

G.I.-30 SEPT. 1976

GEOCRES No. 3165-89DIST. 9 REGION W.P. No. 10-69-14/15CONT. No. 73-190W. O. No. STR. SITE No. HWY. No. 417LOCATION Green Creek DiversionNo of PAGES -=====OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. T. C. Kingsland, (2) FROM: Foundations Office,
Regional Structural Planning Eng., Design Services Branch,
Eastern Region, West Bldg., Downsview.
Kingston, Ontario.

ATTENTION: DATE: September 26, 1972.

OUR FILE REF.

IN REPLY TO

OCT 2 1972

SUBJECT:

3165-89

FOUNDATION INVESTIGATION REPORT
For
Proposed Structures
At the Crossings of Hwy. #417 (E.B.L. and W.B.L.)
And the Green Creek Diversion
Regional Municipality of Ottawa-Carleton
District No. 9 (Ottawa)
W.O. 72-11092 -- W.P.'s 10-69-14 (E.B.L.)
10-69-15 (W.B.L.)

Attached we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above-mentioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

AGS/ao
Attach.

cc: E. J. Orr
B. R. Davis
A. Rutka
S. J. Markiewicz
J. E. Callaghan
B. J. Giroux
E. R. Saint
G. A. Wrong
B. A. Singh
M. M. Dillon & Co. Ltd.

Foundations Files
Documents

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

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For

At the Crossings of Hwy. #417 (E.B.L. and W.B.L.)

Regional Municipality of Ottawa-Carleton

W.O. 72-11092

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W.P.'s 10-69-14 (E.B.L.)

10-69-15 (W.B.L.)

The Foundations Office was requested to carry out a subsurface investigation at the site of the proposed structures at the crossings of Hwy. #417 (E.B.L. and W.B.L.) and the rechannelized Green Creek, in the Regional Municipality of Ottawa-Carleton. The request was contained in a memo from Mr. T. C. Kingsland, Regional Bridge Planning Engineer, Eastern Region, dated July 12, 1972. An investigation was subsequently carried out by this Office to determine the subsoil, bedrock and groundwater conditions at this site.

This report contains the factual results obtained from the investigation, together with the recommendations pertaining to the foundations of the proposed structures as well as the stability and settlement considerations associated with the approach fills.

The area under investigation is located in the vicinity of Ridge Rd. and Green Creek in the Township of Gloucester, Regional Municipality of Ottawa-Carleton. The terrain is flat to gently undulating in relief between about elevations 207 and 215. Some of the land is being used for farming purposes while other portions have been developed as residential areas. The

south to north flowing Green Creek is approximately 25 feet wide and 2 feet deep. A north-south running C.N.R. line is located about 800 feet west of the proposed crossings.

The present physical features of the region are of varied origin and are the result of erosion and deposition by various agencies. During a long period of time, previous to Pleistocene or Glacial time, the region was above sea level.* During this time the major features of the bedrock topography were formed by processes of weathering and stream erosion. During Pleistocene time the region was invaded by one or more ice sheets advancing from the north. The pre-Glacial land surface was modified by glacial erosion and deposition, in places, of material eroded by the ice sheet. Near the close of Pleistocene time, when the ice began to retire, the area was, in large part, below sea level so that as the ice retired or melted back, the sea entered and overspread the Ottawa Valley to depths of several hundred feet. In this arm of the sea, known as the Champlain Sea, thick deposits of sand, silt and clay were laid down. As the ice sheet still further retired, uplift took place, the land gradually emerged from the sea. This area is now commonly called "The Ottawa Valley Plain."** Here extensive sensitive clay deposits are interrupted by ridges of sand and/or bedrock. The clay is generally underlain by glacial till which in turn is followed by Collingwood and Gloucester shale of the Billings formation, Ordovician period.

3. FIELD AND LABORATORY WORK:

Four sampled boreholes, all of which were accompanied by a dynamic cone penetration test, were put down at the site during the field investigation, using a C.M.E. machine adapted

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* Johnston, W.A., "Pleistocene and Recent Deposits in the Vicinity of Ottawa, with a Description of Soils", Geological Surveys #84, Dept. of Mines.

**Chapman, L.J. and Putnam, D.F., "Physiography of Southern Ontario." University of Toronto Press, 1967.

for soil sampling purposes.

Samples of the overburden were obtained by using a 2-inch O.D. split-spoon sampler, which was hammered into the soil in accordance with the specifications for the Standard Penetration Test. The same method was used to advance the dynamic cone penetration tests. In the cohesive portions of the overburden samples were obtained in 2-inch I.D. Shelby tubes which were pushed down hydraulically. Wherever possible, the undrained shear strength of the cohesive stratum was measured by performing field vane tests. Bedrock was proven in all the boreholes by obtaining BX size rock core samples.

The soil, bedrock and groundwater conditions encountered at the boring locations, are presented on the Record of Borelog sheets appended to this report. The locations and elevations of the boreholes were provided by the personnel from the Eastern Region Engineering Surveys Section. The elevations in this report are referenced to a Geodetic datum. The boring locations and elevations are shown on Drawing No. W.O. 72-11092A. A stratigraphical profile, inferred from the boring data, is also presented on the aforementioned drawing.

All the samples were subjected to a careful visual examination in the field and subsequently in the laboratory. Following this examination, laboratory testing was carried out on selected representative samples to determine the following physical properties of the overburden:

- Natural Moisture Content
- Atterberg Limits
- Grain-size Distribution
- Consolidation Characteristics
- Undrained Shear Strength

The results of this testing are plotted on the Record of Borehole sheets and summarized on Figures No. 1, 2 and 3, all of which are contained in the Appendix of this report.

