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GEOCRES No. 41K-48DIST. 62 REGION _____W.P. No. 7819-95-01

CONT. No. _____

W. O. No. _____

STR. SITE No. 552

HWY. No. _____

LOCATION Bellevue Creek Culvert
ReplacementNo of PAGES - =====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____



Ministry
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REMARKS _____

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FOUNDATION DESIGN SECTION

foundation investigation and design report

**ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION**

WP 7819-95-01
HWY 552

DIST 62
STR SITE 38S-355

Hwy. 552 and Bellevue Creek Culvert Replacement

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GEOCRES 41K-48

DATE JUN 04 1996

FOUNDATION INVESTIGATION REPORT

For:

Hwy. 552 and Bellevue Creek Culvert Replacement

WP 7819-95-01, Site No. 38S-355

HWY. 552, District 62, Sault Ste. Marie

Introduction

This report summarizes the results of a foundation investigation conducted at the aforementioned site. It is proposed to replace the existing failed open culvert with either an Acrow bridge resting on piles or shallow spread footings or alternatively replacing this culvert with a new one. Approach fills in the order of magnitude of 6-7 metres will be required. This report contains factual information obtained from this investigation pertaining to structural foundations and related earth works.

Site Description

The site is located approximately 8 km east of Hwy. 17 along Hwy. 552 at the Bellevue Creek crossing in the Township of Vankkougnet, District of Algoma.

The topography of the area consists of generally flat to gently rolling lands covered with deciduous and coniferous trees and short wild grasslands. The area is residential with private homes just east and west of the culvert. The terrain dips down sharply into the valley which contains the Bellevue Creek with the area in and around the creek being covered with stones. This area appears to have been washed out on numerous occasions and is within the flood plain. During large rain fall periods the area floods very easily with the previous culvert unable to meet the capacity requirements. As a consequence the fill placed above the culvert has been completely washed out.

Investigation Procedures

Soil data and inherent properties were obtained by in situ and laboratory testing. The procedures employed are discussed below.

Field Investigation

The field work for the investigation was carried out between 95 09 21 and 95 09 24 and consisted of three boreholes which were advanced to a maximum depth of 26.4 m (El. 172.0 m) below the ground surface. A borehole was placed at the location of each proposed footing/pilecap to the east and west of the failure zone along the highway. A third borehole was drilled at the creek elevation, within the flood plain on the shore. Two dynamic cone tests were advanced at the bottom of the borings in order to extend beyond the depths of sampling.

Along the highway surface the two boreholes had elevations of 198.8 m and 198.4 m respectively to the east and west. The third borehole at the creek level had an elevation of 194.3 m.

Track mounted CME 55 equipment employing hollow stem augering techniques was used to advance all boreholes. In general, disturbed subsoil samples were retrieved at 0.75 m intervals for the surficial 4.5 m and 1.5 m thereafter. These samples were supplemented by retrieving 5 undisturbed shelly tube samples for laboratory evaluation and testing. In situ vane tests were also conducted in cohesive materials to determine the undisturbed and remoulded undrained shear strengths of soil. The tests were conducted employing the standard MTO 'N' vane in accordance with ASTM D2573.

Groundwater depths were obtained by monitoring the levels in the open boreholes throughout the duration of the field investigation. All boreholes were backfilled at the completion of the fieldwork.

Survey information related to the location and elevation of boreholes were taken from plans provided by the Structural and Surveys and Plans Section, Northwestern Region.

Laboratory Analysis

The following laboratory tests were carried out on select soil samples.

1. Atterberg Limit Test
2. Grain Size Analysis
3. Natural Moisture Content
4. Unit Weights

Laboratory test results are given in the following section of this report and are illustrated on figures and borehole logs included in the appendix.

Subsurface Condition

General

The subsoil stratigraphy from the highway surface consisted of 1.4 - 2.1 m of fill composed of a mixture of sand and gravel. This layer underlying the pavement also contained traces of organics. The fill deposit is underlain by a silty sand, trace gravel, and an organic silt. The large presence of organics indicates that this area and depth was part of the original flood plain. To the east (BH1) the silty sand is 2.3 m thick, with the organic silt lying underneath having a 3.4 m thickness. To the west (BH2) the organic silt was not encountered to the same extent with a 0.7 m thick layer sandwiched within the silty sand. Beneath the above non-cohesive strata was a red/brown clay, trace/some silt which extended beyond the scope of this investigation. The borehole located within the creek bed (BH3) confirmed that the culvert rests on a silty sand, with a trace of gravel and organics. Within this borehole the silty sand layer had a 4 m thickness. The cohesive clay strata was confirmed at a depth of 4.0 m with the organic silt deposits on the banks not encountered. Throughout the site, the clay deposit was encountered at an elevation of 188.3 to 191.7 m with a depth of 7.1 - 10.1 m at the proposed abutment location and a depth of 4 m beneath the culvert.

The plan and location of borings and the stratigraphical profile are shown on Drg.

No. 78199501-A in the attached appendix. The field and laboratory test results are plotted on the record of borehole sheets also included in the appendix of this report. A brief description of the different soil types is given below.

Mixture of Sand and Gravel, trace Organics (FILL)

Beneath the highway surface the fill material was found to be composed of a mixture of sand and gravel, trace organic which extended down below the pavement to depths of 1.4 m to 2.1 m, north and south respectively. There appears to be an additional layer of pavement beneath the surficial one, with a 0.3 m thickness of granular separating the two. The original grade of the highway appears to have been raised.

