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STR. SITE No. 37-95

HWY. No. 400

LOCATION Hwy 400 & Kirby Side Rd.
Overpass

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

CONT 92-95

WP 95-85-01

DIST 6

HWY 400

STR SITE 37-95

Proposed Extension of
Kirby Side Road Overpass

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FOUNDATION INVESTIGATION REPORT

For

Proposed Extension of

Kirby Side Road Overpass

W.P. 95-85-01, Site No. 37-95

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 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/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 Wisconsin 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 maximum 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.

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

	<u>Range</u>	<u># of Tests</u>
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.

SILT

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 in the Appendix. 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.

DISCUSSION AND RECOMMENDATIONS

It is proposed to widen the existing rigid frame structure at the Kirby Side Road - 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 Kirby Road will either be replaced with new slope toe retaining walls or eliminated by increasing the length of the 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 wingwall and retaining wall parallel to Hwy. 400.

The existing rigid frame structure, approximately 30.5 m in width and 11.3 m in length, was constructed in 1950. Although the superstructure reveals evidence of concrete spalling and delamination, concrete shrinkage cracking and also corroded exposed rebar, there appears to be no visible signs of distress caused by an unsound substructure. The approach fills also seem to be performing satisfactorily.

The structures' closed-type abutment are founded on spread footings within the surficial clayey silt deposit at an elevation of approximately 249.7 m. This corresponds to approximately 2 m below the natural ground surface. The footing width is approximately 4 m. The existing forward toe retaining walls are approximately 3.75 m in length and taper in height from 0.6 m to 0.3 m. These retaining walls are also founded on spread footings at the same elevation in the clayey silt deposit.

The existing alignments and profile grades of Hwy. 400 and Kirby Side Road will be maintained without any changes. The profile grade of Hwy. 400 at the structure is approximately 257.5 m whilst the profile grade of Kirby Side Road is approximately 251.5 m. Consequently, existing approach fills in the order of magnitude of 6 m will also be required for the extended approaches.

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 below. It can be observed from the table that the new footings will be founded at the same elevation as the existing spread footings. The underside of all footings should be provided with a minimum 1.2 m of earth cover for frost protection.

Table 2 - Structure Foundations

<u>Structure</u>	<u>Founding Elevation (m)</u>	<u>Factored Capacity at U.L.S. (kPa)</u>	<u>Bearing Capacity at S.L.S. Type II (kPa)</u>
Abutments	<249.7	375	250
Retaining Walls	<249.7	375	250

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 should 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. If necessary, sliding resistance can be supplemented by constructing shear keys in the founding soil below the base of the footings.

Construction of the 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 contain 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.

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:

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Angle of Internal Friction (ϕ)	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_0)*	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 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 50 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 placement.

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 system 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

<u>Soil</u>	<u>Elevation (m)</u>	<u>Saturated Unit Weight (kN/m³)</u>	<u>Effective Shear Strength Parameters ϕ°</u>
Clayey Silt	258-244	22.5	30
Silt	<244	20	30

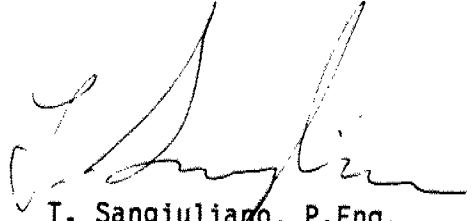
Earth pressures shall be computed in accordance with section 6.6.1. of the O.H.B.D.C. Appropriate consideration for sloping soil surfaces shall be included in the design. Buoyant unit weights of soil shall be employed below the prevailing groundwater table.

The soldier piles can be installed in preaugered holes or driven using conventional pile driving methods. For preaugered holes advanced in the silt stratum, in which soil ravelling can be anticipated, the method selected shall ascertain that the shaft of the holes are protected against caving. One method of achieving this is by installing a steel liner and constructing the caisson within the steel liner. After the liner has been cleaned out and the required reinforcing installed, the concrete should be placed in the dry. Alternatively mud drilling techniques may be required. The method of installation selected by the Contractor shall be in accordance with OPSS 903.07.03 and OPSS 538-539 and subject to review by this office.

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.

