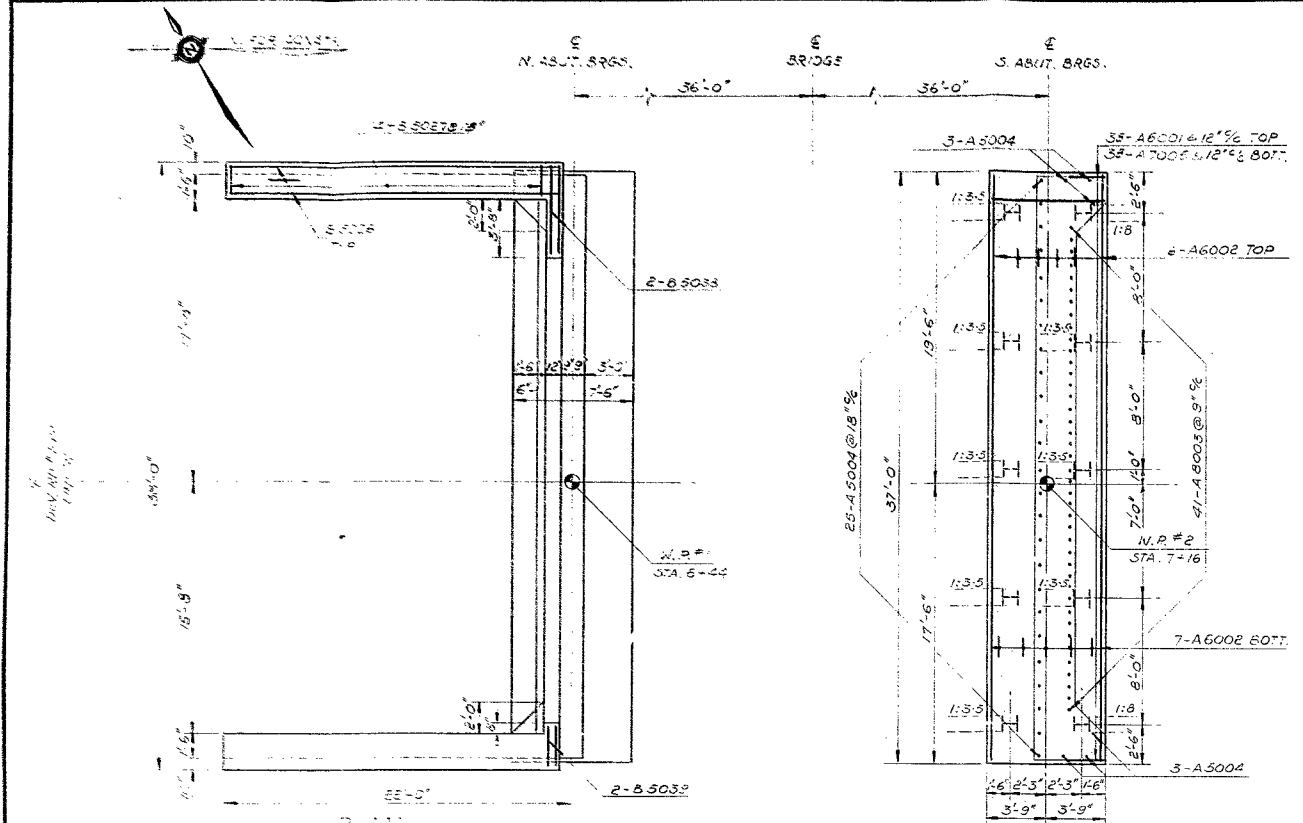


#68-F-80
W.P. #700-68-1
DEV. RD. #1012
MAGNETAWAN
RIVER BRIDGE



NOTE
MINOR ELEVATIONS TO FOLLOW
HORIZONTAL ALIGNMENT

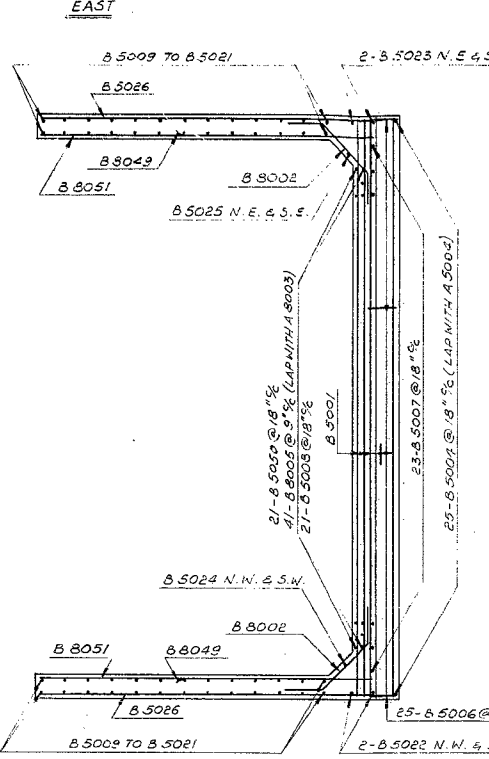
FOR DETAILS OF PARAPET
SEE D-6687-5

FOR DETAILS OF PARAPET
SEE D-6687-5

FOR DETAILS OF PARAPET
SEE D-6687-5

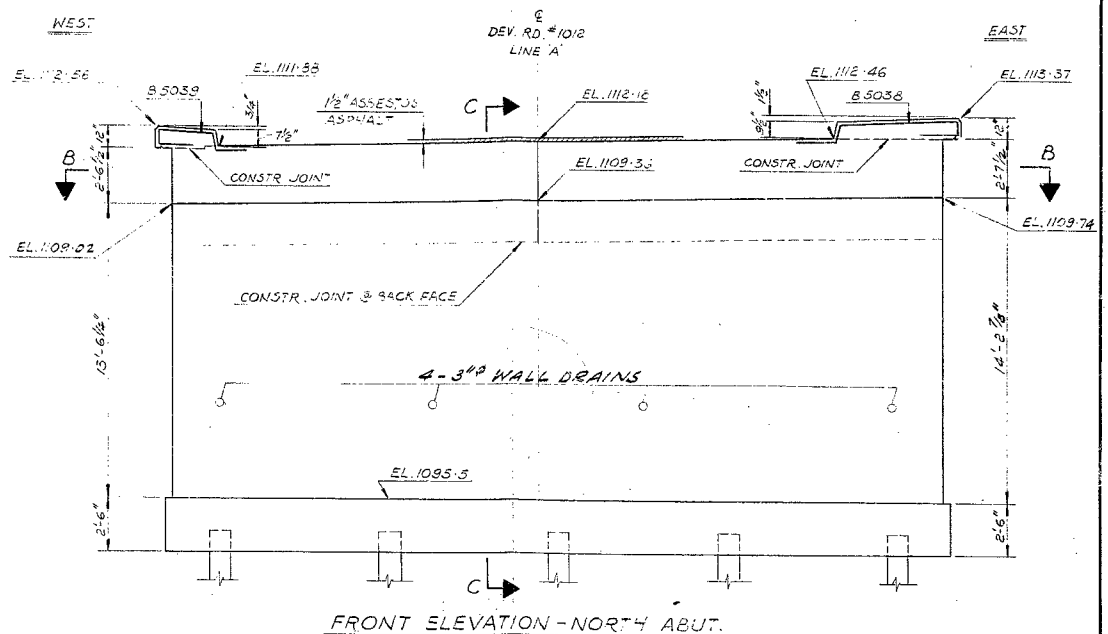
SECTION A-A
NORTH ABUT. SIMILAR
SCALE: 3/16" = 1'-0"

LOCATION	NO. OF PILES	LENGTH
N. ABUT.	10	76'-0"
S. ABUT.	10	76'-0"

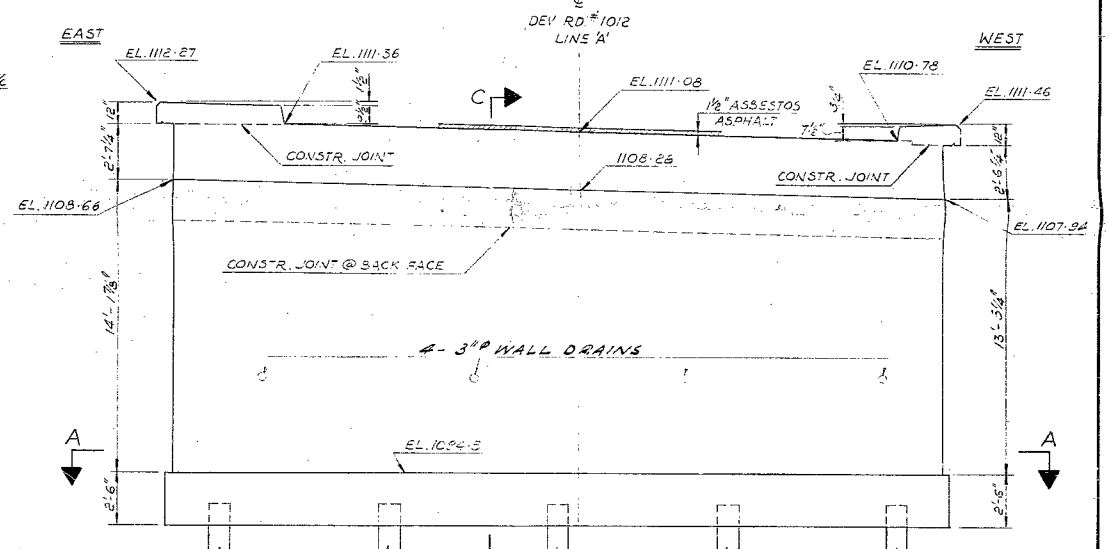


SECTION B-B
SCALE: 3/16" = 1'-0"

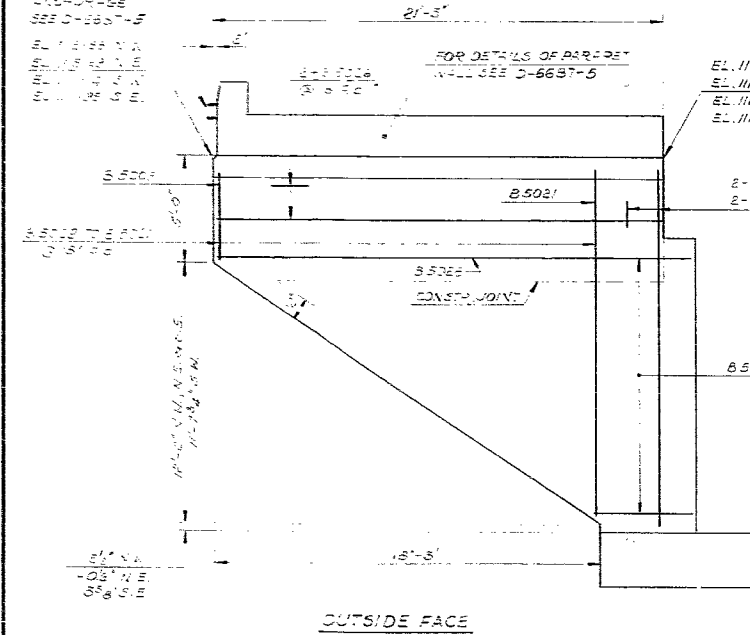
SCALE: 1/4" = 1'-0"
UNLESS NOTED OTHERWISE



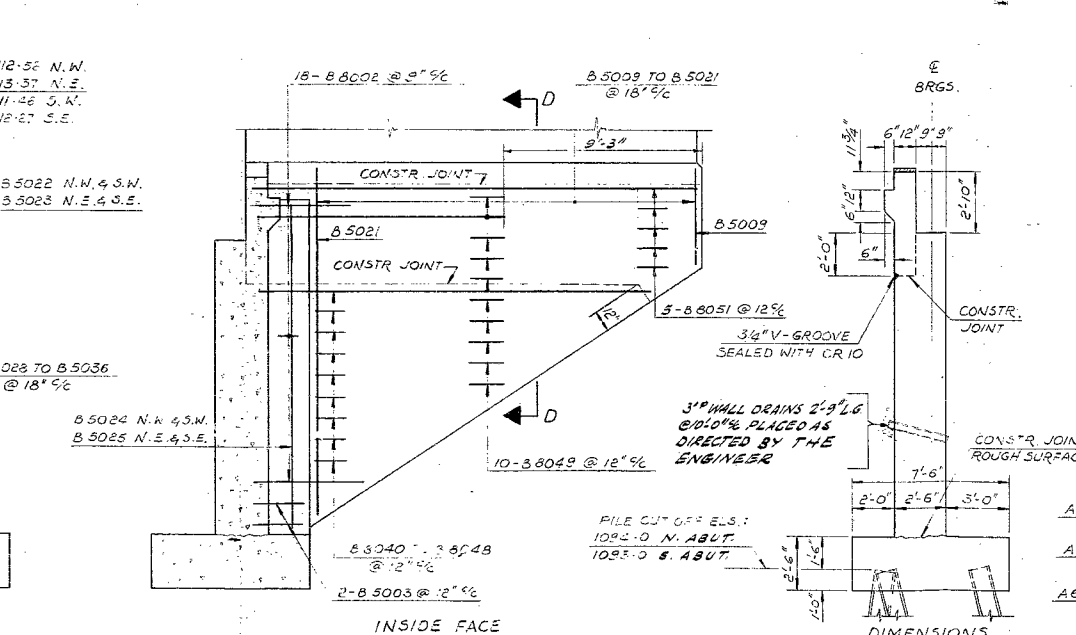
FRONT ELEVATION - NORTH ABUT.