The standard penetration resistance 'N' values ranged from 6 to 53 blows/0.3 m.

Silty Sand, trace Gravel, trace Organics

Underlying the fill was a silty sand, trace gravel, trace organics which was found to extend beneath the creek. At the south (BH 2) location an organic silt deposit was sandwiched within this layer. The silty sand deposit had an upper thickness of 1.6 m and a lower thickness of 5.7 m. Elsewhere this layer had a thickness of 2.3 m to 4 metres.

Results of grain size distribution tests carried out on select samples are shown on Figure 1 in the appendix. The results indicate the material comprises of 1 - 3 % gravel, 58 - 82 % sand, 12 - 35 % silt and 3 - 4 % clay.

Standard penetration tests carried out in this deposit revealed 'N' values of 2 to 17 blows/0.3 m having a very loose to compact state of denseness.

Organic Silt, trace Sand

Encountered only within the two boreholes at the abutment locations was an

organic silt, trace sand. To the south (BH 2) the layer had a thickness of 0.7 m, while to the north (BH 1) it had a thickness of 3.4 m. A plasticity test (Figure 2) shows the material to fall below the 'A' line indicating the presence of organics.

Standard penetration resistance 'N' values ranged from 2 to 3 blows/0.3 m, having a very loose to loose state of denseness.

Clay, trace/some Silt

Underlying all the layers above was a cohesive clay, trace/some silt which was found to be at a depth ranging from 7.1 to 10.1 metres.

Results of grain size distribution tests carried out on select samples are shown in Figure 3 in the appendix. The results indicate the material contained a large percentage of clay with little sands and gravels. The deposit is comprised of 0 % gravel, 0 % sand, 15 - 30 % silt and 70 - 85 % clay.

The results from the Atterberg Limit Tests performed on the fine fraction of this deposit is summarized as follows:

	<u>Range</u>	<u>No. of Tests</u>
Natural Moisture Content (w)	42 - 55 %	5
Liquid Limit (W_L)	53 - 65 %	5
Plastic Limit (W_p)	22 - 24 %	5
Plastic Index (I_p)	31 - 41	5
Undrained Shear Strength C_u (kPa)	40 - 76	12
Sensitivity	3 - 10	12
Unconfined Undrained (UU - kPa)	43 & 52	2
Unit Weight (kN/m^3)	16.1 - 16.9	4

From the plasticity chart (Figure 4), the layer can be classified as a clay with high plasticity.

In this stratum the shear strength was fairly consistent with depth increasing only slightly. Values ranged from 40 - 76 kPa with an average of 59 kPa, indicating a firm to stiff state of consistency. In situ sensitivity ranged from 3 - 10 corresponding to a sensitive to very sensitive classification.

Consolidation testing in the laboratory was also performed to determine settlement characteristics of this deposit. The results indicate an initial void ratio of (e_o) 1.42 and 1.75 and a compression index of (c_c) 0.81 and 1.0.

In this stratum the standard penetration resistance 'N' values ranged from 1 to 9 blows/0.3 m.

Groundwater Conditions

Groundwater levels obtained at the time of the investigation revealed that the groundwater table is at a depth of 4 - 4.5 m below the highway surface at the abutment locations and at the surface for the borehole drilled on the creek bed.

Discussion and Recommendations

Inspection of the failure area indicate the culvert was undermined due to erosion of the Silty Sand deposit beneath the foundation. It is estimated that the culvert was built in the 1930's with the erosion process occurring over a long period of time. Recent flooding in the region apparently hastened this process with the flow volume exceeding the capacity of the culvert. As a result the entire embankment was washed away.

Structure Foundations

The structure foundations can be founded on conventional spread footings or end-bearing steel H-piles as described below.

OPTION ONE - ACROW BRIDGE

The following pertains to the implementation of an Acrow two lane truss structure.

a) Deep Foundation Units - Steel H-Piles

The structure foundations can be founded on steel H-piles. As an end bearing stratum was not encountered capacities were calculated relying on the skin friction of the piles. For the purposes of the O.H.B.D.C., design axial capacity for vertical piles are summarized below for various lengths of piles.

<u>Axial Capacities - Driven Steel H-Piles</u>			
<u>Pile Type</u>	<u>Pile Length</u>	<u>Bearing Capacity at SLS type II (kN)</u>	<u>Factored Capac. at ULS (kN)</u>
HP310X110	10 m	185	125
	20 m	485	325
	30 m	790	525
HP310X79	10 m	180	120
	20 m	475	320
	30 m	770	515

As settlement of the Clay is expected to be of already occurred due to the embankment loading, negative skin friction is expected to be minimal and is therefore not included in these values.

b) Shallow Foundations - Spread Footings

Due to the presence of a relatively weak surficial non-cohesive deposit and organics, shallow spread footings can be accommodated only on the condition this material be removed down to an elevation of 191 m and a granular pad be utilized to distribute the load. Removal would be quite extensive as the weak deposits extend 7 - 10 m. This option may not be the most practical.

Capacities are provided below based on the thickness of the granular pad.