4. SUBSOIL AND BEDROCK CONDITIONS:

4.1) General:

The surficial stratum across the site is composed of

a firm to very stiff silty clay to clay. The thickness of this cohesive stratum varies from 19 to 29 feet. Underlying this deposit is a 11 feet to 15 feet thick loose to very dense glacial till deposit. The glacial till is followed by shale bedrock.

The boundaries of the various deposits, as determined in the boreholes, are shown on the accompanying Record of Borehole sheets. The stratigraphical profile, shown on Drawing No. 72-11092A, has been inferred from this data. From ground surface downward the various soil and bedrock types encountered are as follows.

4.2) Silty Clay to Clay:

Directly beneath a nominal topsoil cover (8 inches) is a 19 to 29 feet thick sensitive marine stratum composed of a grey silty clay to clay with a trace of sand throughout. The upper 6 to 8 feet of this stratum is brown in colour which indicates that this zone has been desiccated. Occasional silt and sand seams, up to 2 inch thick, were encountered throughout the deposit. Grain-size distribution tests were carried out on samples of the clay stratum, the results are shown in envelope form on Figure No. 1, in Appendix I of this report.

The engineering properties of the deposit, as determined by field and laboratory testing, are presented in Table 1.

TABLE I

<u>Tests</u>	<u>Range</u>
Bulk Density (p.c.f.)	102-115
Liquid Limit (W_L) (%)	33-64
Plastic Limit (W_p) (%)	20-27
Natural Moisture Content (W) (%)	37.5-60
Liquidity Index (I_L)	0.6-1.5
Initial Void Ratio (e_o)	1.24 - 1.55
Compression Index (C_c)	0.53 - 0.87
Degree of Preconsolidation (p.s.f.) ($P_c - P_o'$)	2,600 - 5,700

Undrained Shear Strength (p.s.f.) (C_u)

i) Field Vanes	800 - >2,000
ii) Lab. Testing	800 - >2,000

The Atterberg Limit tests, summarized on Table I are also plotted on the Plasticity Chart, Figure No. 2. These results indicate that cohesive stratum is inorganic with a plasticity ranging from intermediate to high.

Based on the undrained shear strength testing carried out it is estimated that the consistency of the major portion of the stratum varies from firm to very stiff. The upper desiccated zone, however, is very stiff.

The consolidation characteristics of the stratum were determined by carrying out four laboratory tests, the results of which are shown as Void Ratio vs. log of pressure plots on Figure No. 3. This testing indicates that the clay stratum is preconsolidated by anywhere from 2,600 to 5,700 p.s.f. in excess of the existing overburden pressure.

4.3) Heterogeneous Mixture of Silt, Sand and Gravel, with Some Clay - Glacial Till:

Underlying the cohesive stratum is a deposit of glacial origin consisting of a heterogeneous mixture of silt, sand and gravel, with some clay. This glacial till deposit, which varies from 11 to 15 feet thickness is basically granular in nature. There are, however, random cohesive zones throughout; in these areas the till has a matrix of clayey silt binding sand and gravel. Boulders up to 6 inches in size were encountered below elevation 180 in B.H. #4. Typical grain-size distribution curves, obtained from samples of the till are plotted on Figure No. 1.

The Standard Penetration Tests carried out gave 'N' values which ranged from 1 to 7 blows/ft., immediately below the clay stratum, increasing to 12 to 29 blows/ft. in the lower portion of the deposit. Based on these results, it is estimated that the relative density of the upper "reworked" zone varies from loose to compact, while the lower zone is in the compact range.

4.4) Shale Bedrock:

The glacial till deposit is directly underlain by bedrock, which was proven in each of the boreholes by obtaining up to 5 feet of BX size rock core samples. The bedrock surface was found to range from elevation 173.5 (B.H. #1) to elevation 180.5 (B.H. #6), which corresponds to depths below ground surface of from 30.5 to 43.5 feet.

The bedrock core samples were examined by Mr. K. W. Ingham, Geologist, Ministry of Transportation and Communications. Mr. Ingham presented the result of his bedrock examination in a memo to this Office, dated September 14, 1972. The bedrock is composed of a grey calcareous shale which is in a sound state as evidenced by the high percentage of rock core recovered.

5. GROUNDWATER CONDITIONS:

The groundwater level conditions across the site, during the period of investigation (August, 1972), were observed by taking readings in the open boreholes. The results of the readings are summarized on Drawing No. 72-11092A.

The observations indicate that the groundwater level is located between elevations 204 and 212.5 which corresponds to depths below ground surface of from 3 feet to 6 feet.

6. DISCUSSIONS AND RECOMMENDATIONS:

6.1) General:

In the vicinity of Ridge Rd., in the Regional Municipality of Ottawa-Carleton, the east and westbound lanes of Hwy. #417 will cross the diverted Green Creek. It is understood that a 25 feet span, 260 feet long culvert structure will be placed at each of the crossings; the culverts will be approximately 135 feet apart. The profile grade of the E.B.L. and W.B.L. of Hwy. #417 will vary between elevations 242 and 243, while the invert of the creek will be at elevation 202. At these grades the associated embankments will extend up to 37 feet above the level of the Green Creek valley floor.

Two structure types are being considered, namely:

- i) a rigid frame reinforced concrete structure,
- or ii) a structural plate pipe arch.

The subsoil at the site consists of a 19 to 29 feet thick, firm to very stiff, stratum of silty clay to clay, which is underlain by a 11 to 15 feet thick glacial till deposit, which in turn is followed by sound shale bedrock.

Either of the alternate schemes would be practical from a foundation point of view. The recommendations pertaining to the two schemes are discussed separately in the subsections to follow.

6.2) Embankment Fill:

The E.B.L. and W.B.L. of Hwy. #417 will be carried across this area on embankments. The E.B.L. and W.B.L. embankments will have a maximum height of 32 and 37 feet, respectively. The two embankments will be placed directly on the sensitive cohesive stratum. The critical condition for stability of an embankment on slightly overconsolidated clays, as is the case with this stratum, generally occurs during or immediately after construction. This being the case a total stress analysis ($\phi = 0$) provides a suitable means of assessing the stability of the embankment sections. In this method of analysis, stability is governed by the applied loads and the stress-strain and undrained shear strength properties of the foundation and embankment soils.

