G.I.-30 SEPT. 1976

GEOCRES No. 41K-47

DIST. 18 REGION

W.P. No. 84-90-01

CONT. No. 94-205

W. O. No.

STR. SITE No. 38S-211

HWY. No. 548

LOCATION Hwy 548 @ Two Tree River

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

Ministry
of
Transportation

Ontario

FILE No. ______________ **DATE** ______________

Tom Mills 705-942-8120
Glen Carson 705-746-0707
(C.S. mtu)

REMARKS See WP 1515-68-00 for
Richardson Creek / Hwy 548
([illegible] th of site)

SETTLEMENT PLATE
PLOTS

ADM-G-24 79-06

DIST. 18 HWY. 548
CONT. No. 94-205
WP No. 84-90-01

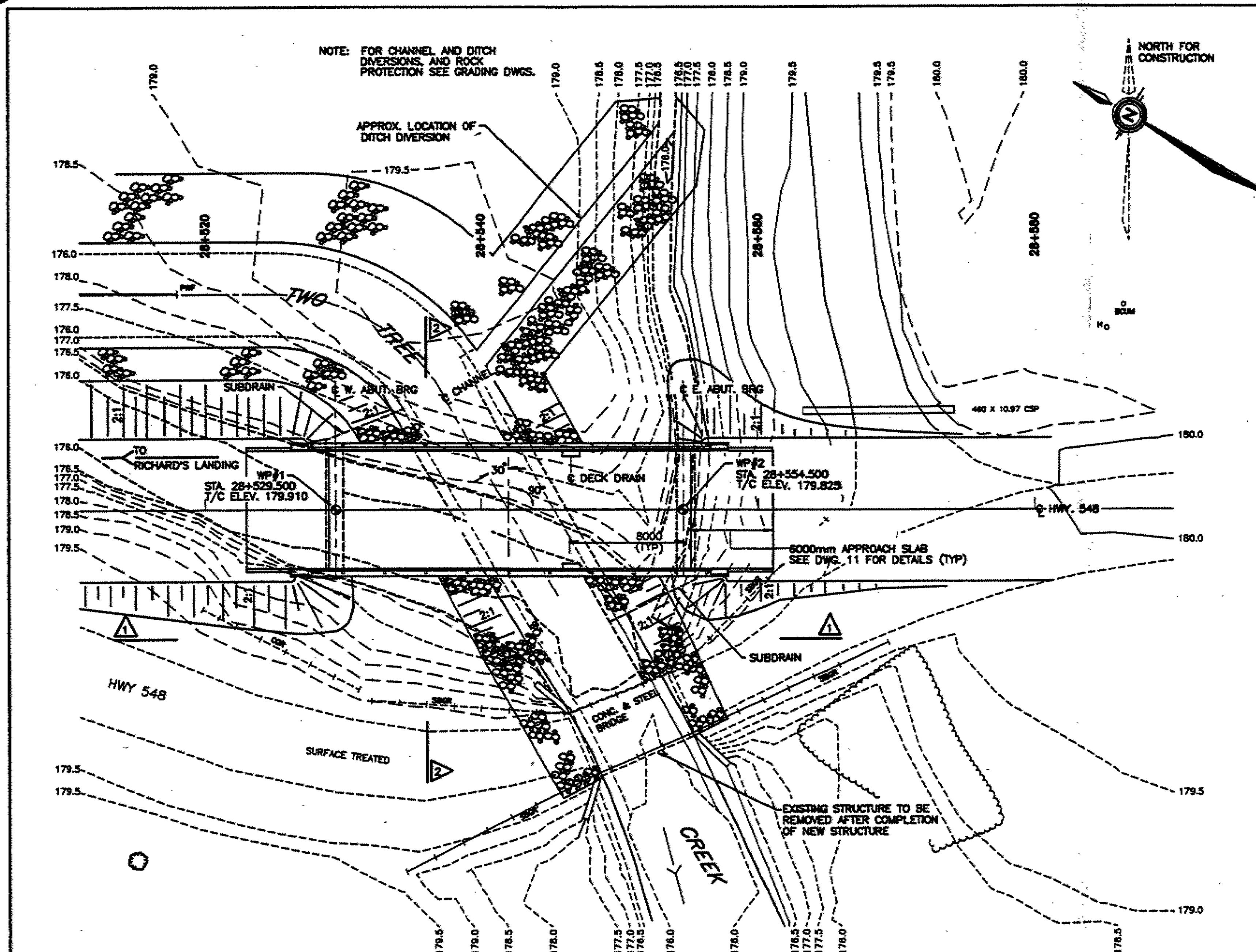
TWO TREE RIVER BRIDGE

GENERAL ARRANGEMENT

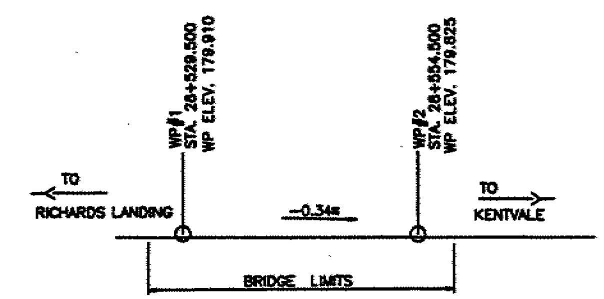
SHEET 12

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE NOTED



PLAN
1:200



PROFILE OF HWY. 548
NTS

GENERAL NOTES

1. CLASS OF CONCRETE
   ALL CONCRETE 30MPa
2. CLEAR COVER TO REINFORCING STEEL
   ABUTMENTS, WINGWALLS & RETAINING WALLS
   FRONT FACE .... 70 ± 20
   BACK FACE .... 70 ± 20
   DECK
   TOP .... 80 ± 20
   BOTTOM .... 40 ± 10
   REMAINDER .... 70 ± 20 UNLESS OTHERWISE NOTED
3. REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX C DENOTE COATED BARS.

CONSTRUCTION NOTES

- GIRDER SEATS SHALL BE FINISHED LEVEL AND TO THE SPECIFIED ELEVATIONS.
- NO BACKFILL SHALL BE PLACED UNTIL DECK CONCRETE HAS REACHED A STRENGTH OF 20 MPa.
- BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.

CONSTRUCTION SEQUENCE

- THE TOP PORTION OF THE ABUTMENT WALL ABOVE THE CONSTRUCTION JOINT SHALL BE CAST INTEGRALLY WITH THE DECK. THE DECK SHALL BE PLACED PRIOR TO CASTING THE ABUTMENT WALLS.

LIST OF DRAWINGS

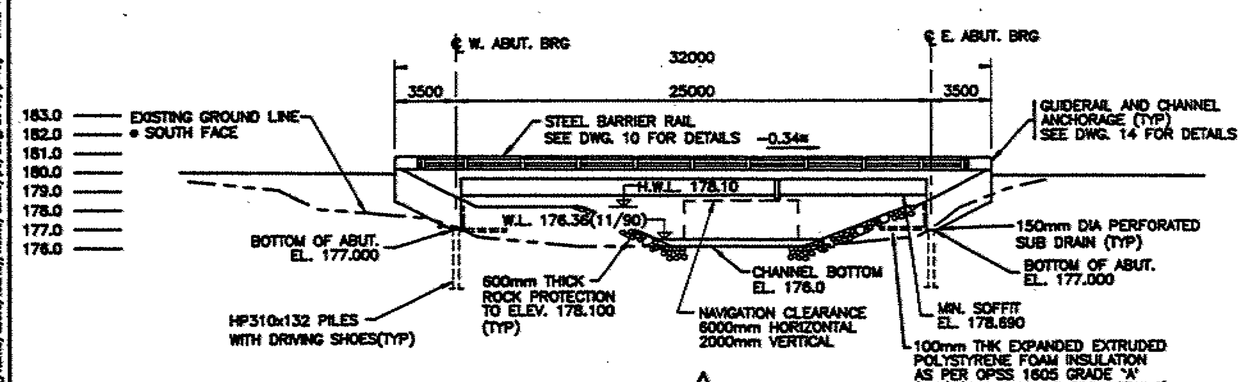
1. GENERAL ARRANGEMENT
2. BORE HOLE LOCATION AND SOIL STRATA
3. PILE LAYOUT
4. ABUTMENT DETAILS
5. WINGWALL DETAILS
6. STRUCTURAL STEEL I
7. STRUCTURAL STEEL II
8. DECK DETAILS
9. SCREED ELEVATIONS
10. STEEL BARRIER RAIL DETAILS
11. 6000mm APPROACH SLAB
12. PILE DRIVING – STEAM & DIESEL HAMMERS
13. MISC. DETAILS I
14. MISC. DETAILS II

LIST OF ABBREVIATIONS

WP – WORKING POINT
WL – WATER LEVEL
T/C – TOP OF CONCRETE

LIST OF APPLICABLE STANDARD DRAWINGS

OPSD 3501.00 – GRANULAR BACKFILL REQUIREMENTS



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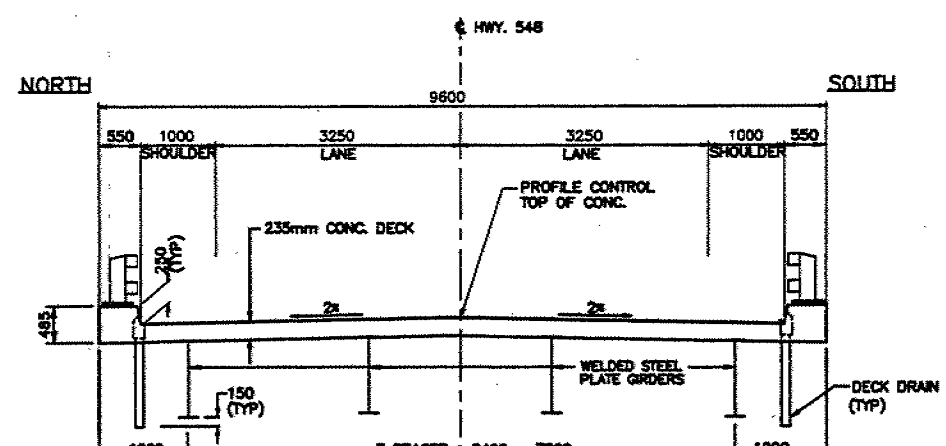

2
1:50

J.B. CONNELL
W.C. PRYSTANSKI

BM 179.514
GEODETIC DATUM
RR SPIKE IN HYDRO POLE
13.8 LT. 28+254.2

AUTO-TROL FILE : ttl(/home/users/jas/dwg/2tree/p1.dg DATE : 10/28/93 14:22:33

REVISIONS

DESIGN JBC	CHK WCP	CODE OHBDC-83	LOAD C1	DATE OCT/92	
DRAWN LP	CHK JBC	SITE 38S-211	STRUCT	SCHEME	DWG 1

FOUNDATION INVESTIGATION REPORT

CONTRACT NO. 94-205

RECEIVED
MAR 2 1 1994
FOUNDATION DESIGN SECTION

Ministry of Transportation
Ontario

INDEX

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

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	$-^\circ$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	$-^\circ$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

FOUNDATION INVESTIGATION REPORT
FOR PROPOSED STRUCTURE REPLACEMENT, HWY 548
TWO TREE RIVER, SITE NO. 38S-211
DISTRICT 18 - SAULT STE. MARIE
W.P. 84-90-01

1.0 INTRODUCTION

This report summarizes the results of a foundation investigation completed at this bridge crossing site. It is understood that the existing bridge crossing will be realigned over Two Tree River on Highway 548 on St. Joseph Island, Ontario. The present crossing structure, consisting of a relatively short span, concrete and timber bridge structure, incorporates an abrupt curve on the southwest side in order to make a perpendicular approach to the river.

2.0 SITE DESCRIPTION AND GEOLOGY

The project site is located along a straight section of Hwy. 548 between stations 28+475 and 28+525 at the intersection of the Two Tree River, in St. Joseph Township near the western edge of St. Joseph Island, Ont. St. Joseph Island is located along the north shore of Lake Huron to the west of Manitoulin Island and some 45km to the south-southeast of Sault Ste. Marie, Ont.

The terrain in the immediate area of the project site is relatively flat farmland with bedrock outcroppings several kilometres away to both the northwest and southeast. The river flows from north to south adjacent to the north side of the highway turning sharply to the southwest at the existing bridge structure. It then flows in a generally southwesterly direction of Munuscong Lake. Because of the general orientation of the river, both existing approaches to the bridge curve sharply in order to provide a more perpendicular approach to the river crossing. The resulting sudden change in present alignment of the highway reportedly requires traffic to slow abruptly at both the north and south bridge approaches.

Physiographically, St. Joseph Island straddles the boundary between the extreme northern edge of the Michigan Basin and the southern edge of the Southern Province. The Southern Province lithology found on the extreme northern end of the island consists of an assemblage of Middle Precambrian rock belonging to the Cobalt Group of the Huronian Formation. Rock types consist of conglomerate, greywacke, orthoquartzite, siltstone and argillite.

The rock formations underlying and exposed across the majority of the island, including the project site, are Middle to Upper Ordovician in age, and lie unconformably over the aforementioned Cobalt Group. The principal formation is the Lindsay formation. The most notable feature of this formation is the presence of oil shale. The remainder of the formation is an assemblage of limestone, shales and dolomite. The southern tip of the island is Lower Silurian in age, being lithographically comprised of sandstones, shales and dolomites of the Whitby formation.

Much of St. Joseph Island is covered with glacio-lacustrine deposits which are remnants of a post-Algonquin glacial lake formed during the recession of the Wisconsin glacier that occupied parts of the basins of present day Lake Huron and Lake Superior. Lowlands (less than 300m in elevation) adjacent to the present shoreline of St. Joseph Island were also inundated by this lake. Lacustrine plains of silt and clay and lacustrine beaches of gravelly sand were deposited in many of these lowland areas. The clays found at the project area are generally homogeneous, red in colour, and commonly stoney and bouldery. Underlying the clays, glacial till deposits are found, some of which have been reworked by the action of the ancient lake.

3.0 INVESTIGATION PROCEDURES

3.1 Field Investigation

The field work was undertaken between Sept. 7, 1990 and Sept. 11, 1990, inclusive and consisted of 3 sampled boreholes advanced to depths ranging from 9.6m to 28.1m below the existing ground surface. In addition, one dynamic cone penetration test was carried out from ground surface between boreholes 1 and 2, and a second cone was driven from the bottom of borehole 3. Boreholes 1 and 2, and both cone tests were advance to practical refusal (i.e., greater than 50 blows per 25mm). The elevation of the ground surface at the borehole and cone test locations ranged from 178.3m to 180.1m. The locations of the boreholes and cone tests relative to the proposed bridge abutments are shown on Drawing 849001-A*

A track mounted CME 55 drill rig, employing hollow stem augers, wash boring and diamond drilling methods, was used to advance the boreholes. In general, subsoil samples were retrieved at 0.9m intervals for the upper 3.0m and at 1.5m intervals below 3.0m depth. Sample retrieval was conducted in accordance with the Standard Penetration Test (ASTM D1586) except at selected depths where 50mm diameter thin walled shelby tube samples were obtained in accordance with ASTM D1587. Field vane testing in accordance with ASTM 2573, was performed in undisturbed cohesive soil at 1.5m or 3.0m intervals between sample locations. All samples were identified in the field and then returned to our laboratory for further examination and laboratory testing.

Groundwater levels were obtained throughout the duration of the field investigation by monitoring the levels in the open boreholes . A 50mm diameter plastic filtered standpipe was installed in borehole 3. The remaining boreholes were backfilled at the completion of the fieldwork.

The ground surface elevations of the boreholes were surveyed with reference to a temporary benchmark provided by MTO surveyors. The benchmark, with elevation 179.514m, is a spike in a hydro pole located on the west side of the road at Station 28+454.2m, offset 13.8m left. The borehole locations were referenced to the existing bridge structure.

Details regarding the subsoil encountered are referenced to the Unified Soil Classification System and are included on the attached borehole logs, Drawing 2 to 5, inclusive, in Appendix "A".

3.2 Laboratory Analyses

A series of laboratory tests were performed to identify the gradation, behaviour, and pertinent geotechnical properties and characteristics of the recovered soil samples. The tests included:

1) Atterberg Limits
2) Grain Size Analyses
3) Unit Weights
4) Moisture Contents
5) Consolidation Test

* Dwg. No. 2, Sheet 13 of the Contract Drawings.

The results of the various laboratory tests are summarized in section 4.0 and are illustrated on the corresponding borehole logs, Drawings 2 to 5, Appendix A and attached Figures 1 to 5, inclusive, in Appendix B.