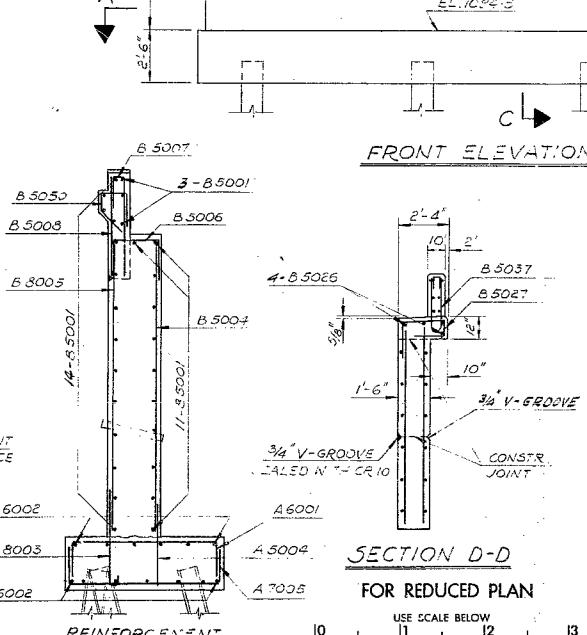
FRONT ELEVATION - SOUTH ABUT.



OUTSIDE FACE



INSIDE FACE



SECTION C-C

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

68-F-80

MAGNETA RIVER BRIDGE
TOWN OF KEARNEY

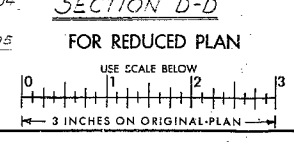
ONTARIO HIGHWAY No. 10/12 DEV. RD. #10/12 DIST. No. 11
OF PARRY SOUND

TWP. PERRY LOT CON.

ABUTMENTS

APPROVED: [Signature] BRIDGE ENGINEER

DESIGN: C.F.F. CHECK: P.O.L. CONTRACT: [Signature]
DRAWING: H.N. CHECK: C.F.F. NO.: [Signature]
DATE: AUG 1969 LOADING: HS 20-44 DRAWING No.: D-6687-3



MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Office,
Admin. Bldg.

FROM: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

ATTENTION Mr. S. McCombie

DATE: March 14, 1969

OUR FILE REF:

IN REPLY TO

MAR 18 1969

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed Crossing at Magnetawan River
And Dev. Road 1012, Line 'A'
Village of Kearney, Township of Perry
District of Parry Sound
District No. 11 (Huntsville)
W.J. 68-F-80 -- W.P. 700-68-1

Attached, we are forwarding to you, our detailed
foundation investigation report on the subsoil conditions
existing at the above structure 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/MdeF
Attach.

cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farlen
H. McArthur
W. S. Aitken
E. R. Saint
J. C. McAllister
B. A. Singh

Foundations Files
Gen. Files

A. G. Sternac
A. G. Sternac
PRINCIPAL FOUNDATION ENGINEER

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 2. DESCRIPTION OF SITE.
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 - 4.3) Fine Sand.
 5. GROUNDWATER CONDITIONS.
 6. DISCUSSION AND RECOMMENDATIONS:
 - 6.1) General.
 - 6.2) Foundations.
 7. MISCELLANEOUS.
-

FOUNDATION INVESTIGATION REPORT
For
Proposed Crossing at Magnetawan River
And Dev. Road #1012, Line 'A'
Village of Kearney, Township of Perry
District of Perry Sound
District No. 11 (Huntsville)
W.J. 68-F-80 -- W.P. 700-68-1

1. INTRODUCTION:

A request to carry out a foundation investigation for the proposed new bridge to carry Dev. Rd. #1012 over the Magnetawan River, was received from Mr. J. C. McAllister, Regional Bridge Location Engineer, in a memo dated October 30, 1968.

An investigation was subsequently carried out by this Section to determine the subsoil conditions existing at the site of the proposed bridge.

This report contains the results of our field and laboratory investigation, together with our recommendations for the foundations of the new structure.

2. DESCRIPTION OF SITE:

The new structure is proposed to be located at the same site as the existing one, but at a slightly greater skew to the centre-line. The site is situated in the middle of the Village of Kearney on Dev. Rd. #1012. The existing bridge is a one-lane 77.0 ft. clear span steel truss bridge. The original bridge was constructed in 1908. It has outlived its useful life and is in very poor condition. The traffic is presently carried over a Bailey bridge which has been placed on top of the existing bridge.

The site is located at the junction of Hassard Lake and the Magnetawan River. The flow of water at this point is from the lake southward to the river. The area in the vicinity of the bridge site, is built up and the topography is flat to undulating.

3. FIELD AND LABORATORY WORK:

The field work at the site consisted of two sampled boreholes and one dynamic cone penetration test. In addition, cone penetration tests were carried out commencing from the bottom of each sampled borehole. All boreholes were advanced using conventional diamond drilling equipment adapted for soil sampling purposes. A driving energy of 350 ft.-lbs. per blow was used for the dynamic cone penetration tests.

Disturbed samples were obtained using a 2-inch O.D. split-spoon sampler driven according to the specifications of the Standard Penetration Test. Where it was not possible to recover the samples with a split-spoon sampler, they were recovered by a 2-inch O.D. slotted tube sampler.

Samples were visually examined in the field and subsequently in the laboratory. Grain-size analysis tests were carried out on selected samples:

The results of field and laboratory tests are summarized on the Record of Borehole sheets, which are contained in the Appendix to the report.

The locations and the elevations of boreholes are given on Drawing No. 68-F-30A, which is also contained in the Appendix to this report.

The borehole locations and elevations were surveyed by the Huntsville District Office of the D.H.O.

4. SUBSOIL CONDITIONS:

4.1) General:

The subsoil consists of a deposit of fine sand, some silt and traces of clay. Granular fill has been used behind the abutments.

The boundaries between the two deposits are shown on the attached Record of Borehole sheets. The estimated stratigraphical profiles shown on Drawing No. 68-F-30A, are based upon this information.

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.1) General: (cont'd.) ...

From ground level downwards, the different soil deposits are described as follows:

4.2) Fill Material:

This material was encountered only in Borehole 1, which was put down behind the west abutment. The material consists of gravel and sand with occasional small boulders. The depth of the fill in the borehole is 13 ft. The 'N' values indicate a compact density.

4.3) Fine Sand:

This is the only subsoil deposit found at this location in both boreholes, which were terminated in this stratum. The material essentially consists of fine sand with small amounts of silt and traces of clay. The grain-size analyses carried out, show the following distribution and are plotted on Fig. 1.

Sand	65	-	92%
Silt and Clay	8	-	35%

The 'N' values range from 2 blows/ft. to 14 blows/ft. - except for the last sample in each borehole. It is inferred that most of the 'N' values recorded are probably indicative of some unavoidable loosening of the sand below the casing due to drilling operations. Based on the above, and on the dynamic cone penetration tests, the relative density of the deposit is considered to be loose to compact.

5. GROUNDWATER CONDITIONS:

The water level in the river, at the time of investigation, was at elevation 1099.7. Groundwater level in borehole 2 was found to be at elevation 1100.3. It may be assumed that the groundwater level in the vicinity of the river, is equal to or slightly higher than the prevailing river water level.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

It is proposed to construct a new, single-span bridge to carry Dev. Road #1012 over the Magnetawan River. The new structure will be at the same site, but will have a greater skew. The new grade will be the same as the existing one.

6.2) Foundations:

Due to the generally loose density of the upper zone of the sand deposit at the site, which extends from the river bed to as much as 20 ft. in depth, it is not considered a suitable bearing stratum for the support of the bridge abutments on spread footings. For this reason, and because of the susceptibility of the sand to scour, it is recommended that a piled foundation be provided at this site.

From our experience at other sites having similar subsoil conditions - particularly Hwy. 35 and Beech River - it has been found that, for such strata, steel H-piles are the most suitable type of pile. Therefore, it is recommended that the proposed abutments be supported by means of 12 BP @ 53 H-piles. Based on the experience at the above mentioned site, it is considered that, for a pile 75 ft. long, an allowable working load of 70 tons per pile would be realized. This figure may be taken for preliminary design, but it is recommended that at least one pile loading test be carried out on a representative pile to determine the allowable load prior to final design.

It is anticipated that the settlement of the pile foundation would be in the order of 1 inch.

For frost protection purposes, foundations for the abutments should be protected by at least 6 feet of earth cover.

No stability problems are anticipated for 2:1 side slopes.

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Foundations: (cont'd.) ...

No dewatering problems are foreseen if the cut-off level for the piles is above the river water level, but if the cut-off level for the piles is below the water level, a dewatering scheme will be necessary.

Reference should be made to the Hydrology Section for the recommendations regarding the provision of rip-rap for scour protection.

7. MISCELLANEOUS:

The field work for this project was carried out during the period November 20 to 23, 1968, under the supervision of Mr. A. Prakash, Project Foundation Engineer, who also prepared this report.

The equipment used was owned and operated by Canadian Longyear Co. Ltd.

This report was reviewed by Mr. K. G. Selby, Supervising Foundation Engineer.

March 1969.

APPENDIX 1

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 68-F-80

LOCATION Sta. 6 + 27 o/s 9' Lt.

ORIGINATED BY AP

W.P. 700-68-1

BORING DATE Nov. 20 - 22, 1968

COMPILED BY _____ AP

DATUM Geodetic

BOREHOLE TYPE Washboring, NX & BX Casing & Cone

CHECKED BY AK

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

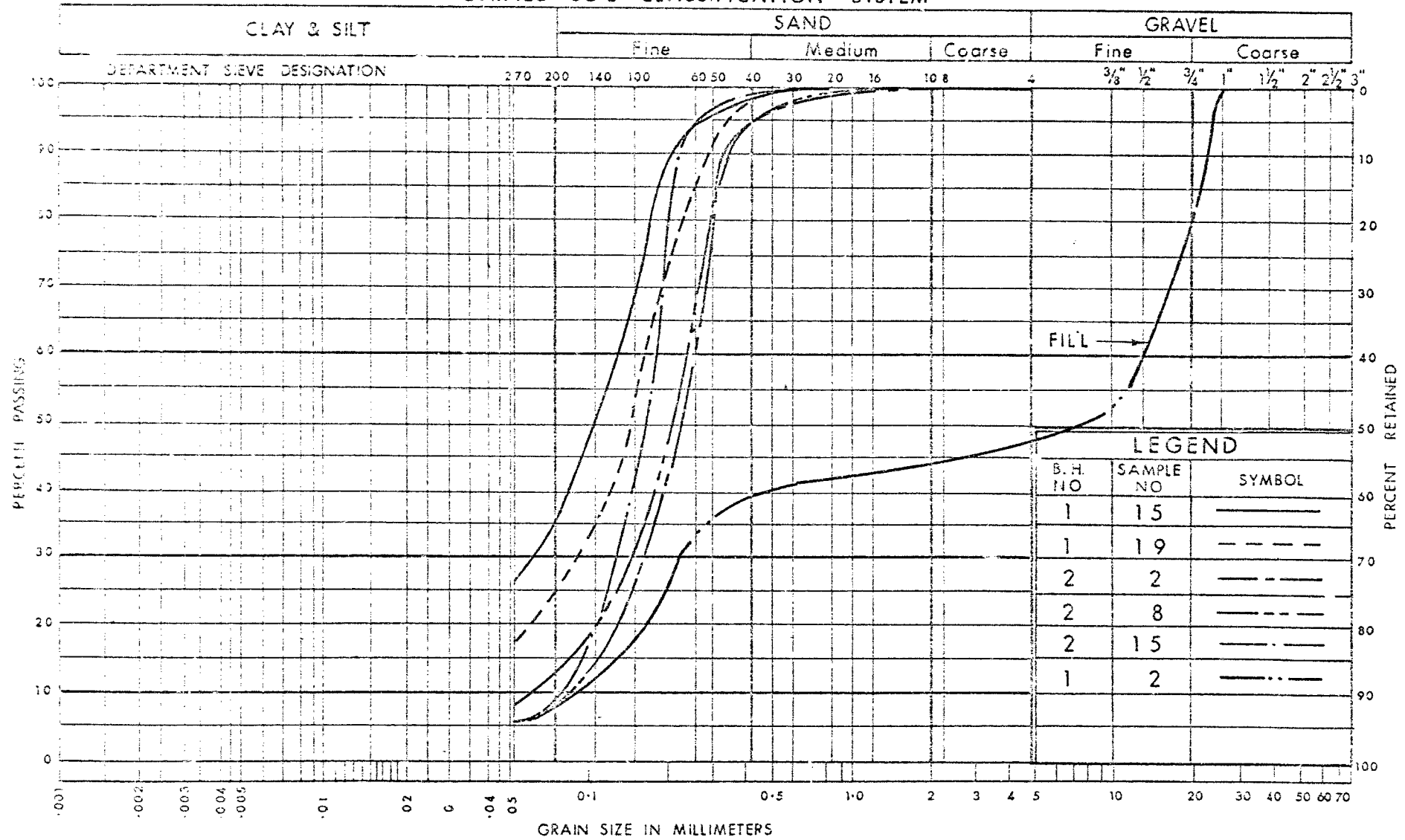
FOUNDATION SECTION

JOB 68-F-80 LOCATION Sta. 7 + 43.5 O/S 24' Rt. ORIGINATED BY AP
 W.P. 700-68-1 BORING DATE Nov. 25 - 27, 1968 COMPILED BY AP
 DATUM Geodetic BOREHOLE TYPE Washboring, NX & BX Casing & Cone CHECKED BY AK