Analyses have been carried out, therefore, in terms of total stresses to determine the stability of the fill sections. The following assumptions were made.

1) Soil Properties

<u>Elevation</u>	<u>Soil</u>	<u>Density</u> <u>(p.s.f.)</u>	<u>Strength Parameters</u>	
			<u>C_u (p.s.f.)</u>	<u>ϕ (°)</u>
	Embankment Fill	125	0	30
210-202	Clay	115	1,200	0
202-194	Clay	105	900	0
194-188	Clay	110	1,000	0
188-182	Granular Till	115	0	35
182-176	Granular Till	120	0	42

2) Groundwater Level - Elev. 205.

The results of the stability computations are summarized in tabular form.

Stability Considerations

<u>Height of Embankment</u>	<u>Berm Requirements</u>
31 feet	Nil
34 feet	10 feet wide - mid-height
37 feet	20 feet wide - mid-height

- NOTE:
- a) All slopes 2:1.
 - b) The surface of all berms required should slope away from the fill at a gradient of 20:1 for drainage purposes.
 - c) The requirements listed provide a minimum factor of safety of 1.3.

The cohesive subsoil will settle due to the embankment fill loading. Computations were carried out in order to estimate the magnitude of this consolidation settlement. The results are tabulated below.

Estimated Maximum Consolidation Settlement
Beneath Centre-Line of Embankments

<u>Hwy. #417 Lane</u>	<u>Max. Ht. of Fill</u>	<u>Est. Total Settlement</u>
W.B.L.	37'	6" to 8"
E.B.L.	28'	4" to 5"

The major portion of this consolidation settlement should occur within seven years following fill placement, while 50 percent should be realized within 18 months. In addition, the granular glacial till will settle approximately 1 inch; this settlement will be elastic - i.e., take place during or immediately following fill placement.

The fills, in the immediate vicinity of the culverts, should be protected against the scour action of Green Creek. A properly designed and placed rip-rap could be used for this purpose.

6.3) Structure Schemes:

6.3.1) Twin Rigid Frame Structures:

Rigid frame structures can be supported on shallow foundations located in the upper cohesive stratum. A minimum of 4 feet of earth cover should be provided to the underside of the foundations for frost protection purposes. This would place them at or below elevation 198. Foundations founded as recommended, could be designed using an allowable bearing value of 1.0 t.s.f.

Settlement will be induced in the cohesive subsoil by the applied mat pressure. This settlement will be of the order of 2 to 3 inches. Further, it will occur within 18 months after construction of the culvert.

If the bearing value quoted above is insufficient or if the settlement estimated exceeds tolerable limits, then the structures could be supported on end-bearing piles driven to bedrock. For estimating purposes it can be assumed that the pile tips will be located at the following elevations.

<u>Structure Location</u>	<u>Estimated Pile Tip Elev.</u>
E.B.L.	173 to 174
W.B.L.	176 to 180

The allowable pile load would be dependent on the section chosen. The underlying cohesive subsoil will settle due to the embankment load; therefore, some negative skin frictional loads may be imposed on the piles supporting the foundations. The negative skin frictional component should be allowed for by employing a design value which is 85 percent of the allowable structural capacity of the pile section. For instance 12BP74 steel H-piles should be designed for 80 tons/pile rather than the 95 tons/pile usually employed.

The foundation excavations will extend some 3 to 4 feet below the groundwater level recorded at the time of the investigation. Since the cohesive subsoil is relatively impervious, no major dewatering problems are anticipated. In places, the diversion will be located within the confines of the existing channel of Green Creek. In order to prevent seepage from this source into the

SS 0-17

excavations the creek should be temporarily diverted during the construction period. Any minor seepage emanating from other sources such as the water bearing granular seams located within the cohesive stratum, could be controlled using conventional techniques (pumping from sumps, etc.)

If the structures are designed as rigid frames then a coefficient of earth pressure at rest (K_0) of 0.5 should be assumed for the granular material placed behind the wall, when designing the wall sections. However, if some movement of the top of the wall is permitted, then a coefficient of active earth pressure (K_a) of 0.33 can be used. In all cases the design should incorporate the full effect of the surcharge located above the walls.

In order to relieve the build up of excess hydrostatic pressure behind the walls, suitable drainage measures should be provided. Weep holes, located at the base of the walls, could be employed for this purpose; these holes should be spaced not more than 10 feet apart.

6.3.2) Twin Structural Plate Pipe Arches:

As an alternative twin structural plate pipe arches could be employed at this site. No major complications are envisaged with regard to the placement and performance of the structures. The bedding and backfilling for the culvert should be carried out in accordance with current M.T.C. practices. The pertinent standard is No. DD-808-B (Type 5). It is recommended that the structural pipe arches be placed on a mat of granular fill with a thickness of 4.5 feet (12 inches specified in DD-808-B). This provision will distribute the critical corner bearing pressure (maximum predicted 9,000 p.s.f.) so that the stress increase, induced in the cohesive foundation subsoil, will not exceed the allowable bearing capacity of this stratum.

The pipe arches will settle differentially due to the consolidation of the underlying cohesive stratum. In order to allow for this settlement it is recommended that the two

structures be cambered as follows:

<u>Pipe Arch</u>	<u>Recommended Camber</u>
W.B.L.	6"
E.B.L.	4"

The culvert excavation will extend below the groundwater level recorded during the period of the investigation. Dewatering considerations will be similar to those discussed in Subsection 6.3.1.

7. MISCELLANEOUS:

The field work for this project was carried out on August 1 and 2, 1972, under the supervision of Mr. S. A. Ahmad, Project Foundations Engineer.

