

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 279-85-01 DIST 18
HWY 17 STR SITE 38S-88

Root River Bridge

CONT 91-218

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FOUNDATION INVESTIGATION REPORT

For

W. P. 279-85-01

Root River Bridge, Hwy. 17

District #18, Sault Ste. Marie

INTRODUCTION

This report summarizes the information obtained from a foundation investigation carried out at the above-noted site.

The fieldwork for this project was carried out during the period from 86 04 01 to 86 04 11 utilizing a continuous flight auger machine equipped with 82 mm I.D. hollowstem augers and 60 mm I.D. B-Casing. Also, a diamond drill equipped with 60 mm I.D. B-Casing and 76 mm I.D. N-Casing was utilized. The investigation consisted of 7 sampled boreholes [numbered BH#1 to BH#7] supplemented with Cone Penetration Tests. The boreholes ranged in depth from 9.1 m [BH#1] to 34 m [BH#3].

SITE DESCRIPTION AND GEOLOGY

The site is located about 1 km east of the Sault Ste. Marie City Limits, within Garden River Indian Reserve #14 in the District of Algoma. The Ministry's Right-of-Way is bounded by the Garden River Indian Reserve on the south side and the Canadian Pacific Railway Right-of-Way on the north side of the highway.

Topography at the site is gently rolling with some hills in the surrounding vicinity. The Root River is a shallow, medium to fast flowing river at the site crossing. It has a relatively wide valley, but is not well incised. Water elevation was fairly constant during the time of the investigation, but may fluctuate depending on the season.

Geologic references indicate that the site lies within a lacustrine deposit. In the Pleistocene epoch a glacio-alluvial belt of "beach like" sand was deposited across this region. This belt abuts the hills of the Precambrian Shield to the north and the St. Mary's River to the south. Sedimentary bedrock of the Jacobsville Formation underlies the lacustrine deposit at a depth ranging between 50-100 m.

In the vicinity of the site, land use is primarily rural residential with some limited industrial development.

SUBSURFACE CONDITIONS

General

The subsurface conditions can be summarized briefly as follows: Underlying a veneer of topsoil, the area is predominantly covered with deposits of poorly graded sand with a thickness of about 30 m. This granular deposit is underlain by a stratum of silty clay to clay whose thickness was not established in this investigation.

The boundaries of the subsoil types, in-situ and laboratory test results, as well as groundwater levels are shown on the Record of Borehole Sheets ^{H1 to H7 inclusive} in the Appendix. The location of each borehole is shown in plan on Dwg. No. 2798501-A together with 3-stratigraphical sections.

The various soils encountered at this site are described as follows:

Fill

A boundary between the native soil and the embankment fill was not distinctly apparent. However, the upper zones of the fill material consisted of sand with occasional isolated pockets of gravel, clay, or organics. Occasional cobbles were also encountered.

It is evident that the material used for the embankment construction was local material obtained from a location relatively close to the site.

Topsoil

The site is covered by a veneer of topsoil generally in the order of 150 mm in thickness. However, the thickness of this organic deposit may vary across the site. No sampling or testing of this material was carried out.

Sand

The predominant deposit across this site consists of sand. The thickness of this deposit ranges between a minimum of 6.6 m in the area of BH#6, to 28.3 m in the vicinity of BH#3. These thicknesses include the depth of the fill which essentially is the same composition.

Grain size distribution testing was carried out on 14 samples of this non-cohesive deposit. Three slightly different compositions are evident: sand trace silt; sand some silt; and sand trace silt, gravel. Generally, the silt content increases slightly with depth. In the vicinity of BH#7, the sand deposit was found to include trace gravel.

Of the 14 samples tested, 10 were composed of sand trace silt. The results of these tests are shown in envelope form on Figure #1 in the Appendix and can be summarized as follows:

	<u>Gravel</u>	<u>Sand</u>	<u>Silt</u>	<u>Clay</u>
Range %	0	85 - 99	(1-15)*	
Average%	0	95	(5)*	

* % of silt and clay size particles combined

Figure #2 in the Appendix illustrates the results of gradation tests carried out on 2 samples of the sand deposit [BH#7, #1 and #6] which contained trace gravel, silt.

Figure #3 in the Appendix illustrates the results of gradation tests carried out on 2 samples of the sand deposit [BH#4, #15 and BH#6, #3] which contained some silt (up to 21%).

It should be noted, however, that the composition of the sand deposit may vary randomly with depth and location. Figures #1, #2 and #3 serve only to illustrate the results of the samples tested. Cohesive seams may also be encountered occasionally and randomly throughout the deposit.

Based on the results of the grain-size distribution tests it is evident that the sand sized particles in this deposit can be described as medium to fine size.

Based on the interpretation of Standard Penetration Test "N" values, this non-cohesive deposit is generally in a loose to compact state. With depth, however, the deposit becomes more dense.

It should be noted that when this material is subjected to an unbalanced hydrostatic pressure, "boiling" may result.

Silty Clay to Clay

In BH#3, a deposit of silty clay to clay was encountered at a depth of 28.3 m below the ground surface. The full lateral or vertical extent of this deposit was not established.

The results of 3 Atterberg Limits Tests carried out on samples from BH#3 are shown on Figure #4 in the Appendix and are summarized as follows:

<u>Sample #</u>	<u>W %</u>	<u>W_L %</u>	<u>W_p %</u>	<u>I_p %</u>
8	37.5	53	19	34
9	29.5	42	14	28
10	41	62	20	42

Based on these results, the deposit can be described as a silty clay of intermediate plasticity (CI group) to a clay of high plasticity (CH group).

The results of laboratory shear strength tests indicate that with the area investigated, the consistency of the deposit ranges from stiff to firm.

The unit weight of this cohesive material was measured to be 18.7 and 17.9 kN/m³ in two "undisturbed" samples.

Occasional seams of sand were encountered within this deposit.

Groundwater Conditions

Stabilized groundwater levels were measured in open boreholes. The measurements indicate that the groundwater elevation, at the time of investigation, varied between 177.3 m and 179.6 m. Generally, it can be assumed that the groundwater level across the site is governed by the water level in Root River. At the time of the investigation, the water level in the river was found to be at Elevation 177 m.

DISCUSSION AND RECOMMENDATIONS

The existing 3 span structure (12.2 m - 24.4 m - 12.2 m) at this site was constructed in the early 1940's. The end spans are independent steel stringers, with the centre span having a steel deck truss with steel beams and stringers. Limited substructure information indicates that the piers are probably supported on timber piles and the abutments are supported on spread footings.

A bridge deck condition survey was carried out in September 1984 and the results indicate that the bridge deck is in poor condition and beyond repair. Four alternatives are proposed for this structure in the Preliminary Design Report prepared by the Northwestern Region Engineering and Right-of-Way Office:

- ALT. 1 Complete bridge replacement
- ALT. 2 Deck replacement and foundation stabilization
- ALT. 3 Deck replacement
- ALT. 4 No action

Bridge inspection addendums dating back to 1957 indicate that abutment rotation and approach settlement are occurring. Consequently, only alternatives ALT. 1 and 2 are addressed in this report.