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BLOWS / FOOT 20 40 60 80 100				SHEAR STRENGTH P.S.F.		WATER CONTENT % w_p — w — w_L		
100.3	Ground Level														
0.0															
			1	SS	6										
			2	SS	4	1090									0 92 (8)
			3	SS	3										
			4	SS	6	1080									
			5	SS	7										
			6	SS	9	1070									
	Fine sand, traces		7	SS	8										
	of silt and clay		8	SS	7	1060									0 88 (12)
	Loose to compact.		9	SS	11										
			10	SS	10	1050									
			11	WS											
			12	SS	9										
						1040									
			13	SS	3										
						1030									
			14	SS	12										0 92 (8)
						1020									
			15	SS	19										
1014.8						1010									
85.5	End of Borehole														
						1000									
997.5															
102.8	End of Cone Test														

200/10"

UNIFIED SOIL CLASSIFICATION SYSTEM



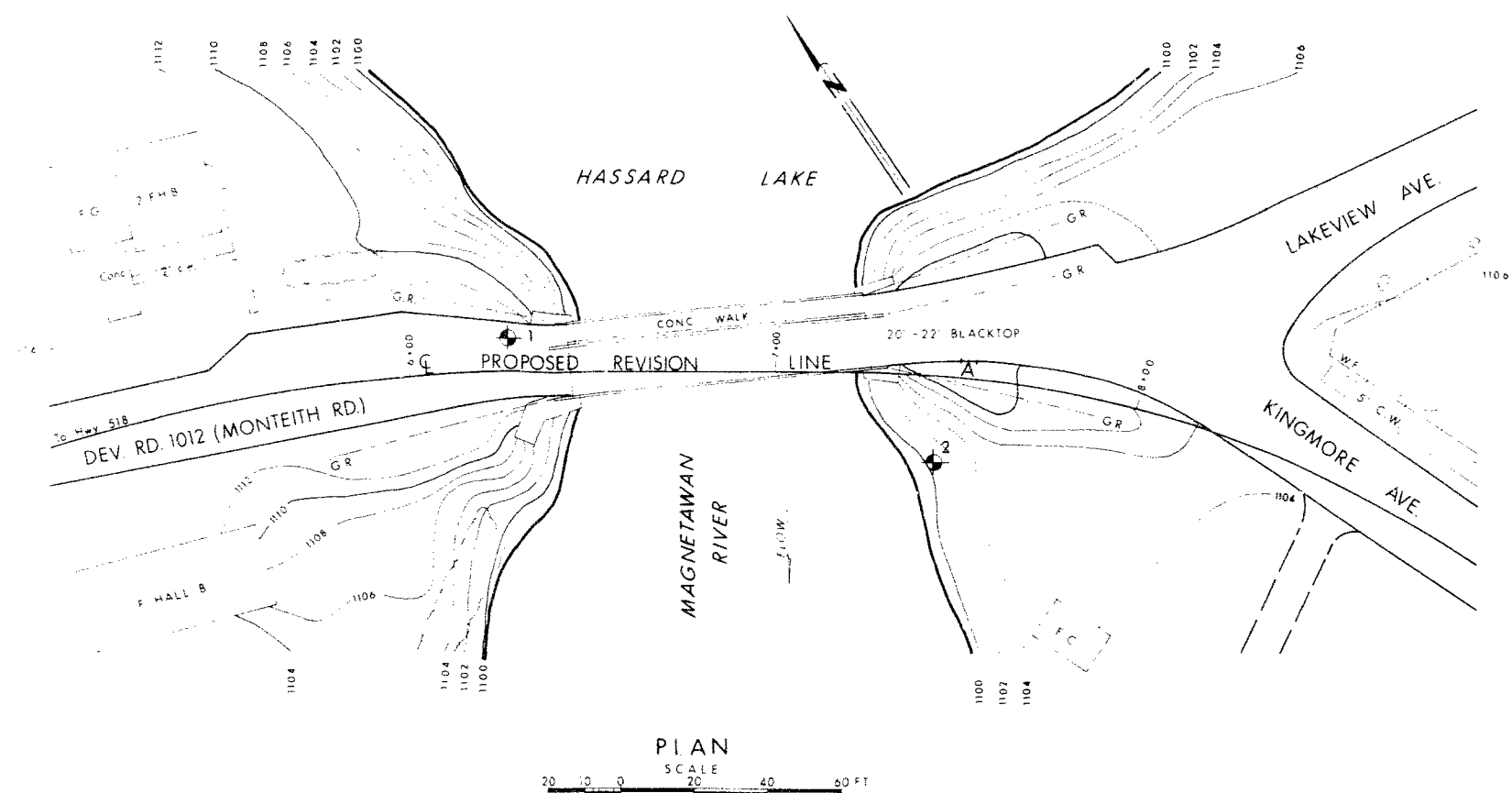
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION FINE SAND

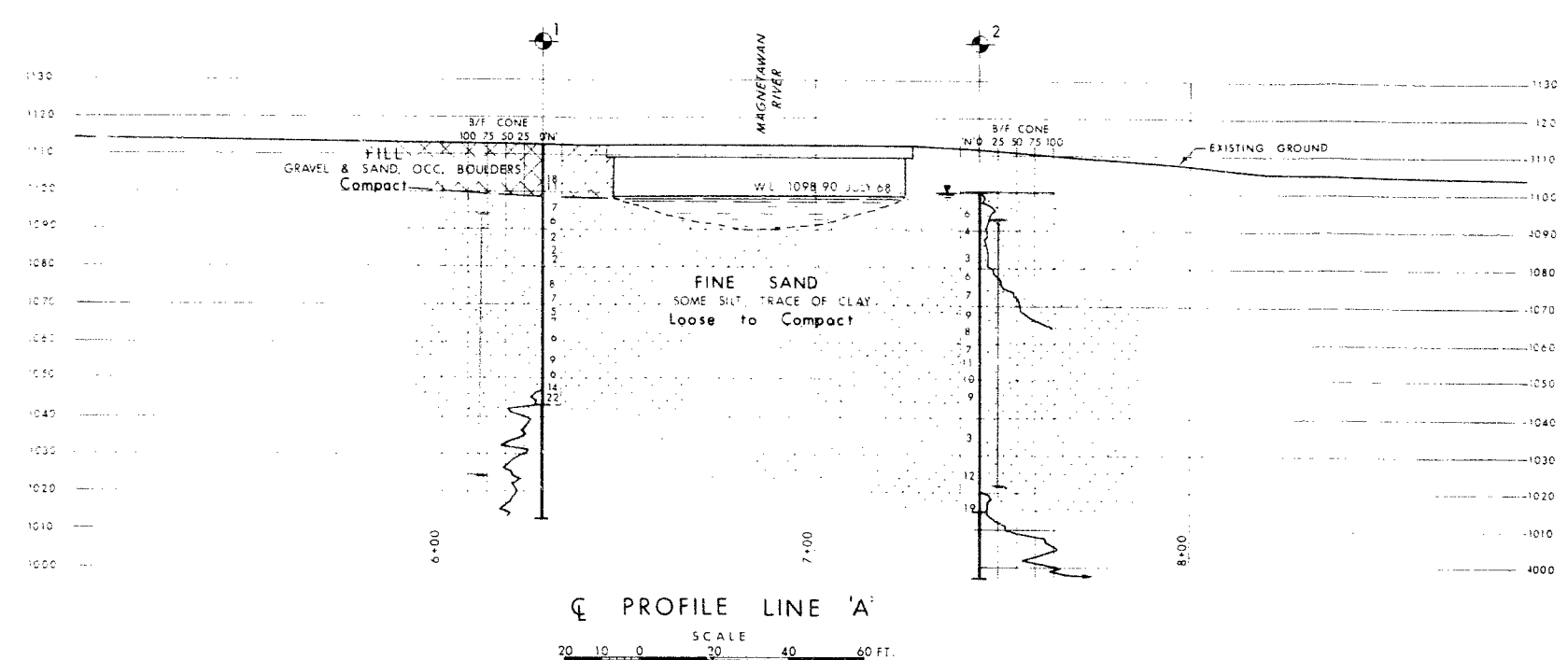
W.P. No. 700 - 68 - 1

JOB No. 68 - F - 80

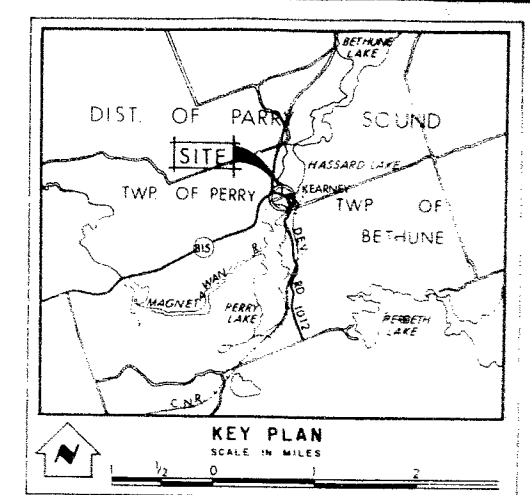
FIG. No. 1



PLAN
SCALE
0 20 40 60 FT.



PROFILE LINE 'A'
SCALE
0 20 40 60 FT.



KEY PLAN
SCALE IN MILES

LEGEND

- Bore Hole
- ⊕ Cone Penetration Hole
- ⊕ Bore & Cone Penetration Hole
- Water Levels established at time of field investigation, NOV. 1968

NO.	ELEVATION	STATION	OFFSET
1	1112.6	6+27	9' LT
2	1100.3	7+43.5	14' RT

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

MAGNETAWAN RIVER

DEV. RD. 1012 (MONTEITH RD.) LINE A DIST. NO. 13
DIST. OF PARRY SOUND TOWN OF KEARNEY
TWP. PERRY LOT CON.

BORE HOLE LOCATIONS & SOIL STRATA			
SUBWD. A.P.	CHECKED <input checked="" type="checkbox"/>	WP NO. 700-68-1	M.B.T. DRAWING NO.
DRAWN A.N.	CHECKED <input checked="" type="checkbox"/>	JOB NO. 68-F-80	68-F-80A
DATE JAN. 4, 1969	SITE NO.	BRIDGE DRAWING NO.	
APPROVED <i>[Signature]</i>	CONT. NO.		