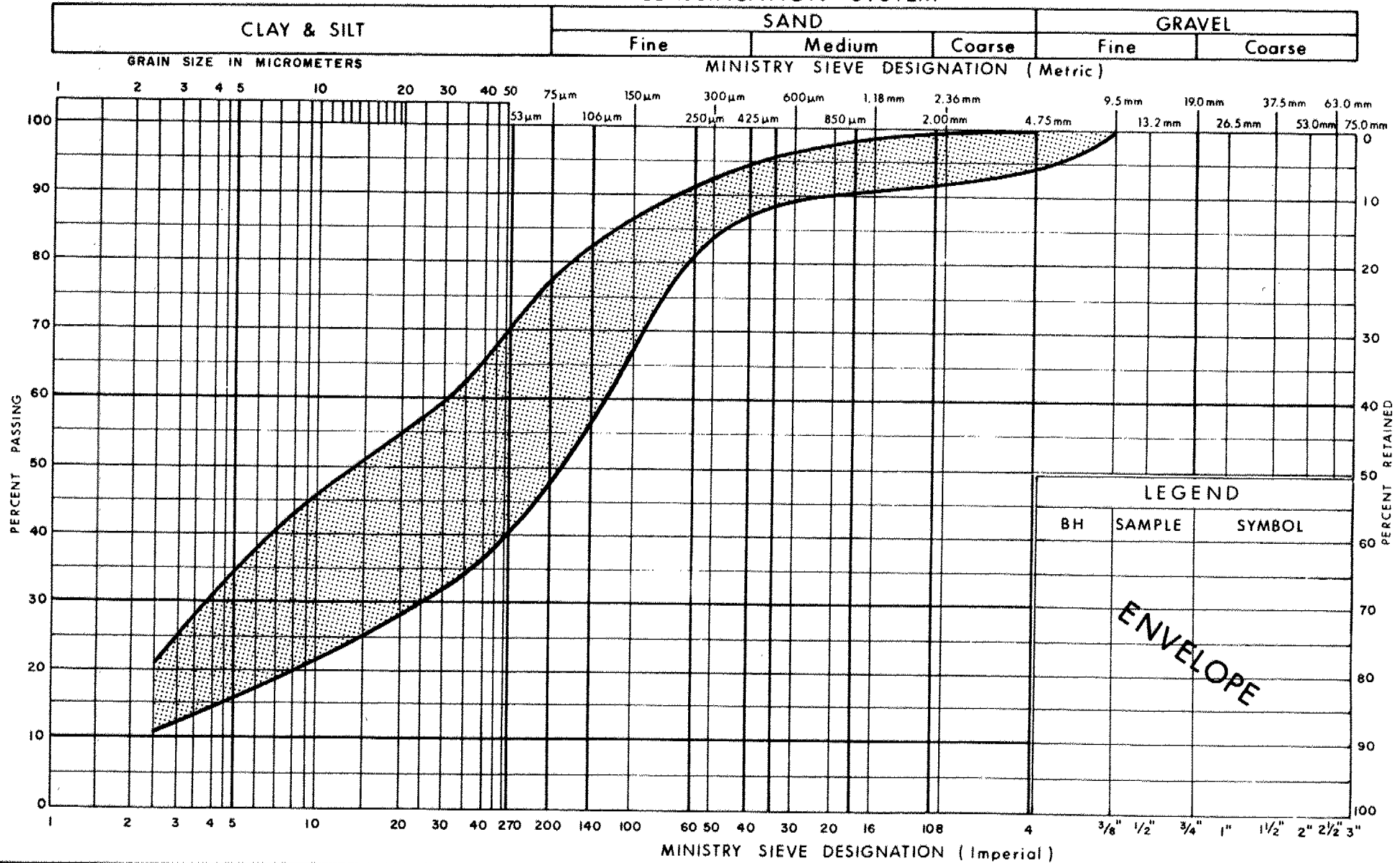

T. Sangiuliano, P.Eng.
Foundation Engineer



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

APPENDIX

UNIFIED SOIL CLASSIFICATION SYSTEM

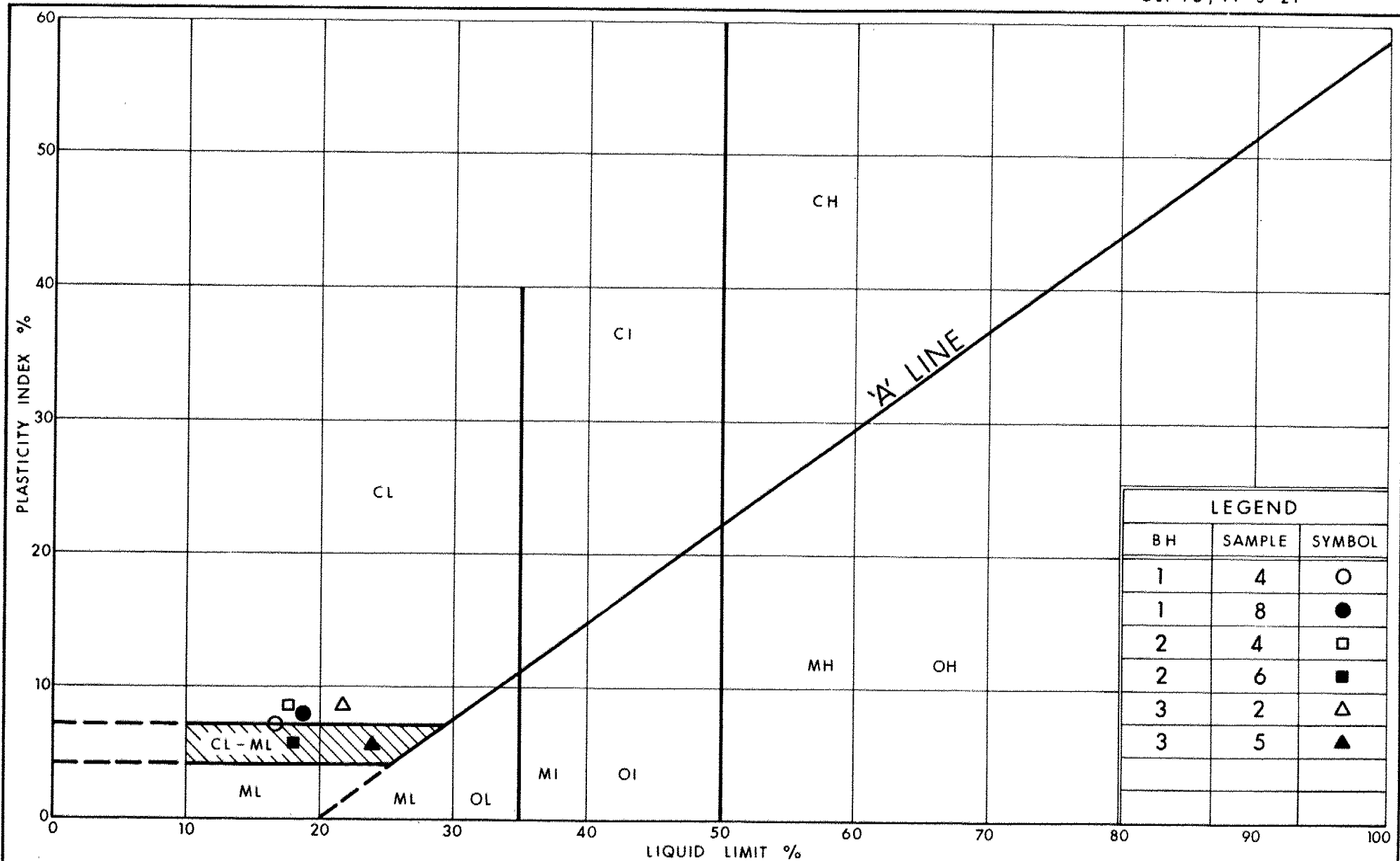


Ministry of
Transportation

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

FIG No 1

W P 95-85-01



Ministry of
Transportation

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

FIG No 2

W P 95-85-01

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 63.5 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N}

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

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

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
251.7 0.0	Ground Surface												
	Clayey Silt Some Sand, Trace Gravel (Glacial Till)		1	SS	9								
	Grey		2	SS	14								
			3	SS	12								
			4	SS	12								
	Stiff V. Stiff		5	SS	12								
			6	SS	25								
			7	SS	24								
			8	SS	24								
243.9 7.8	Silt Some Sand, Trace Gravel		9	SS	7								
	Grey, Compat to Very Dense		10	SS	15								
			11	SS	20								
239.1 12.6	End of Borehole		12	SS	55								

+³, x⁵; Numbers refer to 20
Sensitivity 15 ϕ 5 (%) STRAIN AT FAILURE
10

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

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L	WATER CONTENT (%) W 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES							
251.9	Ground Surface											
0.0	Sand Trace Gravel (Fill)		1	SS	5							
250.1	Brown, Loose		2	SS	6							
1.8	Clayey Silt Some Sand, Trace Gravel (Glacial Till)		3	SS	8							
			4	SS	12							
			5	SS	13							
	Grey Firm to V. Stiff		6	SS	22							
	Occ. Sand Seams		7	SS	16							
			8	SS	22							
244.3												
7.6	Silt Trace Sand, Trace Gravel Grey Compact to Dense		9	SS	24							
			10	SS	34							
			11	SS	28							
			12	SS	18							
			13	SS	19							
236.2			14	SS	4							
15.7	End of Borehole											