4.0 SUBSURFACE CONDITIONS

4.1 General

With the exception of the surficial deposits, the soil stratigraphy encountered in the boreholes was generally similar at all test locations. All boreholes encountered a thin surficial layer of organic topsoil, ranging in thickness from 150mm to 300mm. Beneath the organic topsoil in borehole 1, a 250mm layer of sand and gravel fill was encountered whereas in borehole 3, a 2.2m deposit of natural silt and sand was found.

Underlying the surficial organic topsoil layer in borehole 2, and the granular fill and silt and sand deposits in boreholes 1 and 3, respectively, a deposit of very stiff to firm clay was encountered at the thickness ranges from 1.5m thick in borehole 2 to approximately 3.5m in borehole 3. This clay crust was underlain by approximately 17m of generally soft to firm, red-brown clay. Generally dense to very dense, grey sand and gravel (with silt mixture) was encountered at and below depths of 20m to 21m. This stratum was penetrated a maximum of 7.9m. Boreholes 1 and 2 were terminated in this very dense granular stratum due to practical refusal to augering and sampling methods on presumed boulders. Diamond drilling was undertaken within the stratum in the deeper borehole 2 in order to verify whether bedrock or boulders had been encountered at refusal level. This coring proved that boulders had been encountered.

The boundaries between the various soil types, in situ and laboratory test results as well as groundwater levels established at the time of the field work are shown on the attached Record of Borehole sheets, Drawings 2 to 5, inclusive, in Appendix "A". Included on the site plan (Drawing 849001-A*) is a subsoil stratigraphical section.

Detailed descriptions of each of the significant subsoil strata encountered in the boreholes are provided below.

* Dwg. No. 2, Sheet 13 of the Contract Drawings.

4.2 Silt and Sand

The silt and sand stratum encountered in the upper 2.3m of borehole 3 consists of a naturally deposited, stratified silt and sand mixture with a trace to some woody organic particles and a trace of clay. Figure I in Appendix B illustrates a grain size curve for this material, indicating that it consists of predominantly fine sand (54%) and silt (46%) with possibly a trace of clay.

The material changes in colour with increasing depth from a grey-brown, with some orange and some black staining, to a red-brown colour near the interface with the underlying clay. The moisture contents from the two samples obtained were 31% and 28%.

Based on the 'N' values obtained from the Standard Penetration Test, which ranged from 9 to 2, the sand and silt material is considered to have a loose to very loose relative density.

4.3 Clay

The predominant soil stratum encountered in all three boreholes is a clay which is generally homogeneous except for a narrow band (approximately 0.5m to 1.0m in thickness), at about elevation 170m, where thin, 112mm thick layers of interbedded fine sand at approximately 10mm to 20mm intervals were encountered in all three boreholes. The deposit has been subdivided on the borehole logs to show the approximate boundary between the stiff to firm crust and the underlying soft to firm unweathered clay. Typical grain size curves for the deposit are shown on Figure 2. The grain size distribution indicates that the deposit generally consists of from 0% to 4% fine sand, 18% to 26% silt sizes, and 72% to 82% clay sizes (i.e., less than 2 micrometers). The grain size curve shown on Figure 2 for borehole 2, sample SS8, represents the grain size distribution for the zone of silty clay with the fine sand layers encountered at about elevation 170m, where approximately 25% of the sample tested was fine sand.

The clay stratum is generally red-brown in colour, except in the vicinity of elevation 170m where the colouring of both the clay and fine sand layers is predominantly grey with just a trace of redness.

Natural moisture contents in the clay stratum ranged from a low of 21% in the upper crust to a high of 82% in the lower, unweathered zone. The unit weight of the material was found to range from 15.4 kN/m^3 to 17.5 kN/m^3.,

Atterberg Limited tests were undertaken to define the plasticity of the clay. The results of the tests are plotted on Figure 5 and on the individual borehole records. A summary of the indices is provided in Table 1.

Table 1
Routine Test Results

	Range	# of Tests
Natural Moisture Content (W_N%)	21-82	26
Liquid Limit (W_L%)	60-70	4
Plastic Limit (W_P%)	24-26	4
Plasticity Index (I_P%)	35-45	4
Unit Weight (kN/m^3)	15.4-17.5	7

Based on these test results, the clay stratum has a medium to high plasticity.

A consolidation test was carried out on an undisturbed sample from borehole 1 at a depth of 6.2m. The results of the consolidation test are plotted on Figure 4, Appendix B. A summary of the indices obtained is provided in Table 2 below.

Table 2
Consolidation Test Results
Hole 1 depth 6.2m

Ground Elevation	180m
Estimated Water Table Elevation	177.3
Present Overburden Pressure (p_o)	74 kPa approximate
Anticipated Pressure at 6.2m under weight of granular fill to El. 180	96 kPa approximate
Pre-consolidation Pressure (p_c)	90 kPa approximate
Co-efficient of Volume Compressibility (m_v) (for the stress range of 20 kPa to 100 kPa)	8.3 x 10^{-4} kPa^{-1} approximate
Coefficient of Consolidation (c_v)	5.3 x 10^{-9} m^2/sec
Coefficient of Permeability (k)	4.7 x 10^{-9} cm/sec

The results indicate that the clay stratum is lightly overconsolidated and highly compressible.

Field shear vane tests carried out between sample intervals in the clay yielded shear strengths ranging from 12 kPa to 50 kPa, with the majority of values falling between 20 kPa and 40 kPa. The measured sensitivity ranged from 3.0 to 7.0 indicating that the clay has a low to medium sensitivity. All of the field vane tests have been plotted on Fig. 6. The estimated median line agrees well with the approximate c/p ratio =.26 indicated by Bjerrum for a plasticity of 40 percent.* For the estimated preconsolidated pressure of 90 kPa indicated at 6.2m depth in the consolidation test result, Fig. 4 the corresponding undrained shear strength for a c/p ratio = 0.26 is computed to be 23.4 kPa, which value agrees reasonably well with the estimated median line. For this reason it is our opinion that the vane test results near the El. 175 and 173 levels in hole 1 err on the low side.

Standard Penetration Test 'N' values in the upper clay crust above river level ranged from 16 blows/0.3m to 5 blows/0.3m, while in the lower unweathered zone, the 'N' values ranged from less than 1 (i.e., pushed manually or by weight of rods) to 4 blows/0.3m. Based on both the results of the 'N' values, the upper crust is considered to be generally very stiff to firm while vane tests in the underlying unweathered zone of clay indicate a soft to firm consistency, with the strength gradually increasing with depth.

4.4 Heterogenious Mixture of Silt, Sand and Gravel (Glacial Till)

The sand and gravel stratum encountered at the base of boreholes 1 and 2 consists of a heterogenous mixture of predominantly sand and gravel sizes with a trace of silt and clay sizes. Cobble and boulder sizes were also encountered in the boreholes and probably constitute a significant proportion of the deposit. The results of grain size analyses carried out on two recovered samples from this stratum are presented on Figure 3. The grain size curves, which are considered representative of the glacial till soil matrix, indicate gravel contents ranging from 37% to 52%, a sand content of 38% to 58%, and a silt content, which may include some clay sizes, of 5% to 10%. Because of the variable nature of this type of deposit, the proportions of clay, silt, sand, gravel and boulder sizes within the deposit will vary. The curves do not reflect the influence of any cobble or boulder sizes because these materials could not be sampled.

* Geotechnical Properties of Marine Clays, L. Bjerrum, Geotechnique, Vol. IV, 1954

The granular mixture was grey in colour with moisture contents ranging from 6% to 12%.

Based on the 'N' values obtained from the Standard Penetration Tests, which ranged from 23 blows/0.3m to 82 blows/0.15m, and the difficulty encountered while augering, this material is considered to have a dense to very dense, relative density. Tactile examination of recovered samples indicated possible weak cementation.

5.0 GROUNDWATER CONDITIONS

Observations of the groundwater level in boreholes 1 and 2 were undertaken by measuring the water level in open boreholes. A 25mm diameter plastic standpipe, perforated in the lower end and wrapped with filter fabric, was installed in borehole 3 to the bottom of the hole (within the clay stratum).

At the time of the field programme, the water level in the river was approximately 3.3m below the existing bridge deck (approximately elevation 176.7), with a depth of approximately 400mm.

The groundwater level, at the time of the field work, in boreholes 1 and 2 was estimated at elevation 177.3m and 178.1m, respectively or approximately 0.6m and 1.4m higher than the level of the adjacent river. However, the water level in the standpipe in borehole 3 was observed at ground surface (elevation 179.7m) one day after completion of the hole. It is considered that the relatively high water level observed in the standpipe in borehole 3 can be attributed to either excess hydrostatic pressure in the confined fine sand layer, which were encountered within the clay at about elevation 170m, or more probably, to a temporarily highter level resulting from recent wet weather conditions.

The groundwater levels, in general, are subject to seasonal fluctuations as well as being influenced by the level of the adjacent river. Thus, the levels that will be encountered during construction can be expected to vary from those reported.

Note: The preceding report is a copy of the factual information from the Foundation Report prepared by Trow Consulting Engineers Ltd. (consulting geotechnical engineers for this project), under the technical supervision of the MTO Foundation Design Section.

REGISTERED PROFESSIONAL ENGINEER
P. PAYER
PROVINCE OF ONTARIO

P. Payer

P. Payer, P. Eng.
Senior Foundation Engineer

REGISTERED PROFESSIONAL ENGINEER
D. H. DUNDAS
94 03 02
PROVINCE OF ONTARIO

D. Dundas

D. Dundas, P. Eng.
Chief Foundation Engineer
(Acting)

APPENDIX A

DRAWINGS

Ministry of Transportation Ontario

Drawing 2

RECORD OF BOREHOLE No 1

METRIC

W P 84-90-01 LOCATION STA. 28+489.4 o/s 2.1 m LT. ORIGINATED BY S.M.

DIST 18 HWY 548 BOREHOLE TYPE Hollow Stem Auger COMPILED BY M.P.

DATUM Geodetic DATE September 7 and 8 1990 CHECKED BY A.S.

ELEV DEPTH	SOIL PROFILE DESCRIPTION
180.0	Ground Surface
0.0	Topsoil ±150 mm over
0.4	Sand & Gravel Fill
177.3 / 2.7	Clay, some silt, moist Very Stiff to Firm Red-Brown
	Clay, some silt, Soft to Firm Red-Brown (interbedded grey Clay, some silt and fine sand encountered between elevation 170 to 171)
160.9 / 19.1	Heterogeneous mixture of silt, sand and gravel (Glacial Till) compact to very dense, with numerous cobbles and boulders, grey.
156.1 / 23.9	End of borehole due to refusal to auger on probable boulders

Sample Number	Type	'N' Values	Shear Strength / Remarks	Unit Weight γ kN/m³	Grain Size Distribution (%) GR SA SI CL
1	SS	16			
2	SS	7			
3	TW	PM	s=3.3	17.5	6 22 72
4	SS	PM			
5	SS	PM	s=3.0		
6	TW	PM	s=3.0	17.2	
7	SS	PM			
8	SS	PM	s=5.0		
9	SS	PM			
10	TW	PM	s=4.0	15.4	
11	SS	PM	s=4.0; water content >82%		
12	SS	PM			
13	TW	PM	s=3.5	16.9	
14	SS	23			37 58 5

Ground water level at elevation 177.3 (depth 2.7).

Elevation scale: 178, 177.3 / 2.7, 176, 174, 172, 170, 168, 166, 164, 162, 160, 158

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

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

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

OFFICE REPORT ON SOIL EXPLORATION

Drawing 3

RECORD OF BOREHOLE No 2 METRIC

W P 84-90-01 | LOCATION STA. 28+535.0 o/s 10.8 m LT. | ORIGINATED BY S.M.

DIST 18 HWY 548 | BOREHOLE TYPE Hollow Stem Auger & B.Q. Core | COMPILED BY M.D.

DATUM Geodetic | DATE September 10 and 11, 1990 | CHECKED BY A.S.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/M³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.6	Ground Surface									
0.0	Topsoil ±500 mm over Clay, some silt									
178.1	Stiff Red-brown	1	SS	7	▽	178.1 / 1.5		o		
1.5	Clay, some silt Soft to Firm	2	SS	PM				o		
		3	SS	PM			+ s=6.0	o		
	Red-brown	4	TW	PM		176		o	15.7	
	(interbedded grey, Clay, some silt and fine sand encountered at elevation 170.0 to 170.5)						+ s=6.0			
		5	SS	PM		174		Wp–W >W	82%	0 23 77
		6	SS	PM				o		
						172	+ s=7.0			
		7	SS	PM				o		
		8	SS	PM		170		o		24 46 30
							+ s=6.0			
		9	SS	1		168		o		
		10	TW	PM				o	15.5	
						166	+ s=6.7			
		11	SS	PM				o		
		12	SS	PM		164		Wp–W		0 17 83
							+ s=4.0			
		13	SS	PM		162		o		
		14	SS	PM		160		o		
159.5 / 20.1	Heterogeneous mixture of silt, sand and gravel (Glacial Till) compact to very dense, with numerous cobbles and boulders, grey.	15	SS	4				o		
		16	SS	54		158		o		52 38 10
		17	R.C B.Q	8%		156				
		18	SS	68/150 mm				o		
		19	R.C B.Q	13%						
		20	SS	78/150 mm		154		o		
		21	SS	82/150 mm				o		
						152				
151.6		22	SS	65/25 mm						
28.0	End of borehole									

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

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

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

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Drawing 4

RECORD OF BOREHOLE No 3

METRIC

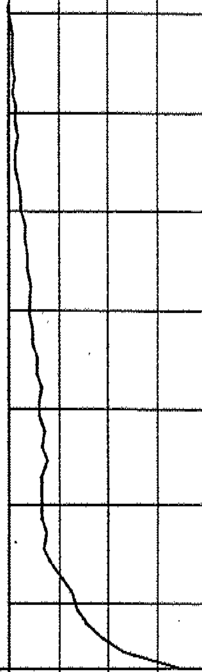
W P 84-90-01 — LOCATION STA. 28+563.6 o/s 3.4 m LT. — ORIGINATED BY S.M.

DIST 18 HWY 548 — BOREHOLE TYPE Hollow Stem Auger & Cone Test — COMPILED BY M.D.

DATUM Geodetic — DATE September 9, 1990 — CHECKED BY A.S.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.7	Ground Surface							
0.0	Topsoil ±150 mm over Silt & Sand Loose to Very Loose Grey-brown, Stratified	1	SS	9	179.7			
		2	SS	2	178			54 46
177.4 / 2.3	Clay, some silt Firm, red-brown	3	SS	5				
176.2 / 3.5		4	SS	5	176			
	Clay, some silt Soft Red-brown (interbedded grey, Clay, some silt and fine sand encountered between elevation 170.1 and 170.5)	5	SS	1	174	+s=5.3		
		6	SS	PM		+s=5.0		
		7	TW	PM	172		17.2	
						+s=5.0		
		8	SS	PM	170			
170.1 / 9.6	End of Borehole Probable Clay, some silt				168, 166, 164, 162, 160, 158			
156.5 / 23.2	End of cone test							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100



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

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

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

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

OFFICE REPORT ON SOIL EXPLORATION

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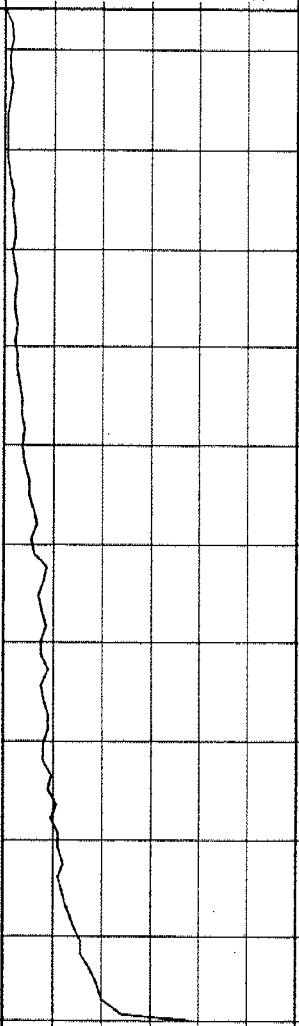
Drawing 5

RECORD OF BOREHOLE No 4 METRIC

W P 84-90-01 LOCATION STA. 28+510.3 o/s 101 m LT. ORIGINATED BY S.M.

DIST 18 HWY 548 BOREHOLE TYPE Cone Test COMPILED BY M.D.