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

SS	SPLIT SPOON	TW	THINWALL OPEN
WS	WASHED SAMPLE	TP	THINWALL PISTON
SB	SCRAPER BUCKET SAMPLE	OS	OESTERBERG SAMPLE
AS	AUGER SAMPLE	FS	FOIL SAMPLE
CS	CHUNK SAMPLE	RC	ROCK CORE
ST	SLOTTED TUBE SAMPLE		
	PH	SAMPLE ADVANCED HYDRAULICALLY	
	PM	SAMPLE ADVANCED MANUALLY	

SOIL TESTS

Qu	UNCONFINED COMPRESSION	LV	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	FV	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_r	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_0	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

— 5 —

Abstract

BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 11 CONTRACT NO. 70-85 STRUCTURE MAGNETAWAN RIVER BRIDGE
CONTRACTOR BERMINGHAM CORP. LTD. DESIGN LOAD OF PILE 60 TON
HAMMER DETAILS: TYPE BERMINGHAMMER WEIGHT 6,800 LB HEIGHT OF FALL OR ENERGY 25.000 FT.
TYPE OF ANVIL OR CAP H. PILE WEIGHT OF ANVIL OR CAP 1,100 LBS
PILE DETAILS 12 BP 53 H. PILES BATTER 1:8
PILE NO. 1 LOCATION N/W CORNER OF NORTH ABUTMENT DATE DRIVEN MAY 19/71

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.
35'	1		26	4		51	25		76		
	2		27	4		52	23		77		
	3		28	4		53	28		78		
	4		29	5		54	30		79		
	5		30	5		55	36		80		
	6		31	5		56	41		81		
	7		32	6		57	41		82		
	8		33	6		58	39		83		
	9		34	6		59	41		84		
	10		35	7		60	50		85		
	11		38'	7		61	58		86		
	12		76'	12		62	68		87		
	13		38	12		63	86		88		
	14		39	15		64	108		89		
	15		40	15		65	121		90		
38'	16	1	41	18	76'	66	177		91		
38'	17	1	42	16		67	203		92		
	18	1	43	15		68	236		93		
	19	2	44	16		69	285		94		
	20	2	45	19		70	340		95		
	21	2	46	19		71			96		
	22	2	47	20		72			97		
	23	3	48	22		73			98		
	24	3	49	23		74			99		
	25	3	50	24		75			100		

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	15	16	17	19	23	28
MEASURED REBOUND IN INCHES	.5	.5	.5	.6	.75	.75
FINAL LENGTH OF PILE	70.1'			FINAL CUT OFF ELEVATION		
				1094.0		

REPORT TO BE SENT TO: - PRINCIPAL FOUNDATION ENGINEER
MATERIALS & RESEARCH DIVISION
DEPARTMENT OF HIGHWAYS
PARLIAMENT BUILDINGS
TORONTO, ONTARIO

SIGNED Harvey J. Rickward
NAME (PRINT) HARVEY J. RICKWARD
DATE MAY 25/71

ATTACH SKETCH OF PILE NUMBERING SYSTEM

BRIDGE CONSTRUCTION - PILE DRIVING RECORD

DISTRICT NO. 11 CONTRACT NO. 70-AS STRUCTURE MAGNETAWAN RIVER BRIDGE
CONTRACTOR BERMINGHAM CORP. LTD. DESIGN LOAD OF PILE 60 Ton
HAMMER DETAILS: TYPE DELMAG D22 WEIGHT 10,033 HEIGHT OF FALL OR ENERGY 39,700
TYPE OF ANVIL OR CAP H. PILE WEIGHT OF ANVIL OR CAP 1,323 LBS
PILE DETAILS 12 BP 53 H PILE BATTER 1:3.5
PILE NO. 15 LOCATION N/E CORNER OF SOUTH ABUTMENT DATE DRIVEN MAY 26/71

TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.	TOTAL LENGTH BEING DRIVEN	LENGTH IN GROUND	PENETRATION BLOWS/FT.
38	1		38	26	5	76	51	17		76	
	2			27	6		52	17		77	
	3			28	6		53	17		78	
	4			29	7		54	18		79	
	5			30	9		55	18		80	
	6			31	9		56	22		81	
	7		38	32	10		57	22		82	
	8		38	33	10		58	24		83	
	9		38	34	10		59	27		84	
	10		76	35	11		60	27		85	
	11		76	36	11		61	27		86	
	12		76	37	11		62	28		87	
	13		76	38	11		63	32		88	
38	14	1	76	39	12		64	31		89	
38	15	1		40	13		65	35		90	
38	16	2		41	13		66	36		91	
	17	2		42	13		67	39		92	
	18	2		43	12		68	39		93	
	19	2		44	13		69	46		94	
	20	3		45	13		70	53		95	
	21	3		46	14	76	71	62		96	
	22	3		47	15		72			97	
	23	3		48	15		73			98	
	24	4		49	15		74			99	
	25	4		50	16		75			100	

DETAILS FOR FINAL SIX INCHES OF PENETRATION	1	2	3	4	5	6
BLOWS PER INCH	4	5	5	5	6	7
MEASURED REBOUND IN INCHES	.5	.5	.5	.5	.75	.75
FINAL LENGTH OF PILE	71.4			FINAL CUT OFF ELEVATION 1093.0		

REPORT TO BE SENT TO: - PRINCIPAL FOUNDATION ENGINEER
MATERIALS & RESEARCH DIVISION
DEPARTMENT OF HIGHWAYS
PARLIAMENT BUILDINGS
TORONTO, ONTARIO

SIGNED Harvey J. Richards
NAME (PRINT) HARVEY J. RICHARDS
DATE MAY 27/71

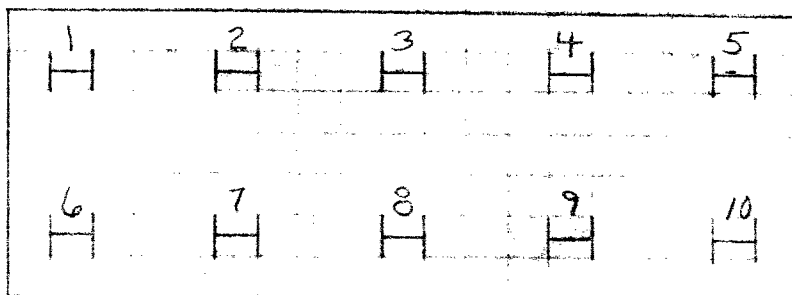
ATTACH SKETCH OF PILE NUMBERING SYSTEM

SUPER IMPOSED DOCUMENT MAY
APPEAR AS MULTI-FEED ON FILM.

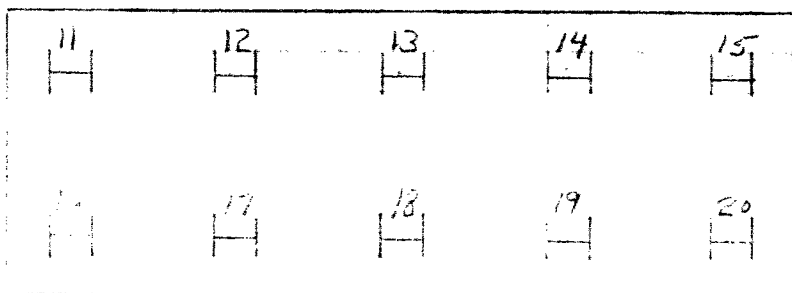
NORTH FOOTING
"H" PILES



NORTH LOS CONST.



South Footing
"H" Piles



MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Building.

FROM: C. S. Grebski,
Bridge Office.

ATTENTION:

DATE: August 28, 1969.

OUR FILE REF.

IN REPLY TO

SUBJECT: Magnetewan River Bridge
at Town of Kearney,
W.P. 700-68-1, Site #44-187,
Hwy. No. Dev. Rd. #1012, District #11.

68-F-80

Attached herewith we are submitting the final bridge
drawings which show the foundation design for this
structure.

Kindly give us your comments at your earliest
convenience.

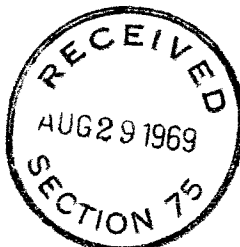
CSG/kr

Attache.

c.c. Foundation Section.

C. S. Grebski,
Bridge Design Engineer.

for comments. Pile length OK, load 60% pile. Scale 1/2" = 1',
found. 1/2" = 1' 11" 5/2/69



JK

November 27th, 1969.

GEOTECHNICAL INVESTIGATION

MAGNETAWAN RIVER BRIDGE

ONTARIO DEPARTMENT OF HIGHWAYS

W.P. 700-68-01

SUBMITTED BY

Professional Services Division

JARROCK HERSEY INTERNATIONAL LIMITED



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230-515

1.0 - INTRODUCTION

The Professional Services Division of Warnock Hersey International Limited was authorized by Mr.A.Rutka, Ontario Department of Highways to carry out a Geotechnical study of the subsoils at the site of the proposed Magnetawan River Bridge.

The site is located on Montielth Road, Line "A", Town of Kearney, Township of Perry as shown on the enclosed site plan.

The purpose of the investigation was to evaluate the ultimate bearing capacity of round steel and H-piles using the in situ pressuremetric test method.

Therefore the anticipated behaviour of the piles are presented in graphical form to facilitate the choice of pile type in function of load and depth of embedment.



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All results calculated in this report are a direct function of pressurometric values as obtained from the tests and are presented in the form of Limit Pressure and Standard Pressurometric Modulus of Deformation. No attempt has been made to break down the parameters into the normal components used in standard soil mechanics terminology (ϕ K_0 etc).

The field work was carried out as a joint venture by Warnock Hensley International Limited and Geoprobe (Quebec) Limited.

One Borehole No. 3 was drilled to a depth of 81'6" for the purpose of Geoprobe pressuremeter testing.



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2.0 - FIELD TESTING PROGRAMME

2.1 - Soil Borings

A sketch showing the locations of the boreholes may be found in the Appendix of this report.

The borehole was advanced according to standard wash boring procedures using a Boyles Brothers screw feed type drill. Standard EX casing was advanced in 5 foot lengths with a 350 lb. hammer and then washing inside the casing followed. The hole was kept full of Bentonite at all times to prevent caving.

Some Standard Penetration tests were taken and split spoon samples recovered to facilitate description of soil stratigraphy.

The surface elevation of the borehole was taken with respect to a benchmark as shown on the enclosed plan.

2.2 - Geoprobe Pressuremetric Tests

The Geoprobe test is an in situ pressuremetric test that is performed inside a borehole at various elevations.



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After the probe is set at a test elevation, the expandable rubber pressure cell is inflated with gas pressure to a given value and held for a sufficient length of time, such that the change in diameter versus time is very small (at pressures below the creep pressure). In stiff to hard soils, it is usually necessary to hold the test pressure for only one minute before increasing pressure to a higher value. By performing the tests at various pressures, a stress-deformation curve can be plotted and a number of soil properties may be derived, such as the lateral earth pressure at rest, the modulus of deformation, the creep pressure, the limit pressure and the shear strength of the soil. Normally, the pressuremeter test is performed at a number of elevations in the borehole so that a complete profile of the aforementioned properties may be determined.

The modulus of deformation of the soil is determined mathematically as a function of the slope of the stress-deformation curve.




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The limit pressure, P_L , is the pressure at which complete failure of the soil occurs if the tests are carried to this point. This is the pressure at which an infinite expansion of the borehole takes place.

The creep pressure P_f , is limit of pseudo-elastic zone where the soil ceases to behave as a nearly elastic material and plastic deformations take place.

Geoprobe tests were carried out in Borehole 3 and representative test curve samples are included in Appendix C of this report.


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3.0 - SOIL CONDITIONS

In the area of the borehole, there was 6 inches of organic topsoil over 5 feet of fill material. Under the fill was various stratified layers of fine sands and with silt and gravel.

The N-values were in the range of 4 - 5 blows/foot to a depth of 50 feet below grade. Below this depth, the density of the material increased.

The water level was recorded at 6.7 feet below grade.

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4.0 - GEOTECHNICAL DISCUSSION

4.1 - General

Figure 1 in Appendix C illustrates the variation in Limit Pressure with depth. To a depth of 30 feet the P_L value varied between 4.5 and 7.3 tsf where it decreased to a low value of 2.3 tsf. At 42 feet below grade, the P_L increased to an average value of 7.0 tsf and extended at this strength to 65 feet. From 65 feet to 80 feet the P_L increased at a constant rate to 16.0 tsf.

The standard pressuremetric modulus of deformation, E , followed a similar trend from 60 tsf near the surface to 23 tsf at 35 feet.

The ratio $E/P_L = 8 - 10$ indicates that the material is normally consolidated.

4.2 - Bearing Capacity of Piles

Most of the correlation data available is based on concrete piles and therefore in the calculations for steel H-piles and round steel piles, some assumptions have been made.

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In all cases the contribution from the first five feet from the surface has been neglected in the calculation of friction.

The ultimate bearing capacity (q_{ult}) referred to in this report corresponds to a deflection of approximately 1 cm and corresponds further to a load exceeding the pseudo-elastic zone by 33%.

4.2.1 - Round Steel Piles

The end bearing value is taken as equivalent to that of a concrete pile and the formulae used is as follows: (Menard, Sept. 1965).

$$q_{ult} = P_o + K (P_L - P_o) \quad (1)$$

Where K is a coefficient which varies with depth of embedment and nature of the soil. In the case of the sands at this site, the value of K varied between 3.6 and 4.0.

The friction factor between soil and steel is assumed to be 80 percent of the soil/concrete value (S_o).