The drilling equipment was owned and operated by Master Soil Investigation Ltd., Toronto.

This report was written by Mr. B. T. Darch, Senior Foundations Engineer, and reviewed by Mr. M. Devata, Supervising Foundations Engineer.

B. T. Darch

B. T. Darch, P. Eng.



M. Devata

BTD/ao

M. Devata, P. Eng.

September 1972.

APPENDIX I

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 1

JOB 72-11092

LOCATION Co-ords. 496,999 N; 232,778 E.

ORIGINATED BY SAA

W.P. 10-69-14

BORING DATE Aug. 2, 1972

COMPILED BY SAA

DATUM Geodetic

BOREHOLE TYPE Washboring, NX, RX Casing, BXL Rock Core

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	w_p	w	w_L		
208.7	Ground Surface															
0.0	Silty Clay		1	SS	15											
	Brown. Very Stiff		2	TW	PM											
	Grey		3	TW	PM											
	Clay, trace of sand		4	TW	PM											
	(silt seams up to 1/8" thick below El. 195)		5	SS	6											
188.7	Firm to Very Stiff		6	SS	5											
20.0	Het. mix. of silt, sand and gravel with some clay (Glacial Till)		7	SS	7											
			8	SS	21											
173.7	Loose to Compact															
35.0	Shale Bedrock		9	RC	100%											
168.9	Sound Grey															
39.8	End of Borehole															

OFFICE REPORT SOIL EXPLORATION

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 3

JOB 72-11092

LOCATION Co-ords. 496,919 N; 233,020 E.

ORIGINATED BY SAA

W.P. 10-69-14

BORING DATE Aug. 2, 1972

COMPILED BY SAA

DATUM Geodetic

BOREHOLE TYPE Cont. Flight Auger, BXL Rock Core, Cone Test

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	w_p	w	w_L		
217.7	Ground Surface															
0.0	Silty clay (silt pockets) Desiccated Brown Very Stiff Grey		1	SS	16											
			2	TW	PH	210										
			3	TW	PH											
	Clay to silty clay, trace of sand (silt layers up to 1" thick below El. 194)		4	TW	PH											
			5	TW	PH	200										
			6	TW	PH											
	Firm to Stiff		7	TW	PH	190										
188.7																
29.0	Het. mix. of silt, sand and gravel with some clay (Glacial Till)		8	TW	PH											
			9	SS	7	180										
			10	SS	29											
174.2	Loose to Compact															
43.5	Shale Bedrock			RC												
169.2	Sound Grey		11	BXL	100%	170										
48.5	End of Borehole															

OFFICE REPORT SOIL EXPLORATION

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 4

JOB 72-11092

LOCATION Co-ords. 496,878 N; 233,146 E.

ORIGINATED BY SAA

W.P. 10-69-15

BORING DATE August 2, 1972

COMPILED BY SAA

DATUM Geodetic

BOREHOLE TYPE Washboring, NX, BX Casing, BXL Rock Core, Cone

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	w_p	w	w_L		
207.0	Ground Surface															
0.0	Silty clay Desiccated Brown Stiff Grey		1	SS	6											
			2	TW	PM	200										
	Clay to silty clay trace of sand (seams of silt up to 2" thick below El. 195)		3	TW	PM											
			4	TW	PM											
			5	TW	PM	190										
188.0	Firm to Stiff															
19.0	Het. mix. of silt, sand and gravel, with some clay (Glacial Till) (boulders up to 6" in size below El. 180.)		6	SS	1											
			7	SS	17	180										
176.5	Loose to Compact		8	RC	10%											
30.5	Shale Bedrock		9	SS	100%											
171.0	Sound Grey		10	BXL	90%											
36.0	End of Borehole					170										

OFFICE REPORT ON SOIL EXPLORATION

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 6

JOB 72-11092

LOCATION Co-ords. 496,794 N; 233,397 E.