DATUM Geodetic DATE September 11, 1990 CHECKED BY A.S.

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	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
178.3	Ground Surface										
0.0	Dynamic Cone penetration test only Probable Clay, some silt						178 176 174 172 170 168 166 164 162 160				
157.7 20.6	End of cone test										

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

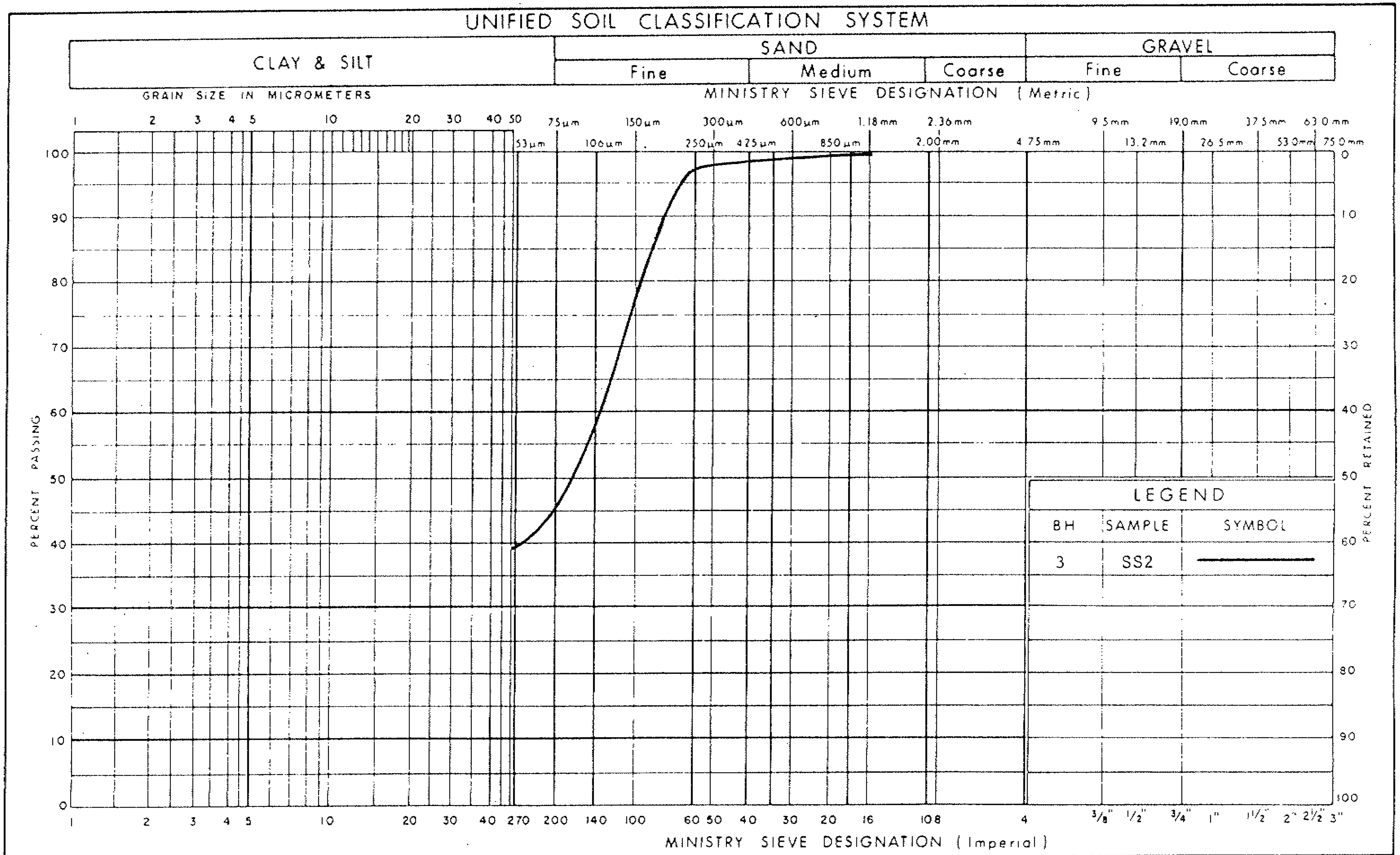
20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX B

FIGURES AND LABORATORY TESTS

78 12 M



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GRAIN SIZE DISTRIBUTION

Grading Curve of Upper Silt and Sand

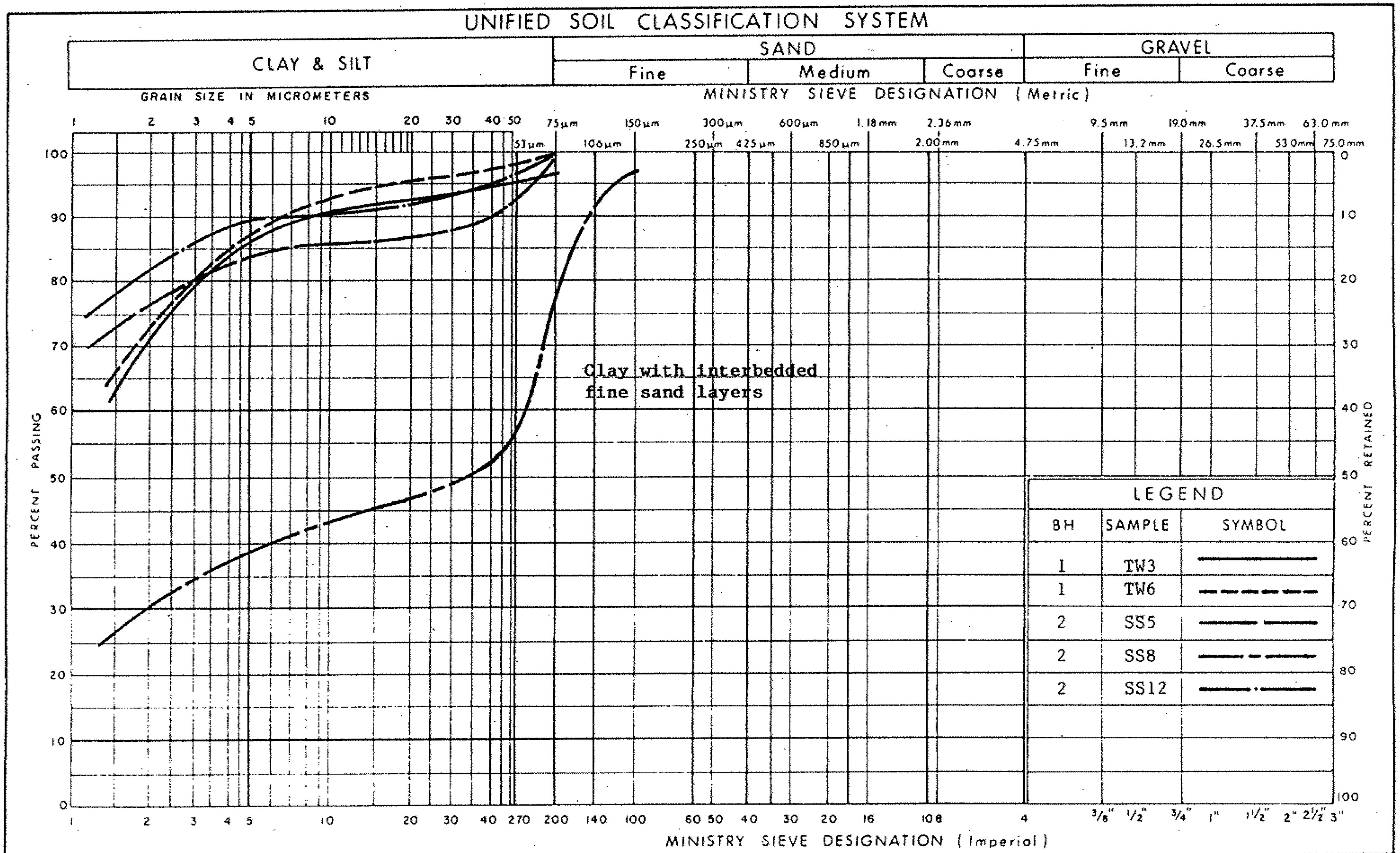
Hole 3 approx. El. 178

FIG No 1

W P 84-90-01

18

78 12 M



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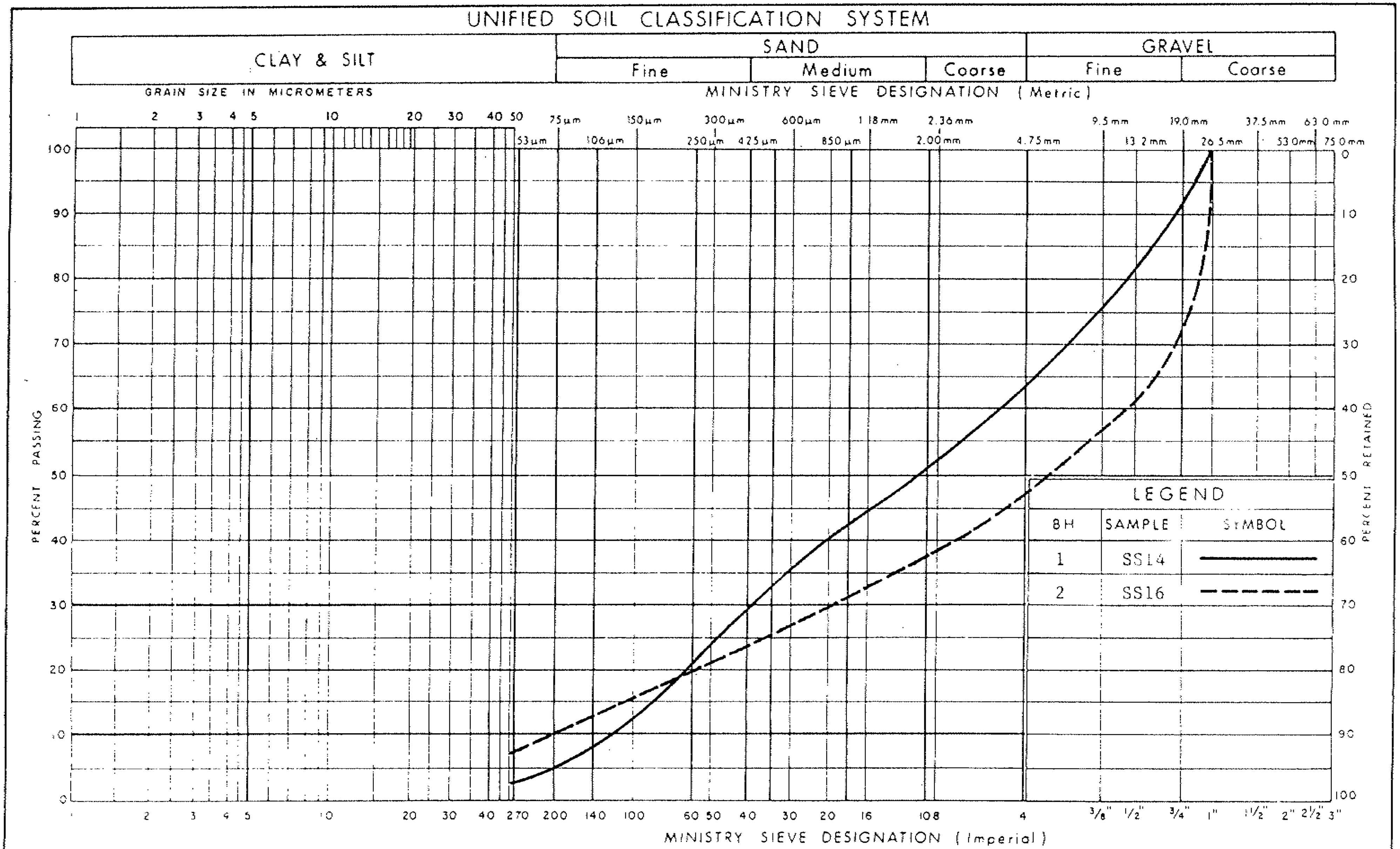
GRAIN SIZE DISTRIBUTION

Grain Size Curves for the clay stratum

FIG No 2

W P 84-90-01

78 12 M



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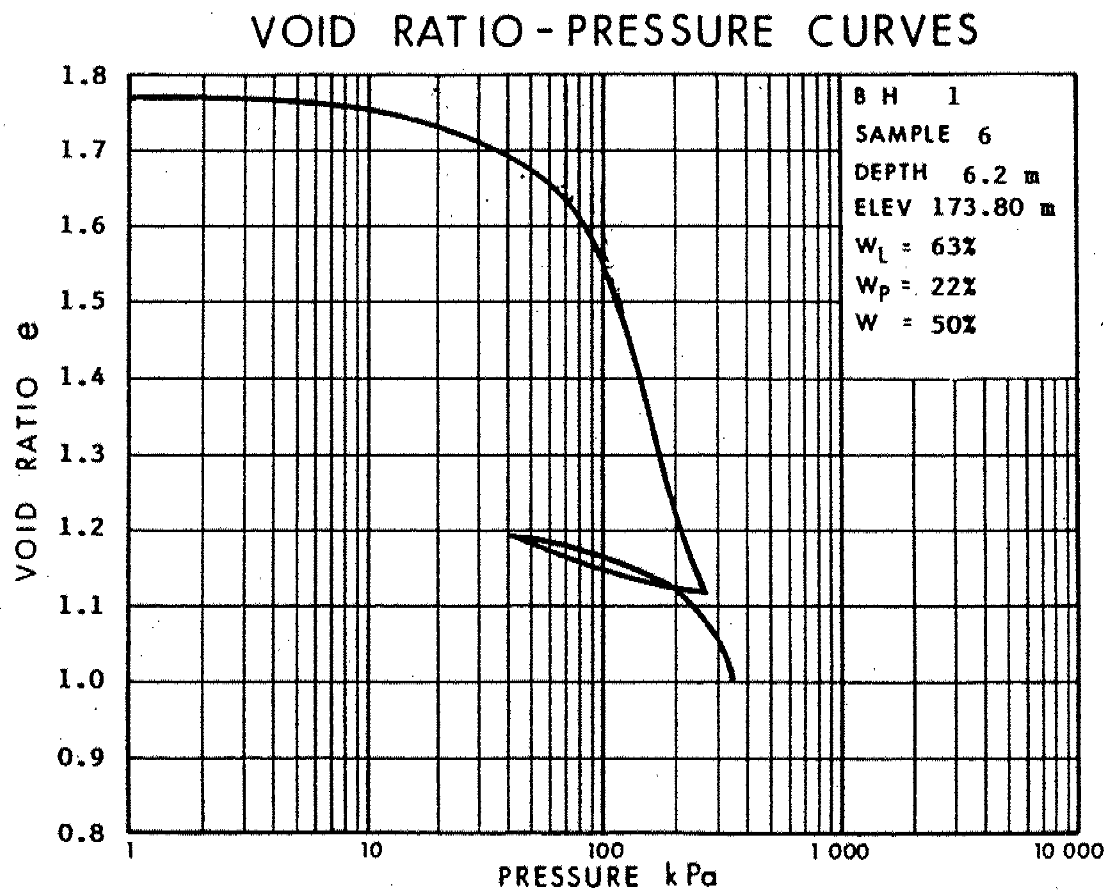
GRAIN SIZE DISTRIBUTION

Grading size curves of the heterogeneous mix of sand and gravel (glacial till) found below approximate El. 160 m