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Therefore the skin friction values for steel ($\frac{S_o}{0.8}$) were derived as follows: (Menard, Sept. 1965)

when

$$P_L < 5.2 \quad \frac{S_o}{0.8} = \frac{P_L - P_o}{8} \quad (2)$$

$$5.2 < P_L < 9 \quad \frac{S_o}{0.8} = \frac{P_L}{20} + 0.28 \quad (3)$$

$$9 < P_L \quad \frac{S_o}{0.8} = 0.8 \quad (4)$$

except for area within 3 diameters of the pile tip
where:


$$P_L < 5.2 \quad \frac{S_o}{0.8} = \frac{P_L - P_o}{8} \quad (5)$$

$$5.2 < P_L < 9 \quad \frac{S_o}{0.8} = \frac{P_L}{7} - 0.10 \quad (6)$$

$$9 < P_L \quad \frac{S_o}{0.8} = 1.2 \quad (7)$$

4.2.4 - Steel H-Piles

For steel H-piles the skin friction value along the pile is calculated by the same formulae as with the round steel piles (Equations No. 1-4) with the exception of the area within ten feet of the tip of the pile.

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Within 10 feet of the pile tip the formulae becomes

$$\frac{S_o}{0.8} = \frac{P_L - P_o}{8} \quad (8)$$

It is assumed that over a 10 to 20 foot length from the pile tip, the sand will form a plug between the flanges of the H-pile and an end bearing capacity based on a cross-section equivalent to the maximum dimensions of the pile will be developed and carried, within limits, by the sand plug.

4.2.3 - Timber Piles

It is assumed that timber piles would behave approximately the same as concrete piles.

4.3 - Settlement

The anticipated settlements have been calculated from formulae proposed by M.Cassan based on a study by H.Cambefort (Sols-Soils No. 18-19).

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4.3.1 - Round Piles

The following is the formulae used in calculating total settlement of round piles.

$$\text{Settlement (w)} = P \frac{4}{\pi D} \frac{1 + \frac{R}{E_s} \frac{h}{D}}{R + 4Bh} \quad (9)$$

Where R and B are coefficients based on the modulus of deformation (E) and in the case of driven piles:

P = Total load on pile head in tons.

R = 13.5E Tons/m²

B = 1.25E Tons/m³


D = pile diameter or width of base in meters.

h = Useful height of pile in meters.

E_s = Modulus of pile material.

E = harmonic mean of standard pressuremetric modulus.

Note: In this formula the unit "Tons" is "Metric" (1000 kilograms). Whereas on the graphs, conversion has been made to Short English Tons (2000 pounds).

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4.3.2 - Square Pile

$$W = \frac{p}{d} \left[1 + \frac{R}{E_s} \frac{h}{d} \right] \quad (10)$$


Where d = width of pile

4.4 - RESULTS & CONCLUSIONS

Using the above formulae and assumptions, the results are presented in the form of five graphs on the following pages. Figure 1 is the ultimate bearing capacity of the different types of piles in function of depth from the existing ground surface.

It can be seen from the figure that the rate of increase of the ultimate bearing capacity, (Q_{ult}) is nearly constant between 50 and 60 feet at which depth a marked increase in the rate becomes evident.

Figures 2A-D illustrate the calculated settlement of the various types of piles at two different depths (30 ft. & 60 ft.).

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Total rupture values have been indicated on the curves, however it is preferable to base the recommendations for working loads of the piles on q_{ult} as defined previously, or better still, on the pseudo-elastic zone which can be determined with greater accuracy.

The final choice of pile type will obviously be one of economics.



ONTARIO DEPARTMENT OF HIGHWAYS

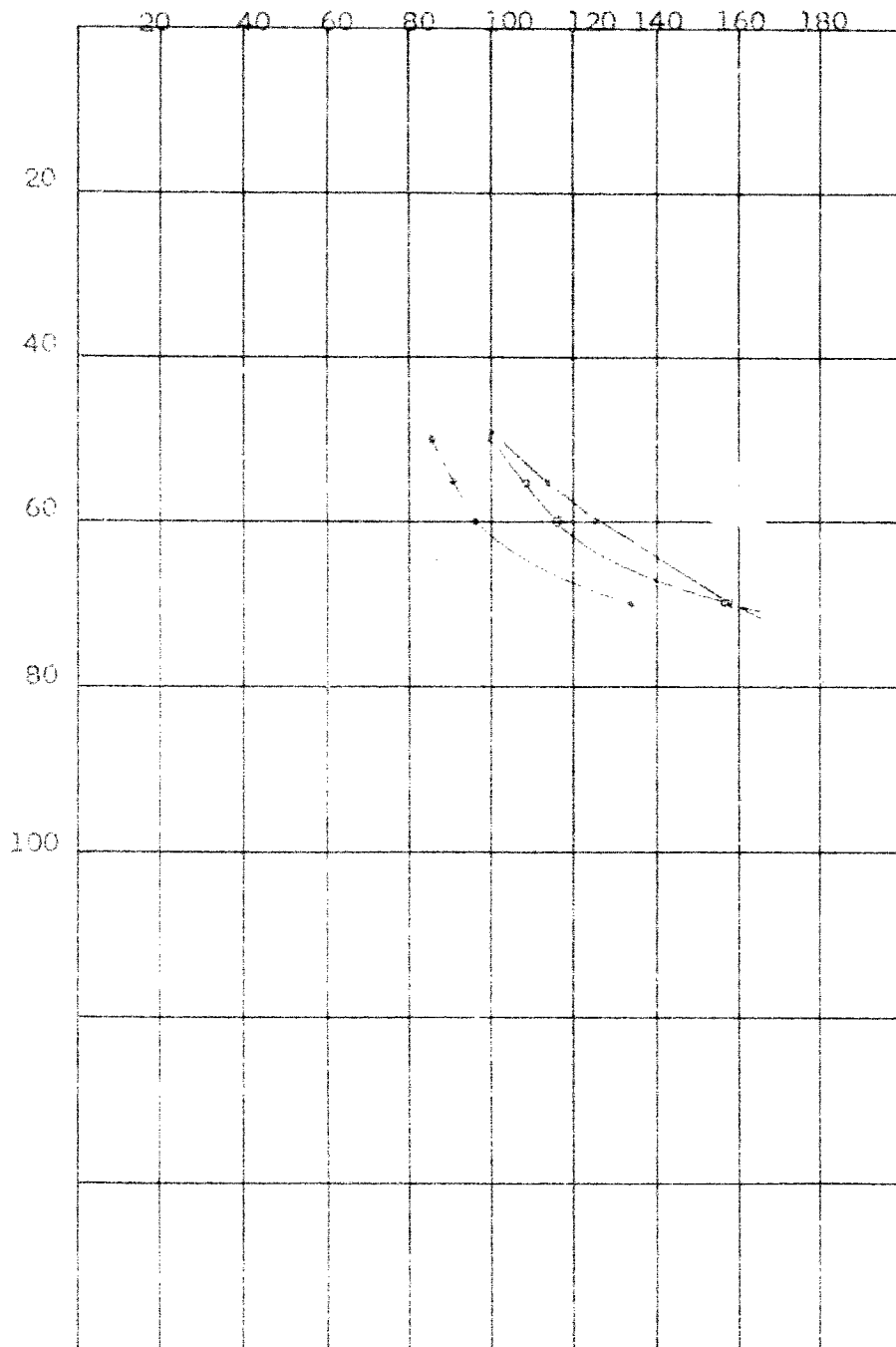
MAGNETAWAN RIVER BRIDGE

Fig 1

230-515

ULTIMATE BEARING CAPACITY (TONS)

DEPTH BELOW EXISTING GROUND LEVEL

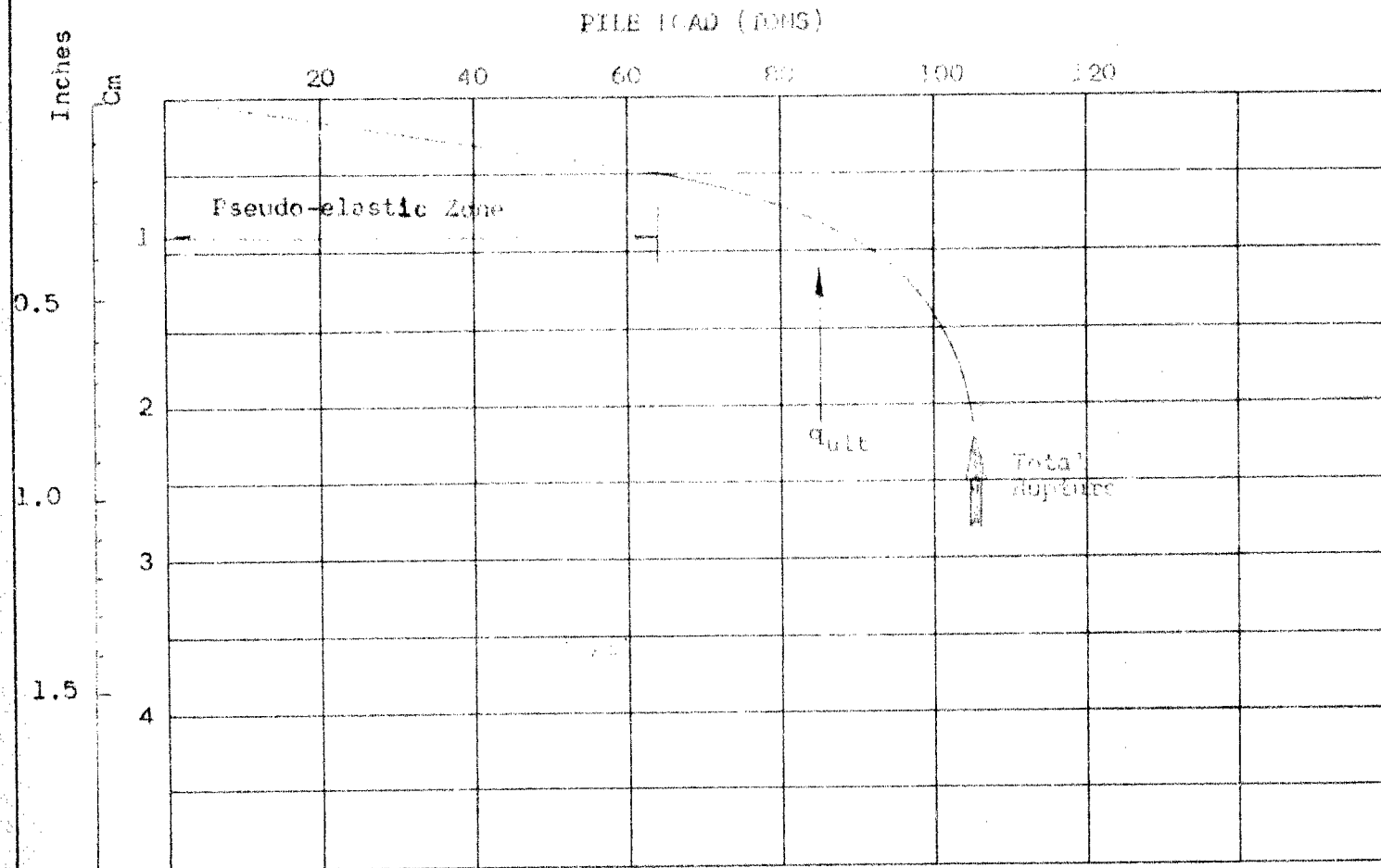


LEGEND

• Round Steel
Pile $\phi=12.75"$

o Round Concrete
Pile $\phi=12.75"$

x Steel H-Pile
12 BP53



Note: Where q_{ult} is load at end of
pseudo-elastic zone plus 33%.