ORIGINATED BY SAA

W.P. 10-69-15

BORING DATE August 1, 1972

COMPILED BY SAA

DATUM Geodetic

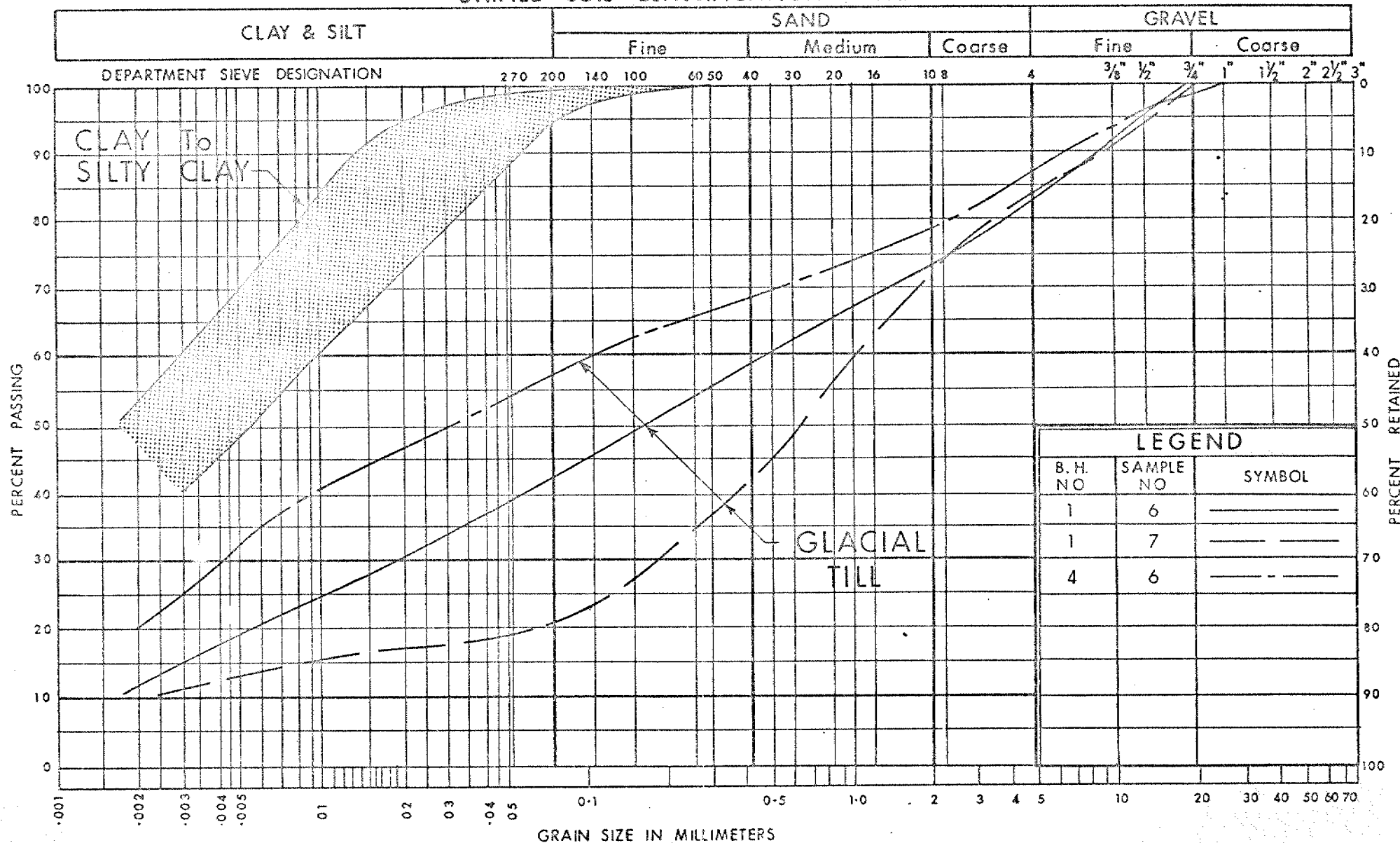
BOREHOLE TYPE Cont.Flight Auger, BX Casing, BXL Rock Core,

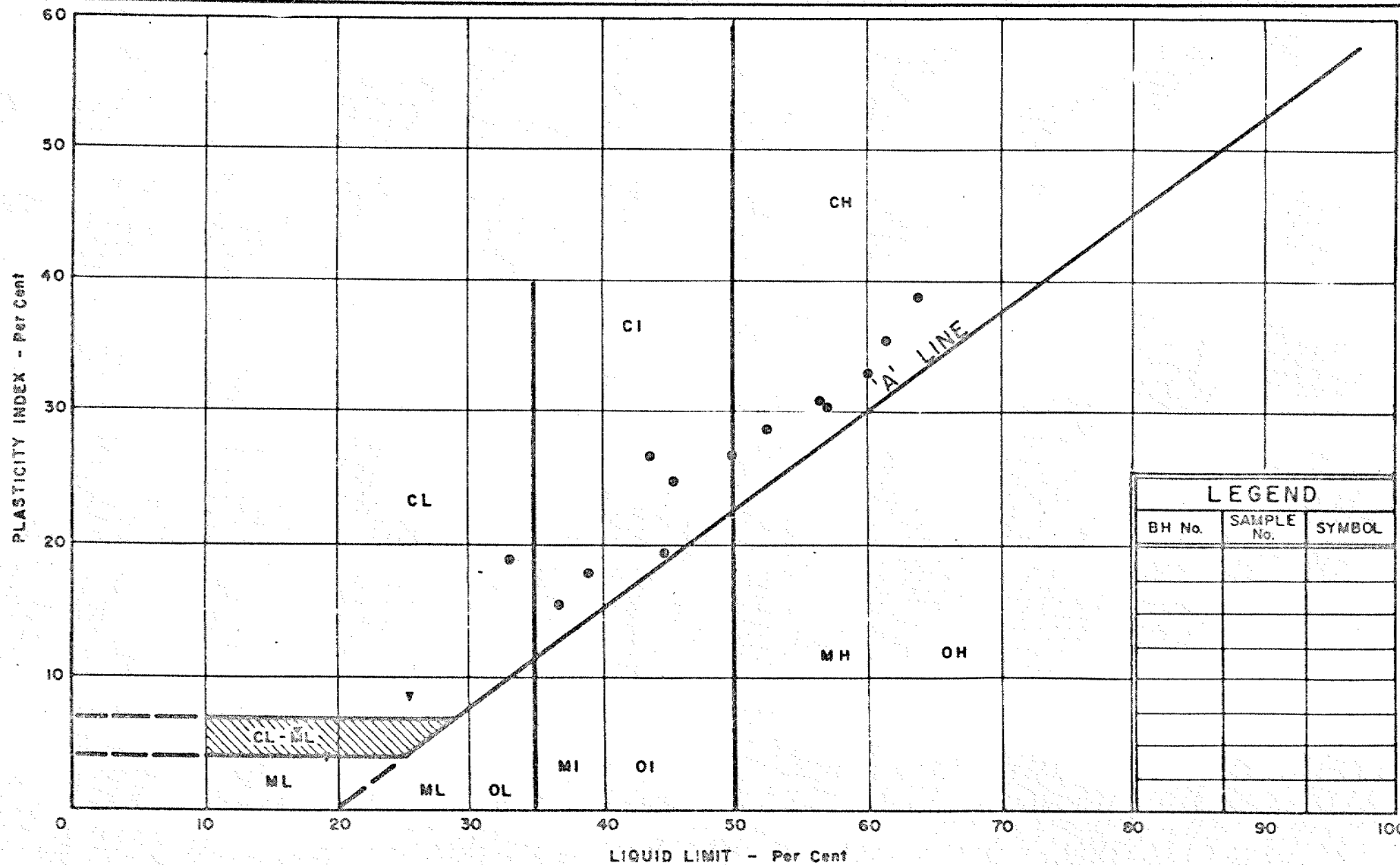
CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					
							20	40	60	80	100+	P.S.F.					
												UNCONFINED	FIELD VANE	QUICK TRIAXIAL			LAB VANE
214.5	Ground Surface																
0.0	Silty clay Desiccated Brown Very Stiff Grey		1	SS	13										in open BH		
			2	TW	PH		210									208.5	
			3	TW	PH												0 4 54 4
	Clay to silty clay, trace of sand (occ. silt seams up to 1/2" thick throughout)		4	TW	PH		200										
			5	TW	PH												
			6	TW	4												
191.5	Firm to Stiff																
23.0	Het.mix.of silt,sand & gravel,with some clay. (Glacial Till)		7	SS	7	190											
			8	SS	12												
180.5	Loose to Compact					180											
34.0	Shale Bedrock		9	RC													
175.5	Sound Grey		BXL	100%													
39.0	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