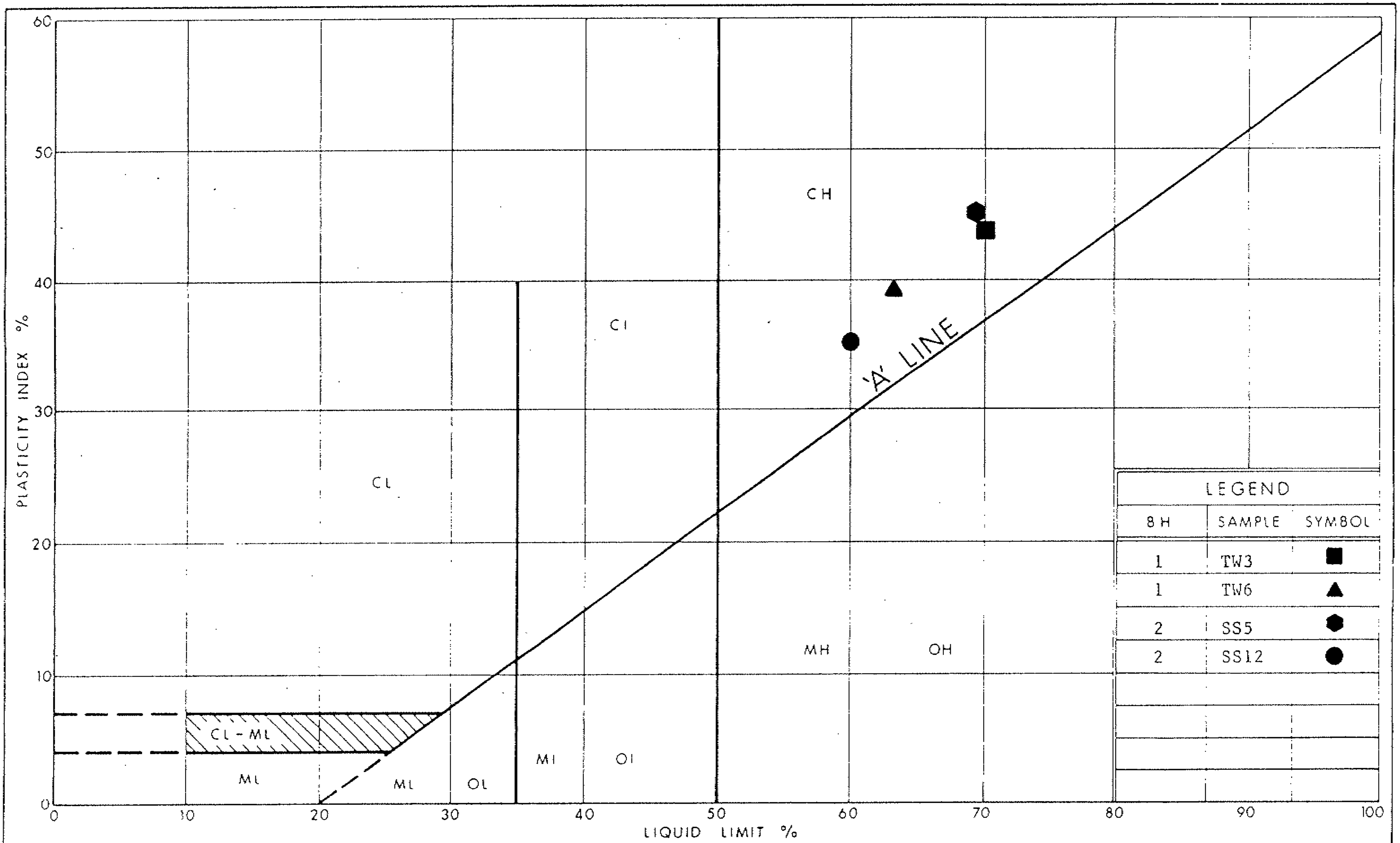
FIG No	3
W P	84-90-01

CONSOLIDATION TEST RESULTS



Po ± 70 kPa
Pc ± 90 kPa
Mv = 8.3 x 10^{-4} kPa^{-1} (20-120 kPa range)
Cv = 5.3 x 10^{-9} m²/sec
k = 4.7 x 10^{-9} cm/sec

W.P. 84-90-01
Fig. 4



LEGEND

BH	SAMPLE	SYMBOL
1	TW3	■
1	TW6	▲
2	SS5	⬢
2	SS12	●

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PLASTICITY CHART

CLAY

FIG No 5

W P 84-90-01

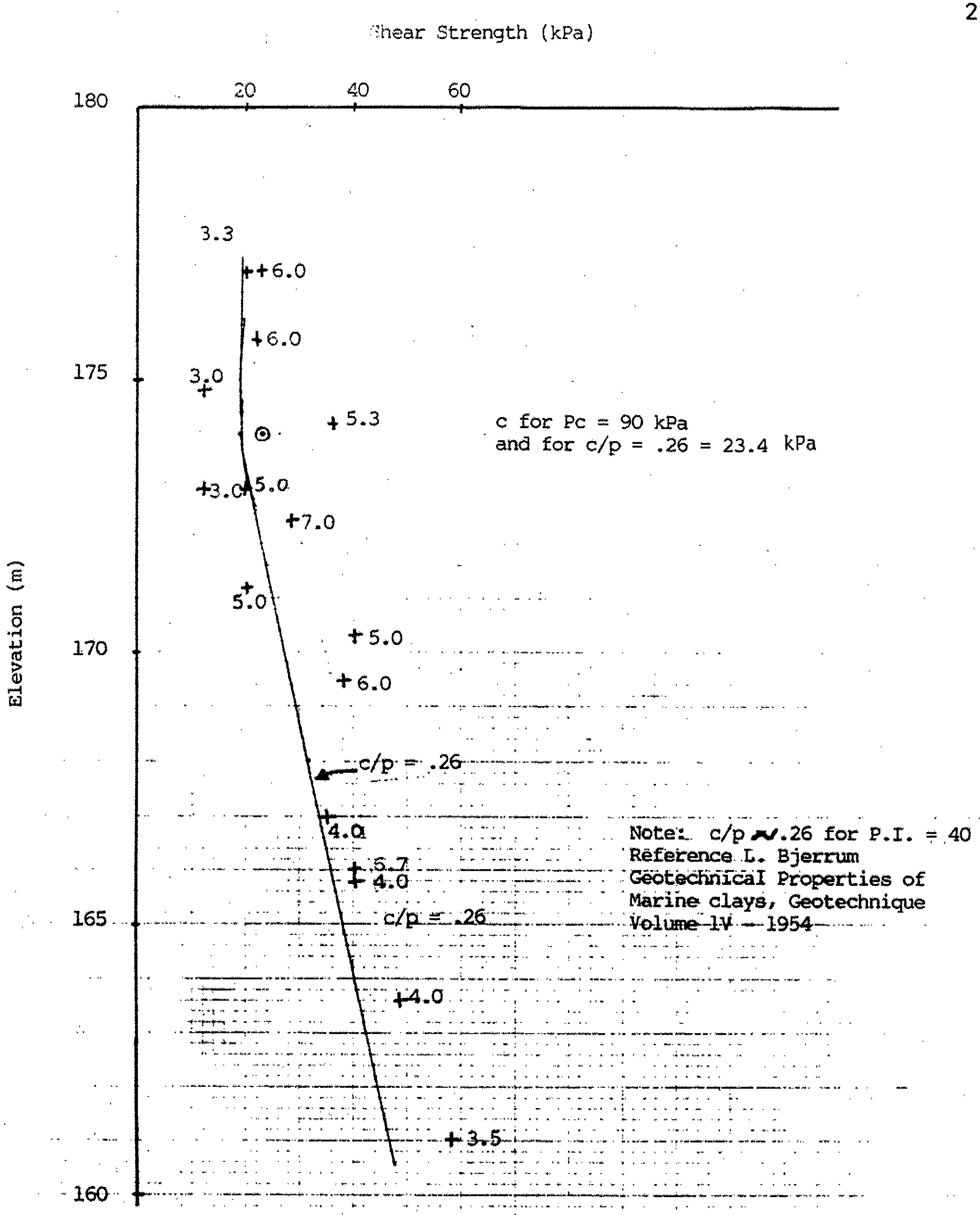

Fig. 6

Field vane test results

Trow

CONT 94-205

FOUNDATION INVESTIGATION
PROPOSED STRUCTURE REPLACEMENT,
HWY. 548
TWO TREE RIVER, SITE NO. 38S-211
DISTRICT 18, SAULT STE. MARIE
W.P. ~~368-90-01~~
84-90-01

PREPARED FOR:

MINISTRY OF TRANSPORTATION OF ONTARIO

GEOCRES # 41K-47

TROW CONSULTING ENGINEERS LTD.
Brampton, Hamilton, Kitchener, London, Markham
North Bay, Oshawa, Ottawa, Sudbury, Thunder Bay

Project: S04377G
Date: July 5, 1991
(Revised July 17, 1991)

1595 Clark Boulevard
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Phone: (416) 793-9800
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FOUNDATION INVESTIGATION REPORT FOR PROPOSED STRUCTURE REPLACEMENT, HWY 548 TWO TREE RIVER, SITE NO. 38S-211 DISTRICT 18 - SAULT STE. MARIE W.P. 368-90-01

1.0 INTRODUCTION

This report summarizes the results of a foundation investigation completed at this bridge crossing site. It is understood that the existing bridge crossing will be realigned over Two Tree River on Highway 548 on St. Joseph Island, Ontario. The present crossing structure, consisting of a relatively short span, concrete and timber bridge structure, incorporates an abrupt curve on the southwest side in order to make a perpendicular approach to the river.

Current plans are to replace the existing bridge structure with a single span structure, more closely aligned with the existing highway. Two options for the bridge abutments are presently being considered, as illustrated on the site plan, Drawing 368-90-01A attached.

2.0 SITE DESCRIPTION AND GEOLOGY

The project site is located along a straight section of Hwy. 548 between stations 28+475 and 28+525 at the intersection of the Two Tree River, in St. Joseph Township near the western edge of St. Joseph Island, Ont. St. Joseph Island is located along the north shore of Lake Huron to the west of Manitoulin Island and some 45km to the south-southeast of Sault Ste. Marie, Ont.

The terrain in the immediate area of the project site is relatively flat farmland with bedrock outcroppings several kilometres away to both the northwest and southeast. The river flows from north to south adjacent to the north side of the highway turning sharply to the southwest at the existing bridge structure. It then flows in a generally southwesterly direction of Munuscong Lake. Because of the general orientation of the river, both existing approaches to the bridge curve sharply in order to provide a more perpendicular approach to the river crossing. The resulting sudden change in present alignment of the highway reportedly requires traffic to slow abruptly at both the north and south bridge approaches.

Physiographically, St. Joseph Island straddles the boundary between the extreme northern edge of the Michigan Basin and the southern edge of the Southern Province. The Southern Province lithology found on the extreme northern end of the island consists of an assemblage of Middle Precambrian rock belonging to the Cobalt Group of the Huronian Formation. Rock types consist of conglomerate, greywacke, orthoquartzite, siltstone and argillite.

The rock formations underlying and exposed across the majority of the island, including the project site, are Middle to Upper Ordovician in age, and lie unconformably over the aforementioned Cobalt Group. The principal formation is the Lindsay formation. The most notable feature of this formation is the presence of oil shale. The remainder of the formation is an assemblage of limestone, shales and dolomite. The southern tip of the island is Lower Silurian in age, being lithographically comprised of sandstones, shales and dolomites of the Whitby formation.

Much of St. Joseph Island is covered with glacio-lacustrine deposits which are remnants of a post-Algonquin glacial lake formed during the recession of the Wisconsin glacier that occupied parts of the basins of present day Lake Huron and Lake Superior. Lowlands (less than 300m in elevation) adjacent to the present shoreline of St. Joseph Island were also inundated by this lake. Lacustrine plains of silt and clay and lacustrine beaches of gravelly sand were deposited in many of these lowland areas. The clays found at the project area are generally homogeneous, red in colour, and commonly stoney and bouldery. Underlying the clays, glacial till deposits are found, some of which have been reworked by the action of the ancient lake.

3.0 INVESTIGATION PROCEDURES

3.1 Field Investigation

The field work was undertaken between Sept. 7, 1990 and Sept. 11, 1990, inclusive and consisted of 3 sampled boreholes advanced to depths ranging from 9.6m to 28.1m below the existing ground surface. In addition, one dynamic cone penetration test was carried out from ground surface between boreholes 1 and 2, and a second cone was driven from the bottom of borehole 3. Boreholes 1 and 2, and both cone tests were advance to practical refusal (i.e., greater than 50 blows per 25mm). The elevation of the ground surface at the borehole and cone test locations ranged from 178.3m to 180.1m. The locations of the boreholes and cone tests relative to the proposed bridge abutments are shown on Drawing 368-90-01A.

A track mounted CME 55 drill rig, employing hollow stem augers, wash boring and diamond drilling methods, was used to advance the boreholes. In general, subsoil samples were retrieved at 0.9m intervals for the upper 3.0m and at 1.5m intervals below 3.0m depth. Sample retrieval was conducted in accordance with the Standard Penetration Test (ASTM D1586) except at selected depths where 50mm diameter thin walled shelby tube samples were obtained in accordance with ASTM D1587. Field vane testing in accordance with ASTM 2573, was performed in undisturbed cohesive soil at 1.5m or 3.0m intervals between sample locations. All samples were identified in the field and then returned to our laboratory for further examination and laboratory testing.

Groundwater levels were obtained throughout the duration of the field investigation by monitoring the levels in the open boreholes . A 50mm diameter plastic filtered standpipe was installed in borehole 3. The remaining boreholes were backfilled at the completion of the fieldwork.

The ground surface elevations of the boreholes were surveyed with reference to a temporary benchmark provided by MTO surveyors. The benchmark, with elevation 179.514m, is a spike in a hydro pole located on the west side of the road at Station 28+454.2m, offset 13.8m left. The borehole locations were referenced to the existing bridge structure.

Details regarding the subsoil encountered are referenced to the Unified Soil Classification System and are included on the attached borehole logs, Drawing 2 to 5, inclusive, in Appendix "A".

3.2 Laboratory Analyses

A series of laboratory tests were performed to identify the gradation, behaviour, and pertinent geotechnical properties and characteristics of the recovered soil samples. The tests included:

1) Atterberg Limits
2) Grain Size Analyses
3) Unit Weights
4) Moisture Contents
5) Consolidation Test

The results of the various laboratory tests are summarized in section 4.0 and are illustrated on the corresponding borehole logs, Drawings 2 to 5, Appendix A and attached Figures 1 to 5, inclusive, in Appendix B.

4.0 SUBSURFACE CONDITIONS

4.1 General

With the exception of the surficial deposits, the soil stratigraphy encountered in the boreholes was generally similar at all test locations. All boreholes encountered a thin surficial layer of organic topsoil, ranging in thickness from 150mm to 300mm. Beneath the organic topsoil in borehole 1, a 250mm layer of sand and gravel fill was encountered whereas in borehole 3, a 2.2m deposit of natural silt and sand was found.

Underlying the surficial organic topsoil layer in borehole 2, and the granular fill and silt and sand deposits in boreholes 1 and 3, respectively, a deposit of very stiff to firm clay was encountered at the thickness ranges from 1.5m thick in borehole 2 to approximately 3.5m in borehole 3. This clay crust was underlain by approximately 17m of generally soft to firm, red-brown clay. Generally dense to very dense, grey sand and gravel (with silt mixture) was encountered at and below depths of 20m to 21m. This stratum was penetrated a maximum of 7.9m. Boreholes 1 and 2 were terminated in this very dense granular stratum due to practical refusal to augering and sampling methods on presumed boulders. Diamond drilling was undertaken within the stratum in the deeper borehole 2 in order to verify whether bedrock or boulders had been encountered at refusal level. This coring proved that boulders had been encountered.

The boundaries between the various soil types, in situ and laboratory test results as well as groundwater levels established at the time of the field work are shown on the attached Record of Borehole sheets, Drawings 2 to 5, inclusive, in Appendix "A". Included on the site plan (Drawing 368-90-01A) is a subsoil stratigraphical section.

Detailed descriptions of each of the significant subsoil strata encountered in the boreholes are provided below.

4.2 Silt and Sand

The silt and sand stratum encountered in the upper 2.3m of borehole 3 consists of a naturally deposited, stratified silt and sand mixture with a trace to some woody organic particles and a trace of clay. Figure I in Appendix B illustrates a grain size curve for this material, indicating that it consists of predominantly fine sand (54%) and silt (46%) with possibly a trace of clay.

The material changes in colour with increasing depth from a grey-brown, with some orange and some black staining, to a red-brown colour near the interface with the underlying clay. The moisture contents from the two samples obtained were 31% and 28%.

Based on the 'N' values obtained from the Standard Penetration Test, which ranged from 9 to 2, the sand and silt material is considered to have a loose to very loose relative density.

4.3 Clay

The predominant soil stratum encountered in all three boreholes is a clay which is generally homogeneous except for a narrow band (approximately 0.5m to 1.0m in thickness), at about elevation 170m, where thin, 112mm thick layers of interbedded fine sand at approximately 10mm to 20mm intervals were encountered in all three boreholes. The deposit has been subdivided on the borehole logs to show the approximate boundary between the stiff to firm crust and the underlying soft to firm unweathered clay. Typical grain size curves for the deposit are shown on Figure 2. The grain size distribution indicates that the deposit generally consists of from 0% to 4% fine sand, 18% to 26% silt sizes, and 72% to 82% clay sizes (i.e., less than 2 micrometers). The grain size curve shown on Figure 2 for borehole 2, sample SS8, represents the grain size distribution for the zone of silty clay with the fine sand layers encountered at about elevation 170m, where approximately 25% of the sample tested was fine sand.