<u>Thickness of Pad (m)</u>	<u>Bearing Capacity at SLS Type II (kPa)</u>	<u>Factored Capacity at ULS (kPa)</u>
1	150	250
2	200	350
3	350	500
4	480	700

These values are based on a 3 m wide footing resting on the cohesive Clay stratum.

Footings should be protected against any potential for undermining caused by erosion with scour protection measures. Due to the potential for large amounts of rainfall and flooding in the area these protection measures should strongly be considered. Regardless of which foundation option is chosen rip rap should be placed along the creek bed and along the forward slopes.

OPTION TWO - CULVERT REPLACEMENT

Alternatively a replacement culvert could be constructed with the embankment filled up to the highway grade. This option may not be feasible as the creek will have to be diverted and an extensive dewatering scheme would be required. In addition, this will raise environmental concerns. The following addresses this option.

Culvert - Shallow Spread Footings

If this option is chosen, it is recommended that appropriate aprons and rip rap be constructed at the culvert inlet and outlet. The design of the scour protection shall be made in conjunction with applicable hydrological parameters. The culvert size should accommodate the greater flows experienced in the regions on occasion. It is additionally recommended to utilize a closed box culvert design as this will provide greater distribution of load and greater resistance to failure due to erosion. The base of the culvert is estimated to be approximately at an elevation of 190 - 191 m, placing it within the loose Sand deposit. The 1 to 2 m of sand beneath this elevation should be removed and replaced with suitable granular backfill.

The corresponding soil capacities are summarized below.

<u>Bearing Capacity at SLS Type II</u>	<u>Factored Capacity at ULS (kPa)</u>
175	250

To facilitate the construction of the culvert, a temporary diversion of the Bellevue Creek will be required. This would require an extensive dewatering scheme, such as sheet piling with well points. As previously discussed, this may not be a very feasible option and is not recommended.

Upon removal of Organic and weak Sand deposits, settlement is expected to be

minimized as consolidation of the underlying clay has already taken place with no additional grade raise proposed. Any settlement will be elastic in nature, approximately 1 % of any granular fill placed upon the removal of the underlying organic layers.

Earth pressure should be computed as per Section 6.1.2.2. of the O.H.B.D.C., and an unyielding foundation condition may be assumed for the computations. The Granular 'A' or 'B' backfill should be in accordance with the Special Provision No. 109F03. The following parameters are recommended for the granular backfill.

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Angle of Internal Friction	35°	30°
Unit Weight (kN/m ³)	22.8	21.2

Approach Embankments

The results of slope stability analysis of approach fills have been conducted using total stress parameters applying the Limit Equilibrium method developed by Sarma (Sarma,1973). The results reveal that fill heights of up to 7 metres can be constructed with 2H:1V slopes. Rock Protection should be placed along the forward slopes if the Acrow bridge is to be constructed and along the side slopes of the constructed embankments.

Construction Considerations

Any footings or pile caps should have a minimum of 2.1 m earth cover to protect against frost protection.

Within the limits of the approach fills, if soft soil is encountered, this should be excavated and replaced by compacted granular fill. Any temporary construction excavations may be carried out at 1.5H:1V slopes.

Miscellaneous

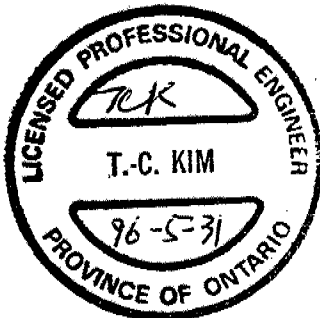
The fieldwork for this investigation was carried out under the supervision of M. Michalek, Jr. Foundation Engineer, utilizing equipment owned and operated by Colbar Resources, Waldan, Ontario.

The project was carried out under the general supervision of T, C, Kim, Sr. Foundation Engineer. The report was written by M. Michalek, reviewed and approved by T. C. Kim.



A handwritten signature in black ink, appearing to read "M. Michalek, Jr.", written in a cursive style.

