIDGE REPAIR & WIDENING HWY. 400 CONSTRUCTION STAGING	SHEET

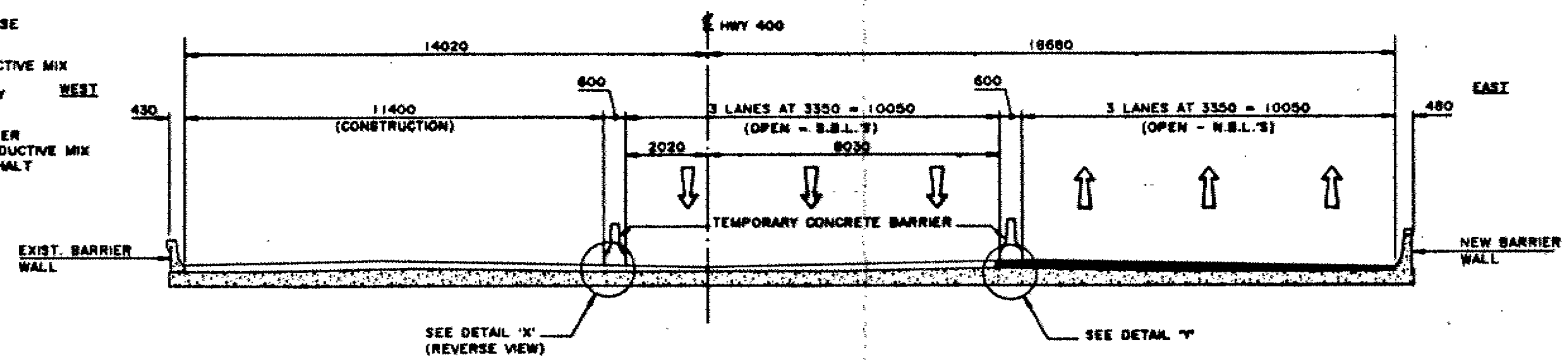
totten sims hubicki associates
ENGINEERS ARCHITECTS AND PLANNERS