ALT. 1, involves the construction of a new bridge. ALT. 2, concerns itself with the rehabilitation of the existing structure and foundations.

Both alternatives require temporary Bailey bridges during the construction period. Recommendations with regards to these temporary structures are presented at the end of this section.

The recommendations that follow utilize piles driven to or constructed at a specific tip elevation. Prior to finalizing the structural design of this structure, the Foundation Design Section would like to carry-out a full-scale pile load test at this site so that the loadings presented in this report can be confirmed or perhaps increased. This Section should be made aware of the project program so that the test can be scheduled.

ALT. 1: BRIDGE REPLACEMENT

If this alternative is selected, the proposed structure would most likely consist of two $25 \pm$ spans, with a 4-lane cross-section, at the same alignment and profile as the existing structure. The following are our recommendations for the foundation design of this structure:

Foundations

The proposed abutments and pier can be founded on Size 36 treated timber piles. TABLE 1, indicates the location, station, factored axial capacity at the U.L.S., axial capacity at the S.L.S. Type II, and pile tip elevation. If significant deviations from the assumed stations occur, this section should be contacted for a review of the recommendations. It should be noted that the piles should not be driven deeper than the tip elevations given, unless this Section is contacted.

TABLE 1

<u>Location</u>	<u>Station</u>	Factored Axial Capacity <u>At U. L. S.</u>	Axial Capacity at S. L. S. <u>Type II</u>	<u>Pile Tip Elevation</u>
West Abutment	10 + 999 \pm	350 kN	225 kN	164 - 165
Pier	11 + 026 \pm	350 kN	225 kN	166
East Abutment	11 + 051 \pm	350 kN	225 kN	162 - 163

Alternatively, the structure can be supported on expanded base concrete piles installed to the tip elevations given in TABLE 1. For the design, 450 mm diameter concrete piles can be used using a loading of 1200 kN/pile at the U.L.S., and 850 kN/pile at the S.L.S. Type II.

The loadings are subject to revision after a pile load test is completed. Similarly, field control measures for the installation or driving will be given after the load test is undertaken.

If timber piles are used, it will be necessary to restrict the particle size of fill material to 75 mm in the area where piles will be driven. In order to further facilitate the pile driving any surficial boulders or cobbles which may obstruct driving should be removed.

If pile caps are constructed below the prevailing groundwater level dewatering will be required in view of the non-cohesive nature of the underlying subsoils.

Liners should be used if the expanded base piles are incorporated in the design.

Lateral Earth Pressures

Abutments should normally be designed for the active earth pressure (k_a) condition. The at-rest (k_o) condition should only be used in cases where the deflection of the abutment is prevented by the propping action of the deck, such as is the case with a rigid frame structure. Similarly, the at-rest (k_o) condition can be used for abutments on spread footings founded on unyielding material unless the abutment can deflect sufficiently to mobilize the active earth pressures.

Backfill to structures should consist of granular material in accordance with MTC Standard Special Provision #121 (dated October 1983). Computation of earth pressures should be in accordance with Section 6.6.1.2 of the O.H.B.D.C. For design purposes, the physical properties of the backfill are as follows:

<u>Material</u>	<u>ϕ</u>	<u>γ</u>
Granular 'A'	35°	22.0 kN/m ³
Granular 'B'	30°	21.2 kN/m ³

Approach Fills

Since the approaches will not be increased in height no stability problems are anticipated in the embankments providing standard 2H:1V slopes are maintained. However if any widening is required the new slopes should be keyed into the existing fill as per current MTC Standards.

Any surficial organic material should be excavated to it's full depth prior to placing any new fill.

A minimum of 2 m frost cover (or equivalent) should be provided to the underside of pile caps.

Random rip-rap over a geotextile should be placed up to the high water table to prevent scour action by the river at the abutment locations.

ALT. 2: DECK REPLACEMENT AND FOUNDATION STABILIZATION

The existing 3 span structure was constructed in the early 1940's. Accurate records of the design and construction were not kept at the time and consequently the information available regarding the foundations of this structure is limited. It is assumed however, that the piers are supported on piles (probably timber) and that the abutments are supported on spread footings.

Maintenance records indicate that the abutments of this structure appear to have settled and significant rotation has been experienced. The abutments exhibit a substantial degree of random cracking and other major structural cracking.

In view of the non-cohesive nature of the subsoils at this site, it appears that any vertical elastic settlement which may have resulted from the structure loadings would have occurred in a relatively short period of time after the initial construction. Consequently, in our opinion, the apparent "settlement" is a result of the forward rotation which is being experienced by the abutments. As the abutments tilt forward the backfill behind the abutments also moves forward while at the same time subsides to fill the larger area created.

The rotation of the abutments may be the result of lateral earth pressures on an inadequate spread footing design.

Since no signs of settlement have been recorded for the piers we assume that the deep foundations are adequately supporting the imposed loads.

Abutments Foundation Stabilization

The existing abutments are founded on spread footings and as previously mentioned, are experiencing rotation, and as a result, settlement of the approaches. Since very little information is available on the design of the structure it is very difficult to predict future distress.

The abutments can be stabilized by constructing "wing-walls" at each abutment so that a "v" shape is formed. The wing-walls should be supported on either timber or expanded base concrete piles using the design parameters given in TABLE 1. The intent would be to transfer as much load as possible currently on the existing spread

footings on to the proposed piles. This option would allow for pile driving to be undertaken from behind the abutment.

Alternatively, the option presented in the Preliminary Design Report (Engineering and Right-of-Way, Northwestern Region) can be used. As described in the Preliminary Design Report, the abutments could be stabilized by driving piles in front of the abutments, onto which end span loads would be transferred via a cap beam.

Piers Foundation Stabilization

As very limited information is available on the design of the pier foundations, it is recommended that any additional load which will result from the deck rehabilitation be supported by means of additional piles at the pier locations. The piles previously described together with the design information given in TABLE 1 can be used in the design.

Lateral Earth Pressures

The information presented under Alternative 1 can be used to compute the lateral earth pressures acting on the abutments. However, if it is deemed necessary to reduce the lateral pressures, consideration could be given to the use of light-weight fill. The existing fill could be excavated down to a depth of 3 or 4 m, to a point 20 m back of the abutments, and replaced with a light-weight fill such as air cooled slag of nominal 10 mm diameter, and with a maximum in-place unit weight of 11 kN/m³.

Alternatively, styrofoam blocks could be used to further reduce the lateral pressures. The limits previously given could be used.

Approach Fills

The recommendations given for Alternative 1 with regards to the approach fills apply for Alternative 2.

BAILEY BRIDGE

It is understood that it is proposed to provide a detour on the south side of the existing structure during construction of this project. The detour will include one or two 40 m single span bailey bridges crossing the Root River. The baileys can be supported on rock-filled timber cribs founded on the ground surface using a design load of 50 kPa.