M. Michalek, P. Eng.
Jr. Foundation Engineer

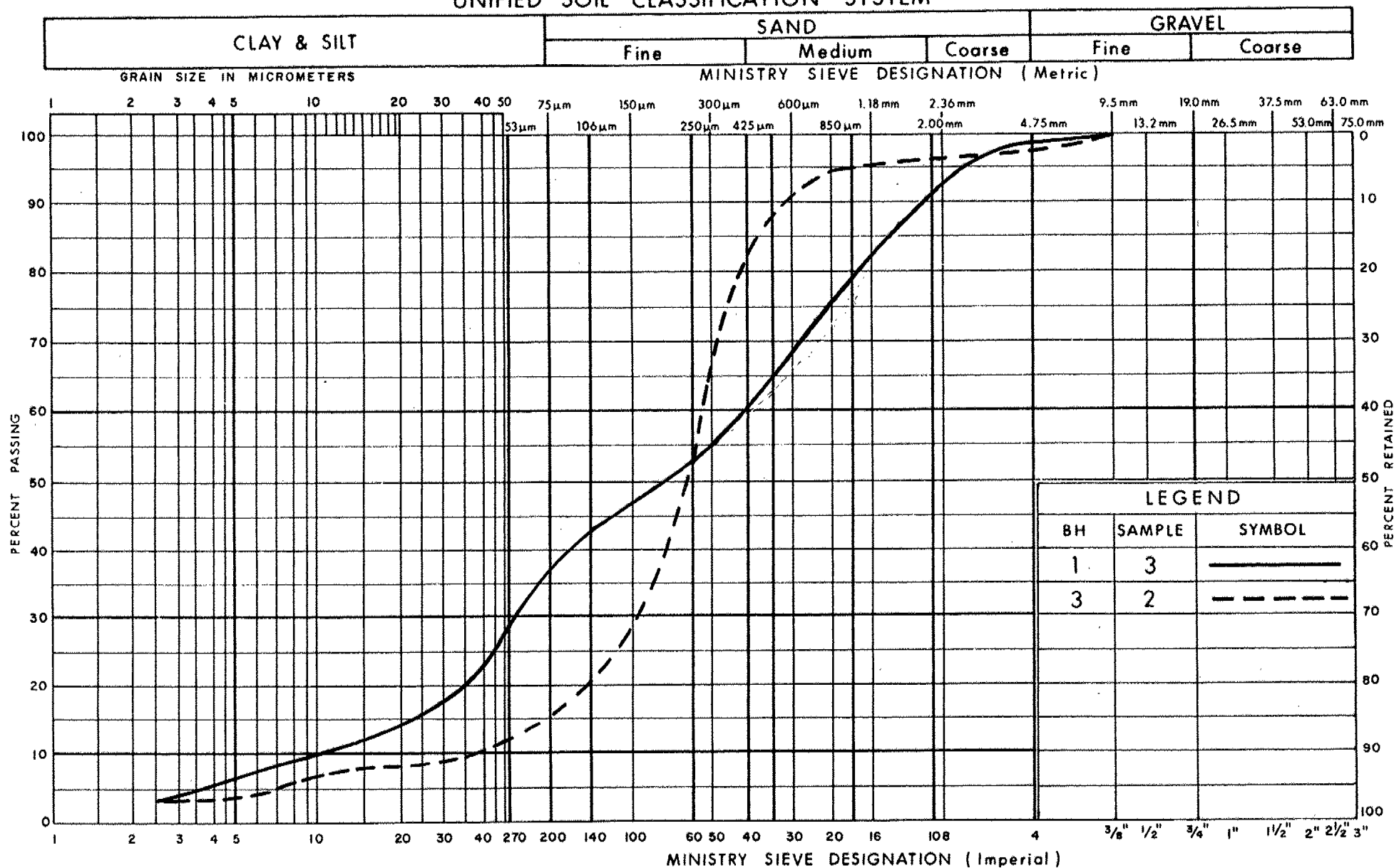


A handwritten signature in black ink, appearing to read "Taechol Kim", written in a cursive style.

