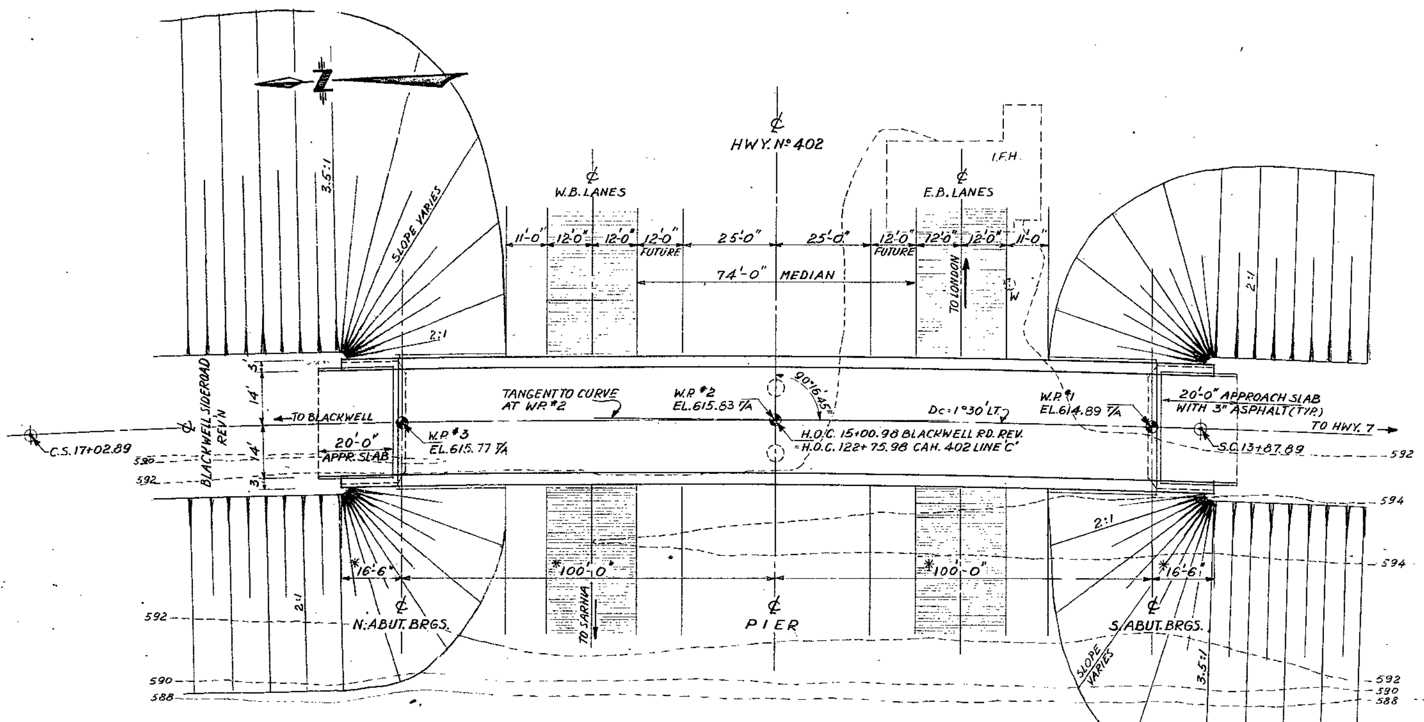
#69-F-91

W.P. 43-66-05

H.W.Y. #402, LINE 'C'

BLACKWELL SIDEROAD

AND C.A.H.

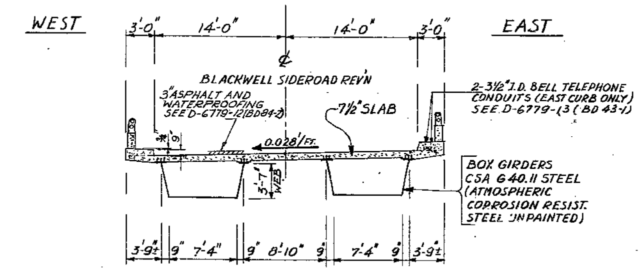


NOTE

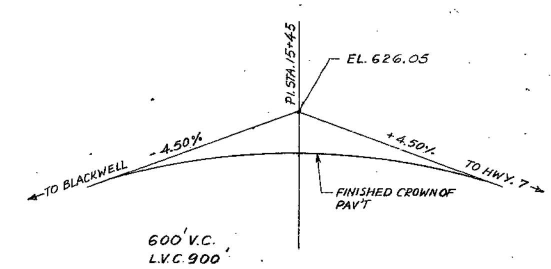
- T/A DENOTES ELEVATIONS ARE TO TOP OF ASPHALT.
- W.P. DENOTES WORKING POINT

✱ DIMENSIONS ARE PARALLEL TO TANGENT TO CURVE AT W.P. #2 STA. 15+00.98

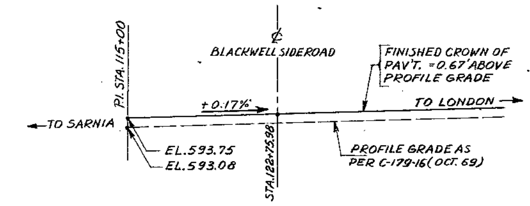
PLAN
SCALE:- 1"=20'-0"



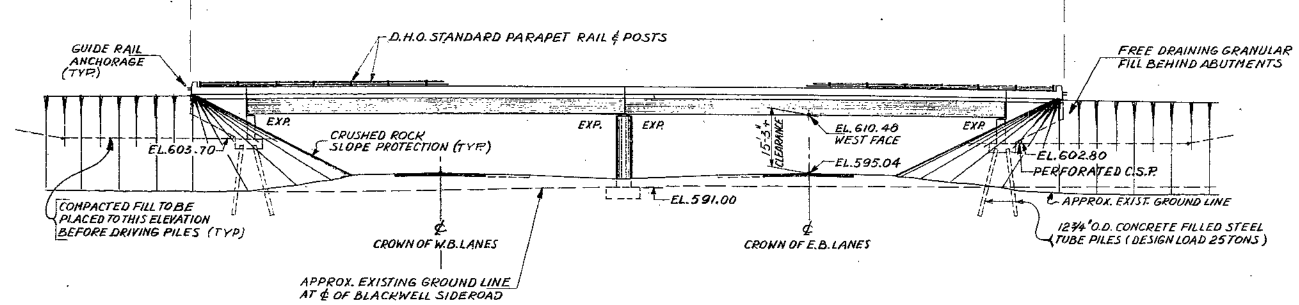
TYP. DECK SECTION
SCALE:- 1/8"=1'-0"