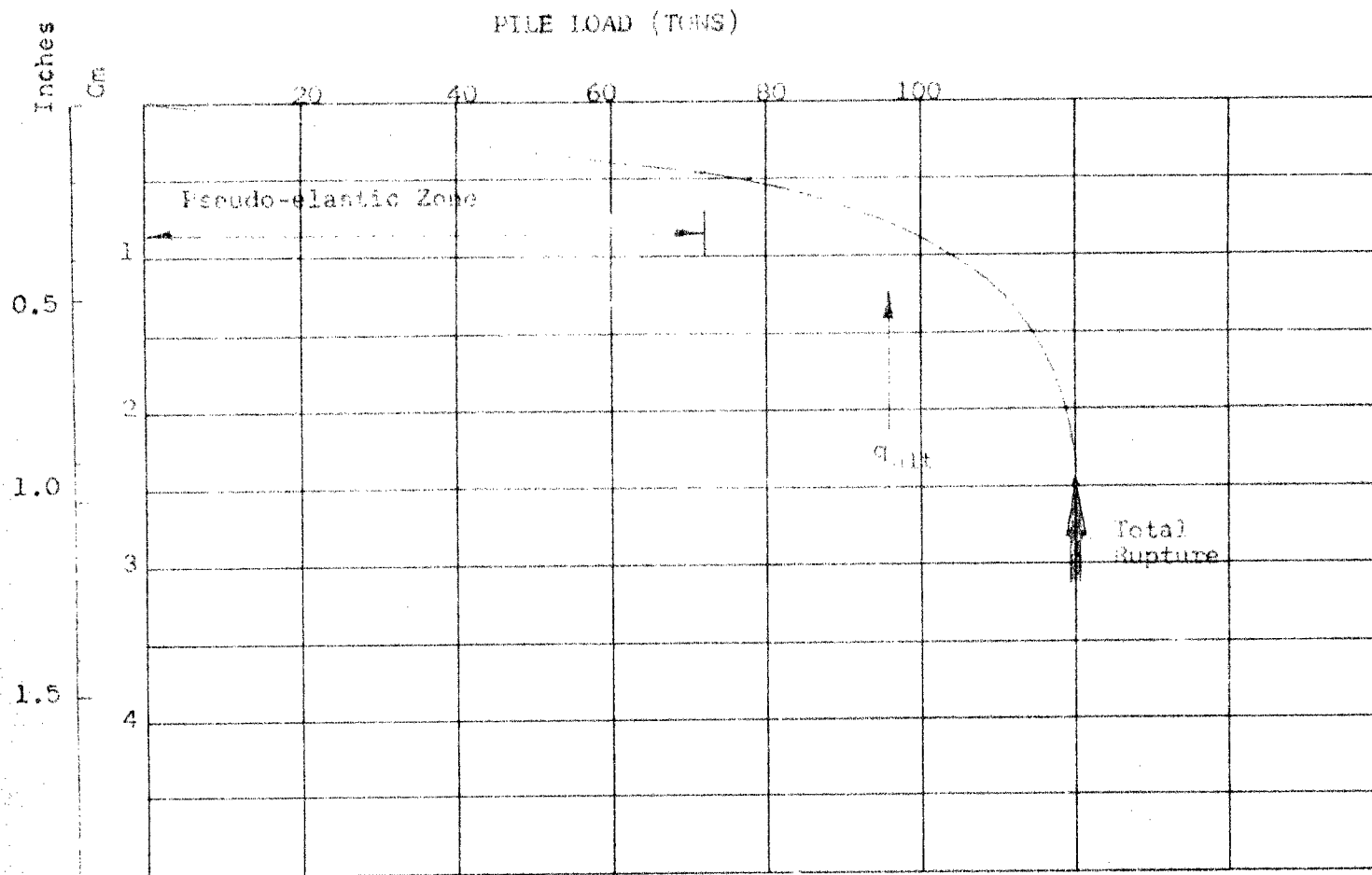
Round Steel Pile at 50'

Settlement



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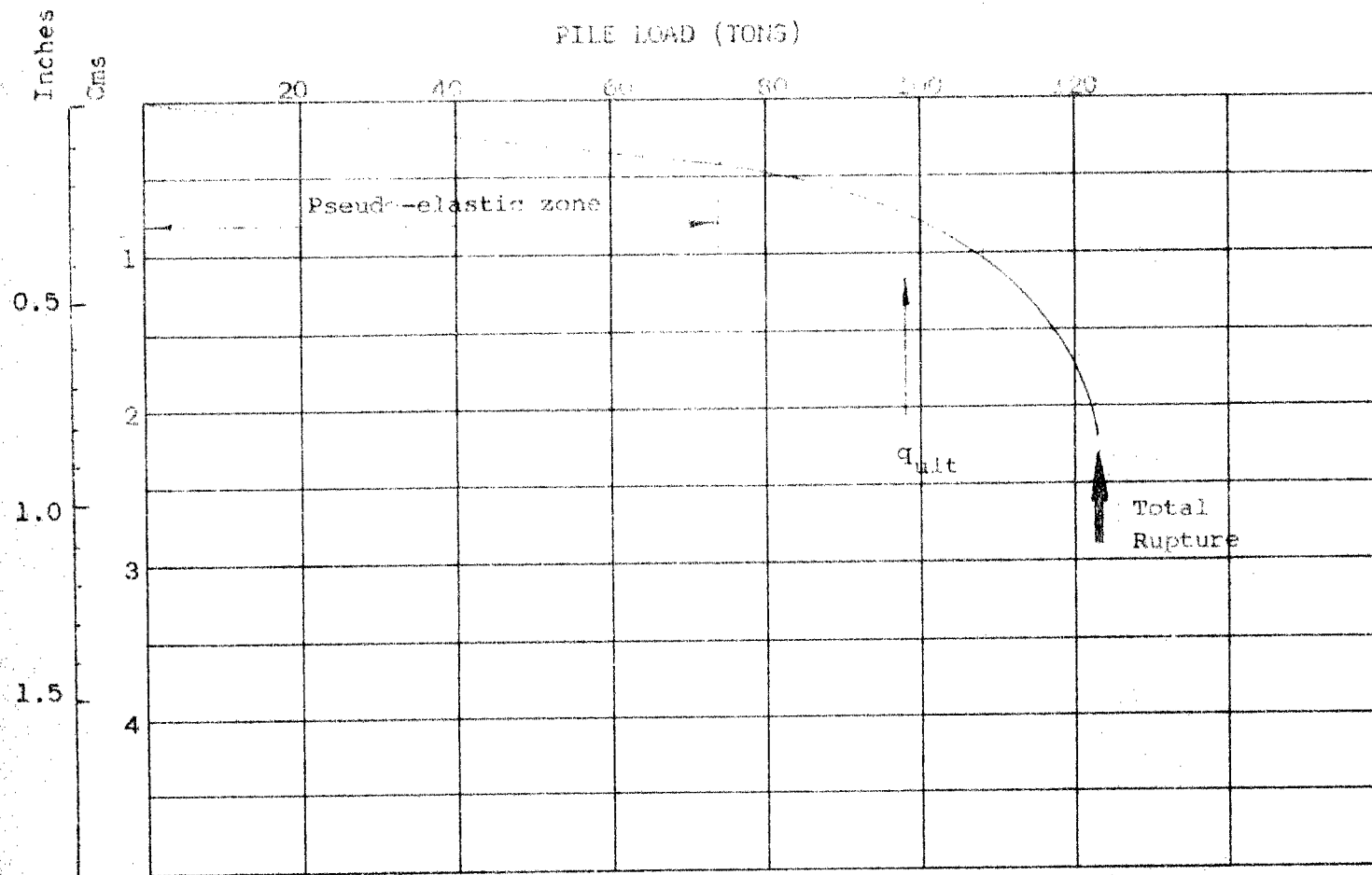
Fig 2B
230-515



Note: Where q_{ult} is load at end of
pseudo-elastic zone plus 33%.

Round Steel Pile at 60'

Settlement



Note; Where q_{ult} is load at end of pseudo-elastic zone plus 33%

H-Pile at 50'



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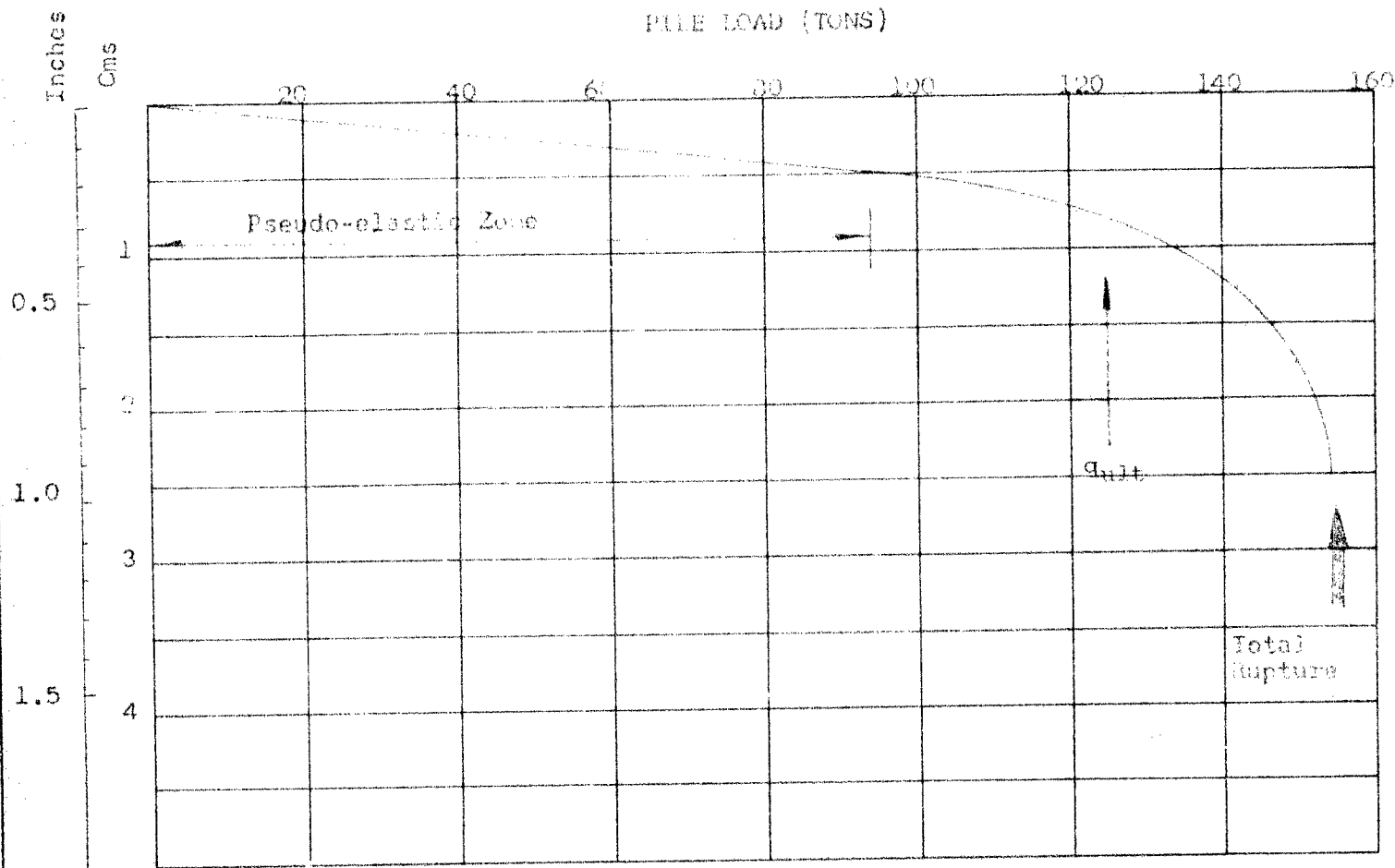
Fig 2C

Settlement



ONTARIO DEPARTMENT OF HIGHWAYS
MAGNETAWAN RIVER BRIDGE

Fig 2D
230-515



Note: Where q_{ult} is load at end of pseudo-elastic zone plus 33%.

H-Pile at 60'

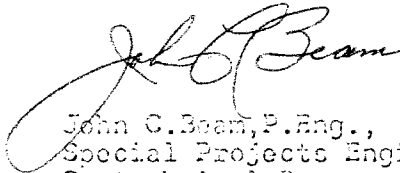
Settlement



This report is respectfully submitted to
Mr. A. Rutka, Ontario Department of Highways.