UNIFIED SOIL CLASSIFICATION SYSTEM





DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART

● - SILTY CLAY to CLAY
▼ - GLACIAL TILL (HET. MIX. OF SILT, SAND & GRAVEL)

WP. No. 10-69-14 & 15

JOB No. 72-11092

FIG. 2

VOID RATIO-PRESSURE CURVES

JOB NO. 72-11092

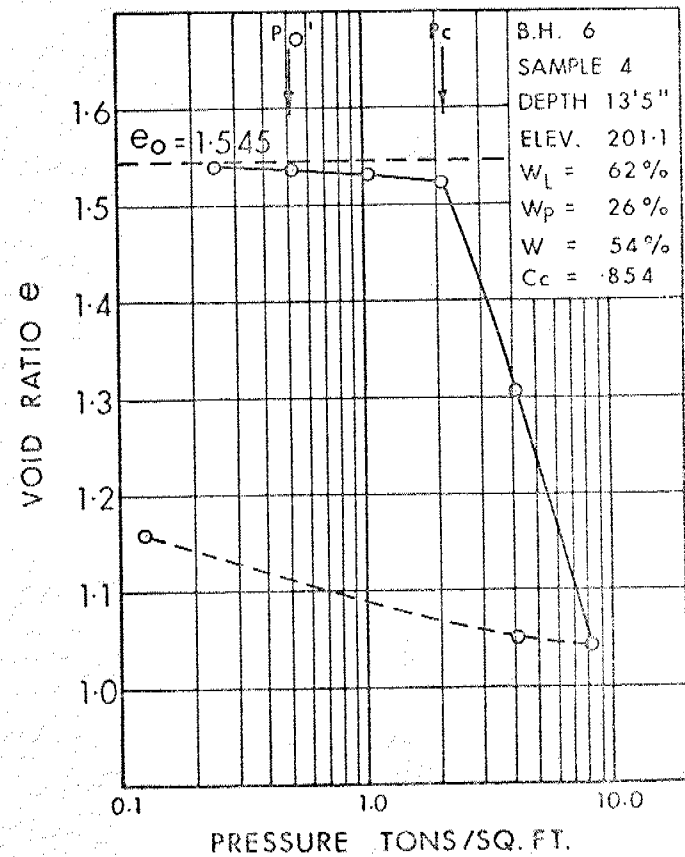
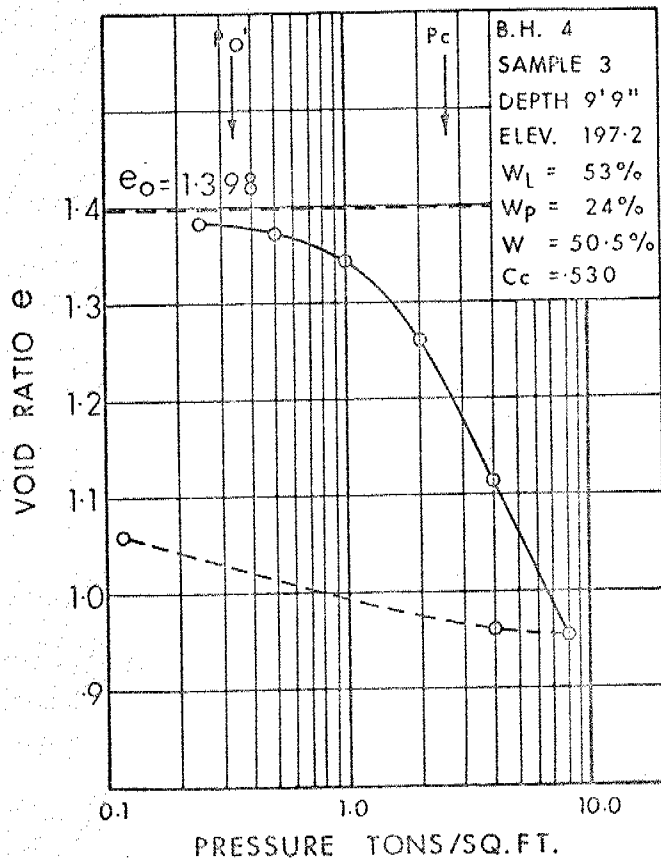
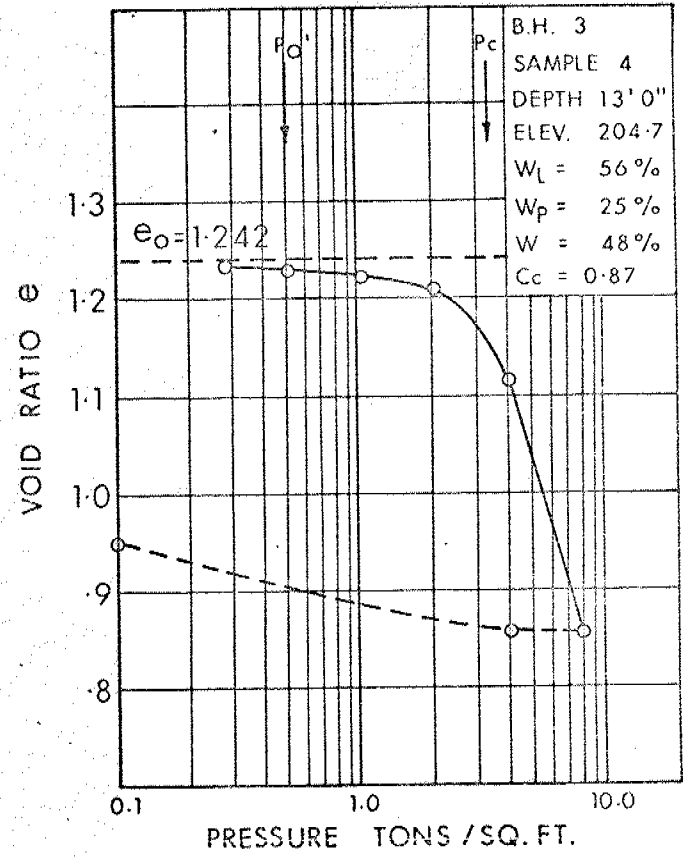
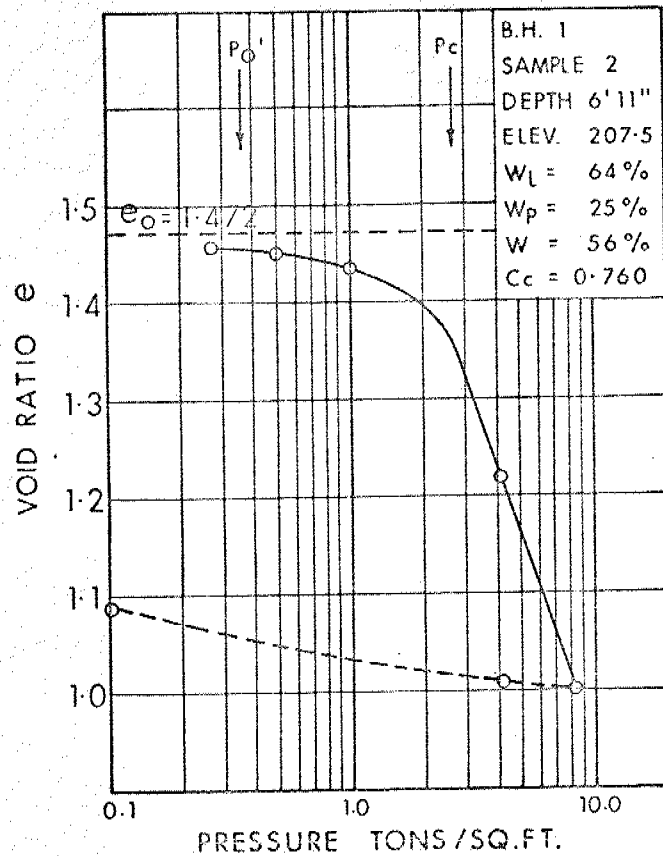