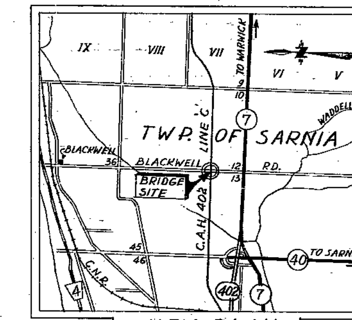
PROFILE OF BLACKWELL SIDEROAD
N.T.S.



PROFILE OF HWY. 402
N.T.S.



ELEVATION
SCALE:- 1"=20'-0"



KEY PLAN
SCALE:- 1 IN = 1 MI.

NOTES

CLASS OF CONCRETE

DECK, CURBS AND PARAPET WALLS____4000 p.s.i.
PIER COLUMNS________________4000 p.s.i.
REMAINDER__________________3000 p.s.i.
AND/OR AS NOTED ON DRAWINGS

CLEAR COVER ON REINF. STEEL

FOOTINGS, ABUTMENTS, PIER COLUMNS, DECK: TOP, BOTT.
3" 3" 3" 1½" 1"
CURBS, PARAPET WALLS, APPROACH SLABS,
2" 1½" 2"
AND/OR AS NOTED ON DRAWINGS

CONSTRUCTION NOTES

THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8 INCH. NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT BEARING SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.

B.M. ELEV. 596.07
GEODETIC DATUM: N.E. CORNER OF CONC. STEP TO IBH 434' RT. 120+98 LINE 'C'

LIST OF DRAWINGS

D-6779-1 GENERAL LAYOUT
" -2 BOREHOLE LOCATIONS AND SOIL STRATA
" -3 FOUNDATION LAYOUT
" -4 ABUTMENTS
" -5 PIER
" -6 STRUCTURAL STEEL I.
" -7 STRUCTURAL STEEL II. & BEARING DETAILS
" -8 DECK
" -9 PARAPET WALL DETAILS
" -10 STANDARD STEEL PARAPET RAIL
" -11 APPROACH SLABS
" -12 STANDARD DETAILS I.
" -13 STANDARD DETAILS II.

REGISTERED PROFESSIONAL ENGINEER CIVIL J. L. KEEN PROVINCE OF ONTARIO

FOR REDUCED PLAN
USE SCALE BELOW
0 1 2 3
3 INCHES ON ORIGINAL PLAN

PRINT RECORD

No.	FOR	DATE
[illegible]	[illegible]	[illegible]

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

69-F-91

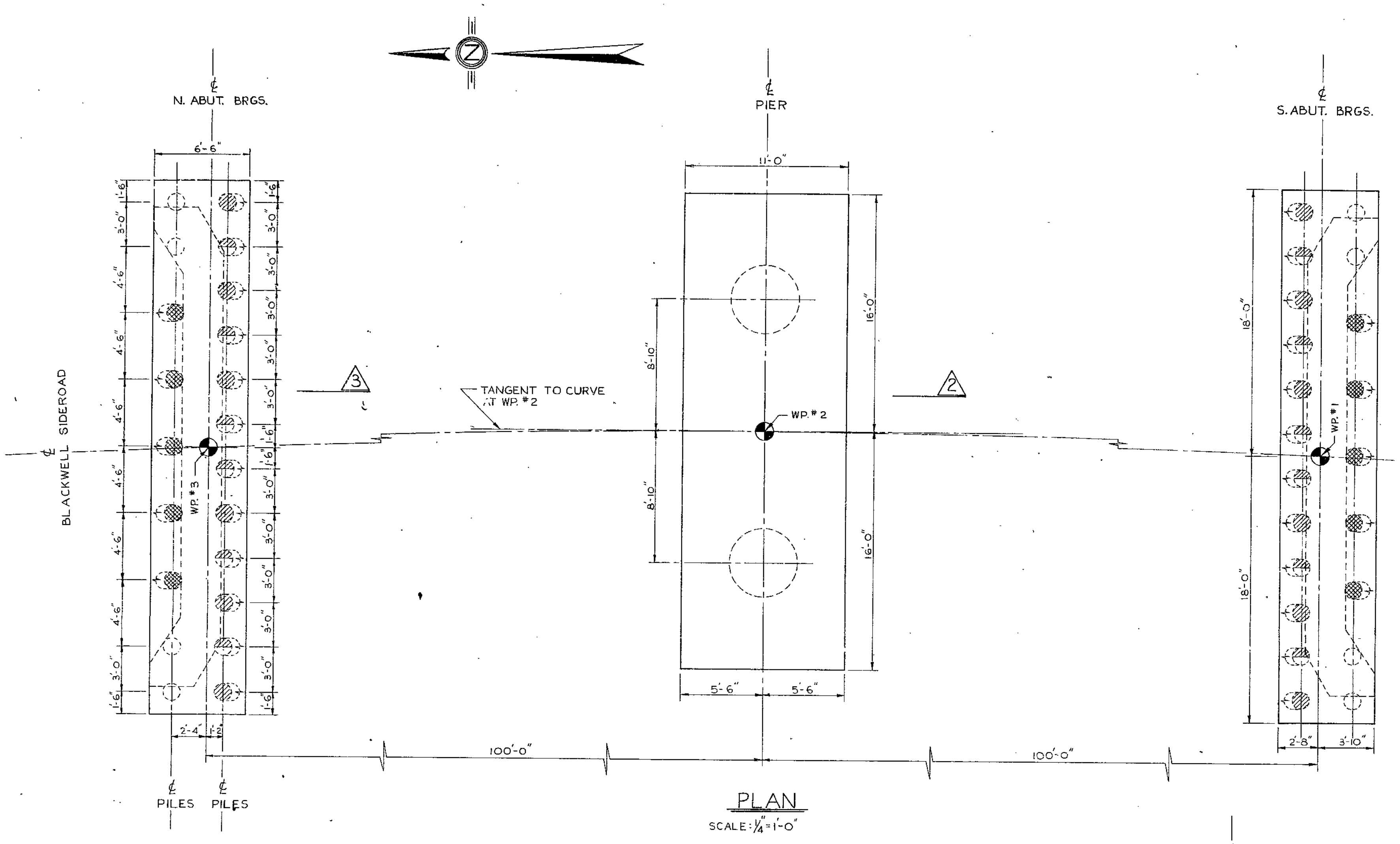
BLACKWELL SIDEROAD U'PASS
1.2 MILES EAST OF MODELAND AVENUE