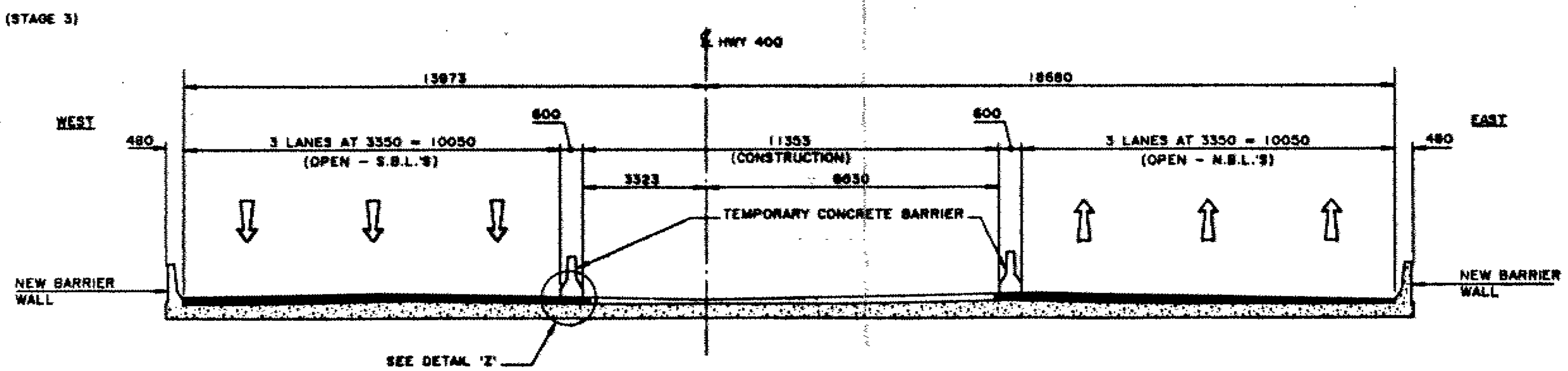
STAGE 1



STAGE 2



STAGE 3



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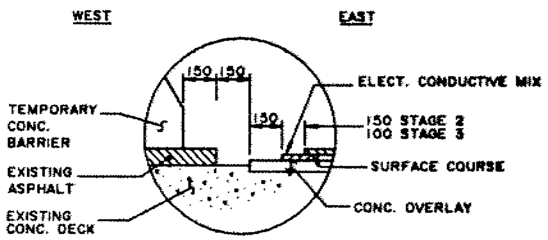
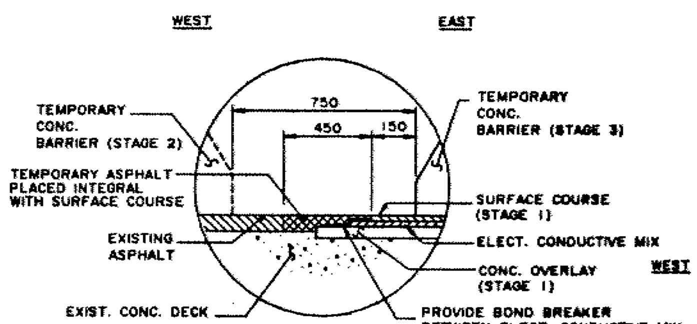
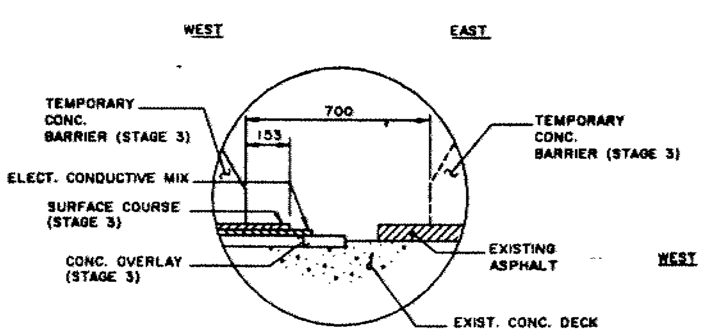


DETAIL 'X'
N.T.S.



DETAIL 'Y'
N.T.S.



DETAIL 'Z'
N.T.S.

SUGGESTED CONSTRUCTION SEQUENCE

STAGE 1

1. MILL ASPHALT ON EXISTING BRIDGE DECK AND APPROACHES TO FINAL CROSS-SECTIONAL PROFILE.
2. INSTALL TEMPORARY CONCRETE BARRIERS.
3. REMOVE EXISTING MEDIAN BOX BEAM AND CONSTRUCT DETOURS.

STAGE 2

1. RELOCATE EAST LINE OF TEMPORARY CONCRETE BARRIER, AND REDIRECT NORTHBOUND TRAFFIC TO MEDIAN DETOUR.
2. REMOVE EXISTING ASPHALT AND WATERPROOFING ON BRIDGE DECK.
3. REMOVE EXISTING BRIDGE WIDENING AT DECK AND OVER WINGWALLS, INCLUDING CONCRETE BARRIER WALL & CONCRETE COUNTERWEIGHTS.
4. INSTALL ROADWAY PROTECTION.
5. REMOVE EXISTING CONCRETE RETAINING WALLS.
6. CONSTRUCT NEW BRIDGE WIDENING, (SEE DECK POURING SEQUENCE, DWG No. 1) INCLUDING BACKFILL & CONCRETE BARRIER WALLS.
7. SCARIFY EXISTING DECK AND REMOVE LOOSE AND DETERIORATED CONCRETE.
8. PLACE NEW NORMAL SLUMP CONCRETE OVERLAY.
9. INSTALL CATHODIC PROTECTION SYSTEM OVER EXISTING DECK.
10. REMOVE ROADWAY PROTECTION AND CONSTRUCT APPROACH WIDENING. REMOVE A PORTION OF THE WINGWALLS.
11. WATERPROOF NEW BRIDGE DECK.
12. PAVE BRIDGE DECK AND APPROACHES.