RECORD OF BOREHOLE No 3

METRIC

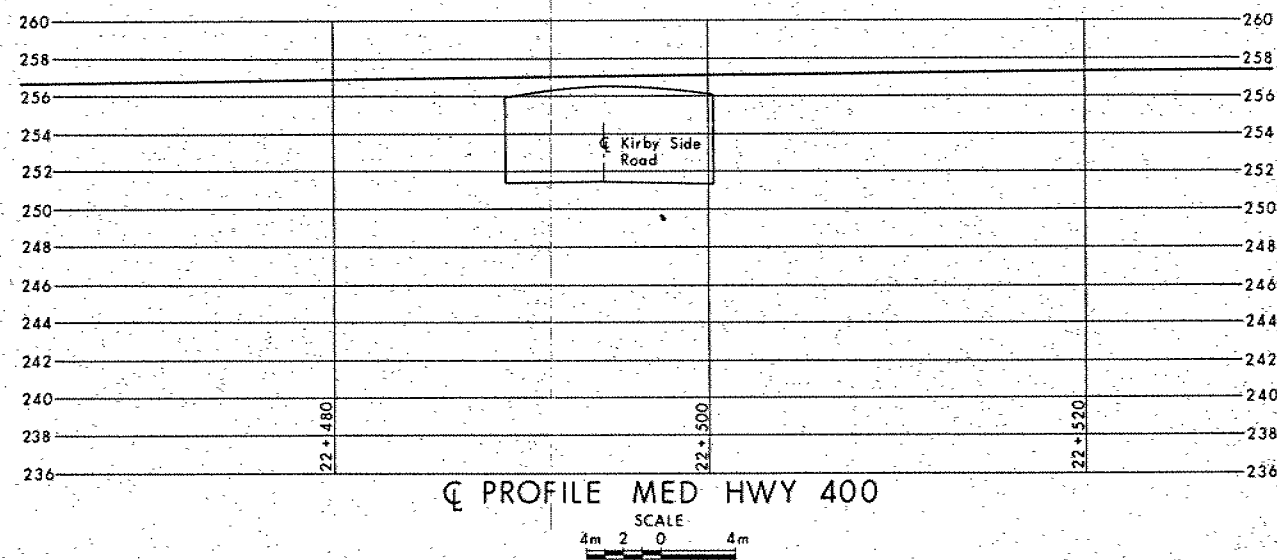
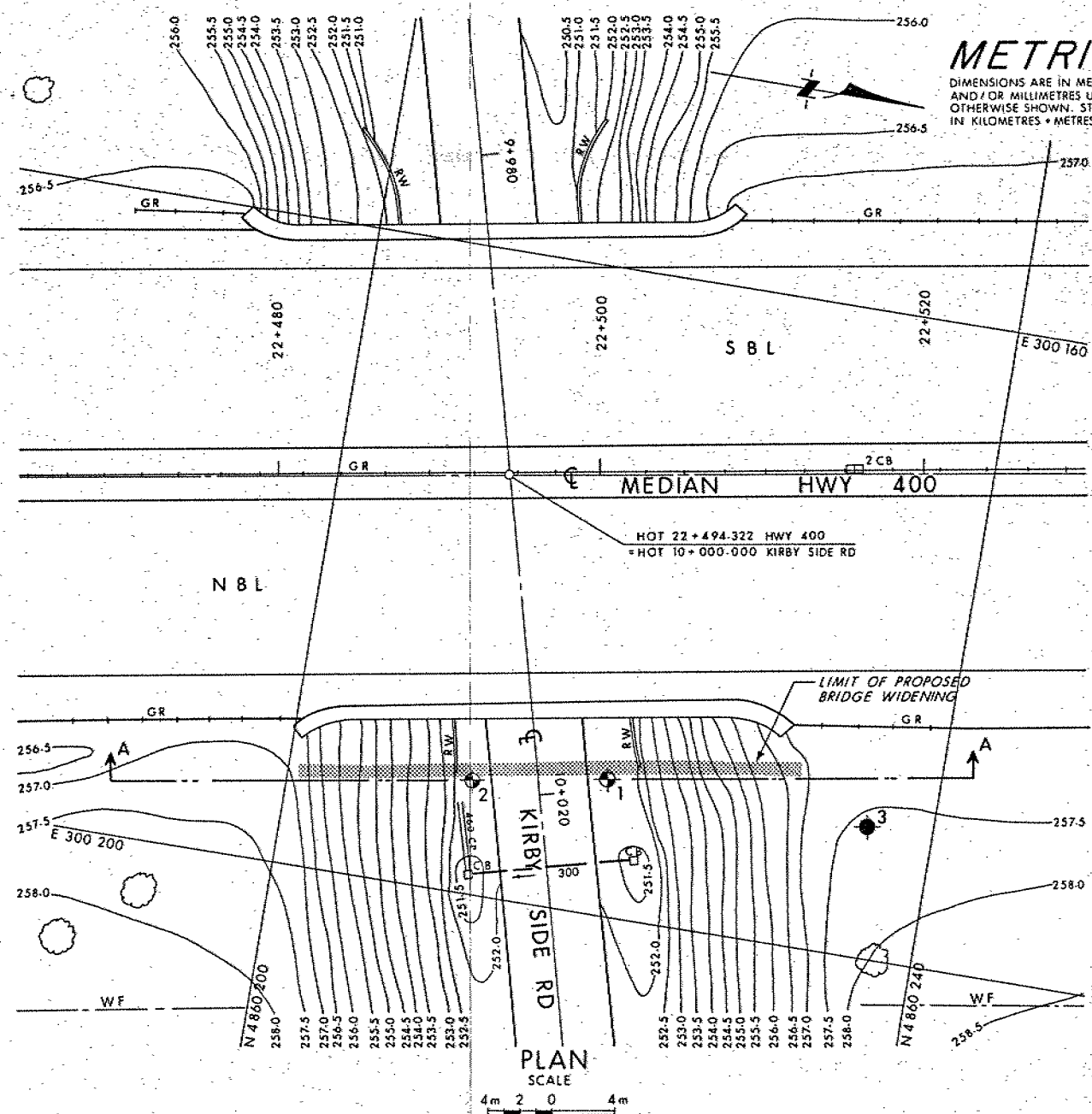
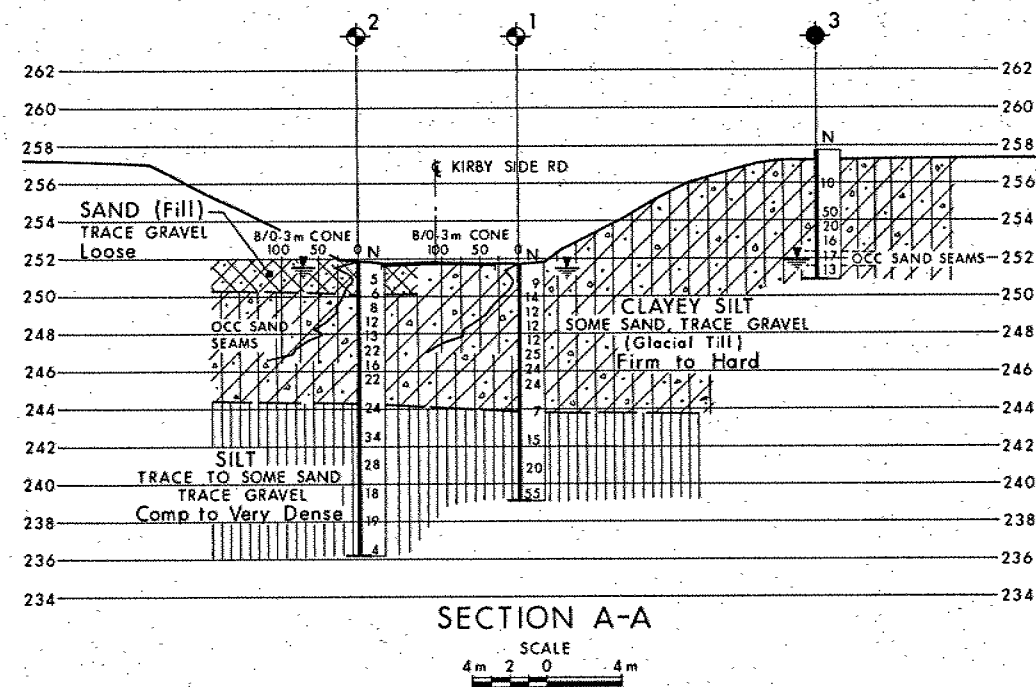
W P 95-85-01 LOCATION Co-ords: N 4 860 236.0; E 300 191.8 ORIGINATED BY AL
DIST 6 HWY 400 BOREHOLE TYPE H. S. Auger COMPILED BY AL
DATUM Geodetic DATE 89 08 27 CHECKED BY TS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
257.8	Ground Surface																
0.0	Clayey Silt Some Sand, Trace Gravel (Glacial Till)		1	SS	10		256									22.5	6 28 56 10
			2	SS	50												
	Brown		3	SS	20		254										
	Grey		4	SS	16											21.9	
	Stiff to Hard		5	SS	17												
	Occ. Sand Seams		6	SS	13		252										0 52 38 10
250.9	End of Borehole																
6.9																	

OFFICE REPORT ON SOIL EXPLORATION

+³, x⁵: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



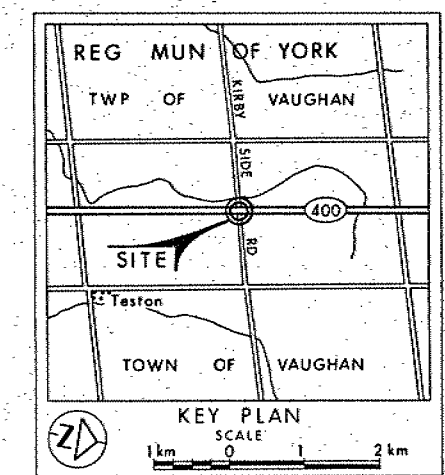
METRIC
DIMENSIONS ARE IN METRES
AND / OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 95-85-01

KIRBY SIDE RD OVERPASS

BORE HOLE LOCATIONS & SOIL STRATA

SHEET



- 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 89 08

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	251.7	4 860 219.7	300 191.5
2	251.9	4 860 211.4	300 193.0
3	257.8	4 860 236.0	300 191.8