KING'S HIGHWAY No. 402 DIST. No. 1
CO. LAMBTON
TWP. SARNIA LOT 12, 13 CON. 7

GENERAL LAYOUT

APPROVED	BRIDGE ENGINEER	SITE No. 14-339	W.P. No. 43-66-05
DESIGN J. L. K.	CHECK J. Sz./K.P.	CONTRACT Nos.	74-56
DRAWING J. Sz.	CHECK J. L. K.	DRAWING No.	D-6779-1
DATE OCT. 1970	LOADING HS20-44		

C



PLAN

SCALE: 1/4"=1'-0"

PILES SUPPLIED			
LOCATION	NO.	LENGTH	TYPE
N. ABUTMENT	21	20'-0"	12 3/4" O.D. x 0.25" WALL THICKNESS STEEL TUBE PILES
S. ABUTMENT	21	19'-0"	

LEGEND

- PILE BATTER 3:1
- PILE " 5:1
- PILE " 8:1
- PILE DRIVEN VERTICALLY

NOTES

DIMENSIONS AND PILE LAYOUT SIMILAR FOR BOTH ABUTMENT FOOTINGS.

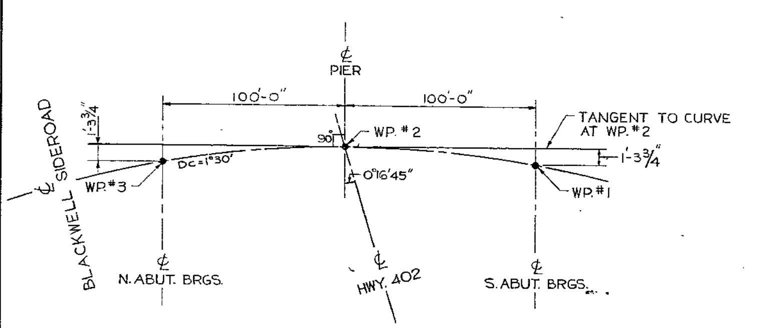
ABUTMENT PILE SPACING TO BE MEASURED AT UNDERSIDE OF FOOTING.

ALL PILES ARE 12 3/4" O.D. x 0.25" WALL THICKNESS STEEL TUBE PILES

TUBE PILES TO BE FILLED WITH 3000 P.S.I. CONCRETE AFTER INSTALLATION AND INSPECTION.

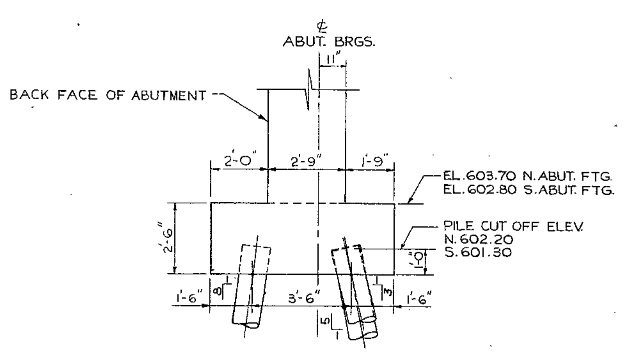
PILES SHALL BE DRIVEN IN ACCORDANCE WITH BD 82-7 USING DESIGN LOAD 25 TONS/PILE.

DESIGN LOAD = 25 TONS PER PILE.

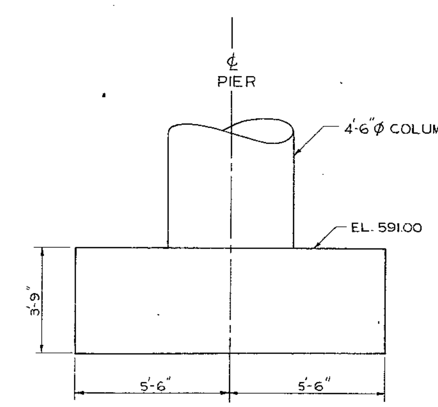


LAYOUT

N.T.S.



SCALE: 3/8"=1'-0"



SCALE: 3/8"=1'-0"

PRINT RECORD		
No.	FOR	DATE

FOR REDUCED PLAN
USE SCALE BELOW
3 INCHES ON ORIGINAL PLAN

REVISIONS		
DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

69-F-91

BLACKWELL SIDEROAD U'PASS

1.2 MILES EAST OF MODELAND AVENUE

KING'S HIGHWAY No. 402 DIST. No. 1
CO. LAMBTON
TWP. SARNIA LOT 12,13 CON. 7

FOUNDATION LAYOUT

				SITE No. 14-339	W.P. No. 43-66-05
APPROVED	BRIDGE ENGINEER			CONTRACT Nos.	74-56
DESIGN	J. Sz.	CHECK	J. L. K.	DRAWING No.	D-6779-3
DRAWING	C. G. T.	CHECK	B. McG.		
DATE	OCT. 1970	LOADING	HS20-44		

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

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

Attention: Mr. G. McCombie

Our File Ref.

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

Date: November 25, 1969

In reply to

Subject:

FOUNDATION INVESTIGATION REPORT
For
Proposed Crossing at
Blackwell Sideroad and C.A.H. #402
Line 'C', Lots 12 & 13, Con. VII,
Twp. of Sarnia - County of Lambton
District #1 (Chatham, Ont.)
W.J. 69-F-91 -- W.P. 43-66-05

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.