STAGE 3

1. RELOCATE TEMPORARY CONCRETE BARRIERS AND REDIRECT NORTHBOUND TRAFFIC TO EAST SIDE OF BRIDGE AND SOUTHBOUND TRAFFIC TO MEDIAN DETOUR.
2. REMOVE EXISTING CONCRETE BARRIER WALL.
3. REMOVE EXISTING ASPHALT AND WATERPROOFING ON BRIDGE DECK.
4. SCARIFY EXISTING DECK AND REMOVE LOOSE AND DETERIORATED CONCRETE.
5. CONSTRUCT NEW CONCRETE BARRIER WALL.
6. PLACE NEW NORMAL SLUMP CONCRETE OVERLAY.
7. INSTALL CATHODIC PROTECTION SYSTEM.
8. PAVE THE BRIDGE DECK AND APPROACHES.

STAGE 4

1. RELOCATE WEST LINE OF TEMPORARY CONCRETE BARRIER. MAINTAIN NORTHBOUND TRAFFIC ON EAST SIDE OF BRIDGE AND DIVERT SOUTHBOUND TRAFFIC TO WEST SIDE OF BRIDGE.
2. REMOVE EXISTING ASPHALT AND WATERPROOFING ON BRIDGE DECK.
3. SCARIFY EXISTING DECK AND REMOVE LOOSE AND DETERIORATED CONCRETE.
4. PLACE NEW NORMAL SLUMP CONCRETE OVERLAY.
5. INSTALL CATHODIC PROTECTION SYSTEM.
6. RECONSTRUCT MEDIAN BARRIER ON BRIDGE DECK.
7. PAVE THE BRIDGE DECK AND APPROACHES.
9. REMOVE TEMPORARY CONCRETE BARRIERS AND RESTORE TRAFFIC TO ORIGINAL NORTHBOUND AND SOUTHBOUND LANES.

MISCELLANEOUS

REPAIR NORTH-WEST RETAINING WALL, ABUTMENT FACES, DECK SOFFIT AND FASCIA.

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN M.G.S.	CHK MGS	CODE OHBDC-83	LOAD CLASS A	DATE JAN., 91	
DRAWN P.S.H.	CHK G.L.A.	SITE 37-59/R2	STRUCT	SCHEME	DWG 3