MISCELLANEOUS

The fieldwork for this investigation was carried out during the period from 86 04 01 to 86 04 11 under the supervision of D. Protulipac and J. Fellenius (Student Engineers). The equipment used was owned and operated by Dominion Soil Investigation Inc. of Toronto, and by Marathon Drilling Inc. of Ottawa.

This report was prepared by L. Politano and D. Protulipac, and was reviewed by M. Devata, Chief Foundations Engineer.



L. Politano, P. Eng.,
Project Foundations Engineer

M. Devata, P. Eng.
Chief Foundations Engineer
(East)

APPENDIX

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

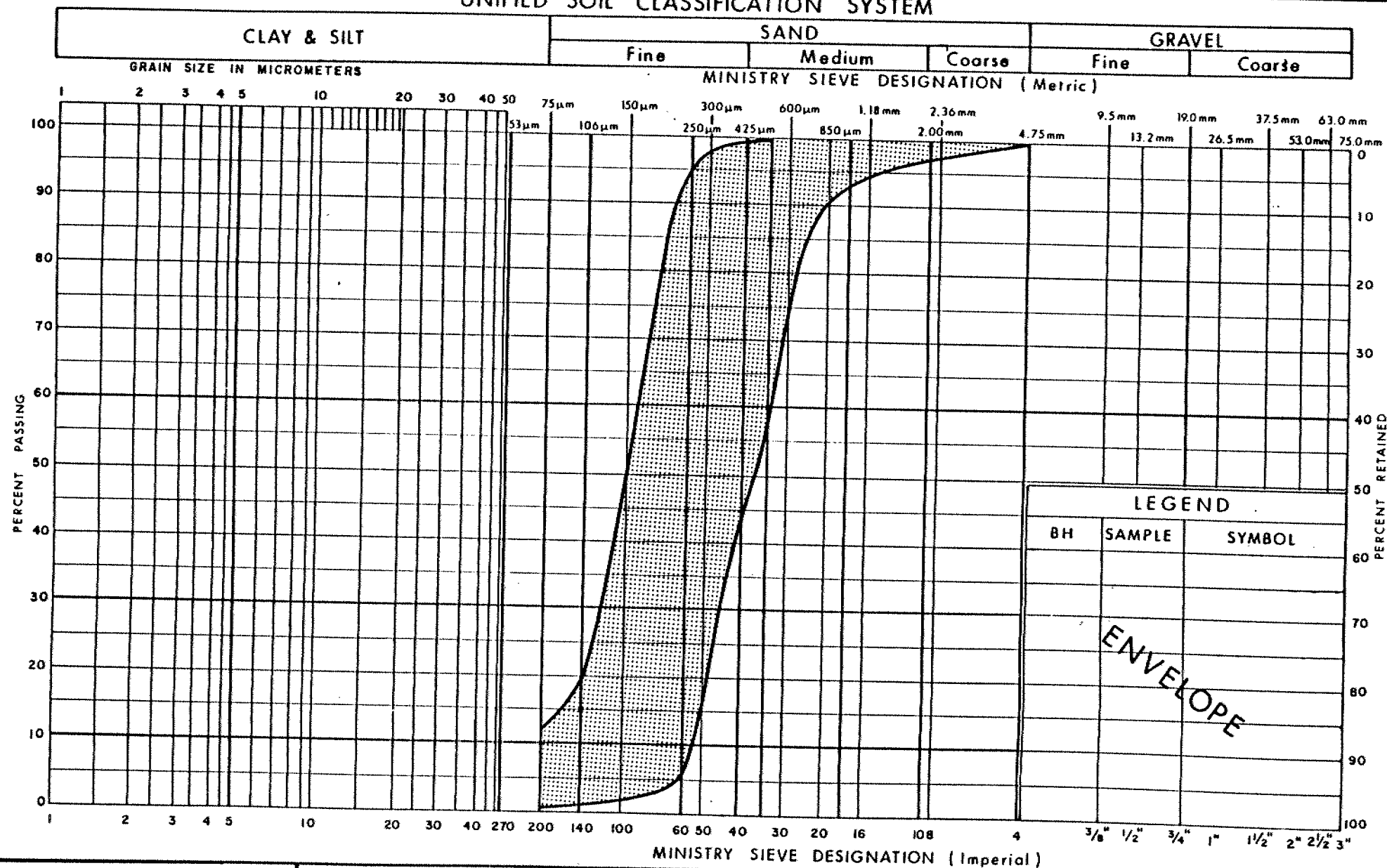
MECHANICAL PROPERTIES OF SOIL

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	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						

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

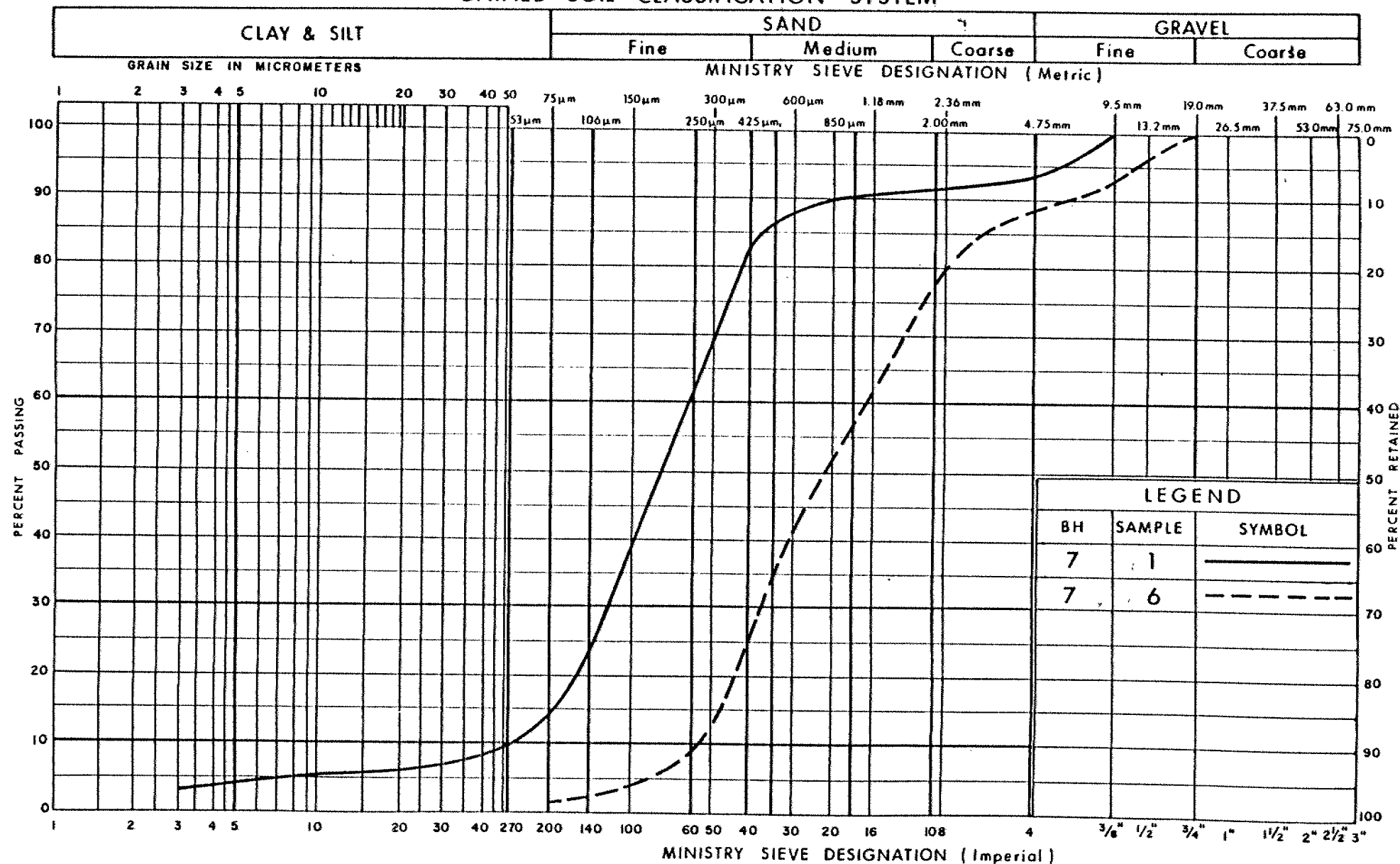
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GRAIN SIZE DISTRIBUTION
SAND, TRACE OF SILT