The clay stratum is generally red-brown in colour, except in the vicinity of elevation 170m where the colouring of both the clay and fine sand layers is predominantly grey with just a trace of redness.

Natural moisture contents in the clay stratum ranged from a low of 21% in the upper crust to a high of 82% in the lower, unweathered zone. The unit weight of the material was found to range from 15.4 kN/m^3 to 17.5 kN/m^3.,

Atterberg Limited tests were undertaken to define the plasticity of the clay. The results of the tests are plotted on Figure 5 and on the individual borehole records. A summary of the indices is provided in Table 1.

Table 1
Routine Test Results

	Range	# of Tests
Natural Moisture Content (W_N%)	21-82	26
Liquid Limit (W_L%)	60-70	4
Plastic Limit (W_P%)	24-26	4
Plasticity Index (I_p%)	35-45	4
Unit Weight (kN/m^3)	15.4-17.5	7

Based on these test results, the clay stratum has a medium to high plasticity.

A consolidation test was carried out on an undisturbed sample from borehole 1 at a depth of 6.2m. The results of the consolidation test are plotted on Figure 4, Appendix B. A summary of the indices obtained is provided in Table 2 below.

Table 2
Consolidation Test Results
Hole 1 depth 6.2m

Ground Elevation	180m
Estimated Water Table Elevation	177.3
Present Overburden Pressure (p_o)	74 kPa approximate
Anticipated Pressure at 6.2m under weight of granular fill to El. 180	96 kPa approximate
Pre-consolidation Pressure (p_c)	90 kPa approximate
Co-efficient of Volume Compressibility (m_v) (for the stress range of 20 kPa to 100 kPa)	8.3×10^{-4} kPa^{-1} approximate
Coefficient of Consolidation (c_v)	5.3×10^{-9} m^2/sec
Coefficient of Permeability (k)	4.7×10^{-9} cm/sec

The results indicate that the clay stratum is lightly overconsolidated and highly compressible.

Field shear vane tests carried out between sample intervals in the clay yielded shear strengths ranging from 12 kPa to 50 kPa, with the majority of values falling between 20 kPa and 40 kPa. The measured sensitivity ranged from 3.0 to 7.0 indicating that the clay has a low to medium sensitivity. All of the field vane tests have been plotted on Fig. 6. The estimated median line agrees well with the approximate c/p ratio =.26 indicated by Bjerrum for a plasticity of 40 percent.* For the estimated preconsolidated pressure of 90 kPa indicated at 6.2m depth in the consolidation test result, Fig. 4 the corresponding undrained shear strength for a c/p ratio = 0.26 is computed to be 23.4 kPa, which value agrees reasonably well with the estimated median line. For this reason it is our opinion that the vane test results near the El. 175 and 173 levels in hole 1 err on the low side.

Standard Penetration Test 'N' values in the upper clay crust above river level ranged from 16 blows/0.3m to 5 blows/0.3m, while in the lower unweathered zone, the 'N' values ranged from less than 1 (i.e., pushed manually or by weight of rods) to 4 blows/0.3m. Based on both the results of the 'N' values, the upper crust is considered to be generally very stiff to firm while vane tests in the underlying unweathered zone of clay indicate a soft to firm consistency, with the strength gradually increasing with depth.

4.4 Heterogenious Mixture of Silt, Sand and Gravel (Glacial Till)

The sand and gravel stratum encountered at the base of boreholes 1 and 2 consists of a heterogenous mixture of predominantly sand and gravel sizes with a trace of silt and clay sizes. Cobble and boulder sizes were also encountered in the boreholes and probably constitute a significant proportion of the deposit. The results of grain size analyses carried out on two recovered samples from this stratum are presented on Figure 3. The grain size curves, which are considered representative of the glacial till soil matrix, indicate gravel contents ranging from 37% to 52%, a sand content of 38% to 58%, and a silt content, which may include some clay sizes, of 5% to 10%. Because of the variable nature of this type of deposit, the proportions of clay, silt, sand, gravel and boulder sizes within the deposit will vary. The curves do not reflect the influence of any cobble or boulder sizes because these materials could not be sampled.

* Geotechnical Properties of Marine Clays, L. Bjerrum, Geotechnique, Vol. IV, 1954

The granular mixture was grey in colour with moisture contents ranging from 6% to 12%.

Based on the 'N' values obtained from the Standard Penetration Tests, which ranged from 23 blows/0.3m to 82 blows/0.15m, and the difficulty encountered while augering, this material is considered to have a dense to very dense, relative density. Tactile examination of recovered samples indicated possible weak cementation.

5.0 GROUNDWATER CONDITIONS

Observations of the groundwater level in boreholes 1 and 2 were undertaken by measuring the water level in open boreholes. A 25mm diameter plastic standpipe, perforated in the lower end and wrapped with filter fabric, was installed in borehole 3 to the bottom of the hole (within the clay stratum).

At the time of the field programme, the water level in the river was approximately 3.3m below the existing bridge deck (approximately elevation 176.7), with a depth of approximately 400mm.

The groundwater level, at the time of the field work, in boreholes 1 and 2 was estimated at elevation 177.3m and 178.1m, respectively or approximately 0.6m and 1.4m higher than the level of the adjacent river. However, the water level in the standpipe in borehole 3 was observed at ground surface (elevation 179.7m) one day after completion of the hole. It is considered that the relatively high water level observed in the standpipe in borehole 3 can be attributed to either excess hydrostatic pressure in the confined fine sand layer, which were encountered within the clay at about elevation 170m, or more probably, to a temporarily highter level resulting from recent wet weather conditions.

The groundwater levels, in general, are subject to seasonal fluctuations as well as being influenced by the level of the adjacent river. Thus, the levels that will be encountered during construction can be expected to vary from those reported.

6.0 DISCUSSION AND RECOMMENDATIONS

Based on the results of the field investigation programme, and the terms of reference previously discussed in Section 1.0, the following geotechnical design comments are provided.

6.1 Bridge Structure

6.1.1 Shallow Foundations

Founding abutments for a bridge structure using conventional shallow spread footings on the weak clay is not considered feasible at this site. Only a minimal net safe bearing pressure in the order of 40 kPa is considered to be available for foundation design. Even with this low design pressure, potentially large, long-term consolidation settlement of the underlying, highly plastic, compressible clay, under the embankment fill weight, are anticipated.

6.1.2 Deep Foundations

Since this site has a deep deposit of soft to firm clay, which could become unstable during the driving of cylindrical piles, it is recommended that the abutments for a bridge structure be founded on low displacement, end bearing, steel 'H' piles driven into the very dense till stratum. The working capacity of the pile is a function of the type, size, and driving energy, and is subject to the design requirements of the 1983 Ontario Highways Bridge Design Code (OHBDC). Since this site has a deep deposit of soft to form clay, in order to minimize the slope stability problem of heaving of the adjacent piles, low disdisplacement type piles, i.e., H-piles are recommended.

The axial capacity of two of the common size H-piles and their estimated termination elevation are presented in Table 3. The required penetration resistance for achieving the recommended capacity should be evaluated by either the Hiley Formula or Wave Equation Analysis once the hammer, pile, capblock combination is known. The Wave Equation Analysis is recommended as indicated in the OHBDC Commentary.

Table 3
Recommended Pile Capacity and Driving Energies

Pile Type & size	Axial Capacity kN		Estimated Termination Elev. (m)	Minimum Driving Energy (kJ)
	S.L.S.	U.L.S.		
HP 310 x 110	1150	1880	153	62
HP 310 x 132	1335	2050	150	62

Negative Skin Friction Allowance of 27 kN/m should be subtracted from the capacities shown.

The capacity of the pile is developed by a combination of frictional resistance and toe-bearing resistance in the till stratum. No positive frictional resistance is considered in the clay. When embankment fill is placed at the abutment, the weight of the fill will cause the clay to consolidate and therefore generate negative skin friction on the pile. The estimated negative friction acting on the piles in the clay above the lower dense granular layer, at this site, 27 kN/m of pile embedment which has to be subtracted from the axial capacities indicated in Table 3.

The actual unfactored axial capacity of the piles can be evaluated by carrying out either static load tests or dynamic field measurements. However, in view of the location of the proposed bridge and the difficulties in testing inclined piles by static loading method, dynamic field measurements are recommended. The recommended factored axial capacity shall be evaluated in accordance with the recommendations in Section 6 of the OHBDC.

Since the piles will be driven into a deposit containing cobbles and boulders, it is recommended that a driving shoe such as a standard MTO driving shoe or equivalent be used to protect the pile toe against damage. The contractor should be cautioned that the piles may have to be supported when penetrating the weak clay layer since the piles may sink under and the weight of the hammer.

It should be noted that, since the lower dense granular deposit is heterogeneous in composition, the penetration lengths within it may vary from the lengths indicated. The quantity calculated from the termination elevation shown in Table 3 could therefore vary considerably.

Inclined piles, in conformance with standard MTO design practice, should be used to resist lateral loads, resulting from earth pressure forces, at the abutments. A minimum spacing of three times the pile diameter should be used for group piles.

If the termination levels of adjacent piles penetrate deeper than a 3 horizontal to 2 vertical line drawn from the toe of the higher piles, the higher piles should be redriven to the established penetration resistance. During the driving of the piles in a group, the vertical elevation of the pile should be monitored. If heaving occurs during the driving of adjacent piles, the heaved piles should be redriven to the established penetration resistance. Provision should therefore be made for restriking piles in the contract documents.

The effective stress stability analyses indicated, in Fig. 8, shows that during construction, if the pore water pressure is allowed to build-up, the factor of safety against failure will be reduced. For example, if the pore pressure ratio Ru is equal to 0.6, i.e., an hydrostatic pressure slightly above ground level, the factor of safety reduces to approximately 1.1. It is therefore recommended that the pore water pressures at different depths be monitored during the installation of the pile foundation in order to confirm the short term stability conditions of the slopes and embankment during the construction stage. This would involve the installation and subsequent measurements of water levels, in a number of pneumatic piezometers placed at strategically locations.

6.2 Approach Fills

6.2.1 Stability Considerations

M.T.O. Alternative 1 - This alternative envisions a 8m single span with the west abutment generally over the existing riverbed and the east abutment into the east bank. An embankment height to El. 181 has been indicated. With the river bed at approximate El. 176, the fill height will be about 5m on the north west side, with the lowest approximately 0.7m below the water surface. The new fill depths in the south easterly approach will vary from approximately 1 to 5m. This design must be modified, as indicated, to a mamimum elevation of 180m.

M.T.O. Alternative 2 - This alternative incorporates a wider single span of approximately 15m with the abutments closer to the banks of the existing river as indicated on the site plan Dwg. 368-90-01A. The embankment fill heights at the abutments approach 5m at some locations but are less where the fill is to be applied over the existing river banks. As stated for Alternative 1, this design must be modified down so that full heights do not exceed El. 180m.

Because of the existence of the weak clay underlying the bridge site, the stability aspects of the approach embankments have been reviewed carefully. In order to evaluate the stability of the approach fills, an average undrained shear strength of 20 kPa has been assumed for the underlying soft clay. A proposed embankment grade at El. 181 has been suggested by M.T.O.

As a first approximation, assuming a rotational failure under the abutments at 5 times the undrained shear strength, the corresponding safe height of fill above river bed elevation 176, incorporating a factor of safety F = 1.3, is about 4m or El. 180. The critical direction of failure is considered to be along the centre line embankment length at the abutments. Failure at right angles to the embankment length is considered to be less likely because of the supporting effect of the existing river banks.

Computer analyses, presented in Fig. 7, indicate the factor of safety for the abutment situation under the addition of 4m of fill over the river bed up to El. 180, for the Alternative I, West approach situation, is about 1.38. The assumptions used in this analysis are indicate on Fig. 7. Consequently, it is recommended that the embankment should not be built higher than El. 180.

6.2.2 Settlement Considerations

The bridge site is underlain by deposits of soft, compressible clay to approximate El. 160m. It is assumed that the approach embankments will be constructed with imported granular fill having a unit weight of approximately 21.2 kN/m3 and constructed with side slopes of 2H:1V. It is also recommended that the river water be temporarily diverted to permit embankment construction in the dry, including the removal of any weak deposits in the river bed.

a) M.T.O. Alternate 1 - Estimates of consolidation settlement have been made assuming the embankment fill to be limited to El. 180m as discussed above. The loading condition for the river bed approach to the north west Alternative 1 abutment is considered to be the most severe. Based on the borehole information, the compressibility test data presented on Figure 4, and approach fill to El. 180, it is estimated that the approach fill will settle as much as 624mm on the north west side where the length of filling in the river will be maximum. The settlement adjacent to the east abutment is expected to be less although it will be differential across the width of the abutment because of the variation in depth of applied fill. Although pre-consolidated, by soil eroded in geologically recent times to the present river bed level, - and therefore essentially representing recompression of the clay, - the estimated settlement is still significant. Because the settlement will result for the most part from recompression, the rate of settlement is expected to occur more quickly than if the clay had never experienced the loading before. However, in order to minimize the effects of variable settlement of the fill, it should be placed well in advance of abutment construction.

If this course of construction is followed for the Alternative 1 proposal, i.e., placement of embankment fill in advance of bridge construction, diversion of the river will be required well beforehand and the exposed portion of the fill protected from river erosion. Monitoring of the settlement of the fill will be required to confirm when settlement adjustment has been reached.

The alternative, which would include construction of the abutment first, - with support on end bearing piles, - would result in high downdrag forces on the piles, as discussed in a previous section, when the approaches are constructed; the long wait for completion of embankment settlement would still apply.

b) M.T.O. Alternate 2 - Alternate 2 will require much less weight of fill on each side of the proposed bridge since the abutments will be closer to the present river banks. Therefore settlements are expected to be less although still of sufficient magnitude to warrant placement of the fill well in advance of bridge construction, as discussed for Alternative 1.

Although detailed settlement calculations have not been carried out it is obvious that the settlements will be greatest where the fill load is heavier and less over the river banks where the fill loading will be less. Consequently, as stated above, in order to give the time to adjust to the variable fill loading and also to minimize differential down drag forces on the piles, placement of the fill well in advance of construction, - preferably in the order of 12 months, - is recommended.

Some trimming back of the presently steep north west river bank will be needed in order to minimize the risk of a localized slope failure during the driving of abutment piles.

From a settlement view, Alternative 2 appears to be preferable to Alternative 1.

6.2.3 Other Alternatives

In view of the limitations on embankment height in order to maintain stability and to control the excessive settlements anticipated even when the embankment heights are reduced, you may wish to consider other construction alternatives such as the introduction of additional spans in order to minimize or eliminate the need for high fill or the use of lightweight approach fill.

6.3 Earth Pressures

Where embankment fill is to be restrained by bridge abutments, the abutments must be designed in accordance with the procedures outlined in Section 6 of the 1983 Ontario Highway Bridge Design Code. A unit weight for the granular fill of 21.2 kN/m^3 can be used.

6.4 Earthquake Considerations

The bridge should be designed to resist earthquake motion in accordance with the recommendations and guidelines indicated in Section 2 of the OHDBC.

6.5 Construction Considerations

Diversion of the river and dewatering of the site will be required during construction. Diversion of the river is expected to be fairly straightforward using a large backhoe excavator to dig a new trench in the existing clay soil. For excavations to a depth of 3m, temporary side slopes are expected to remain reasonably stable at 1H:1V. However, the construction schedule will likely require that the river diversion remain operational for an extended period. Therefore, it is recommended that side slopes be cut at 3H:IV for any medium to long-term excavation period.

Open cut excavations should be limited to a depth of about 4m deep into the soft clay in order to minimize the risk of base heave. For an undrained shear strength of 20 kPa, base heave is expected below an excavated depth of 6 or 7m. If it is necessary to dig deeper, very flat side slopes and/or stepped excavations incorporating sheeting may be necessary. An engineering appraisal is required for any proposal to dig deeper than about 4m.

Once the river is diverted, it is expected that any seepage water can be collected in ditches or sumps dug into the clay. Any very soft alluvium or organize material should be removed from the river bed at this time. Even then the exposed base of the clay is expected to be softened under the action of construction equipment. It will be necessary to use coarser material such as 2 inch stone for the first placement lift and to work it into the clay using the treads of a caterpillar tractor. Compaction of the fill should be to M.T.O. specifications.

Fill must not be stored above El. 180m adjacent to the present river or to the excavated new channel.