T. Kim, P. Eng.
Sr. Foundation Engineer

APPENDIX

UNIFIED SOIL CLASSIFICATION SYSTEM

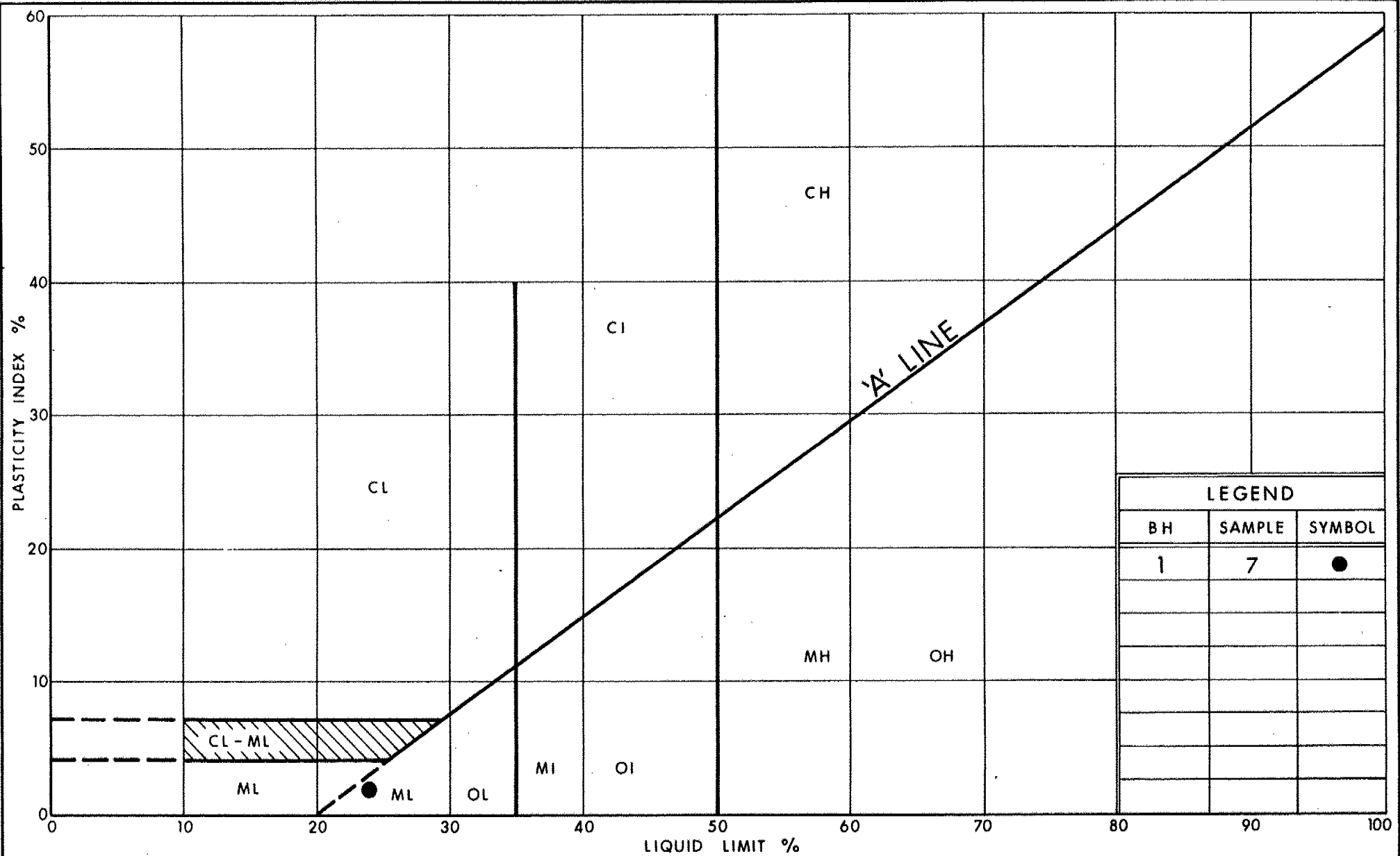


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GRAIN SIZE DISTRIBUTION
SILTY SAND,
TRACE GRAVEL, TRACE ORGANICS

FIG No 1

W P 7819 -95 -01



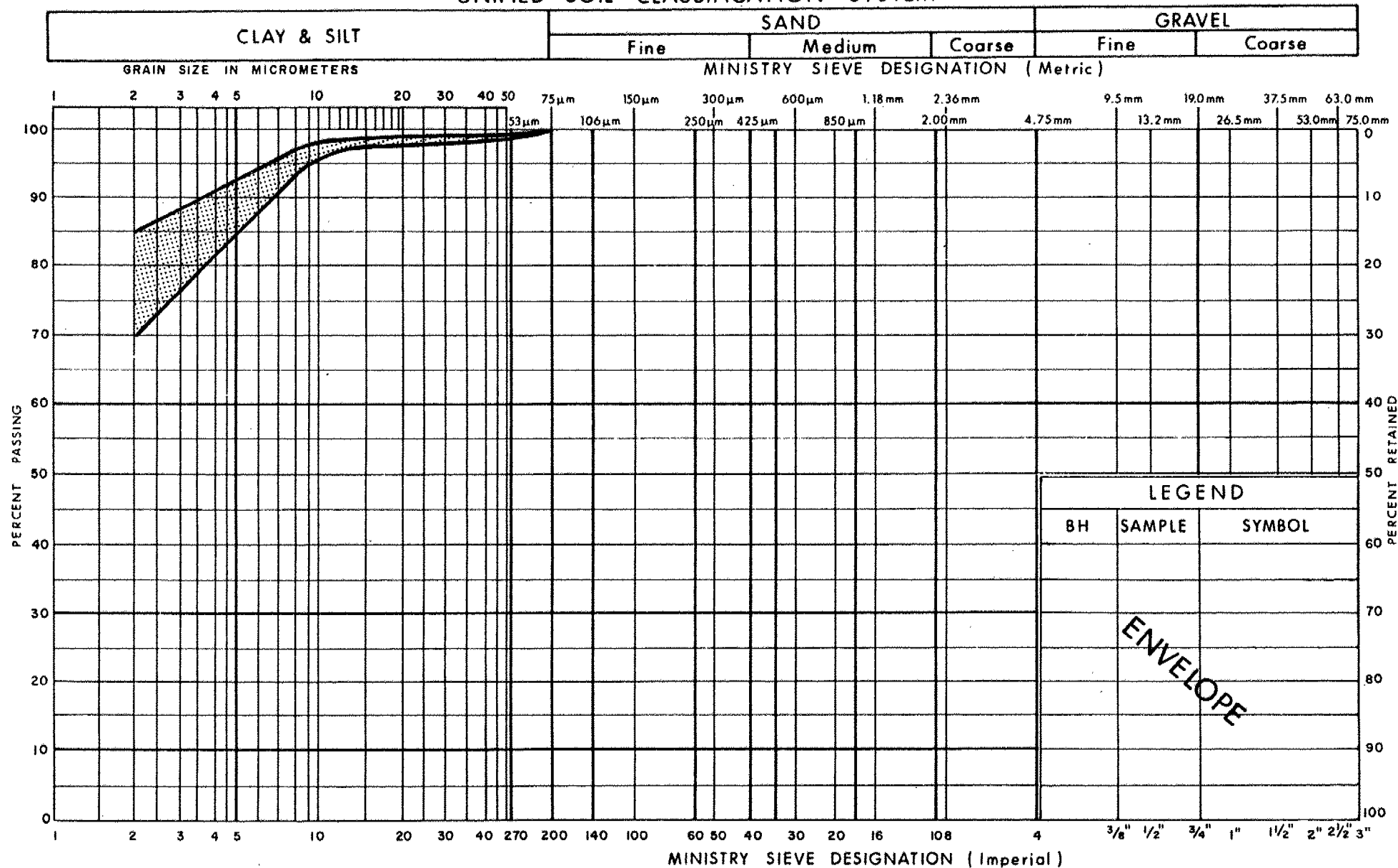
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PLASTICITY CHART ORGANIC SILT, TRACE SAND