A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

AGS/AdeF
Attach.
cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
W. Zonnenberg
F. C. Brown
A. P. Watt
J. Roy
B. A. Singh

Foundations Files
Gen. Files

TABLE OF CONTENTS

FOUNDATION INVESTIGATION REPORT
For
Proposed Crossing at
Blackwell Sideroad and C.A.H. #402
Line 'C', Lots 12 & 13, Con. VII,
Twp. of Sarnia - County of Lambton
District #1 (Chatham, Ont.)
W.J. 69-F-91 -- W.P. 43-66-05

1. INTRODUCTION:

A request for a foundation investigation at the crossing of the proposed C.A.H. #402, Line 'C' and Blackwell Sideroad, was received from Mr. A. P. Watt, Regional Bridge Planning Engineer, in a memorandum dated October 6, 1969.

A field investigation was subsequently carried out by the Foundation Section to determine the subsoil conditions existing at the site. This report contains the results of this investigation and our recommendations pertaining to the design of the proposed structure foundations and approach embankments.

2. DESCRIPTION OF THE SITE:

The site of the proposed underpass structure is situated in the eastern outskirts of the City of Sarnia, approx. 3/4 mi. north of Hwy. #7 on Blackwell Sideroad.

The surrounding area is flat and cultivated farm land. At the west side of the existing sideroad, an approx. 23 ft. wide and 5 ft. deep drainage ditch runs in North-South direction.

Physiographically, the site is located in the region referred to as the St. Clair Clay Plain.

3. FIELD AND LABORATORY INVESTIGATION PROCEDURES:

A total of five sampled boreholes and eight dynamic cone penetration tests was carried out during the course of the field work. Boring was achieved by means of a continuous flight

3. FIELD AND LABORATORY INVESTIGATION PROCEDURES: (cont'd.) ...

auger machine, and conventional diamond drilling equipment adapted for soil sampling purposes. During the field work, disturbed samples were obtained by means of a standard split-spoon sampler: the energy used in driving it, conformed to the requirements of the Standard Penetration Test.

Dynamic cone penetration tests were carried out adjacent to each borehole with the exception of B.H. #9, and also at four other locations. Driving energy to advance the cone was 350 ft.-lbs. per blow. 'Undisturbed' samples were recovered using 2 inch I.D. Shelby Tubes which were pushed into the soil hydraulically or by hand. Where possible, Field Vane Tests were carried out at elevations 12 inches below sample depths.

The bedrock was proved at one borehole location using BXT rock coring equipment.

All boreholes were surveyed in the field by personnel from London Region Engineering Surveys Section. The locations and elevations of the borings are shown on Drawing No. 69-F-91A which accompanies this report.

All samples were visually examined and classified at the site as well as in the laboratory. Following this inspection laboratory tests were carried out on selected samples to determine the following physical properties:

Atterberg Limits
Moisture Content
Grain-size Distribution
Organic Content
Undrained Shear Strength
Bulk Density
Consolidation Characteristics

The test results are summarized on the Record of Borehole sheets contained in the Appendix of this report.

4. SOIL TYPES AND SOIL CONDITIONS:

4.1) General:

Generally uniform subsoil conditions were found to prevail over the site area. The subsoil consists of a deep deposit of cohesive material (clayey silt and silty clay) with some sand and traces of gravel, followed by a shallow zone of very dense sandy till material, followed by limestone bedrock. The boundaries between different deposits are shown on the Record of Borehole sheets attached to the Appendix. The estimated stratigraphical profile of Drawing 69-F-91A is based upon this information.

From ground level downward, the various strata are described in some detail with regard to soil types and soil properties, as follows:

4.2) Fill Material:

This material was encountered in B.H.'s 4, 5, 6, 8 and 9 from the existing roadway level (El. 594) to approx. El. 589. The material in the deposits consists of sand and gravel with traces of fines. The relative density may be described as compact.

4.3) Clayey Silt with Some Sand and Traces of Gravel:

This deposit was intersected in all borings and extends from immediately below the topsoil, or the above mentioned fill material, down to approx. El. 543. The material in the deposit consists of clayey silt with some sand and traces of gravel. A plot of Plasticity Index versus Liquid Limit (Fig. 1) shows the points to fall within the CL zone.

The upper 6 - 8 ft. portion of the stratum is a very stiff to hard desiccated surface crust. This zone is brown in colour due to oxidation. The natural moisture content, in general, is at or below the plastic limit. Standard Penetration Test 'N' values ranged from 21 to 39 blows per foot. The undrained shear strength is in excess of 2000 PSF. This zone is highly over-consolidated due to desiccation and/or weathering.

4. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

4.3) Clayey Silt with Some Sand and Traces of Gravel: (cont'd.) .

Below the desiccated layer, the colour of the deposit gradually changes from brown to grey.

The shear strength of the deposit gradually decreases to a minimum value of 600 PSF at approx. El. 570. From this level on, it increases very rapidly to 1850 PSF at approx. El. 565 and remains more or less constant throughout the remainder of the clayey silt layer.

Physical properties of the overall deposit, as determined from field and laboratory tests, are as follows:

Natural Moisture Content (%)	15.0 to 24.5
Liquid Limit (%)	23.0 to 34.5
Plastic Limit (%)	13.0 to 17.5
Organic Content (Desiccated Zone) (%) ...	0.5 to 1.0
Bulk Density (PCF)	128 to 137
Field Vane Test (PSF)	560 to >2000
Unconfined Shear Strength (PSF)	650 to 1950
Quick Triaxial Shear Strength (PSF)	600 to 1250
'N' Value (Blows/ft.)	10 to 39
Sensitivity	1.3 to 3.3

Typical grain-size distribution curves are included in the Appendix of this report (Figure 2).

All of the field and laboratory shear strength measurements are plotted on Figure 3.

The consistency of the overall deposit may be described as firm to hard.

The results of consolidation tests (Figure 4) carried out on selected samples, indicate that the preconsolidation pressure of the undesiccated portion of the stratum is approximately 0.5 to 1.0 TSF.