6.6 Scour and Erosion Protection

The flow rate and level of flow under extreme flood conditions should be evaluated as part of the hydraulic design. The foundations and/or embankments located adjacent to the river must be protected from potential scour and erosion either by installing permanent sheeting, rip rap or gabion baskets, etc. This protection should extend upstream and downstream and be higher than the highest anticipated flood level. For applications using rip rap or gabion baskets, a suitable underlying geotextile should also be incorporated.

7.0 CLOSURE

We trust that the information presented in this report provides you with sufficient data to proceed with the project. The report was prepared by Mr. W.A. Trow, P.Eng. and review by Dr. K.R. Peaker, P.Eng. Should you have any further questions, please do not hesitate to contact the undersigned at this office.

LICENSED PROFESSIONAL ENGINEER W. A. TROW PROVINCE OF ONTARIO

Yours very truly,
TROW CONSULTING ENGINEERS LTD.

W.A. Trow, P.Eng.

K.R. Peaker, P.Eng.

LICENSED PROFESSIONAL ENGINEER K. R. PEAKER PROVINCE OF ONTARIO

WAT/KRP/jc Encls.
Dist.: See attached

Dist.: **Ministry of Transportation Ontario** **(3)**
Mr. M.S. Devata, P.Eng.
Chief Foundation Engineer

APPENDIX A

DRAWINGS

Ministry of Transportation
Ontario

Drawing 2

RECORD OF BOREHOLE No 1

METRIC

W P 368-90-01 | LOCATION STA. 28+489.4 o/s 2.1 m LT. | ORIGINATED BY S.M.

DIST 18 HWY 548 | BOREHOLE TYPE Hollow Stem Auger | COMPILED BY M.P.

DATUM Geodetic | DATE September 7 and 8 1990 | CHECKED BY A.S.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
180.0	Ground Surface									
0.0	Topsoil ±150 mm over									
0.4	Sand & Gravel Fill	1	SS	16						
	Clay, some silt, moist Very Stiff to Firm Red-Brown	2	SS	7		178				
177.3		3	TW	PM	▽	177.3 2.7	+ s=3.3		17.5	6 22 72
2.7	Clay, some silt, Soft to Firm Red-Brown	4	SS	PM		176				
		5	SS	PM			+ s=3.0			
		6	TW	PM		174	+ s=3.0		17.2	
	(interbedded grey Clay, some silt and fine sand encountered between elevation 170 to 171)	7	SS	PM		172				
		8	SS	PM			+ s=5.0			
		9	SS	PM		170				
		10	TW	PM		168	+ s=4.0		15.4	
		11	SS	PM		166	+ s=4.0	>82%		
		12	SS	PM		164				
		13	TW	PM		162	+ s=3.5		16.9	
160.9										
19.1	Heterogeneous mixture of silt, sand and gravel (Glacial Till) compact to very dense, with numerous cobbles and boulders, grey.					160				37 58 5
		14	SS	23		158				
156.1										
23.9	End of borehole due to refusal to auger on probable boulders									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

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

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

OFFICE REPORT ON SOIL EXPLORATION

Drawing 3

RECORD OF BOREHOLE No 2

METRIC

W P 368-90-1 LOCATION STA. 28+535.0 o/s 10.8 m LT. ORIGINATED BY S.M.

DIST 18 HWY 548 BOREHOLE TYPE Hollow Stem Auger & B.Q. Core COMPILED BY M.D.

DATUM Geodetic DATE September 10 and 11, 1990 CHECKED BY A.S.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
179.6	Ground Surface
0.0	Topsoil ±500 mm over Clay, some silt Stiff Red-brown
178.1 / 1.5	Clay, some silt Soft to Firm Red-brown (interbedded grey, Clay, some silt and fine sand encountered at elevation 170.0 to 170.5)
159.5 / 20.1	Heterogeneous mixture of silt, sand and gravel (Glacial Till) compact to very dense, with numerous cobbles and boulders, grey.
151.6 / 28.0	End of borehole

SAMPLES

NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/M³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7		
2	SS	PM		
3	SS	PM		
4	TW	PM	15.7	
5	SS	PM	82%	0 23 77
6	SS	PM		
7	SS	PM		
8	SS	PM		24 46 30
9	SS	1		
10	TW	PM	15.5	
11	SS	PM		
12	SS	PM		0 17 83
13	SS	PM		
14	SS	PM		
15	SS	4		
16	SS	54		52 38 10
17	R.C B.Q.	8%		
18	SS	68/150 mm		
19	R.C B.Q.	13%		
20	SS	78/150 mm		
21	SS	87/150 mm		
22	SS	65/25 mm		

GROUND WATER CONDITIONS: water level at 178.1 (1.5)

ELEVATION SCALE: 178.1, 176, 174, 172, 170, 168, 166, 164, 162, 160, 158, 156, 154, 152

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

s=6.0, s=6.0, s=7.0, s=6.0, s=6.7, s=4.0

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 20 40 60

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

Drawing 4

RECORD OF BOREHOLE No 3

METRIC

W P 368-90-1 LOCATION STA. 28+563.6 o/s 3.4 m LT. ORIGINATED BY S.M.

DIST 18 HWY 548 BOREHOLE TYPE Hollow Stem Auger & Cone Test COMPILED BY M.D.

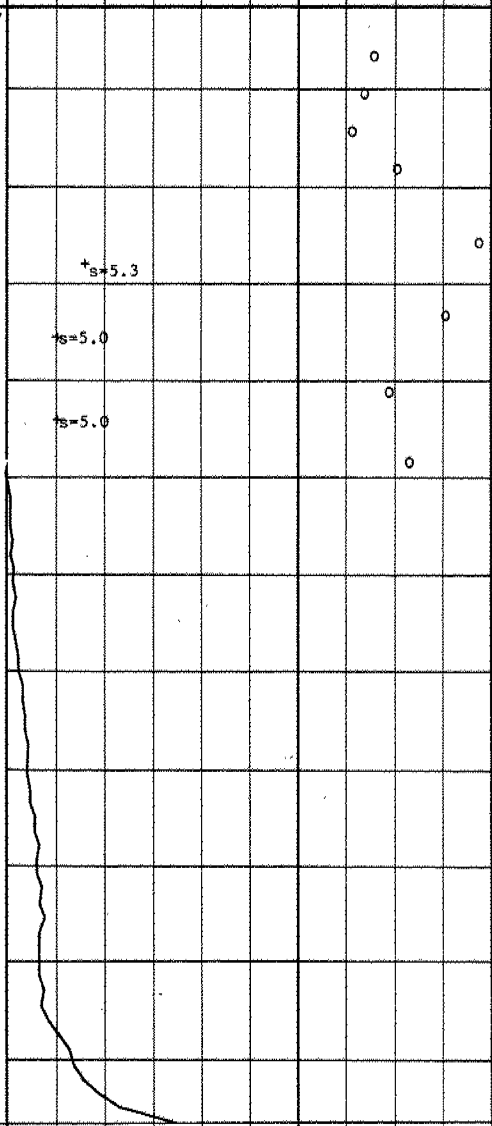
DATUM Geodetic DATE September 9, 1990 CHECKED BY A.S.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.7	Ground Surface										
0.0	Topsoil ±150 mm over Silt & Sand Loose to Very Loose Grey-brown, Stratified		1	SS	9		179.7				
			2	SS	2		178				54 46
177.4 / 2.3	Clay, some silt Firm, red-brown		3	SS	5						
			4	SS	5						
176.2 / 3.5	Clay, some silt Soft Red-brown		5	SS	1		176	+s=5.3			
	(interbedded grey, Clay, some silt and fine sand encountered between elevation 170.1 and 170.5)		6	SS	PM		174	+s=5.0			
			7	TW	PM		172	+s=5.0		17.2	
			8	SS	PM		170				
170.1 / 9.6	End of Borehole Probable Clay, some silt						168, 166, 164, 162, 160, 158				
156.5 / 23.2	End of cone test										

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60



+3, ×5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

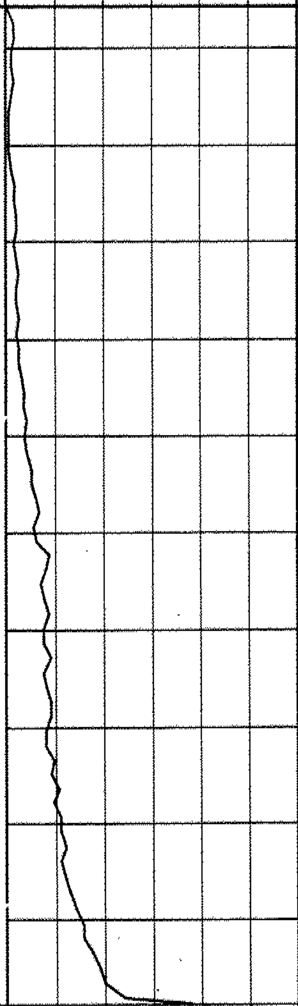
Drawing 5

RECORD OF BOREHOLE No 4 — METRIC

W P 368-90-1 LOCATION STA. 28+510.3 o/s 101.m LT. ORIGINATED BY S.M.

DIST 18 HWY 548 BOREHOLE TYPE Cone Test COMPILED BY M.D.

DATUM Geodetic DATE September 11, 1990 CHECKED BY A.S.

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100)	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
178.3	Ground Surface										
0.0	Dynamic Cone penetration test only Probable Clay, some silt						178 176 174 172 170 168 166 164 162 160				
157.7 20.6	End of cone test										

OFFICE REPORT ON SOIL EXPLORATION

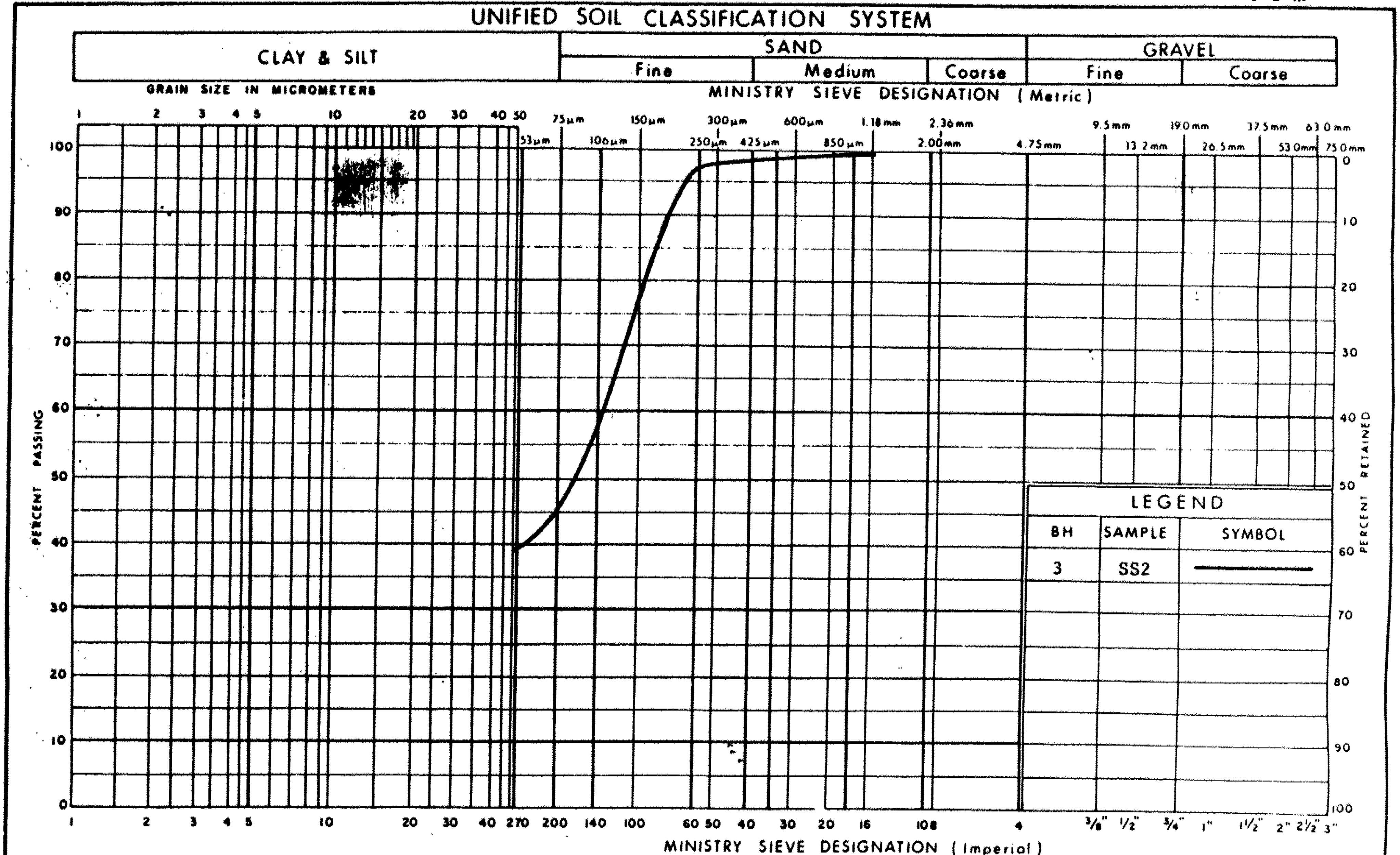
$+^3, \times^5$: Numbers refer to Sensitivity