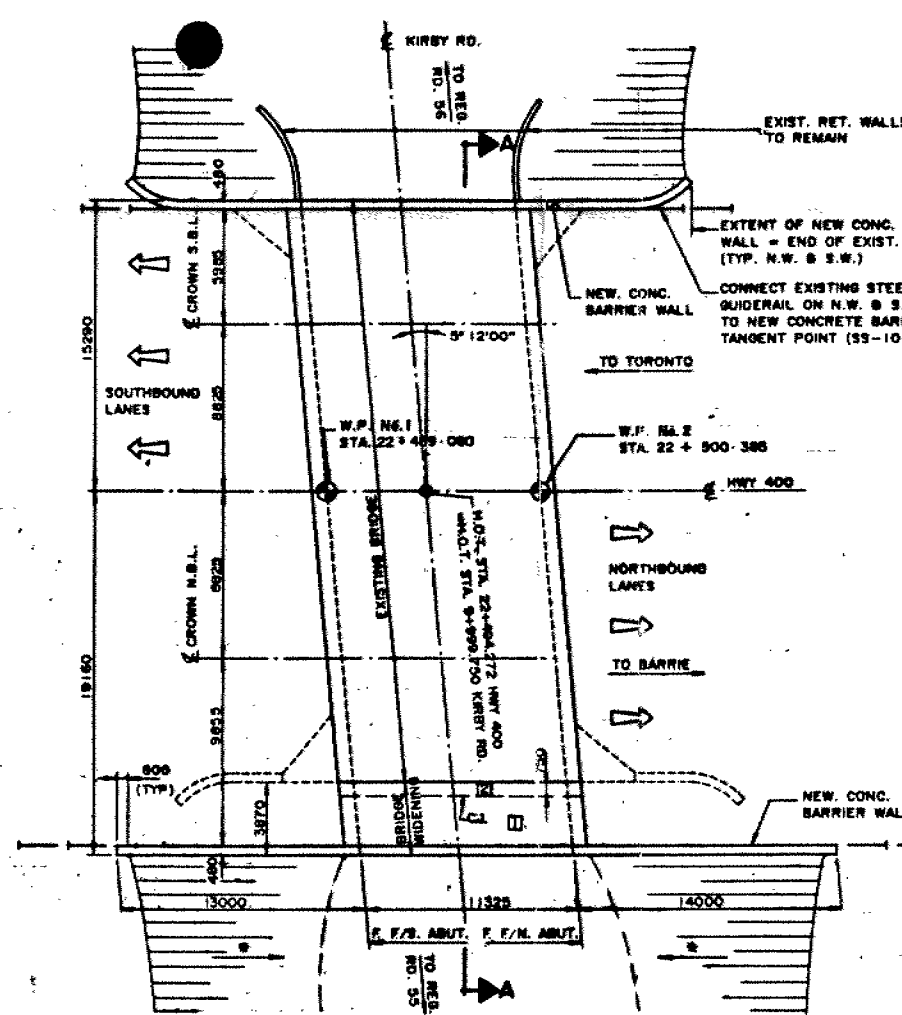
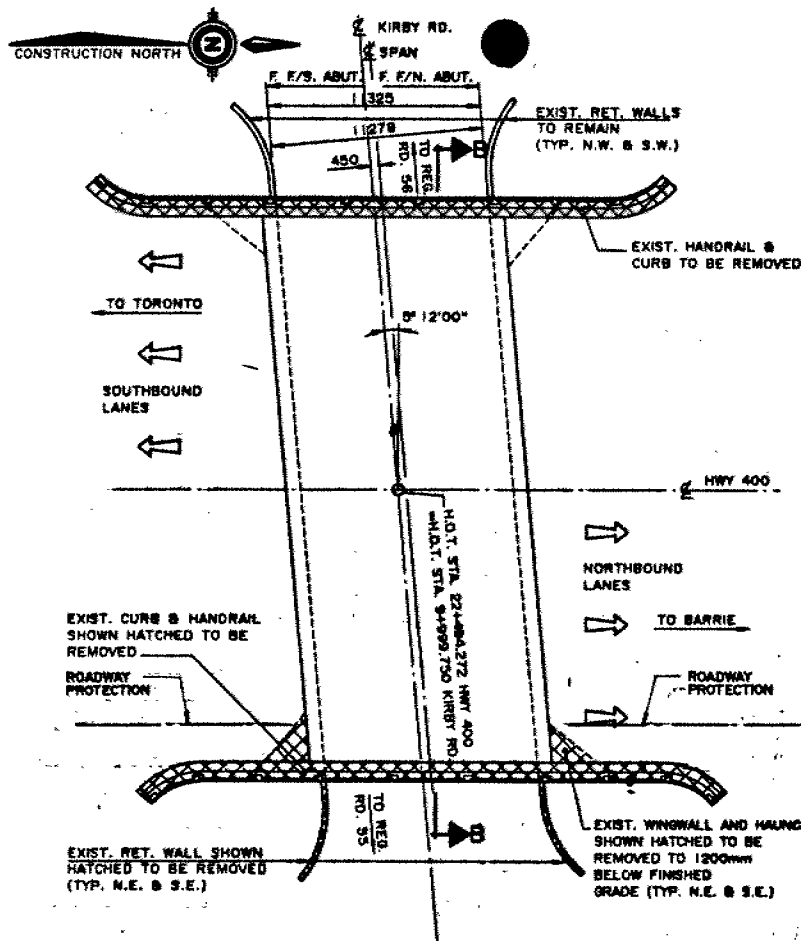
NOTE

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
1	1989 12 14	TS	DATE 1989 12 14
2	1989 12 14	TS	DATE 1989 12 14
3	1989 12 14	TS	DATE 1989 12 14

Geocres No 30M13-93	DIST 6
HWY No. 400	SITE 37-95
SUBMD T S CHECKED	DWG 958501-A
DRAWN	

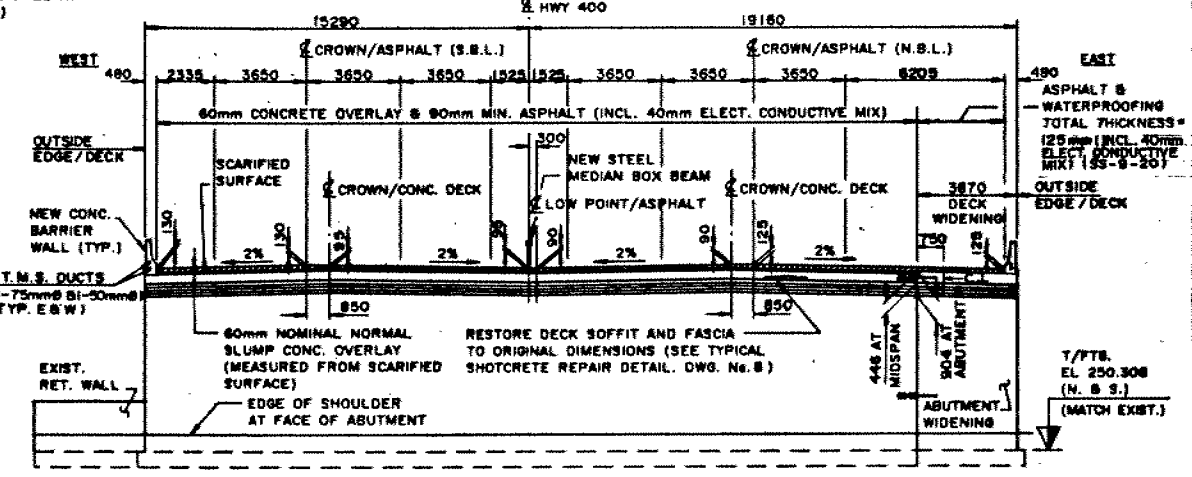


DECK POURING SEQUENCE
 • SEE PLAN - CONSTRUCTION.
 • DENOTES DECK POURING SEQUENCE.
 • STAGE 2 CONCRETE SHALL NOT BE PLACED UNTIL FALSEWORK FROM STAGE 1 POUR HAS BEEN REMOVED.

METRIC
 DIMENSIONS ARE IN METRES
 AND/OR MILLIMETRES
 UNLESS OTHERWISE SHOWN

DISTRICT No. 6
CONT No
WP No 95-85-01
KIRBY ROAD OVERPASS
BRIDGE REPAIR & WIDENING
HWY. 400
GENERAL ARRANGEMENT

totten sims hubicki associates
 ENGINEERS ARCHITECTS AND PLANNERS



NOTE
 • SOUTH ABUTMENT SIMILAR

CONNECT NEW STEEL BEAM
 GUIDERAIL ON N.E. & S.E.
 APPROACHES TO ENDS OF
 NEW CONCRETE BARRIER WALL
 (SS-10-10) (TYP.)

NOTES:
 • W.P. DENOTES WORKING POINT
 • T/A DENOTES ELEVATIONS GIVEN
 TO TOP OF ASPHALT
 • P.F. DENOTES FRONT FACE

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 125mm
• ABUTMENTS, WINGWALLS	
FRONT FACE	80 120mm
BACK FACE	70 120mm
• DECK	
TOP	70 120mm
BOTTOM	50 110mm
• REMAINDER (UNLESS NOTED)	70 120mm

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. 8.

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.

REFERENCE DRAWINGS

- DETAILS OF THE EXISTING STRUCTURE HAVE BEEN OBTAINED FROM DRAWINGS OF THE KIRBY ROAD OVERPASS, TORONTO - BARRIE HIGHWAY, PREPARED BY THE DEPARTMENT OF HIGHWAYS - ONTARIO, DATED OCTOBER, 1949.