..... 5

4. SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

4.4) Silty Clay with Some Sand and Traces of Gravel:

This stratum was found to underlie the clayey silt deposit and extends to approximate El. 474. The material in the deposit consists mainly of clay and silt with some sand and traces of gravel. In some samples, thin layers of sand and silt were found.

The test results are plotted on the Record of Borehole sheets, and also on Figures 1, 3 and 4 of the Appendix. The average physical properties of the material are as follows:

Liquid Limit	: 40%	Plastic Limit:	20%
Moisture Content:	24%	Bulk Density :	123 PCF

Based on laboratory and field measurements, the consistency may be assumed as firm to hard.

4.5) Heterogeneous Mixture of Gravel, Sand and Fines:

A very dense, heterogeneous mixture of gravel, sand, silt and clay layer was found beneath the silty clay zone. The estimated thickness of the deposit is in the order of 6 - 7 ft.

4.6) Limestone Bedrock:

Bedrock at this site was found to consist of generally sound limestone at El. 468 (B.H. #3).

5. GROUNDWATER CONDITIONS:

The following groundwater levels were observed during the field investigation:

B.H. #1	Caved in at El. 509.1
2	El. 583.1
3	El. 573.6
5	El. 587.4
9	El. Dry

..... 6

5. <u>GROUNDWATER CONDITIONS</u>: (cont'd.) ...

It is pointed out, that the foregoing quoted figures may not represent the true groundwater levels, due to the relatively impermeable nature of the subsoil and the short duration of the field work.

Natural gas was observed in B.H. #3 when contact was made with the bedrock.

6. <u>DISCUSSION AND RECOMMENDATIONS</u>:

6.1) <u>General</u>:

It is proposed to build a four-span (35'-73.5'-73.5'-35') underpass structure at the crossing of new Hwy. #402, Line 'C' and Blackwell Sideroad. The proposed profile grade of Blackwell Sideroad will be approximately 20 ft. above the proposed Hwy. #402 grade of elevation 594.

As described in the previous paragraphs of this report, the subsoil at the site consists of a deep deposit of clayey silt and silty clay, containing some sand and traces of gravel. The upper 6 - 8 ft. of the deposit is a very stiff to hard desiccated surface crust. Below this depth the shear strength of the material decreases until a minimum value is reached, then increases again with depth, with some random variation. The desiccated surface crust appears to be suitable for spread footing type foundations.

Because of the compressible nature of the subsoil, it is inevitable that consolidation settlements will occur over a long-term period due to the imposed loads of structure and embankment. Past experience, however, indicates that these settlements will be of a minor nature.

6.2) <u>Foundations</u>:

(a) <u>Spread Footings in Original Ground</u>:

The entire structure may be supported on spread footings placed within the very stiff to hard desiccated zone of the subsoil at or above El. 588'. A safe net pressure of 2.0 TSF may be assumed for design purposes.

..... 7

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

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

(a) Spread Footings in Original Ground: (cont'd.) ...

The desiccated zone is susceptible to softening on contact with water, therefore, it is recommended that the base of the footing excavations be protected by a concrete working slab, immediately on exposure.

All foundations should be protected against frost action by at least 4 feet of earth cover. No dewatering problems are anticipated.

The estimated maximum settlement will be in the order of 1.0 and 1.5 inches under the pier footings.

(b) Spread Footings on Compacted Fill:

As an alternative, the abutments may be supported on spread footings placed on well compacted, suitable granular material within the approach fills. A safe design load of 2.0 TSF may be assumed. The granular material should consist of G.B.C. Class 'A' and should be fully compacted according to the current D.H.O. Standards. A detailed construction scheme is outlined on Figure 5 of the Appendix.

(c) Perched Abutments on Short Piles:

As a second alternative, the abutments may be constructed within the approach fills and supported on short piles driven through the fill and some 5.0 ft. into the desiccated crust. In the case of 12-3/4" O.D. and 1/4" thick wall steel tube piles, a safe design load of 25 tons per pile may be used.

It should be pointed out, that this latter proposal is based on experience with similar structures and similar subsoil conditions in the general area. To obtain more detailed information about pile lengths, pile types and design loads, a full-scale pile loading test would be advantageous and it is strongly recommended that such tests be carried out. Therefore, the recommendations given for this type of foundation are subject to change, depending on the results of the planned pile loading tests.

..... 8

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

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

(c) Perched Abutments on Short Piles: (cont'd.) ...

Regardless of which method is adopted, the structure should be built to accommodate the 3.0 to 3.5 inches differential settlement between the abutments and piers.

(d) End-Bearing Piles:

As another alternative, the abutments and piers may be supported on steel H-piles driven to bedrock. For 12 BP @ 53, a safe design load of 70 tons per pile may be assumed.

6.3) Approach Embankments:

The shear strength of the subsoil is such that it will be able to safely support the 24-ft. high approach embankments constructed with 2:1 side slopes. The fill should consist of well compacted acceptable material. Care should be taken to ensure that no bouldery fill is placed within the approaches through which piles have to be driven, and it is recommended that this portion of the fill contain no larger grain sizes than 3 inches.

Calculations carried out by conventional methods indicated that the settlement due to consolidation of the subsoil caused by embankment loading directly beneath the abutment location will be in the order of 18 inches. Based on the performance of structures and embankments built in the same general area and under somewhat similar subsoil conditions, it is our opinion that the above quoted figure of 18" is grossly overestimated. A maximum settlement of 4 to 5 inches appears to be more reasonable. To minimize the effect of differential settlements between the abutments and pier footings, it is recommended that the approach embankments be built in advance of the structure for as long a period as possible. The topsoil and the soft organic material should be removed in accordance with the pertinent D.H.O. Standards within the construction area.

..... 9

7. MISCELLANEOUS:

The field investigation was carried out during the period October 20 - 25, 1969, under the supervision of Mr. P. Payer, Project Foundation Engineer, who also prepared this report.

Equipment was owned and operated by Dominion Soil Investigation Ltd.