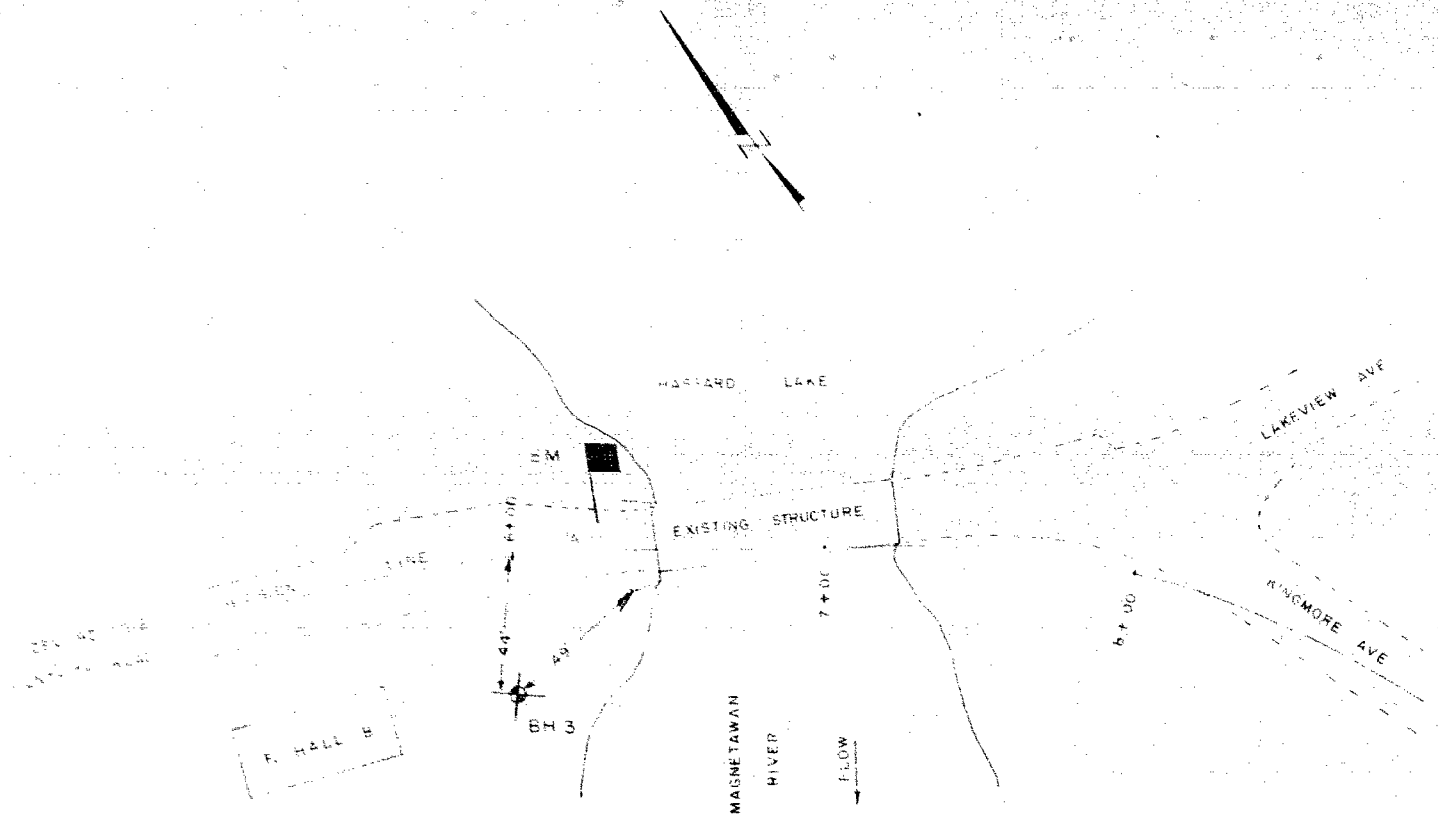
Yours very truly

Professional Services Division
WARNOCK HERSEY INTERNATIONAL LIMITED



John C. Beam, P. Eng.,
Special Projects Engineer,
Geotechnical Department.





BM
 DHO B.H. 1.
 DRAWING NO. 68-F-80-A
 1112.6 GEODETIC

REFERENCES		
DRWG. NO.	DESCRIPTION	DATE

BOREHOLE LOCATION
 MAGNETAWAN RIVER SITE

ONTARIO DEPARTMENT OF HIGHWAYS
 DOWNSVIEW

WARNOCK HERSEY INTERNATIONAL LIMITED
 Professional Services Division

DATE	NOV 1969	SCALE	1" = 40'	DRAWN BY	J.D.
APPROVED BY					
DRAWING NO. 230-515					



OFFICE BOREHOLE RECORD

APPENDIX

PROJECT NO. 230-515

CLIENT Ontario Department of Highways

BOREHOLE NO. 3

LOCATION Magnetawan River Bridge

CASING BX

DATE OF BORING Sept. 13, 14 & 15

DATE OF WL READING Sept. 16/69.

DATUM Geodetic

SOIL PROFILE				SAMPLES						LAB TEST RESULTS		
DEPTH	ELEVATION	DEPTH	SOIL DESCRIPTION	STRAT. PLT	WATER CONDITIONS	CONDITION	TYPE	NUMBER	RECOVERY	N. VALUE	LABORATORY TESTS PERFORMED	WATER CONTENT & ATTERBERG LIMITS
												WP W WL
0	1106.8											DYNAMIC PENETRATION TEST BLOWS PER FOOT...K....
			6" TOPSOIL									0 20 40 60 80
5	1101.8		Fill									
	5.0		SAND									
	6.7		Brown									
			Fine									
10			Loose									
			Changes to									
			grey at									
15			20.0 ft.									
20												
25												
30	1076.8											
	30.0		Sand									
			Grey									
35												
			with some silt seams									
			Fine									
40			Loose									
45												
50	1056.8											
	50.0		SAND									
			Grey									
			Fine									
55			Compact									
			Continued									



OFFICE BOREHOLE RECORD

APPENDIX

CLIENT Ontario Department of HighwaysPROJECT NO. 230-515LOCATION Magnetawan River BridgeBOREHOLE NO. 3 cont'dDATE OF BORING Sept. 13, 14 & 15CASING BXDATE OF WL READING Sept. 16/69.DATUM Geodetic

SOIL PROFILE				SAMPLES						LAB TEST RESULTS			
DEPTH	ELEVATION	SOIL DESCRIPTION	STRAT. PLCT	WATER CONDITIONS	CONDITION	TYPE	NUMBER	RECOVERY	N-VALUE	LABORATORY TESTS PERFORMED	LAB	TEST	RESULTS
DEPTH													
											WATER CONTENT & ATTERBERG LIMITS.		
											W _L	W _P	W _U
55											DYNAMIC PENETRATION FEET BLOWS PER FOOT...K		
											0	20	40
											60	80	
60		SAND Grey Fine Compact											
65													
70	1036.0	SAND & GRAVEL Brown											
70.5													
75													
80	1026.5												
80.25	1025.2	GRAVEL, Coarse											
81.6		End of Hole 81.6 ft.											
85		NOTE: Water Level 6.7 ft. Hole caved 70'-75' losing Bentonite at 70.0 ft.											



ONTARIO DEPARTMENT OF HIGHWAYS

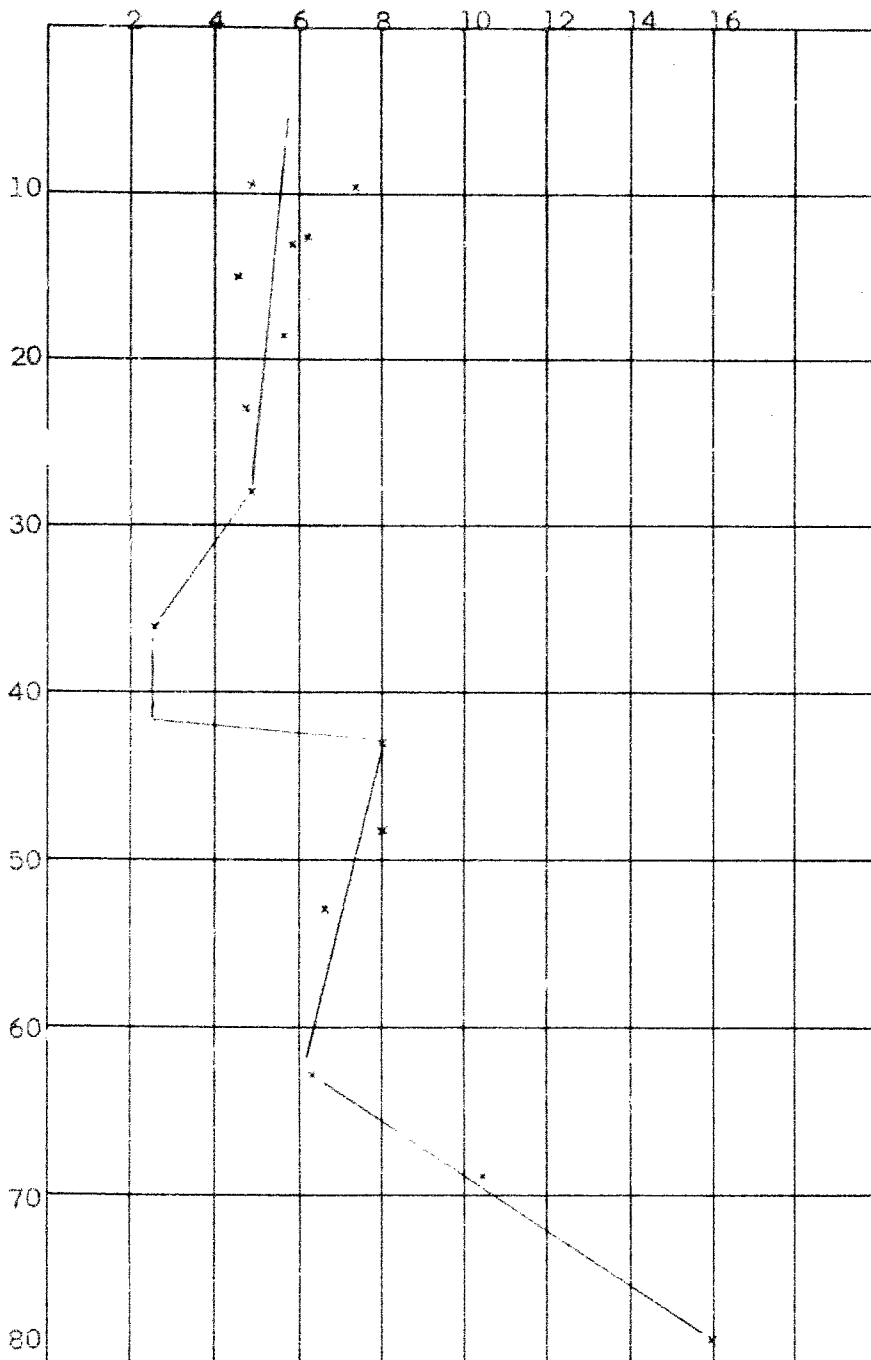
MAGNETAWAN RIVER BRIDGE

APP C Fig 1

230-515

LIMIT PRESSURE (TONS)