METRIC

DIMENSIONS ARE IN METRES AND OR MILLIMETRES UNLESS OTHERWISE SHOWN

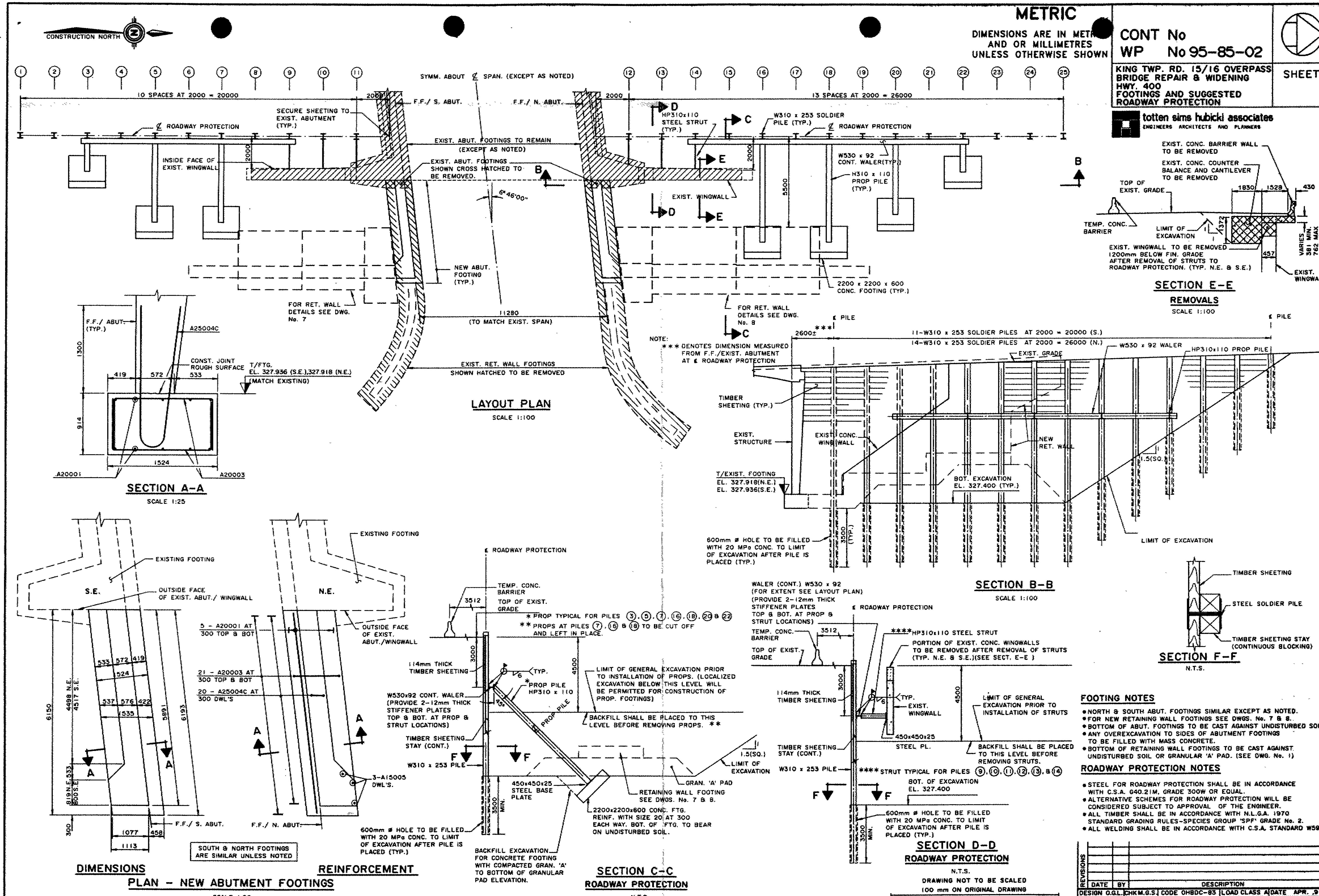
CONT No

WP No 95-85-02

KING TWP. RD. 15/16 OVERPASS
BRIDGE REPAIR & WIDENING
HWY. 400
FOOTINGS AND SUGGESTED
ROADWAY PROTECTION

SHEET

totten sims hubicki associates
ENGINEERS ARCHITECTS AND PLANNERS



FOOTING NOTES

- NORTH & SOUTH ABUT. FOOTINGS SIMILAR EXCEPT AS NOTED.
- FOR NEW RETAINING WALL FOOTINGS SEE DWGS. No. 7 & 8.
- BOTTOM OF ABUT. FOOTINGS TO BE CAST AGAINST UNDISTURBED SOIL.
- ANY OVEREXCAVATION TO SIDES OF ABUTMENT FOOTINGS TO BE FILLED WITH MASS CONCRETE.
- BOTTOM OF RETAINING WALL FOOTINGS TO BE CAST AGAINST UNDISTURBED SOIL OR GRANULAR 'A' PAD. (SEE DWG. No. 1)