FIG No 2

W P 7819-95-01

UNIFIED SOIL CLASSIFICATION SYSTEM

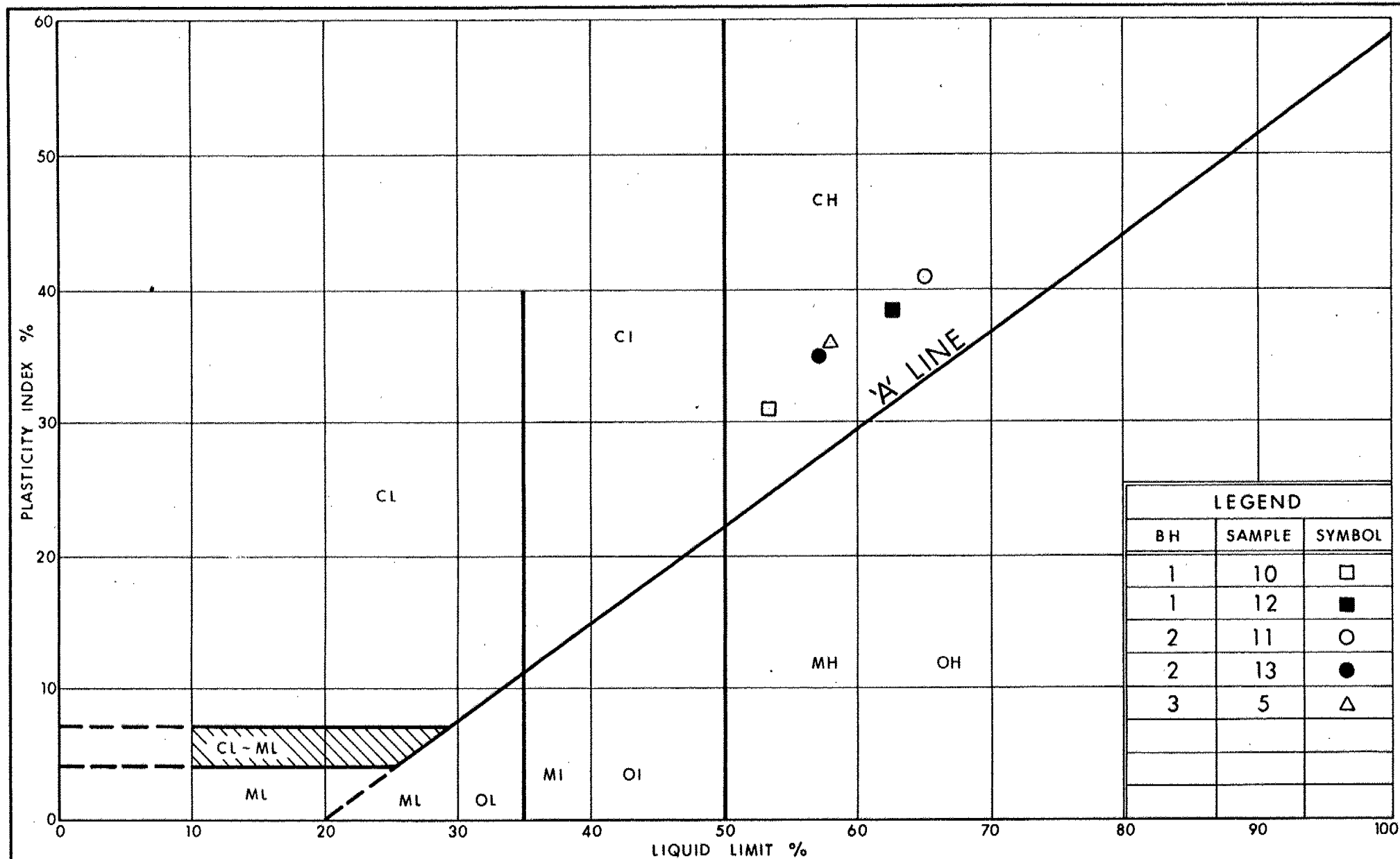


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GRAIN SIZE DISTRIBUTION
CLAY,
TRACE / SOME SILT

FIG No 3

W P 7819-95-01



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PLASTICITY CHART CLAY, TRACE / SOME SILT

FIG No 4

W P 7819 - 95 - 01

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 7819-95-01 LOCATION Sta. 17 + 307.2 Centreline of Hwy. 552 ORIGINATED BY M.M.
 DIST 62 HWY 552 BOREHOLE TYPE Hollow/Solid Stem Augers COMPILED BY M.M.
 DATUM Geodetic DATE 95 09 21 CHECKED BY T.K.

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					
198.8	Highway Surface															
0.0	Mixture of Sand and Gravel [FILL]		1	SS	53											
197.4	Trace Organics		2	SS	9											
1.4	Silty Sand Trace Gravel Trace Organics Very Loose to Loose		3	SS	6											
195.1	Brown		4	SS	4											
3.7	Dark Brown		5	SS	2											
	Organic Silt Trace Sand Very Loose		6	SS	3											
191.7	Dark Brown		7	SS	3											
7.1	Red/Brown		8	SS	3											
	Clay Trace/Some Silt Suff		9	SS	9											
			10	TW	PM											
			11	SS	1	/46cm										
			12	TW	PM											
			13	SS	1	/46cm										
			14	SS	1	/46cm										
			15	TW	PM											
174.0																
24.8	End of Borehole															

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 7819-95-01 LOCATION Sta. 17 + 270.7 Centreline of Hwy. 552 ORIGINATED BY M.M.
DIST 62 HWY 552 BOREHOLE TYPE Hollow/Solid Stem Augers & Cone Test COMPILED BY M.M.
DATUM Geodetic DATE 95 09 22 CHECKED BY T.K.