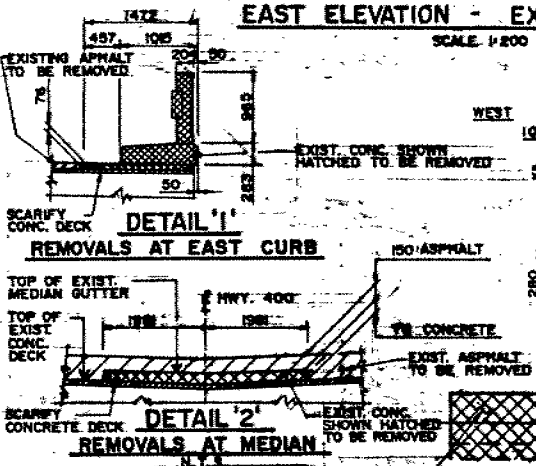
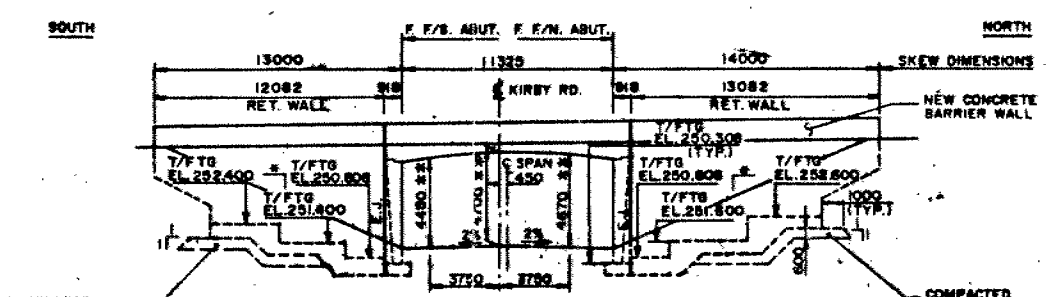
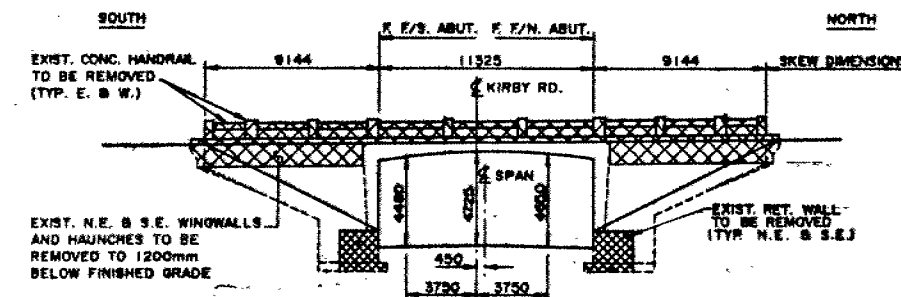
LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS AND SOIL STRATIGRAPHY
3. CONSTRUCTION STAGING
4. FOOTINGS & SUGGESTED ROADWAY PROTECTION
5. FRAME DETAILS
6. S.E. RETAINING WALL
7. N.E. RETAINING WALL
8. BRIDGE REPAIRS
9. EAST BARRIER WALL
10. WEST BARRIER WALL
11. STANDARDS
12. AS CONSTRUCTED ELEV. & DIM.
13. ELECTRICAL EMBEDDED WORK
14. CATHODIC PROTECTION LAYOUT
15. CATHODIC PROTECTION DETAILS
16. QUANTITIES - STRUCTURE
17. QUANTITIES - CATHODIC PROTECTION

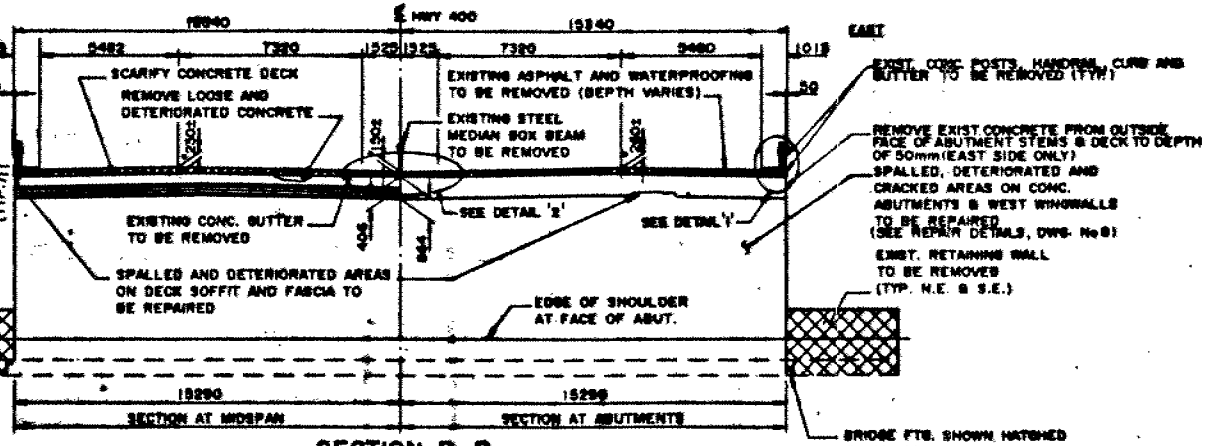
APPLICABLE STANDARD DRAWINGS

- DD - 3503
- DD - 3504
- DD - 3515
- DD - 3522
- DD - 3552
- DD - 4670

DRAWING NOT TO BE SCALED
 100 mm ON ORIGINAL DRAWING



D.M.O. B.M. 63-68
EL. 256.252
 GEODETIC DATUM
 TABLET SET HORIZONTALLY IN W. FACE
 OF S. CONG. ABUTMENT, 3.4m N OF
 S-W CORNER, 52cm BELOW COPING,
 16cm ABOVE GROUND LEVEL.
 18.5 LT. 22+483.1



NOTE
 • DENOTES EXIST. ASPHALT THICKNESS

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 95-85-01



KIRBY ROAD OVERPASS
BRIDGE REPAIR & WIDENING
HWY. 400
CONSTRUCTION STAGING

SHEET

totten sims hubicki associates
ENGINEERS ARCHITECTS AND PLANNERS

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 CONCRETE HANDRAIL POSTS, RAILS, AND CURB.
4. INSTALL ROADWAY PROTECTION.
5. REMOVE CONCRETE RETAINING WALLS.
6. REMOVE EXISTING CONCRETE FASCIA AT EAST EDGE OF BRIDGE DECK TO DEPTH OF 50mm.
7. CONSTRUCT NEW BRIDGE WIDENING (SEE DECK POURING SEQUENCE ON DWG. WP11, INCLUDING BACKFILL & CONCRETE BARRIER WALLS).
8. SCARIFY EXISTING DECK AND REMOVE LOOSE AND DETERIORATED CONCRETE.
9. PLACE NEW NORMAL SLUMP CONCRETE OVERLAY.
10. INSTALL CATHODIC PROTECTION SYSTEM OVER EXISTING DECK.
11. REMOVE ROADWAY PROTECTION AND CONSTRUCT APPROACH WIDENING.
12. REMOVE A PORTION OF THE WINGWALLS.
13. WATERPROOF NEW BRIDGE DECK.
14. PAVE BRIDGE DECK AND APPROACHES.

STAGE 3

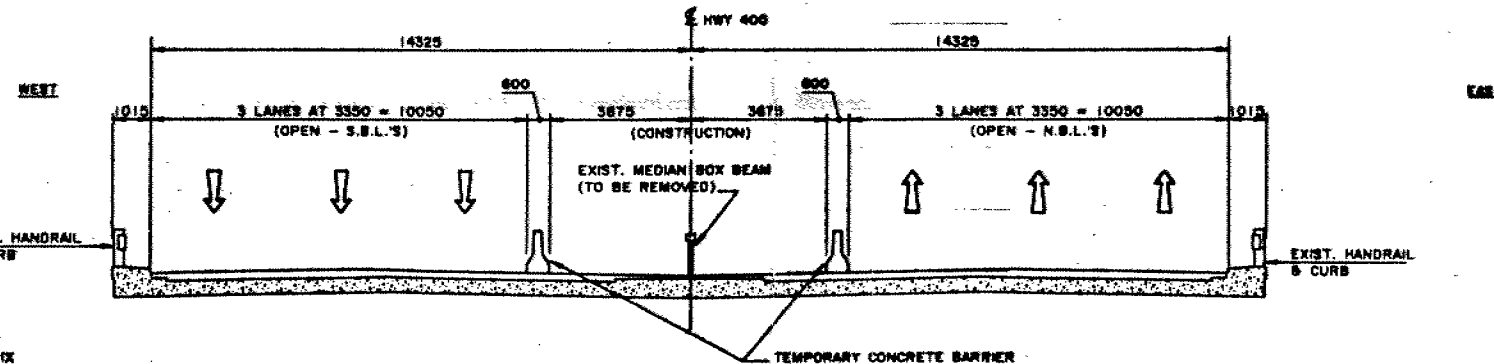
1. RELOCATE TEMPORARY CONCRETE BARRIERS AND REDIRECT NORTHBOUND TRAFFIC TO EAST SIDE OF BRIDGE AND SOUTHBOUND TRAFFIC TO MEDIAN DETOUR.
2. REMOVE EXISTING CONCRETE HANDRAIL POSTS, RAILS, AND CURB.
3. REMOVE EXISTING ASPHALT AND WATERPROOFING ON BRIDGE DECK.
4. SCARIFY EXISTING DECK AND REMOVE LOOSE AND 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