FIG No 1

W P 279-85-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

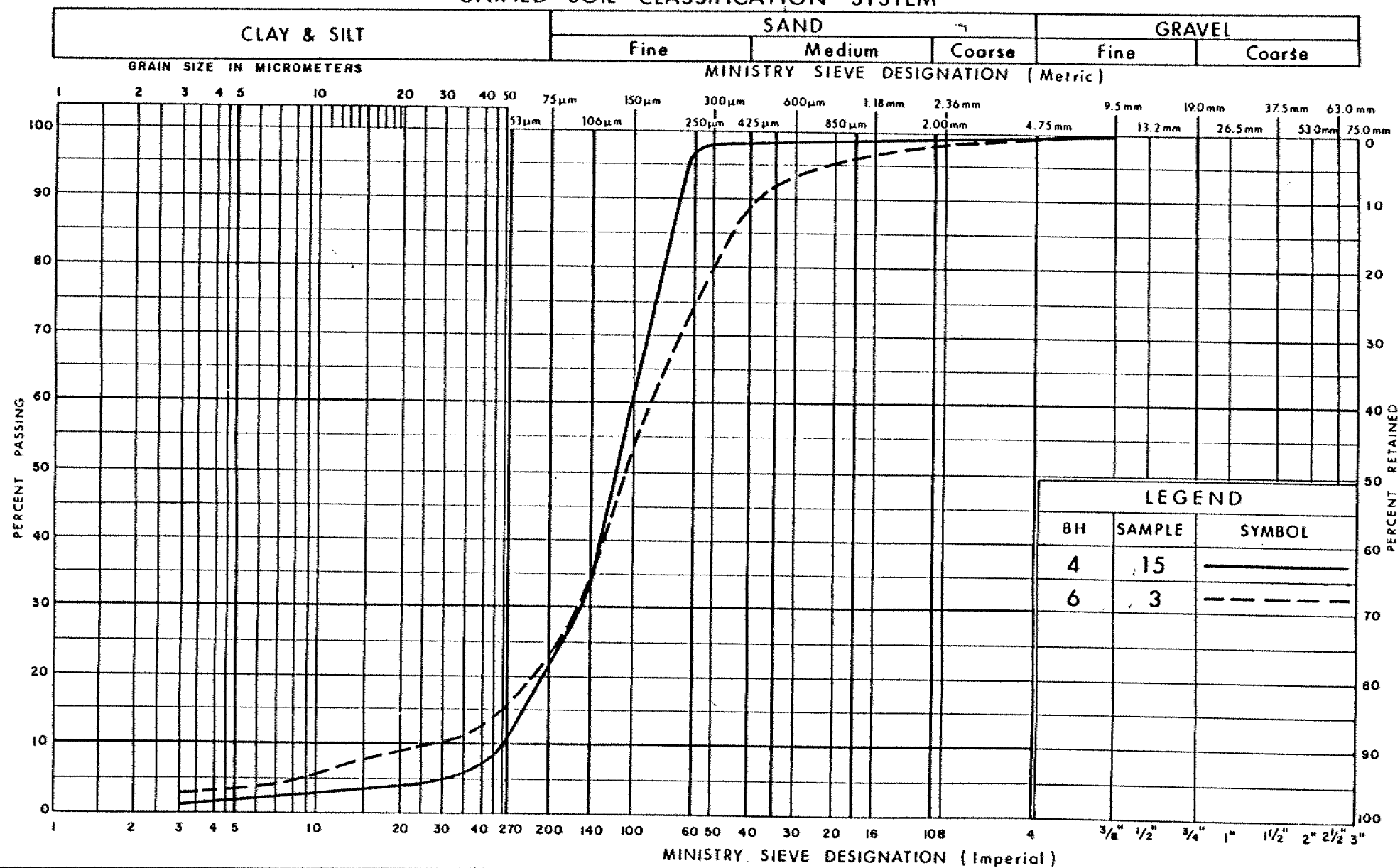
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Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SAND, TRACE OF GRAVEL, SILT