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

APPENDIX B

FIGURES AND LABORATORY TESTS

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

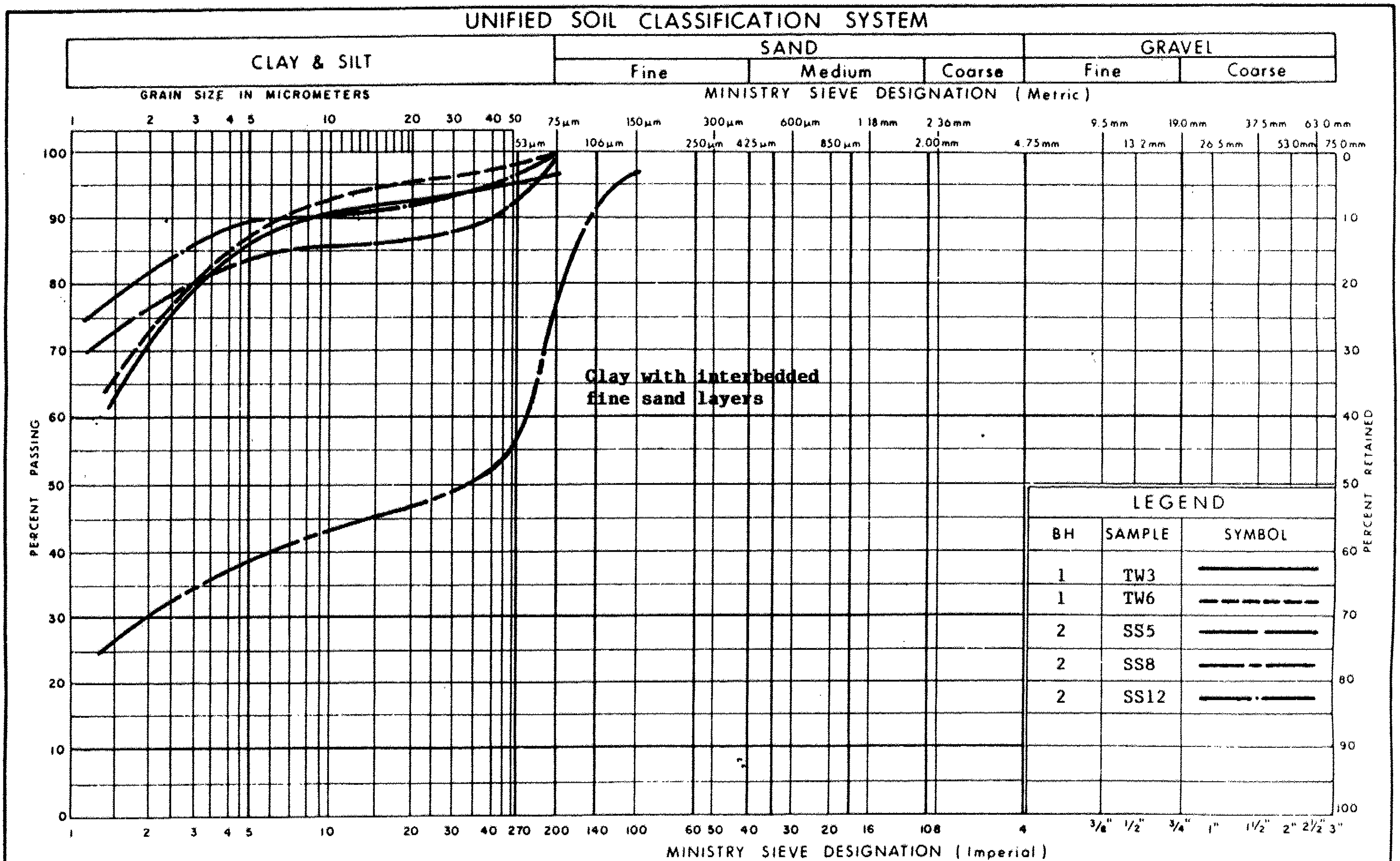
Grading Curve of Upper Silt and Sand

Hole 3 approx. El.178

FIG No 1

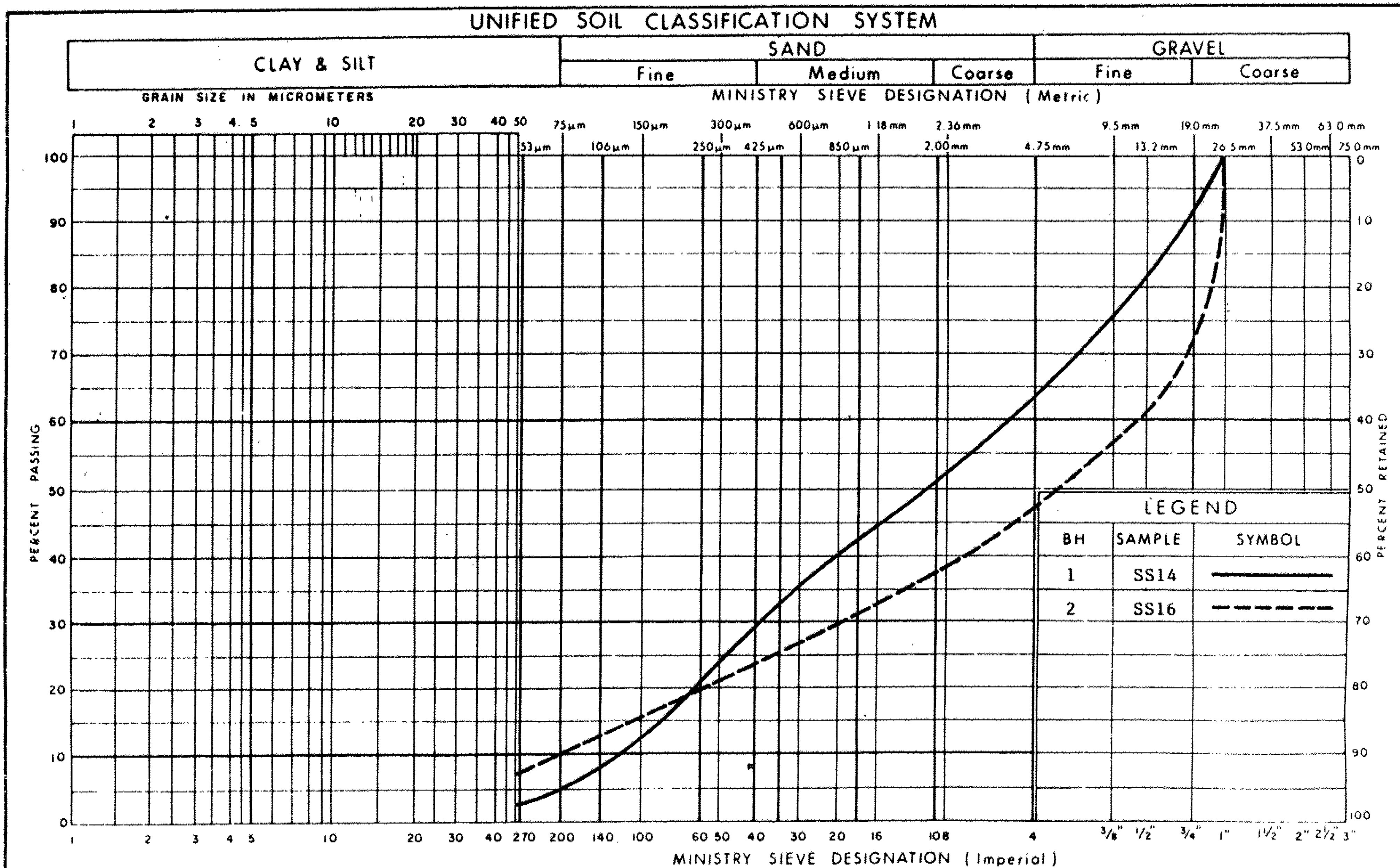
W P 1515-68

78 12 M



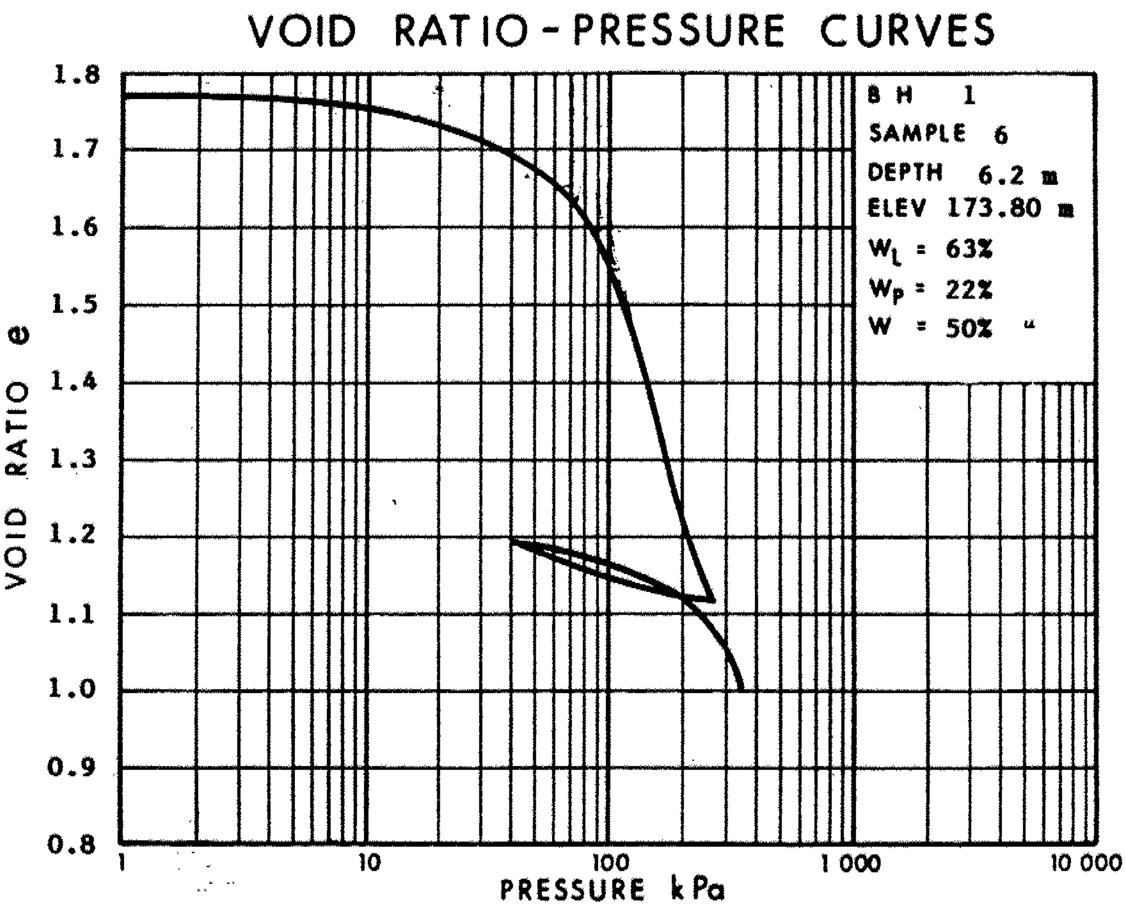
Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION Grain Size Curves for the clay stratum.	FIG No 2 W P 1515-68

78 12 M

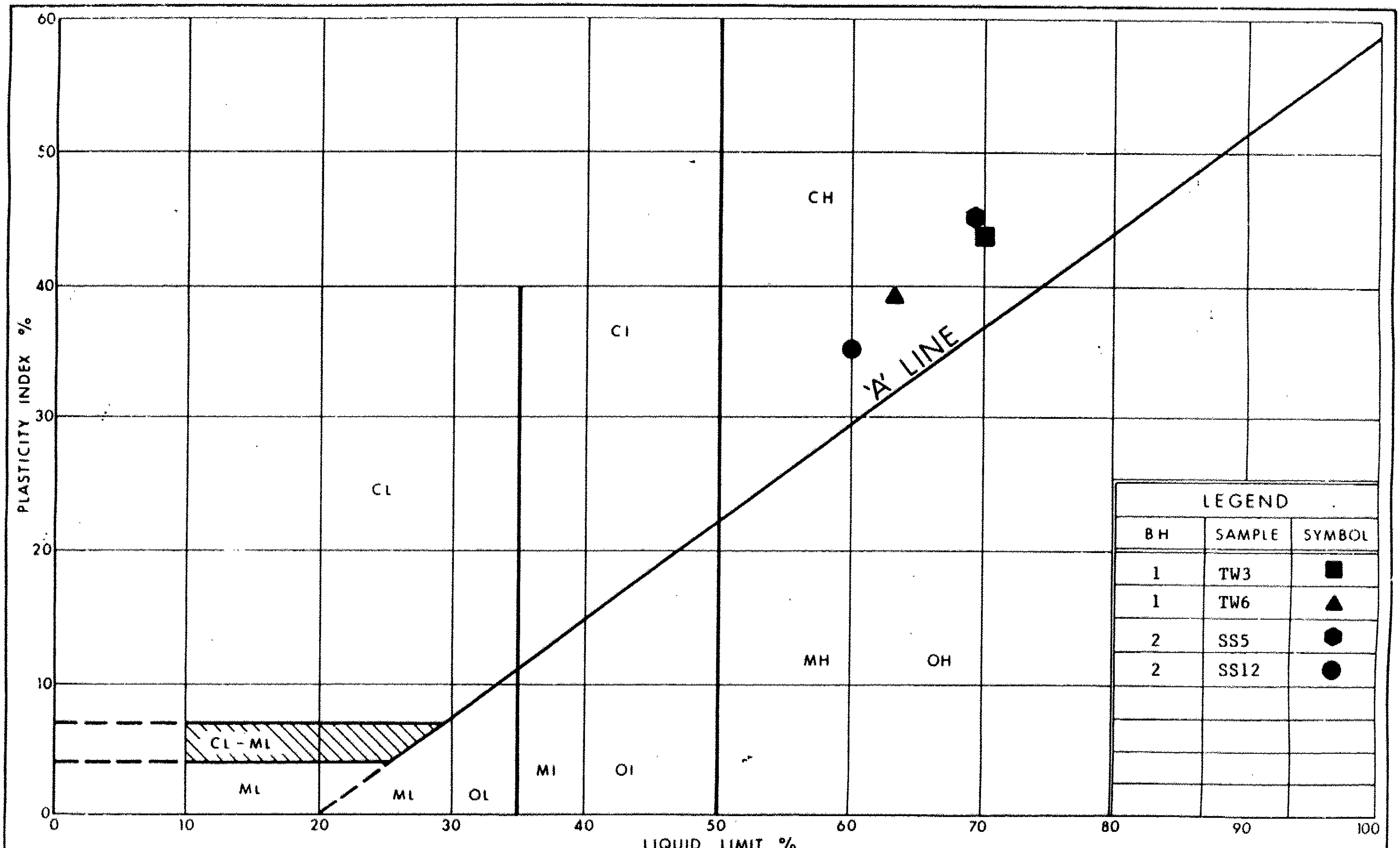


Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION	FIG No 3
	Grading size curves of the heterogeneous mix of sand and gravel (glacial till) found below approximate El. 160m	W P 1515-68

CONSOLIDATION TEST RESULTS



Po ± 70 kPa
Pc ± 90 kPa
Mv = 8.3×10^{-4} kPa^{-1} (20-120 kPa range)
Cv = 5.3×10^{-9} m²/sec
k = 4.7×10^{-9} cm/sec



Ministry of Transportation
Ontario

PLASTICITY CHART

CLAY

FIG No 5

W P 1515-68

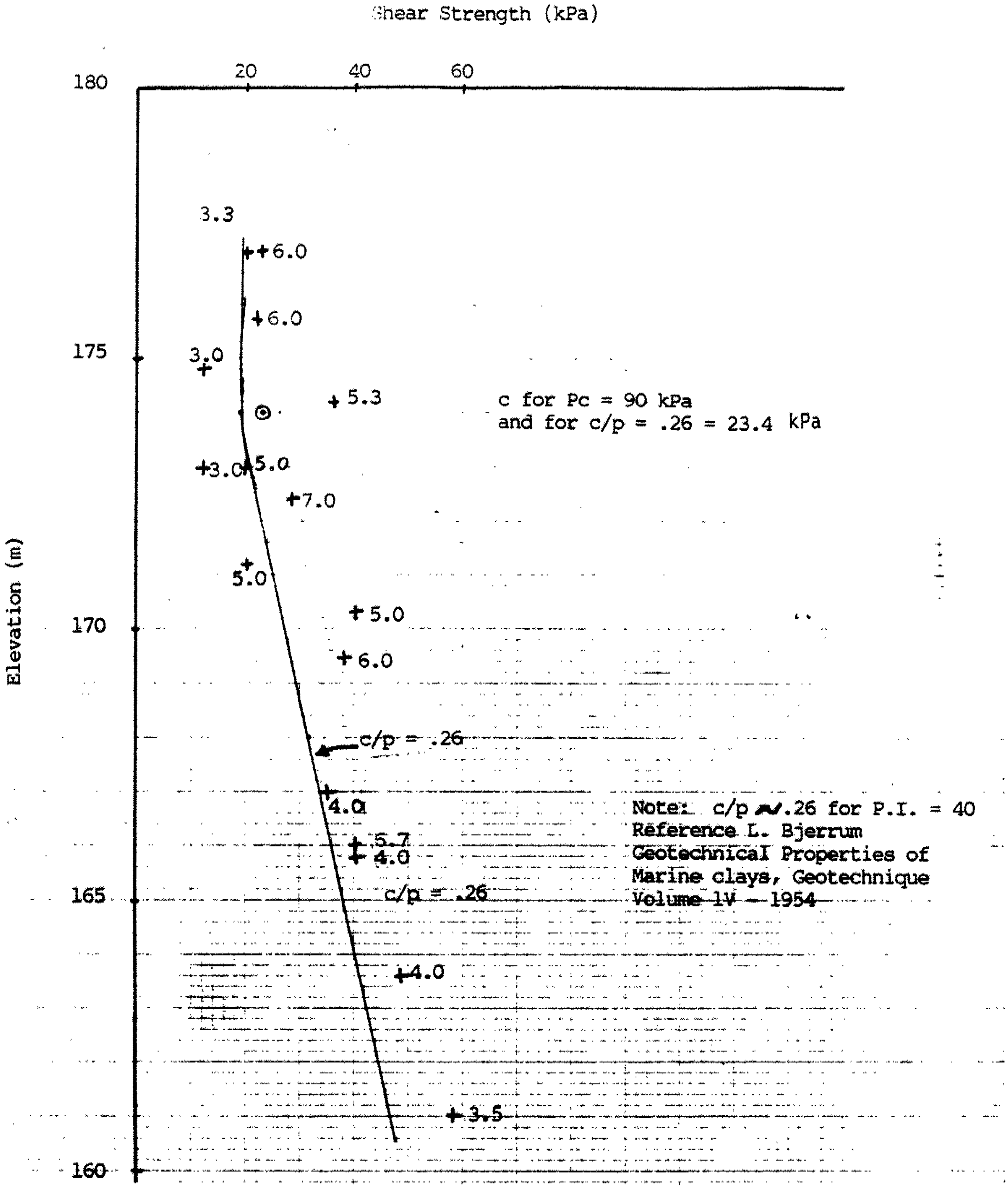
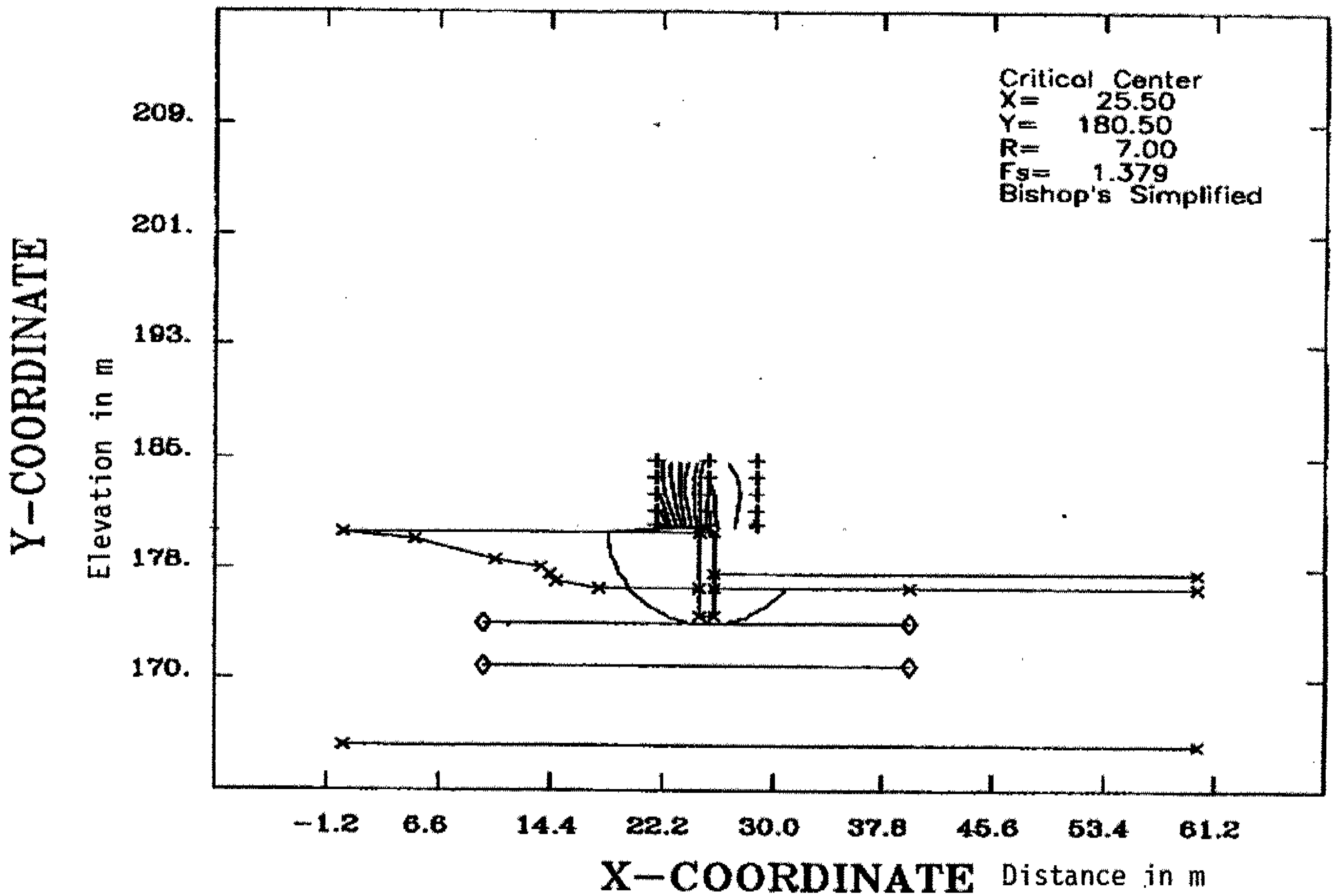