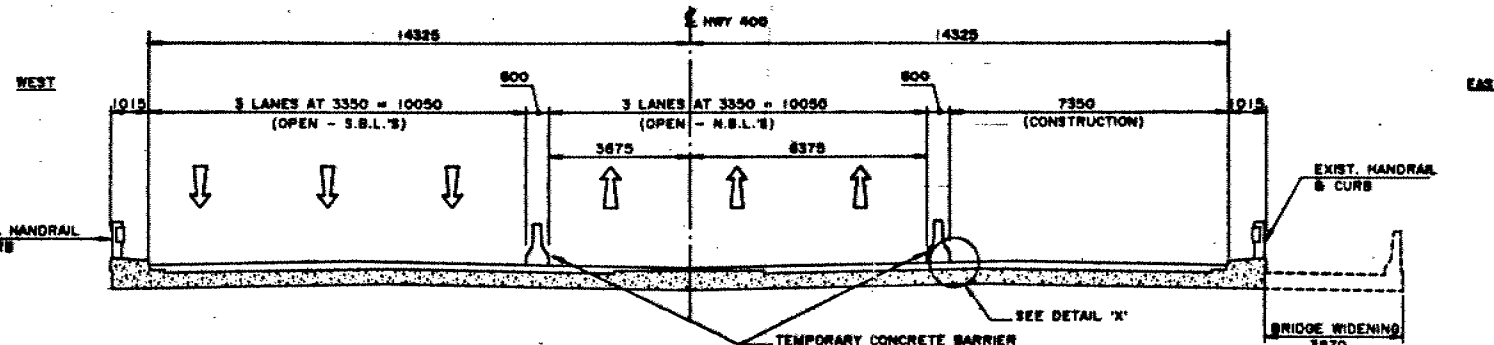
1. RELOCATE WEST LINE OF TEMPORARY CONCRETE BARRIER. MAINTAIN NORTHBOUND TRAFFIC ON EAST SIDE OF BRIDGE AND DIVERT SOUTHBOUND TRAFFIC TO WEST SIDE OF BRIDGE.
2. REMOVE EXISTING ASPHALT AND WATERPROOFING ON BRIDGE DECK.
3. SCARIFY EXISTING DECK AND REMOVE LOOSE AND DETERIORATED CONCRETE, INCLUDING RAISED CONCRETE AT MEDIAN.
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.
8. REMOVE TEMPORARY CONCRETE BARRIERS AND RESTORE TRAFFIC TO ORIGINAL NORTHBOUND AND SOUTHBOUND LANES.

MISCELLANEOUS

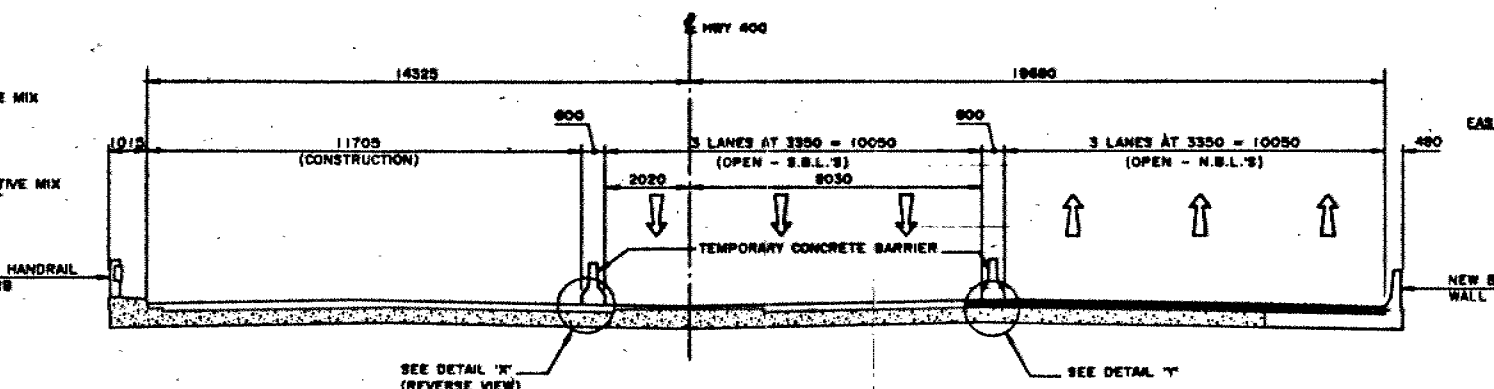
REPAIR ABUTMENT FACES, DECK SOFFIT AND FASCIA.



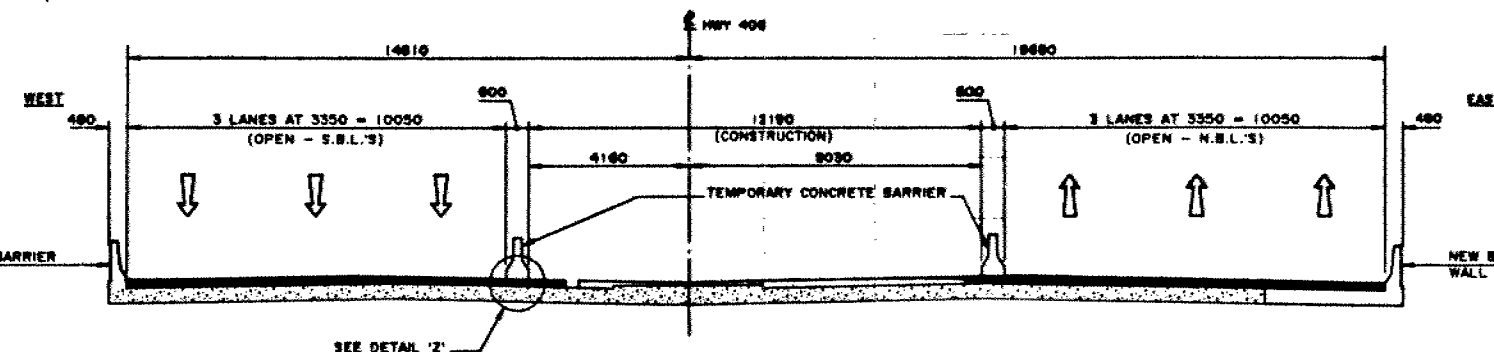
STAGE 1



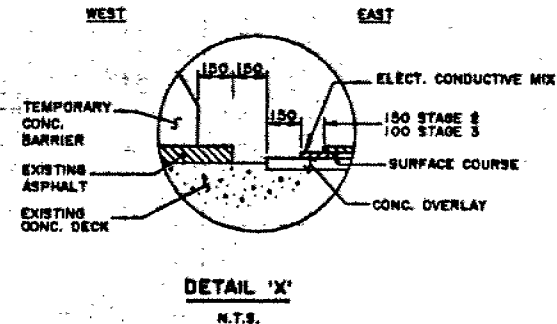
STAGE 2



STAGE 3

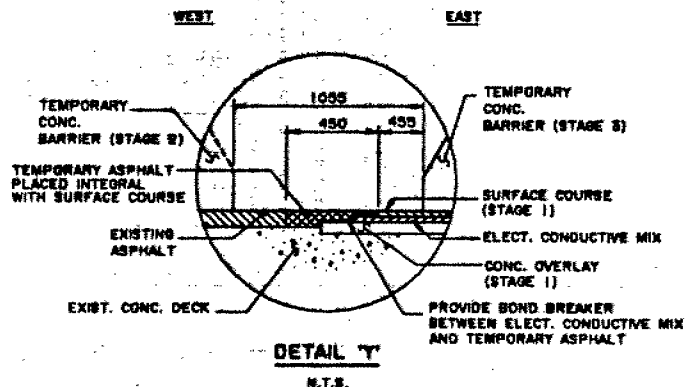


STAGE 4



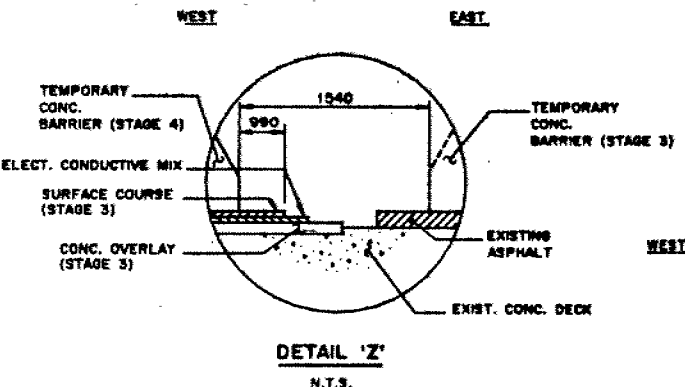
DETAIL 'X'

N.T.S.