FIG No 2

W P 279-85-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

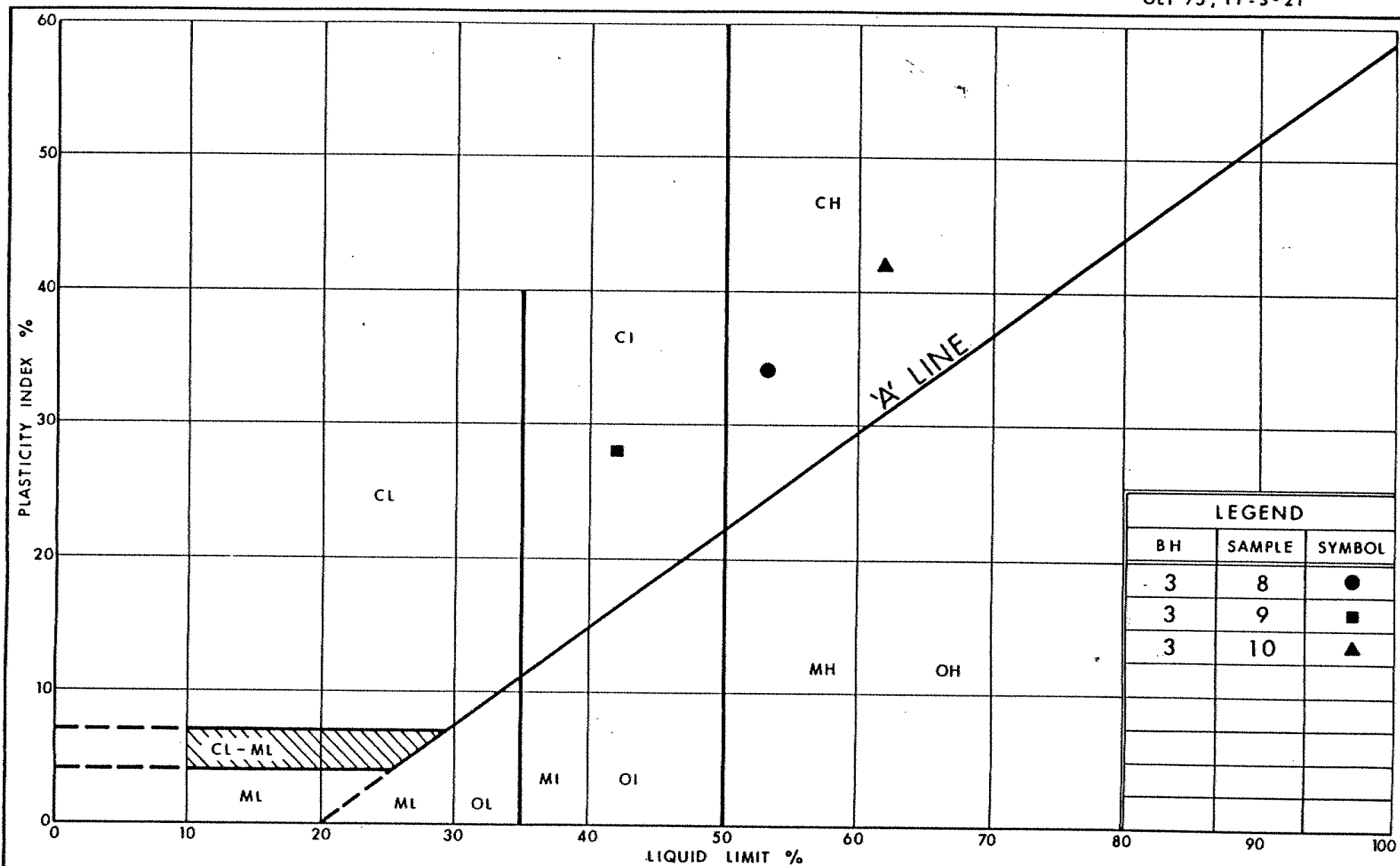
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Communications

GRAIN SIZE DISTRIBUTION

SAND, SOME SILT

FIG No 3

W P 279 - 85 - 01



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PLASTICITY CHART SILTY CLAY TO CLAY

FIG No 4

W P 279-85-01



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RECORD OF BOREHOLE No 1

METRIC

W P 279-85-01 LOCATION STA. 10 + 987.5; 0/s 8.6mLt. C Hwy. 17 ORIGINATED BY JF
DIST 18 HWY 17 BOREHOLE TYPE BW-NW Casing, Washboring and Cone Test COMPILED BY DP
DATUM Geodetic DATE 86 04 07 CHECKED BY GP

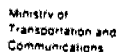
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
184.1	Ground Surface									
0.0	Topsoil Medium to Fine Sand, Trace Silt Compact-Dense Brown		1	AS	*					
			2	SS	18					0 98 (2)
			3	SS	20					
			4	SS	9					
			5	SS	7					
			6	SS	22					
			7	SS	35					0 97 (3)
			8	SS	13					
176.0	End of Borehole		9	SS	9					
175.0	End of Cone Test									
	* Auger Tip Sample									

RECORD OF BOREHOLE No 2

METRIC

W P 279-85-01 LOCATION STA. 10 + 993.2; 0/s 8.6 m Lt. C Hwy. 17
 DIST 18 HWY 17 BOREHOLE TYPE BW-NW Casing, Washboring and Cone Test
 DATUM Geodetic DATE 86 04 01 to 07
 ORIGINATED BY JF
 COMPILED BY DP
 CHECKED BY

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					
184.0	Ground Surface												GR SA SI CL
0.0	Medium to Fine Sand, Trace Silt Loose-Compact Brown		1	SS	2								
			2	SS	1								
			3	SS	1								
			4	SS	2								
			5	SS	6								
			6	SS	16								
			7	SS	10								
			8	SS	8								0 94 (6)
			9	SS	20								
			10	SS	6								
			11	SS	7								
			12	SS	7								0 99 (1)
			13	SS	16								
			14	SS	14								
			15	SS	18								
	Dense to Very Dense		16	SS	56								
			17	SS	56								0 96 (4)
160.7			18	SS	20								
23.3	End of Borehole												



RECORD OF BOREHOLE No 3 (1 of 2) METRIC

W P 279-85-01 LOCATION STA. 11 + 055.2; 0/s 6.6 m Lt. C Hwy. 17 ORIGINATED BY JF
DIST 18 HWY 17 BOREHOLE TYPE H-S Augers, BW Casing Washboring and Cone Test COMPILED BY DP
DATUM Generic DATE 86 04 08 to 10 CHECKED BY CP

[illegible]

Continued

*3, x5: Numbers refer to Sensitivity

15 20 5 (%) STRAIN AT FAILURE Continued



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Communications
Ontario

RECORD OF BOREHOLE No 3 (2 of 2) METRIC

W P 279-85-01 LOCATION STA. 11 + 055.2; °/s 6.6 m Lt. Q Hwy. 17 ORIGINATED BY JF
DIST 18 HWY 17 BOREHOLE TYPE H-S Auger, BW Casing Washboring and Cone Test COMPILED BY DP
DATUM Geodetic DATE 86 04 08 to 10 CHECKED BY GP.

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100						W _p W W _L		
								SHEAR STRENGTH kPa						WATER CONTENT (%)		
							○ UNCONFINED + FIELD VANE									
							● QUICK TRIAXIAL × LAB VANE									
							20 40 60 80 100									
153.6	Continued															
30.2	Silty Clay of Intermediate Plasticity to Clay of High Plasticity Occasional Sand Seams Stiff to Firm		9	TH	PH		σ					18.7	0 49 9 42			
			10	TH	PH			ρ				17.9	0 0 26 74			
149.8			11	SS	16											
34.0	End of Borehole															
	* Groundwater level not established															

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



Ministry of
Transportation and
Communications

RECORD OF BOREHOLE No 4

METRIC

W P 279-85-01 LOCATION STA. 10 + 996.0: 0/s 7.0 m Rt. C Hwy. 17 ORIGINATED BY JF
DIST 18 HWY 17 BOREHOLE TYPE BW-NW Casing, Washboring and Cone Test COMPILED BY DP
DATUM Geodetic DATE 86 03 25 and 26 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES							
184.1	Ground Surface											
0.0	Topsoil						184					
	Medium to Fine Sand,		1	SS	13							
	Trace Silt		2	SS	5		182					
	Very Loose to Loose		3	SS	9							
	Gravel with Sand		4	SS	7		180				65 27 7 1	
	Brown		5	SS	2							
			6	SS	16		178					
			7	SS	7		176					
			8	SS	22		174					
			9	SS	12		172					
	Compact		10	SS	14		170					
			11	SS	15		168					
			12	SS	10		166					
			13	SS	34		164					
	Silty Fine Sand		14	SS	105							
	Very Dense		15	SS	115							
	Brown		16	SS	143		164				0 80 19 1	
162.8												
21.3	End of Borehole											