FIG. 3

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_P	PLASTIC LIMIT
I_P	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	≈ 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e \sigma$ OR $\ln \sigma$	NATURAL LOGARITHM OF σ
$\log_{10} \sigma$ OR $\log \sigma$	LOGARITHM OF σ TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

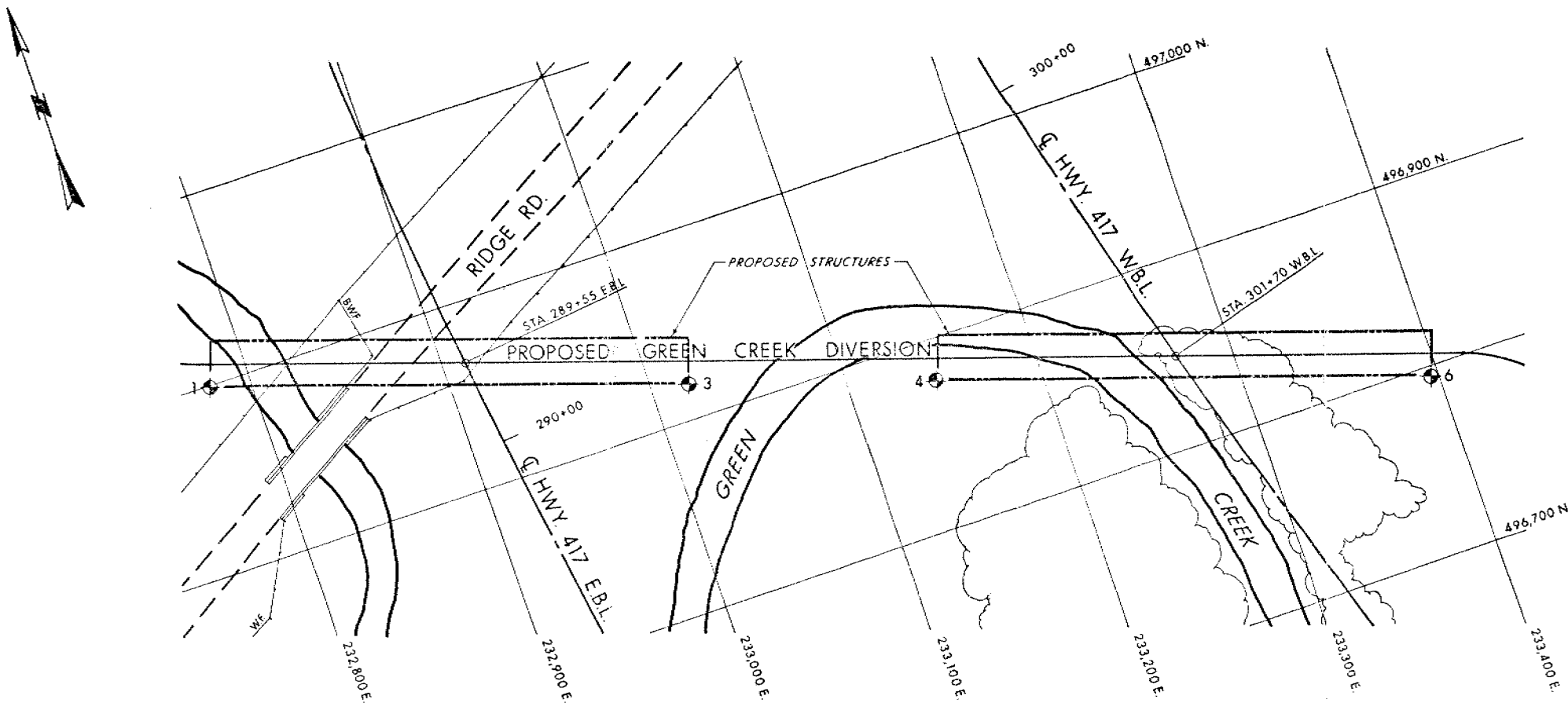
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