DEPTH BELOW EXISTING GROUND LEVEL



Average

 P_L

6.25

Average

E

62

5.5

55

4.8

48

4.2

42

4.0

40

3.1

31

2.3

23

4.2

42

7.7

77

7.3

75

6.9

69

6.7

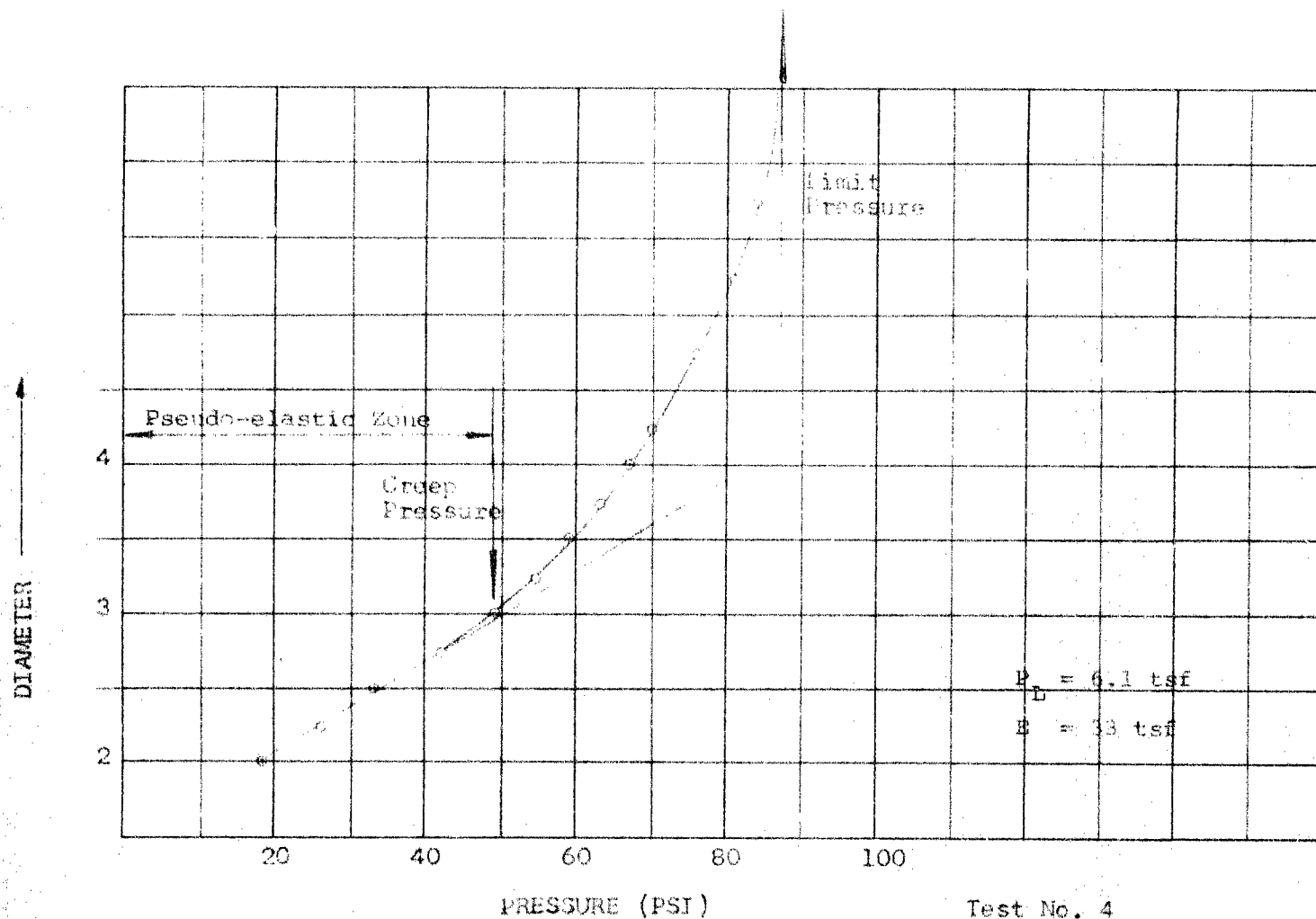
67

7.1

71

10

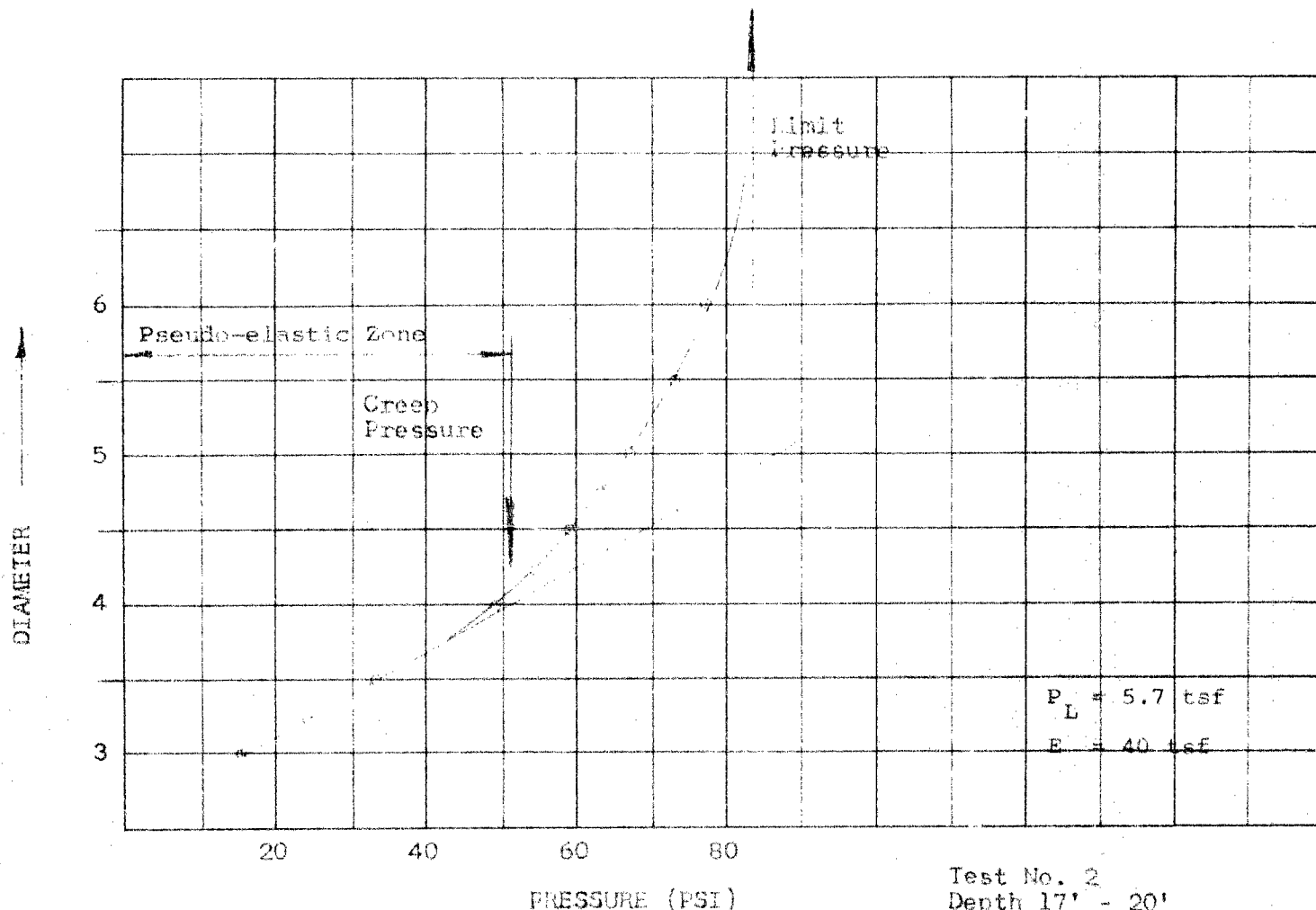
100

ONTARIO DEPARTMENT OF HIGHWAYS
MAGNETAWAN RIVER BRIDGEAPP C Fig. 2
230-515Test No. 4
Depth 11' - 14'
Borehole No. 3



ONTARIO DEPARTMENT OF HIGHWAYS
MAGNETAWAN RIVER BRIDGE

APP C Fig 3
230-515

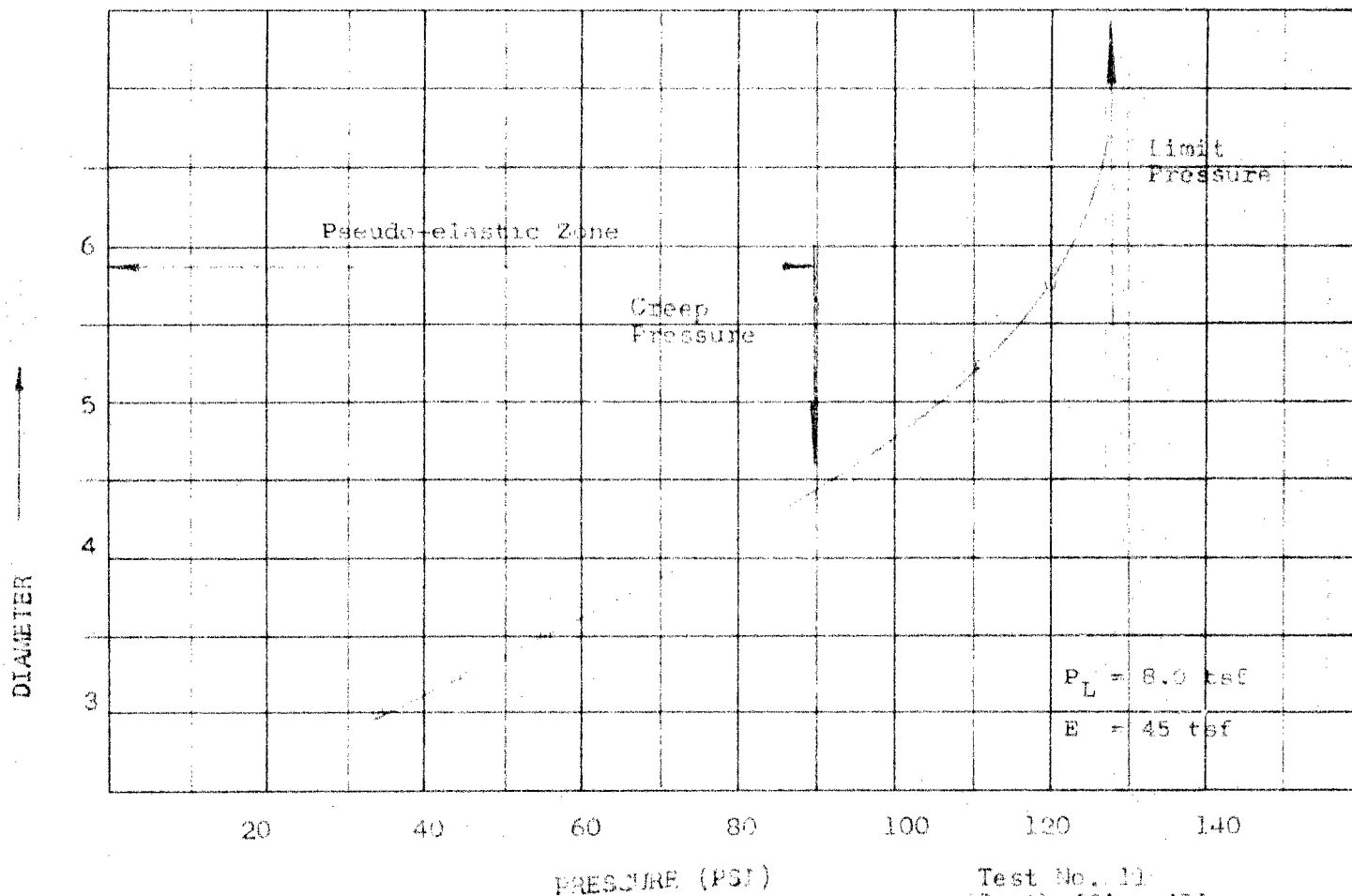


Test No. 2
Depth 17' - 20'
Borehole No. 3



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MAGNETAWAN RIVER BRIDGE

APP C FIG 4
230-515



PRESSURE (PSI)

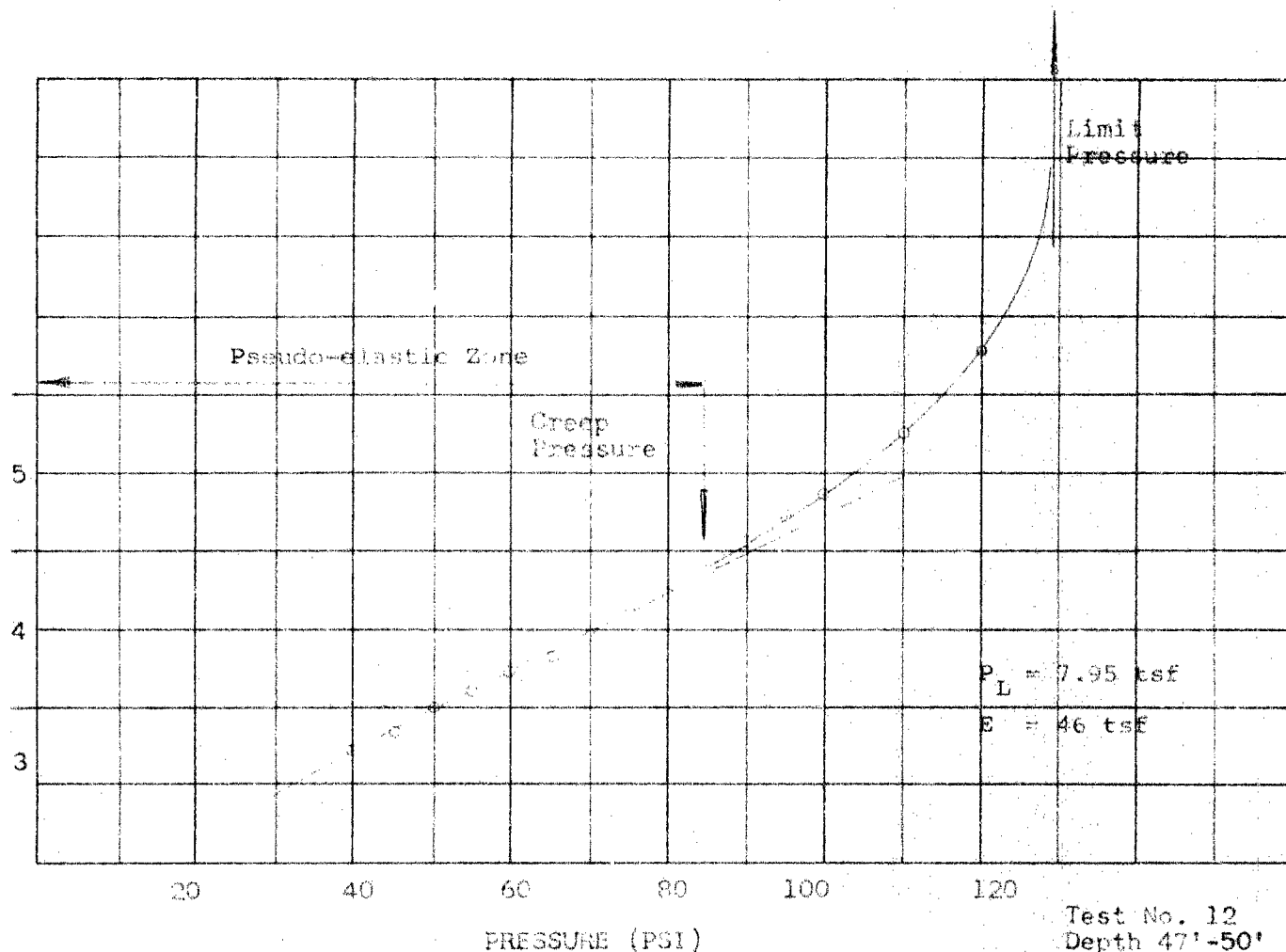
Test No. 11
Depth 42' - 45'
Borehole No. 3



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MAGNETAWAN RIVER BRIDGE

APP C FIG 5
230-515

DIAMETER
↑



Test No. 12
Depth 47'-50'
Borehole No. 3