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

November 1969

APPENDIX I

FORM OS-[illegible]-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

FOUNDATION SECTION

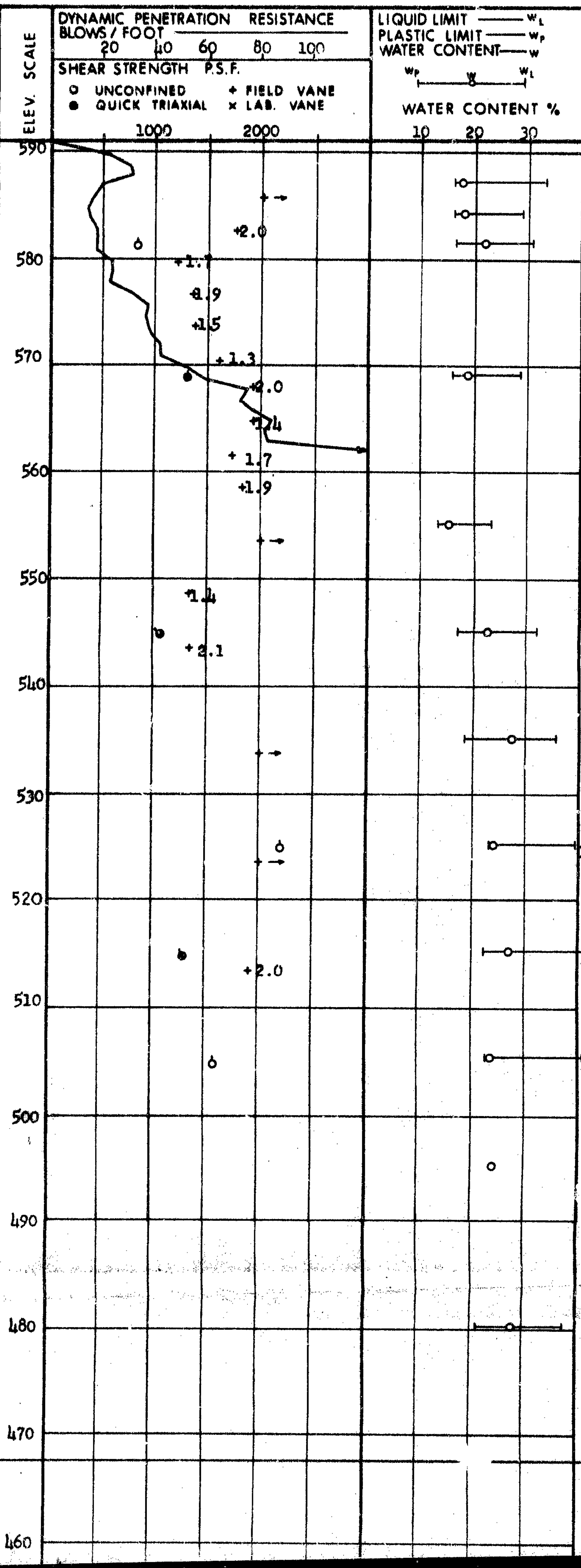
JOB	69-F-[illegible]	LOCATION	Sta. 15+06; 22' Rt.	ORIGINATED BY	PP
W.P.	43-66-05	BORING DATE	October 20 & 21, 1969	COMPILED BY	PP
DATUM	Geodetic	BOREHOLE TYPE	Cont. Flight Auger	CHECKED BY	[illegible]

SOIL PROFILE

ELEV. DEPTH	DESCRIPTION
590.9	Ground Level
0.0	Clayey Silt with some sand and traces of gravel Brown and grey Firm to V. Stiff
543.0 / 47.9	Silty Clay with Some sand and Traces of Gravel Stiff to Hard
474.0	Probably mixture of gravel, sand, silt & clay V. Dense
467.4 / 123.5	Probable Bedrock End of Borehole

SAMPLES

NUMBER	TYPE	BLOWS/FOOT
1	SS	23
2	SS	12
3	TW	PH
4	TW	PH
5	TW	PH
6	TW	PH
7	TW	PH
8	TW	PH
9	TW	PH
10	TW	PH
11	SS	12
12	TW	PH
13	TW	PH
14	TW	PH
15	TW	PH
16	TW	PH
17	TW	PH
18	SS	37
19	SS	31



BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
	0-14-57-29
130	1-14-36-49
132	Water Level Not Establ[illegible]
128	
127.5	
123	
125	
	0-9-44-47