DETAIL 'Y'

N.T.S.



DETAIL 'Z'

N.T.S.

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

DATE	BY	DESCRIPTION
DESIGN M.G.S./CHK M.G.S.	CODE 04BC-83	LOAD CLASS A/DATE JAN. 91
DRAWN P.S.H./CHK G.L.A.	SITE 37-95/R1	STRUCT. SCHEME DWG 3

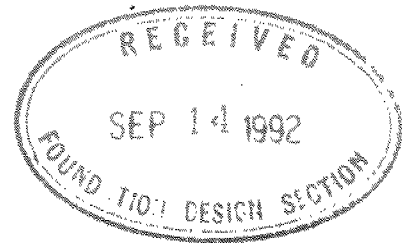
MEMORANDUM

TO: File

FROM: M. G. Shallhorn, P. Eng.

DATE: August 18, 1992

RE: Hwy. 400 - Kirby Sideroad Overpass Structure
Site No. 37-95, W.P. 95-85-01



On Friday, August 14th, 1992 I spoke with Mr. A. Sangiuliano, P. Eng., MTO, Foundation Design Section, regarding the recommendations regarding design of sliding resistance contained in the Geotechnical Investigation Report for the above noted project.

Mr. Sangiuliano advised that current Ministry practice is to consider the shear resistance of the soil rather than adhesion when computing sliding resistance. In this regard he recommended that we calculate the sliding resistance based on an unfactored internal angle of friction of 30°. The resulting coefficient of friction is 0.58.

END OF MEMORANDUM

M. G. Shallhorn, P. Eng.

MGS/la

c.c. Mr. A. Sangiuliano, P. Eng.
Mr. W. Young, P. Eng.

memorandum



To: V. Boehnke
Head, Structural Section
Central Region

Date: 1991 06 11

From: Foundation Design Section
Room 315, Central Bldg.

Re: Kirby Road Overpass
Bridge Widening
W.P 95-85-01
District 6, Toronto

We have reviewed the final drawings for the above project. We find that the general arrangement drawings as well as the granular pad details are as per our recommendations.

We consider that the details of the roadway protection shown as Section C-C of Drawing 4, even though technically acceptable, may cause some construction problems. A soil anchor system could have been used instead of the raker system.

We have no other comments.

A handwritten signature in dark ink, appearing to read "B. Iyer", written over a horizontal line.

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

for

M. Devata, P. Eng.
Chief Foundation Engineer

BI/mmj

Structural Section
Central Region
1201 Wilson Avenue
Atrium Tower, 4th Floor
Downsview, Ontario, M3M 1J8
Telephone: 235-5510

MINISTRY OF TRANSPORTATION

m e m o r a n d u m

TO: A. Wittenberg
Head, Planning & Design
Central Region
5th Floor, Atrium Tower

DATE: May 15, 1992

RE: Revised Final Plans & D4
Hwy 400 Bridge Rehabilitations & Widening
Kirby Side Road O'Pass, W.P. 95-85-01, Site 37-95
King Township Rd 15/16 O'Pass, W.P. 95-85-02, Site 37-59
District 6, Toronto



Due to the deferral of this project last summer, we took the opportunity to revise the design of the bridge rehabilitation. Previously, a concrete overlay with cathodic protection was detailed. The cathodic protection has been deleted and replaced with waterproofing, while maintaining the concrete overlay.

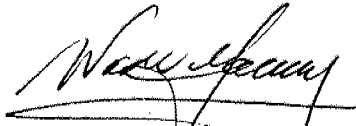
As requested by H. Glass, we have incorporated the use of concrete barrier wall in the median of Hwy. 400 (Tall Wall).

One draft set of modified drawings, D4 and Special Provisions were given to H. Glass on April 21, 1992 to commence work on a composite package. One set of the final package are enclosed and the same are being forwarded to those offices directly affected by the change, namely:

Structural Office	- Review Engineer
Structural Office,	
Bridge Management Section	- Head
Estimating Office	- Manager
District Office	- District Engineer

As well, we are providing 16 sets of full-sized drawings and quantity sheets to H. Glass for the Contract Package Review Meeting. On these items, sheet and item numbers have been changed to reflect the numbering scheme for the combined package.

By copy of this memorandum, Bridge Management Section is requested to advise if the structure can accommodate the additional loading of the barrier wall and to provide comments as usual by June 12, 1992.



Wade F. Young
Senior Structural Engineer
for:
V. F. Boehnke
Head, Structural Section

WFY/
Encl.

cc: R. Reel
H. Jagasia*
M. Devata*
S. Dunham
File 95-85-14.P&D

J. Cullen
G. Al-Bazi
D. Garner*
H. Glass

R. Dorton*
W. Peck
A. Ahmed*
*memo only

Structural Section
Central Region
1201 Wilson Avenue
Atrium Tower, 4th Floor
Downsview, Ontario, M3M 1J8
Telephone: 235-5510

MINISTRY OF TRANSPORTATION

memorandum

TO: R. Reel
Head, Bridge Management Section

DATE: May 8, 1991

RE: Final Plans & D4
Hwy 400 Bridge Rehabilitations & Widening
Kirby Side Road O'Pass, W.P. 95-85-01, Site 37-95
King Township Rd 15/16 O'Pass, W.P. 95-85-02, Site 37-59
District 6, Toronto

Further to the attached memorandum dated May 2, 1991 to A. Wittenberg issuing the Final Plans & D4, kindly provide your comments at your earliest convenience.

For your information, please note the following:

1. The use of silica fume shotcrete will be incorporated into the contract.
2. We have reviewed these drawings and documents previously and provided comments to the consultant. After receiving and further reviewing the revised package on May 2, 1991, we found that additional changes are required. As they are relatively minor in nature and more importantly because we are behind schedule, these changes have not been made on your package. Your understanding in this regard is requested and appreciated.
3. We understand that the remote monitoring unit shown on the drawings is not required and will be removed.
4. The pole mounted assembly for the rectifier, power supply etc., will be replaced by the enclosed cabinet design placed directly on the ground.

By copy of this memo, the Foundation Design Section is requested to insure that the items discussed at the Feb. 13, 1991 meeting with regard to the stepped retaining walls have been incorporated to their satisfaction. As well, Foundation Section had provided comments on Nov. 16, 1990 on the General Arrangement & Staging

Drawings. We believe these comments have been appropriately incorporated. Details of the proposed shoring scheme is part of the enclosed drawings.

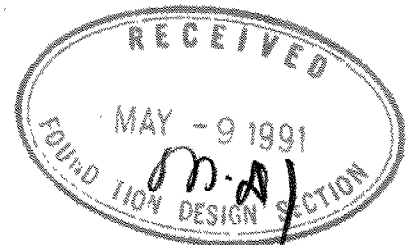


Wade F. Young
Senior Structural Engineer
for:
V. F. Boehnke
Head, Structural Section

WFY/

cc: M. Devata
B. Iyer
H. Jagasia
J. Cullen
G. Al-Bazi

File:95-85-6.BMS



B.T

memorandum



To: V. Boehnke
Head, Structural Section
Central Region

Date: 1990 11 16

Atten: W. Young
Sr. Structural Engineer

From: Foundation Design Section
Room 315, Central Building

Re: General Arrangement and Staging Drawings Review
Kirby Side Road O'Pass, W.P. 95-85-01, Site 37-95
King Township Rd. 15/16 O'Pass, W.P. 95-85-02, Site 37-89

The general arrangement and staging drawings for the proposed widening at the aforementioned sites have been reviewed by this office, and the following comments, applicable to both sites, are provided.

1) Temporary Roadway Protection

It is recommended that in order for this office to implement a thorough review of the temporary shoring scheme required to facilitate the construction of the widening, details of the proposed scheme shall be submitted to this office. The submission shall include the type, longitudinal extent, depth of penetration, method of installation and lateral support of the shoring wall.

2) Perched Retaining Walls on Compacted Granular 'A' Fill

The "East Elevation - New Construction" view on the drawing illustrates perched retaining walls located on approximately 2 metres of compacted granular 'A' fill. To ensure the overall global stability of the retaining wall, it is recommended that the transverse/longitudinal slopes be designed no steeper than 2H:1V. Denotations on the drawing reveal 2H:1V slopes as "maximum" slopes, perhaps implying that steeper slopes will exist.