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



Ministry of
Transportation and
Communications
Ontario

RECORD OF BOREHOLE No 5

METRIC

W P 279-85-01 LOCATION STA. 11 + 054.6; 0/s 7.3 m Rt. C Hwy. 17 ORIGINATED BY JF
DIST 18 HWY 17 BOREHOLE TYPE BW-NW Casing, Washboring and Cone Test COMPILED BY DP
DATUM Geodetic DATE 86 04 03 and 04 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES							
183.8	Ground Surface											
0.0	Topsoil											
	Medium to Fine Sand, Trace Silt		1	SS	3							
			2	SS	2							
	Very Loose - Loose		3	SS	19							
			4	SS	5							
			5	SS	5							
	Brown		6	SS	5							
			7	SS	11							0 92 (8)
			8	SS	6							
			9	SS	15							
			10	SS	13							
			11	SS	12							
			12	SS	10							
			13	SS	14							
			14	SS	27							
	Compact-Dense		15	SS	18							
			16	SS	76							
			17	SS	175							
160.5			18	SS	34							0 93 (7)
23.3	End of Borehole											

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

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 6

METRIC

W P 279-85-01 LOCATION STA. 11 + 065.3; 0/s 7.4 m Rt. C Hwy. 17 ORIGINATED BY JF
DIST 18 HWY 17 BOREHOLE TYPE BW-NW Casing, Washboring and Cone Test COMPILED BY DP
DATUM Geodetic DATE 86 04 02 CHECKED BY GP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
183.8	Ground Surface											
0.0	Medium to Fine Sand, Some Silt		1	SS	5							
			2	SS	8							
			3	SS	4							
	Very Loose - Loose		4	SS	3							
			5	SS	5							
	Brown		6	SS	8							
177.2			7	SS	13							
6.6	End of Borehole											
174.6												
9.2	End of Cone Test											

+3, x5: Numbers refer to
Sensitivity

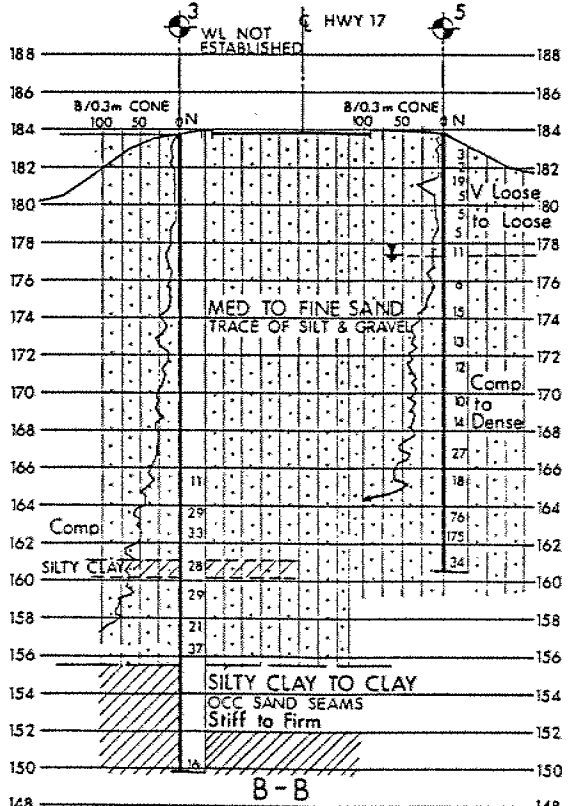
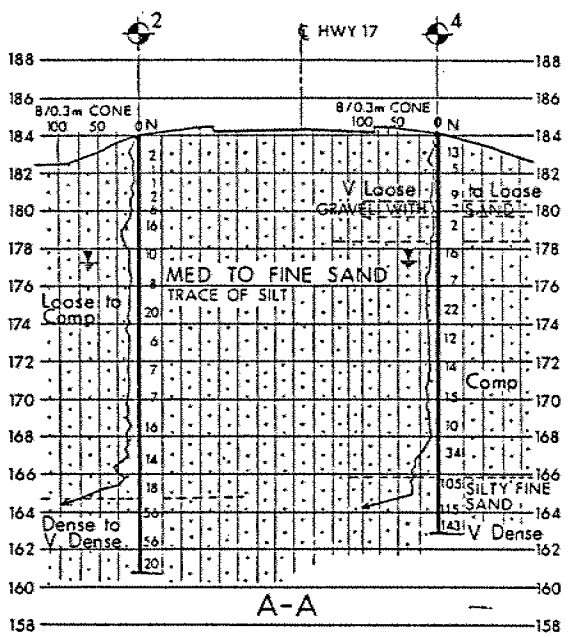
20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 7