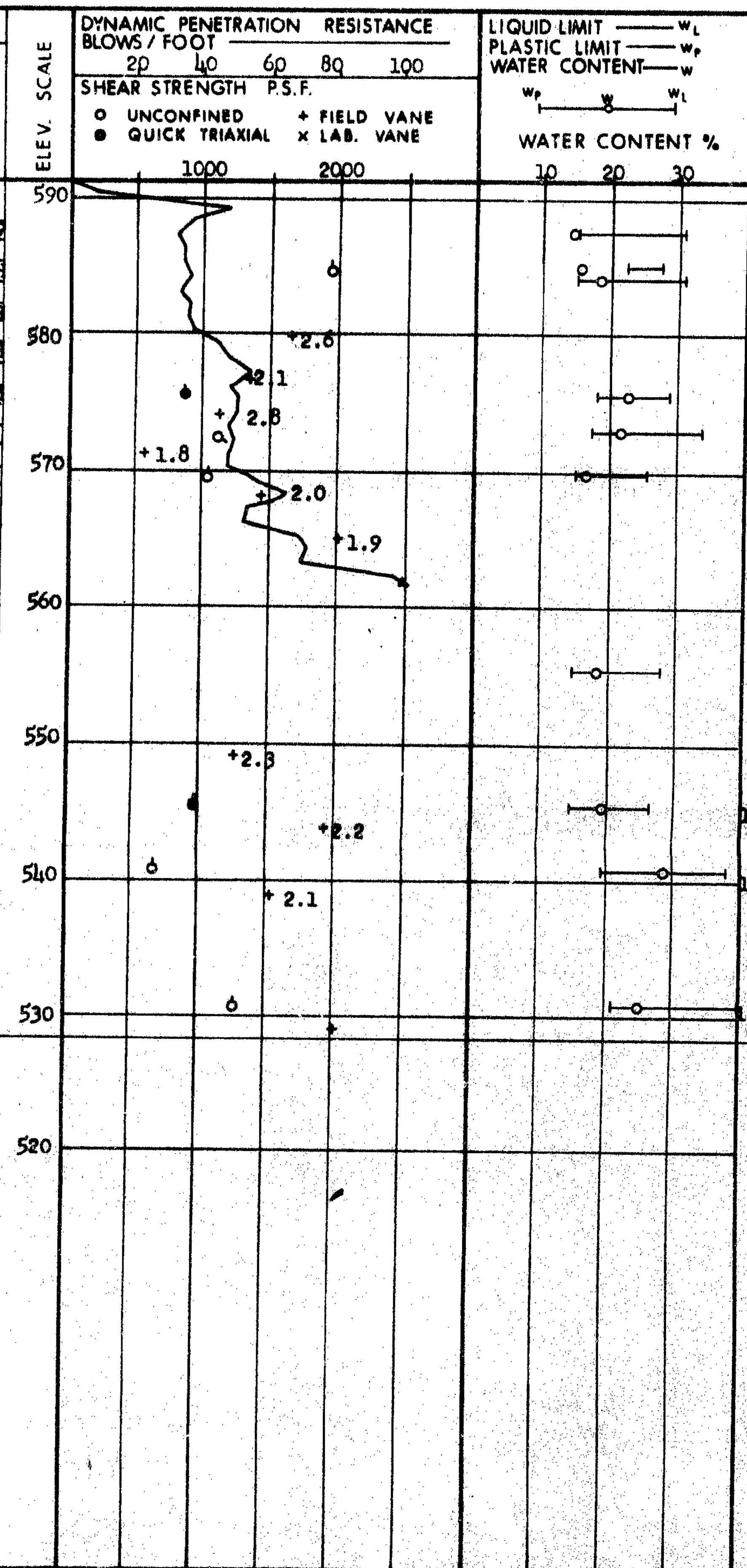
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB	69-F-91	LOCATION	Sta. 13+86; 15' Rt.	ORIGINATED BY	PP
W.P.	43-66-05	BORING DATE	October 20, 21, 1969	COMPILED BY	PP
DATUM	Geodetic	BOREHOLE TYPE	Washboring - NX Casing	CHECKED BY	[illegible]

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
591.1	Ground Level						
0.0	Clayey Silt with some sand and traces of gravel		1	SS	39		
			2	TW	PH	134	6-36-43-15
			3	TW	PH		W.L. El. 583.1
			4	TW	PH		
			5	TW	PM	128.5	
			6	TW	PM	128	
			7	TW	PM	137	3-18-52-27
	Brown and Grey		8	TW	PM		
			9	TW	PM		
	Firm to Hard		10	SS	19		
			11	SS	22		
			12	SS	15		
[illegible].0			13	TW	PM	135	3-17-50-30
[illegible].1	Silty Clay with some sand and traces of gravel		14	TW	PM	120	
	Firm to V. Stiff		15	TW	PM	125.5	
[illegible].1							
[illegible].0	End of Borehole						



20
13 5 % STRAIN AT FAILURE
10

FORM OS-MT [illegible]6 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

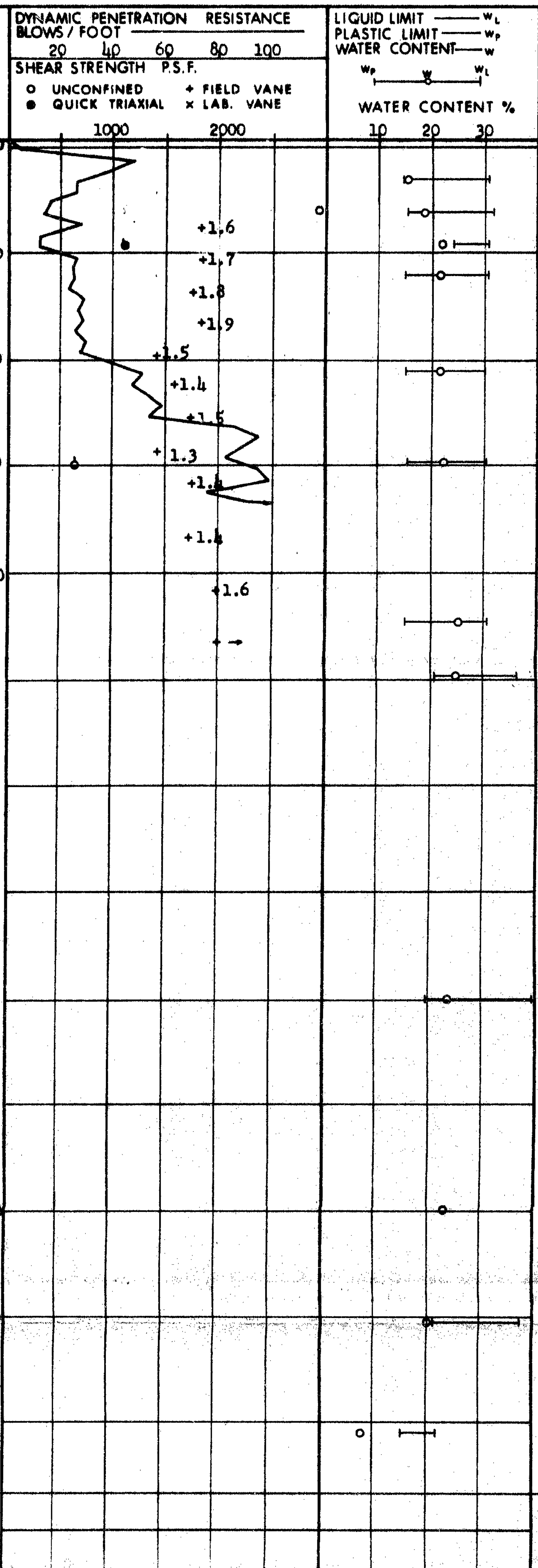
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 69-F-91 LOCATION Sta. 16+10; 18' Rt. ORIGINATED BY PI
W.P. 43-66-05 BORING DATE October 22 & 23, 1969 COMPILED BY PT
DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger & Washboring CHECKED BY [illegible]