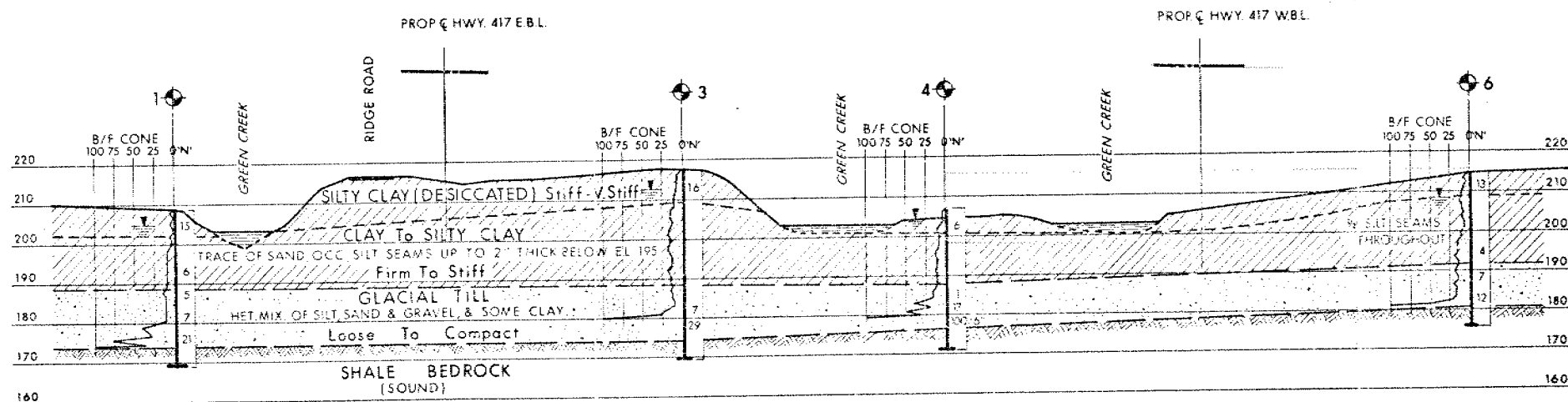
B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

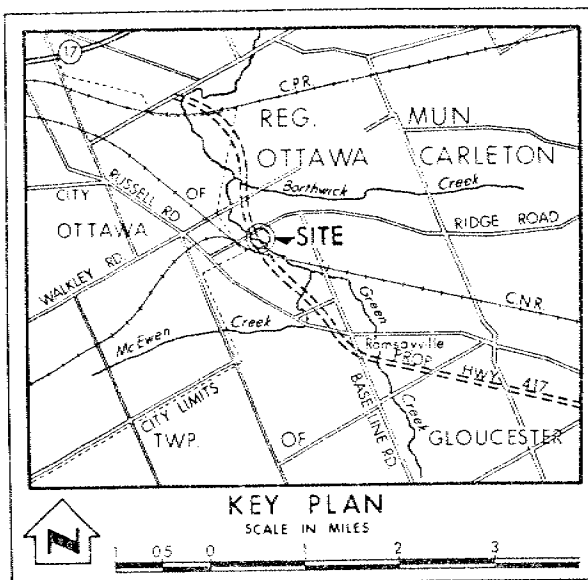
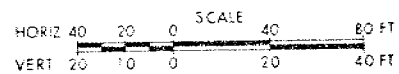
H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL



PLAN



PROFILE GREEN CREEK DIVERSION



LEGEND

- Bore Hole
- ⊕ Cone Penetration Test
- ⊕ Bore Hole & Cone Test
- ⊕ Water Levels established at time of field investigation, AUG 1972

NO.	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	208.7	496,999	232,778
3	217.7	496,919	233,020
4	207.0	496,878	233,146
6	214.5	496,794	233,397

NOTE

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

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
DESIGN SERVICES BRANCH—FOUNDATIONS OFFICE

GREEN CREEK (SOUTH OF RIDGE ROAD)

HIGHWAY NO. 417 E.B. & W.B. DIST. NO. 9
CO. REG. MUN. OTTAWA CARLETON
TWP. GLOUCESTER LOT _____ CON. _____

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD. S.A.	CHECKED	W.F. NO.	DRAWING NO.
DRAWN J.P.G.	CHECKED	W.F. NO. 72-11092	72-11092 A
DATE 11 SEPT 1972	SITE NO.	BRIDGE DRAWING NO.	
APPROVED	CONT. NO.		