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	20 40 60 80 100					
198.4	Highway Surface													
0.0	Mixture of Sand and Gravel [FILL] Trace Organics		1	SS	42		198							
196.3			2	SS	6									
2.1	Silty Sand Trace Organics Very Loose		3	SS	4		196							
194.7			4	SS	3									
3.7 194.0	Organic Silt, Trace Sand Very Loose Dark Brown		5	SS	2		194							
4.4			6	SS	2	/46cm								
	Silty Sand Trace Gravel Very Loose to Compact		7	SS	6		192							3 58 35 4
			8	SS	17		190							
188.3			9	SS	14		188							
10.1	Clay Trace/Some Silt Stiff		10	SS	2		186							
			11	TW	PM		184							
			12	SS	2		182							
			13	TW	PM		180							
			14	SS	2		178							
			15	SS	5		176							
			16	SS	33		174							
172.0							170							
26.4	End of Borehole						168							
	* Cone Test termination depth 32.5m (Elev 165.9). The last two values not shown are 85 & 100 blows.													

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

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:

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

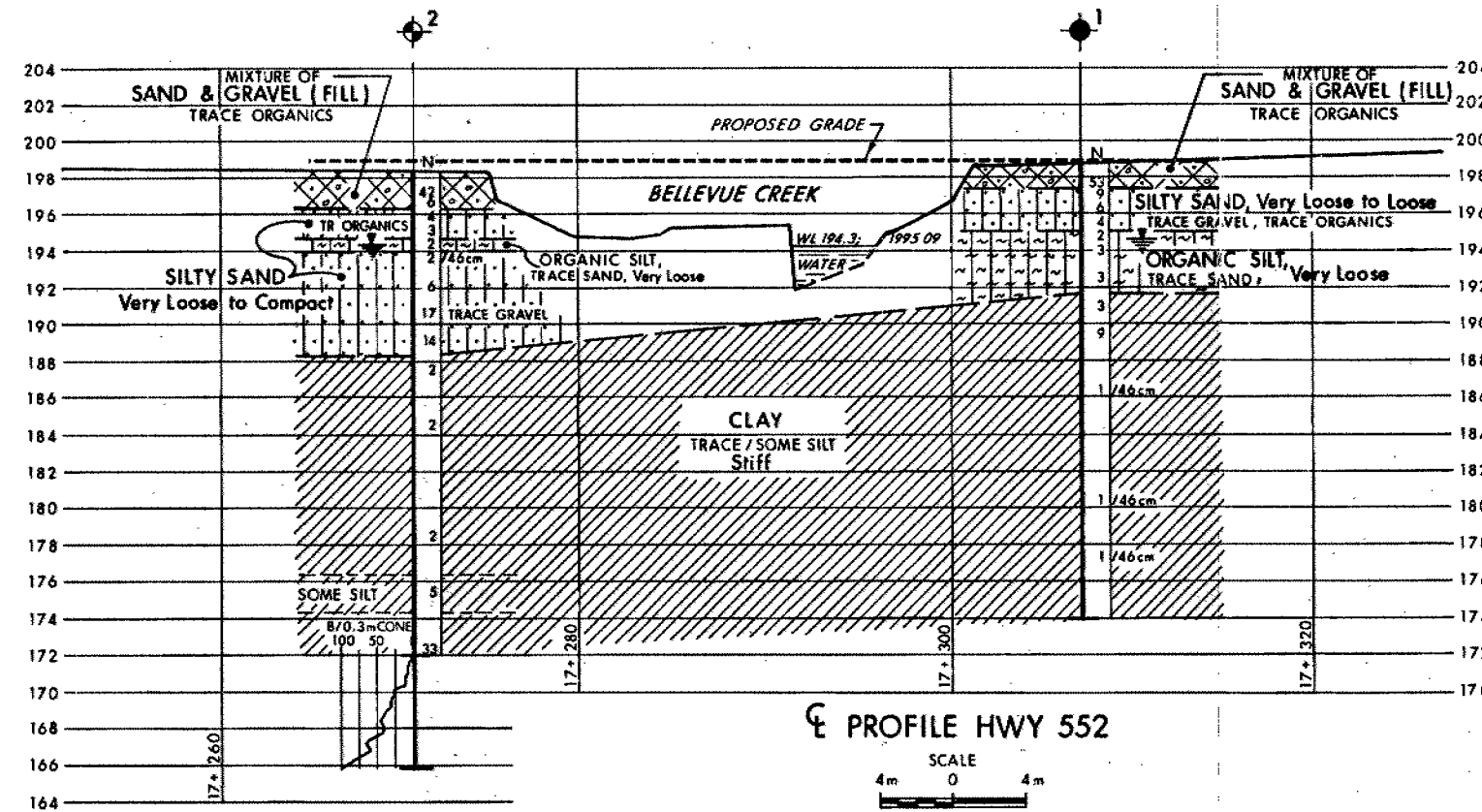
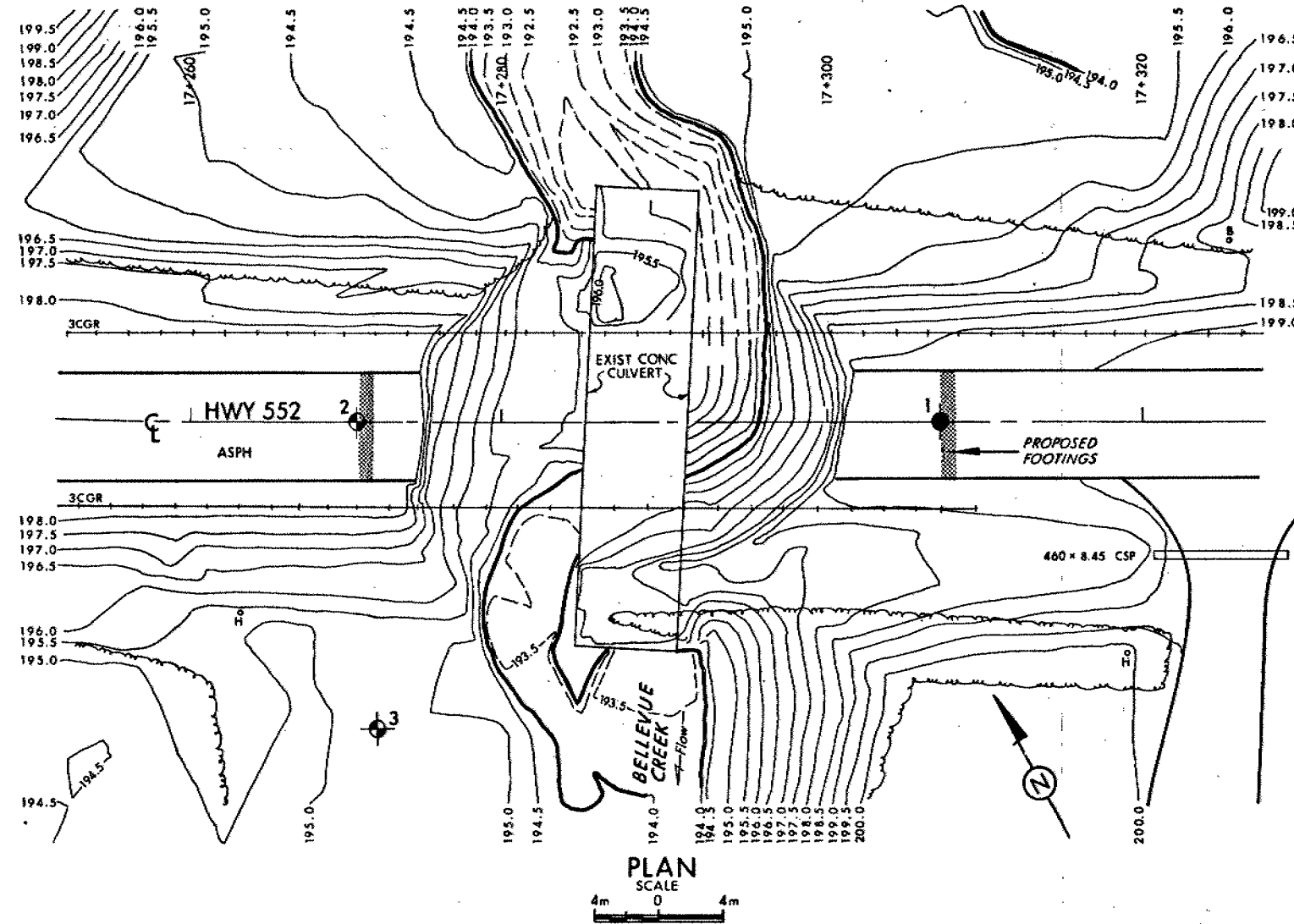
u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	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						



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

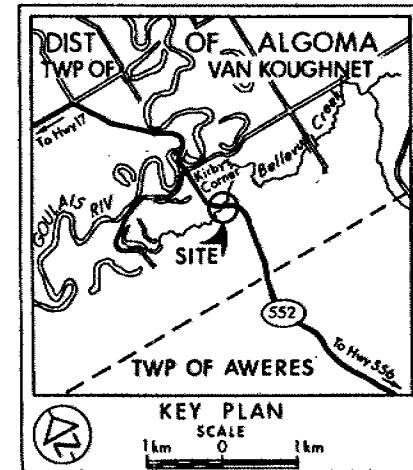
CONT No
WP No 7819-95-01

BELLEVUE CREEK

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 1995 09

NOTE
For Subsoil Information of B.H. No.3
refer to Record of Borehole Sheet.

No	ELEVATION	STATION	OFFSET
1	198.8	17+307.2	ℰ
2	198.4	17+270.7	ℰ
3	194.3	17+272.0	19.0m 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 GC 2.01 of OPS Gen. Cond.



REV.	DATE	BY	DESCRIPTION
1			

Geocres No 41K-48

HWY No 552	DIST 62
SUBMD MM CHECKED MM DATE 1996 01 15	SITE 385-355
DRAWN RS CHECKED RS	DWG 78199501-A