Fig. 6

Field vane test results

TWO TREE CREEK BRIDGE, CU Analysis
19 Ext Rad Con 4/07/91

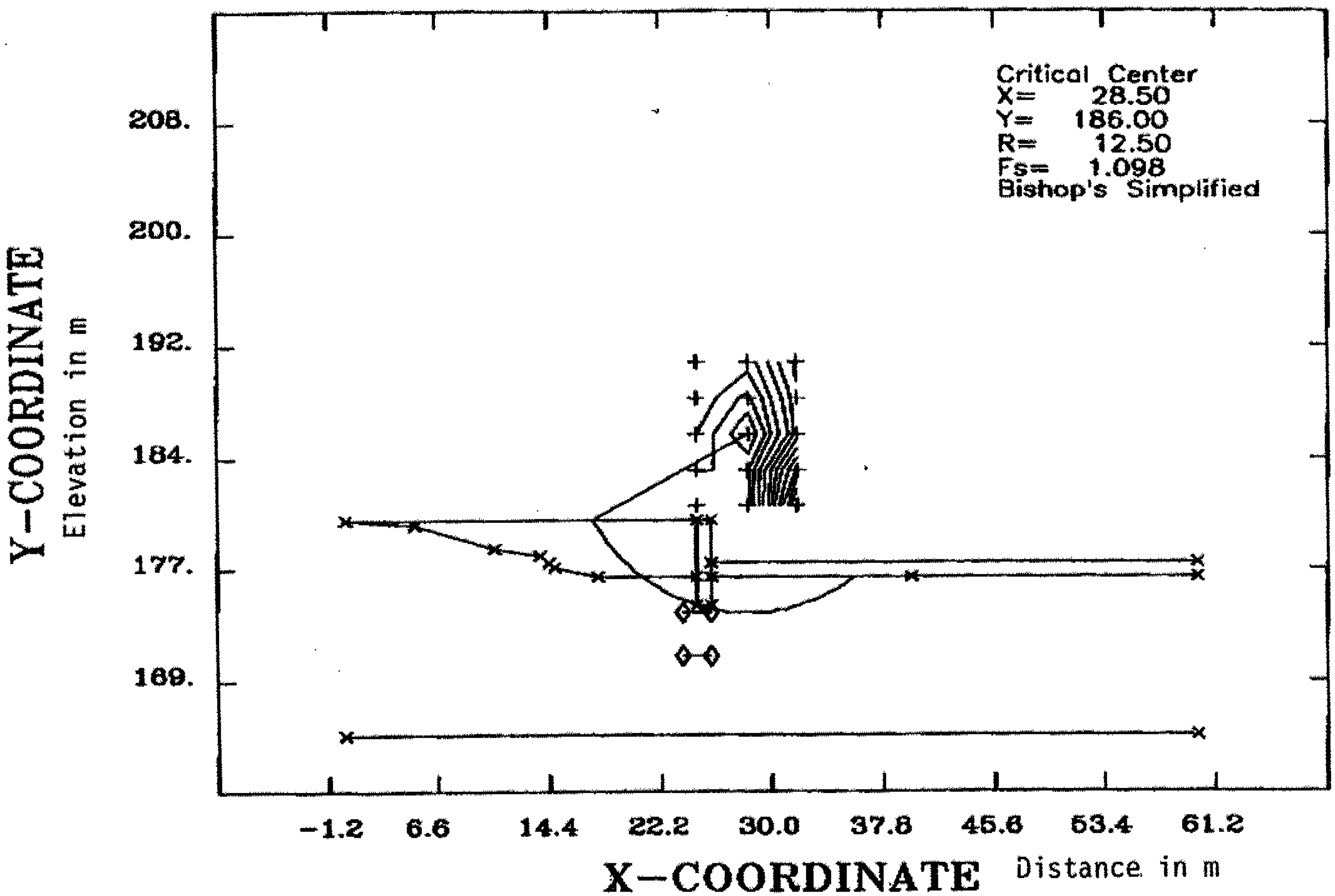


Unit Weight kN/m³	Cohesion KPa	PHI degrees	Description
9.81	.00	.00	WATER
25.00	500.00	40.00	CONCRETE ABUTMENT
21.20	.00	35.00	FILL
17.20	20.00	.00	CLAY
-1.00	.00	.00	Base Of Problem

File name : hy548-19.SET

Fig. 7 Undrained slope stability analysis for west approach of Alternative 2.

TWO TREE CREEK BRIDGE, RU 0.60 New Soil strata
12 4/07/91



Unit Weight kN/m³	Cohesion KPa	PHI Degrees	Description
9.81	.00	.00	WATER
25.00	500.00	40.00	CONCRETE ABUTMENT
21.20	.00	35.00	FILL
17.20	.00	26.00*	CLAY
-1.00	.00	.00	Base Of Problem

File name : hy548-12.SET

* Assumed for P.I. = 40 (After Bjerrum & Simons, Colorado Conference, 1960)

Fig. 8 Slope stability for west approach of Alternative 2 with Ru = 0.6

METRIC

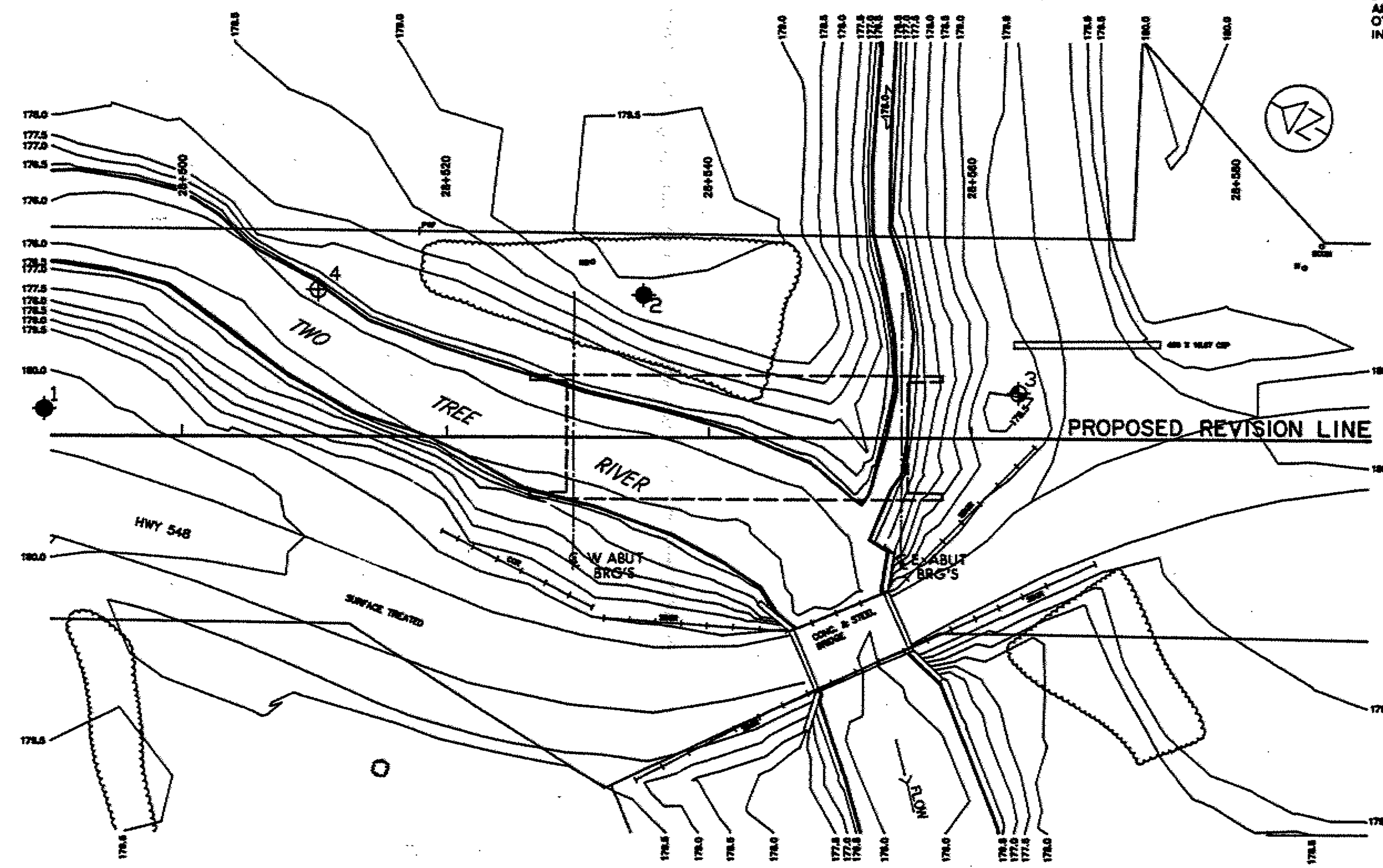
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No 94-205
WP No 84-90-01

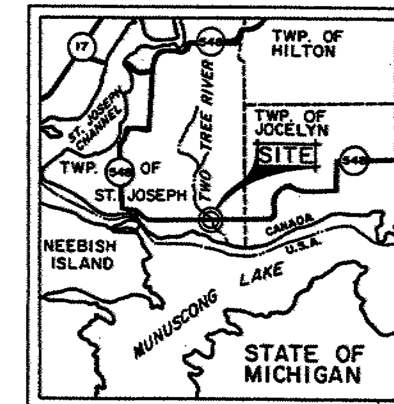
TWO TREE RIVER
(at Hwy 548)
BORE HOLE LOCATIONS & SOIL STRATA

SHEET 13

TROW ONTARIO LTD.



PLAN
SCALE

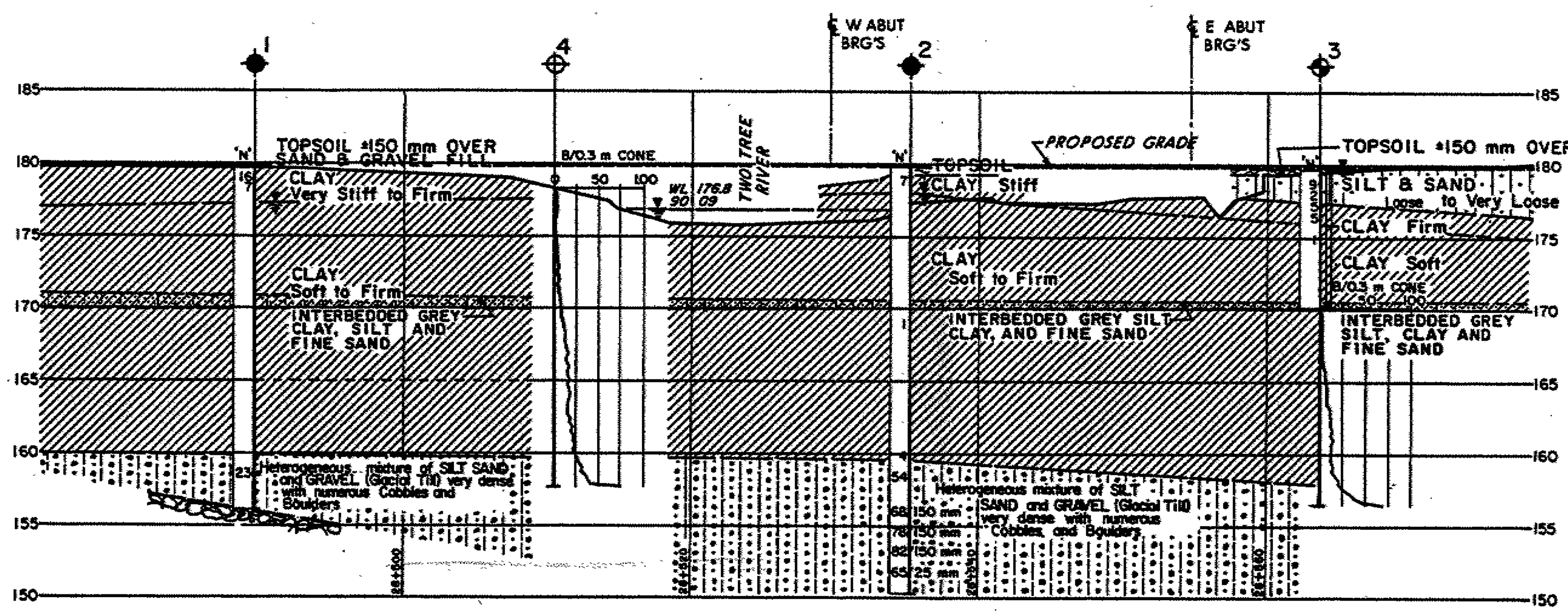


KEY PLAN
SCALE
0 1 2 3 4 5 6 7 8 9 10 km

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 90 09
- NOTE - Boreholes projected onto section.
- Standpipe

No	ELEVATION	STATION	OFFSET
1	180.0	28+489.4	2.1 m LT.
2	179.6	28+535.0	10.8 m LT.
3	179.7	28+563.6	3.4 m LT.
4	178.3	28+510.3	10.1 m LT.



℄ PROFILE REVISION LINE
SCALE
10 5 0 10 20 m

=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

Geocres No 41K-47

HWY No 548			DIST 18
SUBM'D	CHECKED	DATE Oct 19, 1990	SITE 38S-211
DRAWN MD	CHECKED	APPROVED	DWG 2

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