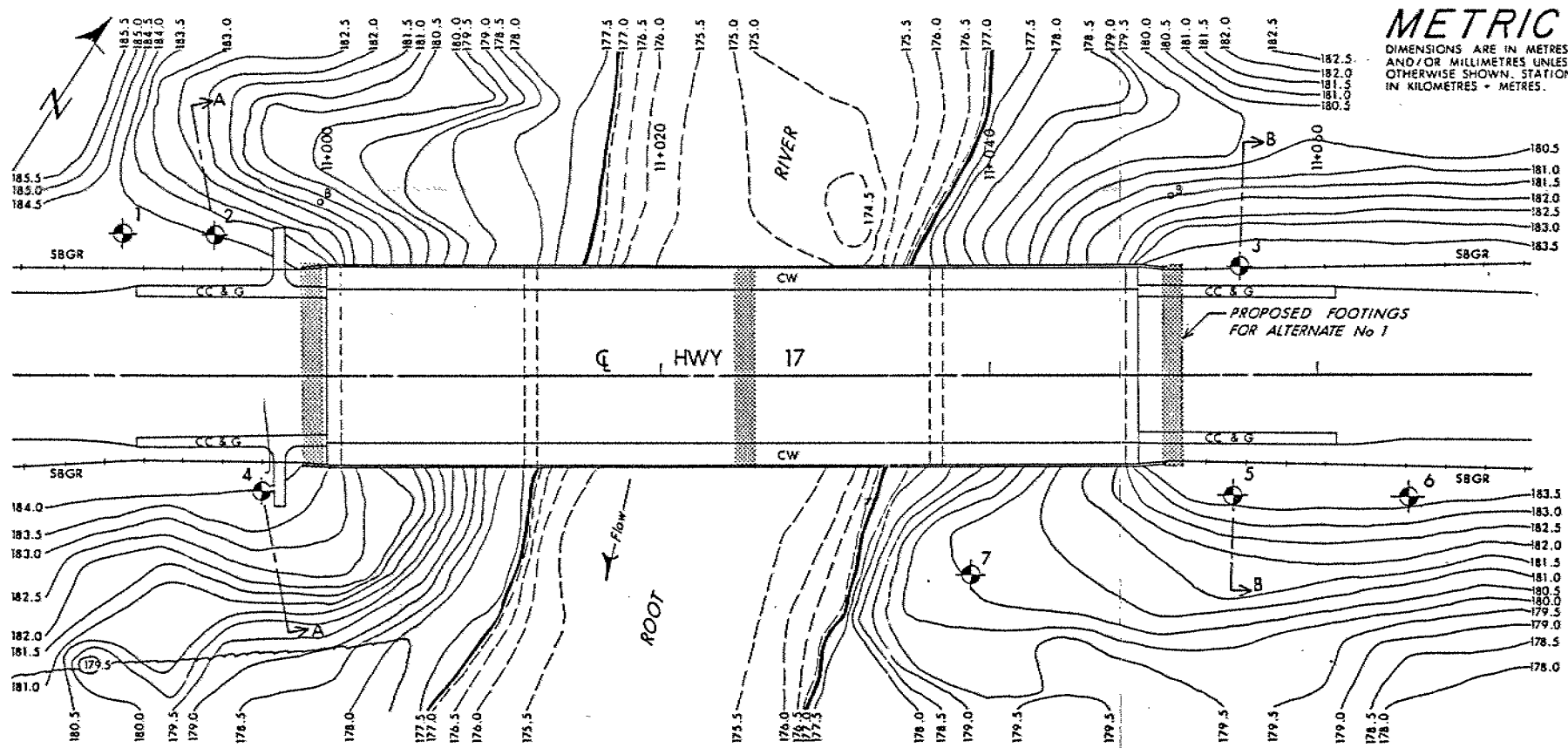
METRIC

W P 279-85-01 LOCATION STA. 11 + 038.8; 0/s 12.2 m Rt. C Hwy. 17 ORIGINATED BY JF
DIST 18 HWY 17 BOREHOLE TYPE BW-NW Casing, Washboring and Cone Test COMPILED BY DP
DATUM Geodetic DATE 86 04 01 and 02 CHECKED BY *GP*

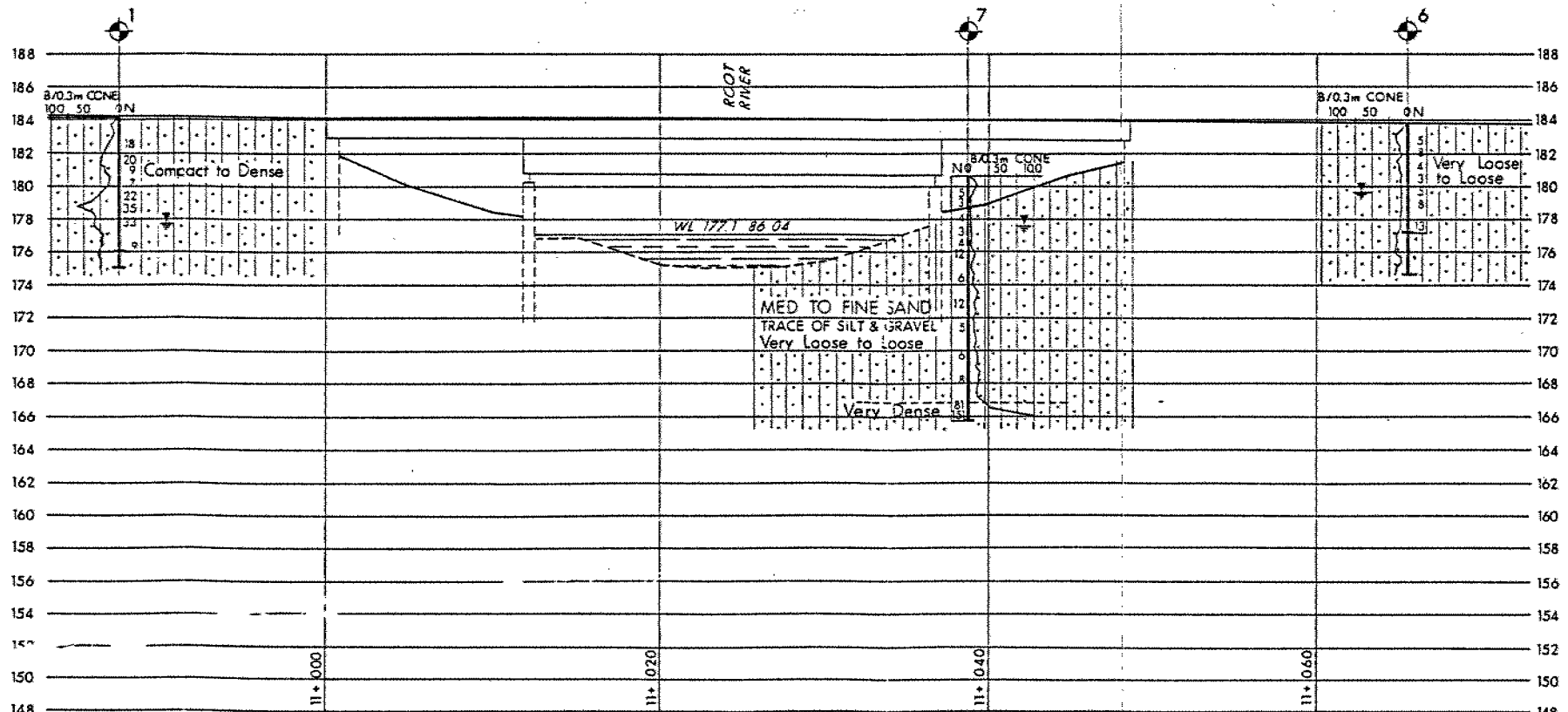
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
180.7	Ground Surface										
0.0	Topsoil										
	Medium to Fine Sand		1	SS	5		180				7 79 11 3
	Trace Silt, Gravel		2	SS	3						
	Very Loose - Loose		3	SS	4		178				0 97 (3)
			4	SS	3						
			5	SS	4						
	Brown		6	SS	12		176				12 87 (1)
			7	SS	6						
			8	SS	12		174				
			9	SS	5						
			10	SS	6		172				
			11	SS	8						
			12	SS	81		170				
	Very Dense		13	SS	151		168				
165.7							166				
15.0	End of Borehole										



SECTIONS
SCALE
4m 2m 0 4m



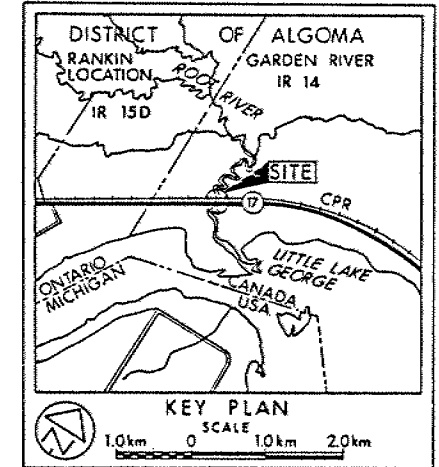
PLAN
SCALE
4m 2m 0 4m



PROFILE HWY 17
SCALE
4m 2m 0 4m

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

CONT NO WP No 279-85-01	SHEET
ROOT RIVER BRIDGE	
BORE HOLE LOCATIONS & SOIL STRATA	



- 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)
 - WL at time of investigation 86 04