ROADWAY PROTECTION NOTES

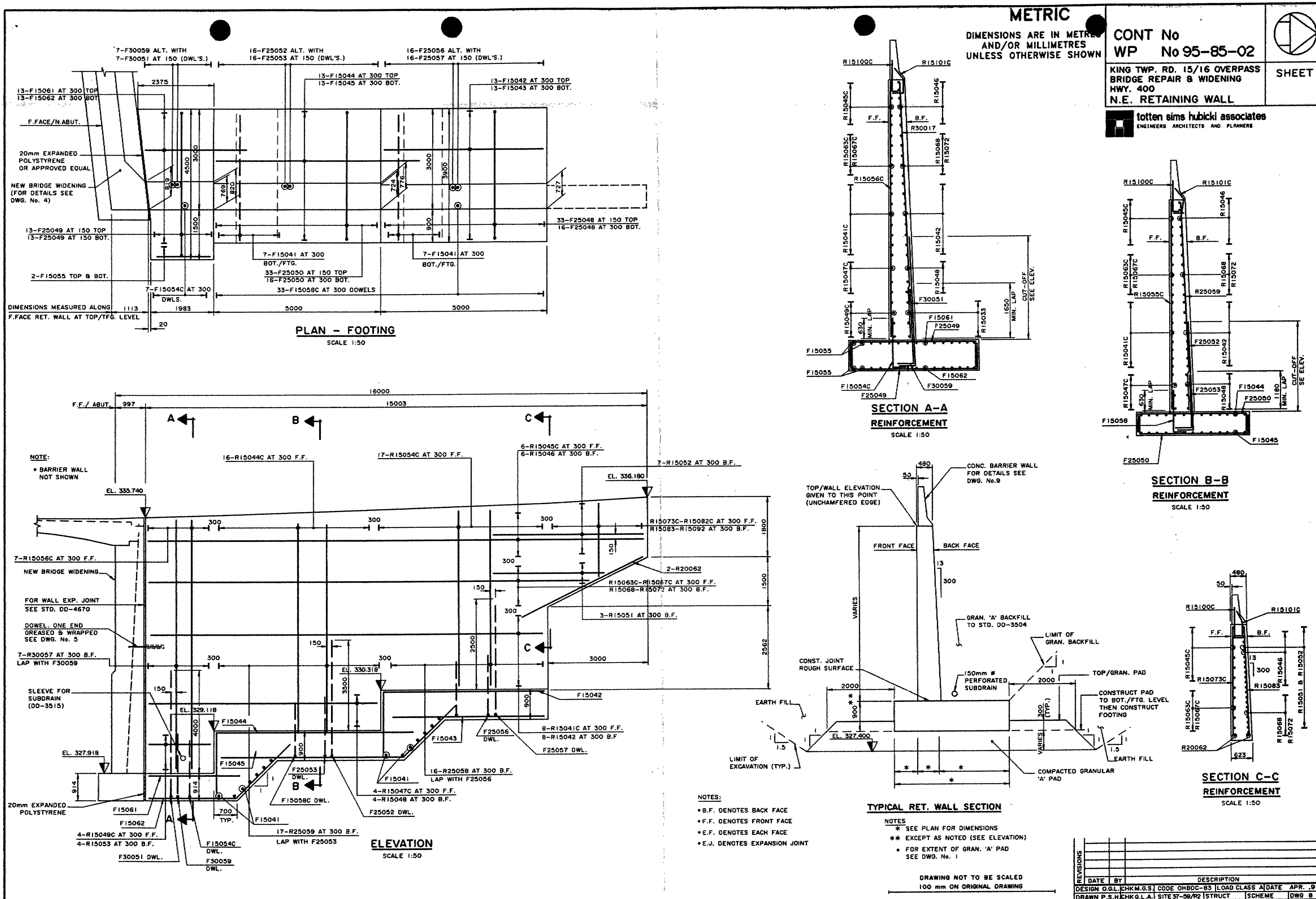
- STEEL FOR ROADWAY PROTECTION SHALL BE IN ACCORDANCE WITH C.S.A. G40.21M, GRADE 300W OR EQUAL.
- ALTERNATIVE SCHEMES FOR ROADWAY PROTECTION WILL BE CONSIDERED SUBJECT TO APPROVAL OF THE ENGINEER.
- ALL TIMBER SHALL BE IN ACCORDANCE WITH N.L.G.A. 1970 STANDARD GRADING RULES-SPECIES GROUP 'SPF' GRADE No. 2.
- ALL WELDING SHALL BE IN ACCORDANCE WITH C.S.A. STANDARD W59.

DRAWING NOT TO BE SCALED

100 mm ON ORIGINAL DRAWING

REVISIONS DATE	BY	DESCRIPTION

DESIGN O.G.L.	CHK M.G.S.	CODE OHBDC-83	LOAD CLASS A	DATE APR. 91
DRAWN P.S.H.	CHK G.L.A.	SITE 37-95/RI	STRUCT	SCHEME	DWG 4



METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN
CONT No
WP No 95-85-02
SHEET
KING TWP. RD. 15/16 OVERPASS
BRIDGE REPAIR & WIDENING
HWY. 400
N.E. RETAINING WALL
totten sims hubicki associates
ENGINEERS ARCHITECTS AND PLANNERS
PLAN - FOOTING
SCALE 1:50
7-F30059 ALT. WITH 7-F30051 AT 150 (DWL'S.)
16-F25052 ALT. WITH 16-F25053 AT 150 (DWL'S.)
16-F25056 ALT. WITH 16-F25057 AT 150 (DWL'S.)
13-F15044 AT 300 TOP
13-F15045 AT 300 BOT.
13-F15042 AT 300 TOP
13-F15043 AT 300 BOT.
13-F15061 AT 300 TOP
13-F15062 AT 300 BOT
F.FACE/N.ABUT.
20mm EXPANDED POLYSTYRENE OR APPROVED EQUAL
NEW BRIDGE WIDENING (FOR DETAILS SEE DWG. No. 4)
13-F25049 AT 150 TOP
13-F25049 AT 150 BOT.
2-F15055 TOP & BOT.
7-F15041 AT 300 BOT./FTG.
33-F25050 AT 150 TOP
16-F25050 AT 300 BOT.
33-F25048 AT 150 TOP
16-F25048 AT 300 BOT.
7-F15054C AT 300 DWLS.
33-F15058C AT 300 DOWELS
DIMENSIONS MEASURED ALONG F.FACE RET. WALL AT TOP/TFG. LEVEL
ELEVATION
SCALE 1:50
NOTE:
* BARRIER WALL NOT SHOWN
F.F./ ABUT.
EL. 335.740
EL. 336.180
EL. 330.318
EL. 329.118
EL. 327.918
16-R15044C AT 300 F.F.
17-R15054C AT 300 F.F.
6-R15045C AT 300 F.F.
6-R15046 AT 300 B.F.
7-R15052 AT 300 B.F.
R15073C-R15082C AT 300 F.F.
R15083-R15092 AT 300 B.F.
2-R20062
R15063C-R15067C AT 300 F.F.
R15068-R15072 AT 300 B.F.
3-R15051 AT 300 B.F.
7-R15056C AT 300 F.F.
NEW BRIDGE WIDENING
FOR WALL EXP. JOINT SEE STD. DD-4670
DOWEL. ONE END GREASED & WRAPPED SEE DWG. No. 5
7-R30057 AT 300 B.F. LAP WITH F30059
SLEEVE FOR SUBDRAIN (DD-3515)
20mm EXPANDED POLYSTYRENE
8-R15041C AT 300 F.F.
8-R15042 AT 300 B.F
F25057 DWL.
16-R25058 AT 300 B.F. LAP WITH F25056
4-R15047C AT 300 F.F.
4-R15048 AT 300 B.F.
F25052 DWL.
17-R25059 AT 300 B.F. LAP WITH F25053
4-R15049C AT 300 F.F.
4-R15053 AT 300 B.F.
F30051 DWL.
F15054C DWL.
F30059 DWL.
F15058C DWL.
SECTION A-A
REINFORCEMENT
SCALE 1:50
SECTION B-B
REINFORCEMENT
SCALE 1:50
SECTION C-C
REINFORCEMENT
SCALE 1:50
TYPICAL RET. WALL SECTION
CONC. BARRIER WALL FOR DETAILS SEE DWG. No.9
TOP/WALL ELEVATION GIVEN TO THIS POINT (UNCHAMFERED EDGE)
FRONT FACE
BACK FACE
GRAN. 'A' BACKFILL TO STD. DD-3504
LIMIT OF GRAN. BACKFILL
CONST. JOINT ROUGH SURFACE
150mm ⌀ PERFORATED SUBDRAIN
TOP/GRAN. PAD
CONSTRUCT PAD TO BOT./FTG. LEVEL THEN CONSTRUCT FOOTING
EARTH FILL
LIMIT OF EXCAVATION (TYP.)
COMPACTED GRANULAR 'A' PAD
EL. 327.400
NOTES:
* B.F. DENOTES BACK FACE
* F.F. DENOTES FRONT FACE
* E.F. DENOTES EACH FACE
* E.J. DENOTES EXPANSION JOINT
NOTES
* SEE PLAN FOR DIMENSIONS
** EXCEPT AS NOTED (SEE ELEVATION)
* FOR EXTENT OF GRAN. 'A' PAD SEE DWG. No. 1
DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING
REVISIONS
DATE BY DESCRIPTION
DESIGN O.G.L. CHK M.G.S. CODE OHBDC-83 LOAD CLASS A DATE APR. 91
DRAWN P.S.H CHK G.L.A. SITE 37-59/R2 STRUCT SCHEME DWG 8