The granular pad shall provide the required load bearing area for the length of the entire footing. The pad geometry illustrated on the drawing at the retaining wall/wing wall interface appears to depict a "void" area between the footing and the granular pad.

Shallow foundations placed adjacent to other shallow foundations, which is the case that is present at both sites shall conform to clause C6-7.2 (h) (i) of the O.H.B.D.C. However, if this is not geometrically feasible, the proper consideration shall be given to address the influence of the load distribution of the upper 'perched' foundation on the lower foundation.

3) Removal of Deleterious Material at Footing Founding Elevation

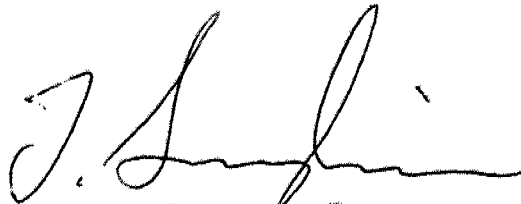
Regarding the removal of deleterious material, the term "softened" shall replace "loosen" to correlate with the cohesive material that may be present at the footing foundation elevation.

4) Construction Sequence - Stage 2

Concrete retaining walls shall not be removed until the roadway protection has been completely installed. Steps 3 and 4, hence, should be modified to reflect this sequence.

Removal of the roadway protection shall conform to OPSS 539.07.02.

If you have any questions regarding the above comments, please do not hesitate to contact this office.



T. Sangiuliano, P. Eng.
Foundation Engineer

for

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

TS/mmj

memorandum



To: V. Boehnke
Head, Structural Section
Central Region

Date: 1990 07 09

Atten: W. Young
Sr. Structural Engineer

From: Foundation Design Section
Room 315, Central Building

Re: Retaining Walls at Kirby Side Road/Hwy. 400
and King Twp. Road/Hwy. 400
Highway 400, W.P. 95-85-01/02
District 6, Toronto

Further to our meeting dated 90 06 11 at which time foundation design soil capacities were sought in conjunction with the proposed retaining walls at the aforementioned sites, the following comments are provided.

It is our understanding that the design and construction of retaining walls parallel to Highway 400 at the ends of the proposed abutment extensions will eliminate the requirement of retaining walls at the toe of the approach forward slopes and hence result in a more economical design. A step foundation has been contemplated for the retaining walls.

The existing King Township Road/Hwy. 400 contains embankment fills at its approaches whereas the Kirby Road/Hwy. 400 structure approaches are situated in excavation cuts. In the absence of soils data for the fill material at the King Twp. Rd. Structure, (acquiring this data was not practical at the site because of access and slope geometry difficulties), the fill material was assumed to be either cohesive or cohesionless with a consistency/denseness equivalent to a SPT "N" value of 10-15 blows/0.3 m.

The retaining walls can be stepped at either site at any elevation above the founding elevations recommended in the original Foundation Report provided the following conditions are satisfied.

- 1) All footings must be founded on a compacted granular 'A' pad with a minimum thickness of 2 metres. The pad must be compacted in accordance with OPSS 501.
- 2) The granular 'A' pad shall extend for a minimum distance of 4 metres beyond the retaining wall and 2 metres within the retaining wall as illustrated in Figure 1. The fill slopes must also be constructed at 2H:1V slopes to assure the overall slope stability.
- 3) All loosened/softened and/or organic material at the founding elevation shall be subexcavated and replaced with granular 'A' material.

.... /2

- 4) Backfill behind the retaining wall should consist of granular 'A' material to prevent hydrostatic pressure build-up.
- 5) The final geometry is reviewed by this office to verify that the location of new footings does not influence any existing foundations.

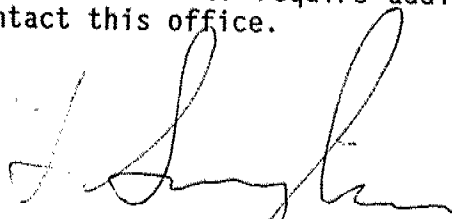
Provided that the above-mentioned conditions are satisfied, the retaining wall foundations can be designed using the following parameters.

Table 1 - Foundation Design Parameters

Bearing Capacity at S.L.S. Type II (kPa)	300
Factored Capacity at U.L.S. (kPa)	450
Unfactored Angle of Internal Friction ($^{\circ}$)	35

No major dewatering difficulties are anticipated during excavation and subsequent placement of the granular 'A' pad. Any 'perched' water can be readily discharged using conventional sump pumping techniques.

If you have any queries regarding the above comments or require additional information, please do not hesitate to contact this office.

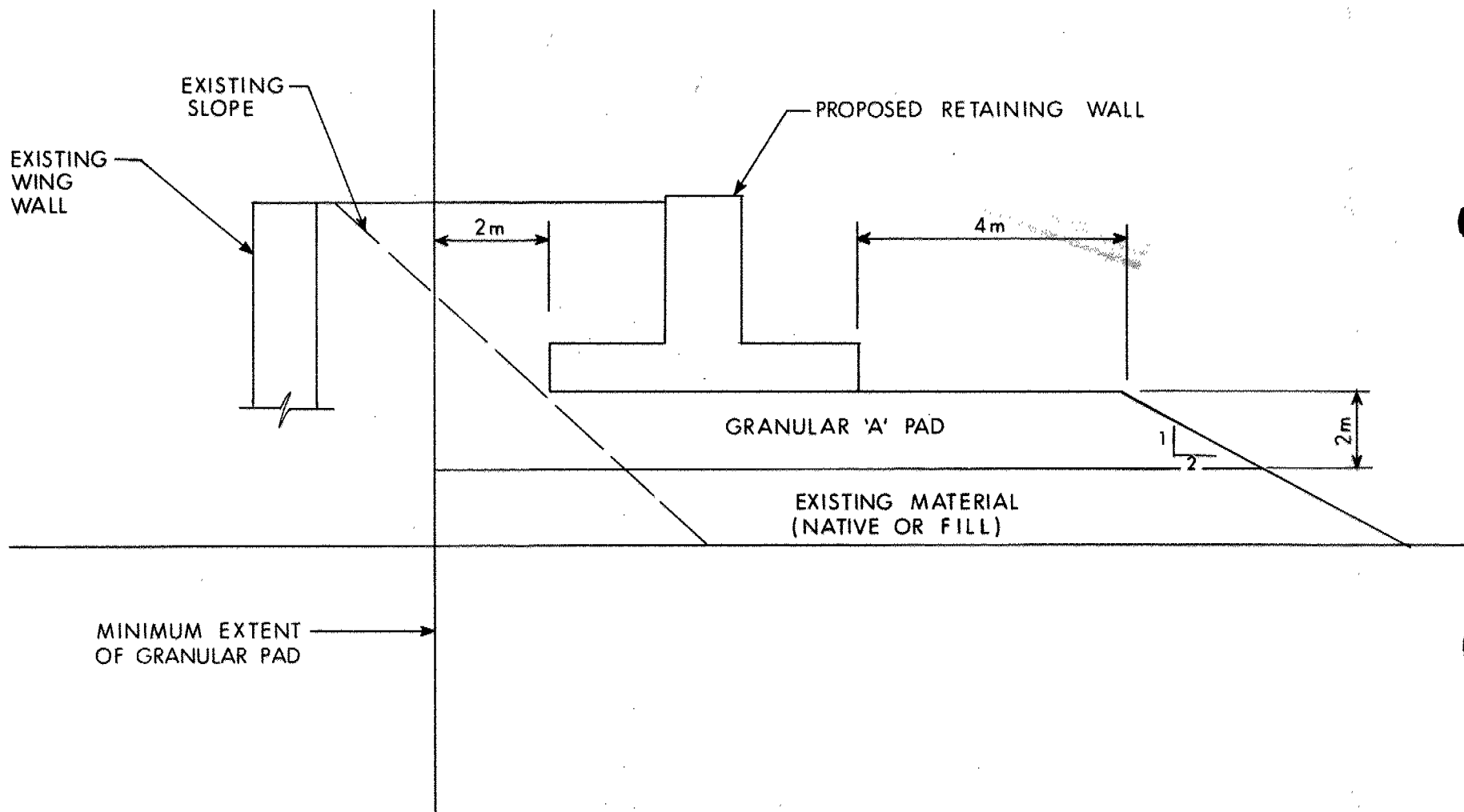


T. Sangiuliano, P. Eng.
Foundation Engineer

for

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

TS/mmj



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