No	ELEVATION	STATION	OFFSET
1	184.1	10+987.5	8.6m Lt
2	184.0	10+993.2	8.6m Lt
3	183.8	11+055.2	6.6m Lt
4	184.1	10+996.0	7.0m Rt
5	183.8	11+055.2	7.3m Rt
6	183.8	11+065.3	7.4m Rt
7	180.7	11+038.8	12.2m Rt

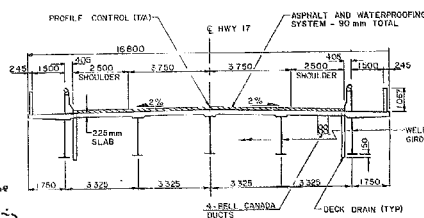
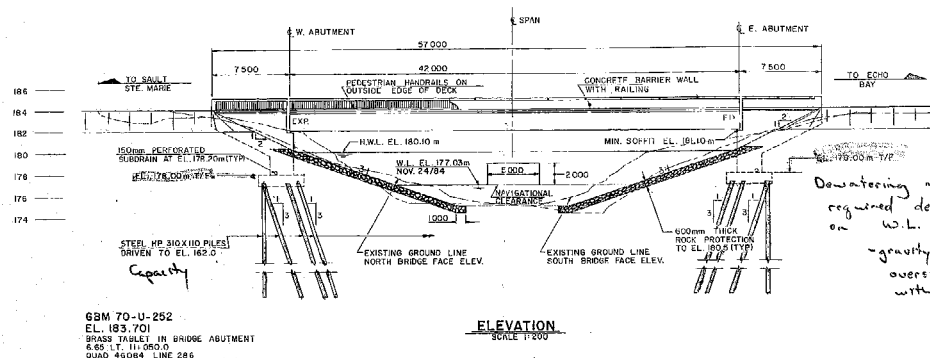
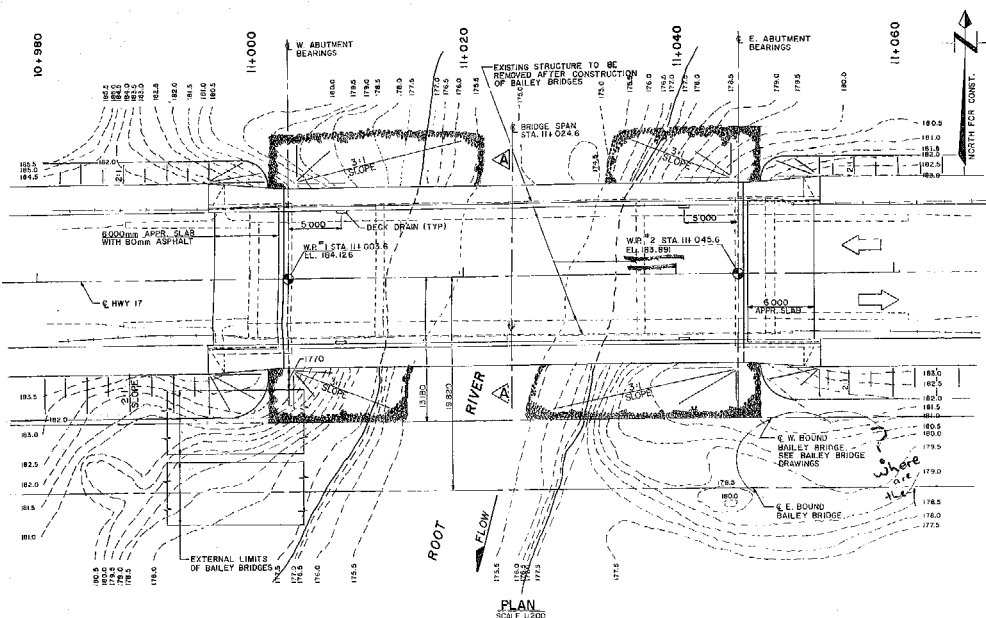
NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

Geacres No 41K-44

HWY No 17	DIST 18
SUBMD DP CHECKED	DATE 86 05 05
DRAWN DT CHECKED	SITE 385-88
	DWG 2798501-A



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

DIST. 18	HWY 17
CONT No	WP No 279-85-01
ROOT RIVER BRIDGE	SHEET
GENERAL ARRANGEMENT	

GENERAL NOTES

CLASS OF CONCRETE
ALL CONCRETE 30 MPa

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

CLEAR COVER TO REINFORCING STEEL
NOTES:
ABUTMENTS AND WINGWALLS: 100+25
FRONT FACE: 70+20
BACK FACE: 70+20
DECK: TOP: 70+20
BOTTOM: 40+10
REMAINDER UNLESS OTHERWISE NOTED: 70+20

CONSTRUCTION NOTES
BEARING SEATS SHALL BE FINISHED LEVEL AND TO THE SPECIFIED ELEVATIONS.

- LIST OF DRAWINGS
- 365-88 1 - GENERAL ARRANGEMENT
 - 2 - BOREHOLE LOCATIONS AND SOIL DATA
 - 3 - FOOTING LAYOUT AND REINFORCING
 - 4 - EAST AND WEST ABUTMENTS
 - 5 - STRUCTURAL STEEL I
 - 6 - STRUCTURAL STEEL II
 - 7 - REBAR
 - 8 - DECK DETAILS AND SCAFFOLD ELEVATIONS
 - 9 - BARRIER WALL WITH RAILING
 - 10 - RAILING FOR BARRIER WALL
 - 11 - PEDESTRIAN RAILING
 - 12 - 600mm APPROACH SLAB
 - 13 - JOINT ANCHORAGE AND ARMOURING
 - 14 - AS CONSTRUCTED ELEVATIONS AND DIMENSIONING
 - 15 - BRIDGE JUNE AND SITE NUMBER DATA
 - 16 - STANDARD DETAILS
 - 17 - QUANTITIES - STRUCTURE I
 - 18 - QUANTITIES - STRUCTURE II

- LIST OF ABBREVIATIONS
- T/A - TOP OF ASPHALT
 - T/F - TOP OF FOOTING
 - W.P. - WORKING POINT
 - W.L. - WATER LEVEL
 - H.W.L. - HIGH WATER LEVEL

APPLICABLE STANDARD DRAWINGS
OP 508 02 - BRIDGE DECK WATERPROOFING
DD 3503-3 - GRANULAR BACKFILL REQUIREMENTS

CROSS SECTION A-A
SCALE 1:100

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

DATE	BY	DESCRIPTION	DATE
DESIGN	CHK	CODE CHGNO 63 1004 A	DATE 12/20/99
DRAWN	CHK	SITE 365-88 DISTRICT	BOC-4, BOB-2AA

Downstreaming may be required depending on W.L. fluctuation - gravity drainage plus oversized excavation with tremies