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
590.6	Ground Level							
0.0	Clayey silt with some sand and traces of gravel Brown and Grey					590		
			1	SS	30			3-17-51-29
			2	TW	PH		133	
			3	TW	PH	580	130	1-16-46-37
			4	SS	12			
			5	TW	PH			W.L. 573.6
			6	TW	PH	570		
			7	SS	13			
			8	TW	PH			
			9	SS	14			
			10	TW	PH	560	128	
			11	SS	15			
			12	TW	PH	550		
			13	SS	15			1-9-45-45
543.0 / 47.6	Silty Clay with sand and traces of gravel Grey V. Stiff to Hard							
			14	SS	22	540		
			15	SS	48	530		
						520		
			16	SS	34	510		
						500		
			17	SS	33	490		
			18	SS	27	480		
474.4 / 116.2	Mixture of gravel, sand, silt & clay V. Dense							
			19	SS	78/6"	470		17-40-31-12
467.9 / 122.7	Bedrock Limestone		20	RC	90%			
462.9 / 127.7	End of Borehole					460		



DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

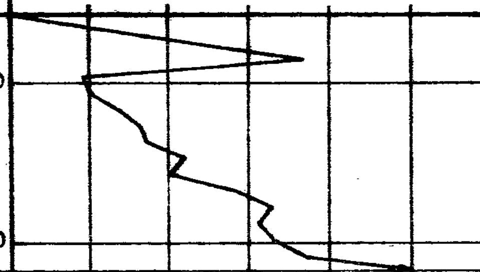
FOUNDATION SECTION

JOB 69-F-91 | LOCATION Sta. 13+86; 19' Lt. | ORIGINATED BY PP

W.P. 43-66-05 | BORING DATE October 22, 1969 | COMPILED BY PP

DATUM Geodetic | BOREHOLE TYPE Cone Test Only | CHECKED BY [illegible]

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT					
594.1	Ground Level									
0.0	Probably									
590.1	Fill Material					590				
4.0	Probably Clayey Silt									
578.2						580				
15.9	End of Cone Test									
						570				



FORM OS-[illegible]-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No.5

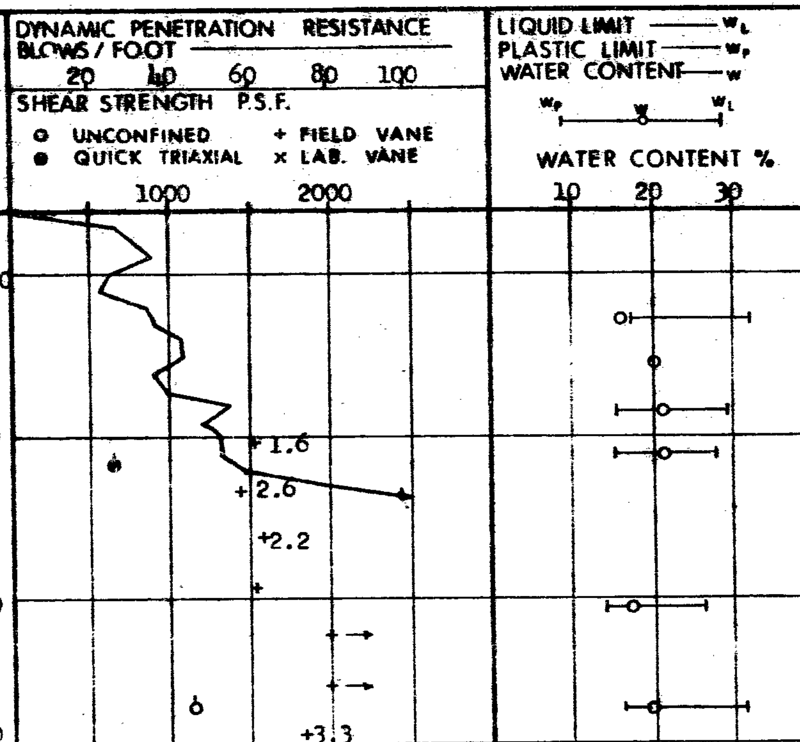
FOUNDATION SECTION

JOB 69-F-91 LOCATION Sta. 14+34; 18' Lt. ORIGINATED BY PP

W.P. 43-66-05 BORING DATE October 22, 1969 COMPILED BY PP

DATUM Geodetic BOREHOLE TYPE Washboring - NX casing CHECKED BY [illegible]

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
593.9	Ground Level						
0.0	Fill Material						
589.6	Sand & Gravel	1	SS	22	590		
4.3		2	SS	21			W.L. 587.4
	Clayey Silt with	3	SS	19			0-14-56-30
	Some Sand and traces of gravel	4	SS	13	580		
	Brown & Grey	5	TW	PM		128	
	Firm to V. Stiff	6	TW	PM			
		7	TW	PM	570		
		8	TW	PM			3-18-48-31
		9	TW	PM			
560.9		10	TW	PM	560	132	
33.0	End of Borehole						



DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

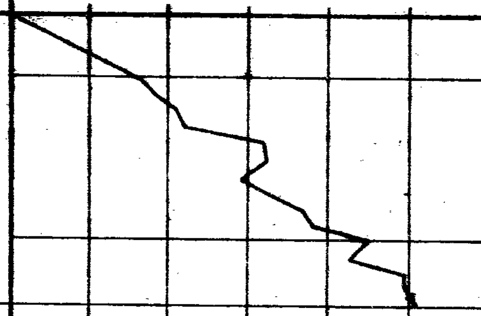
FOUNDATION SECTION

JOB 69-F-91
W.P. 43-66-05
DATUM Geodetic

LOCATION Sta. 15+01; 22' Lt.
BORING DATE October 23, 1969
BOREHOLE TYPE Cone Test Only

ORIGINATED BY PP
COMPILED BY PP
CHECKED BY [signature]

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (20 40 60 80 100); SHEAR STRENGTH P.S.F. (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE)	LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT					
593.9	Ground Level									
0.0	Probably Fill Material									
589.9 4.0	Probably Clayey Silt					590				
						580				
575.9 18.0	End of Cone Test									
						570				



FORM 08-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 7

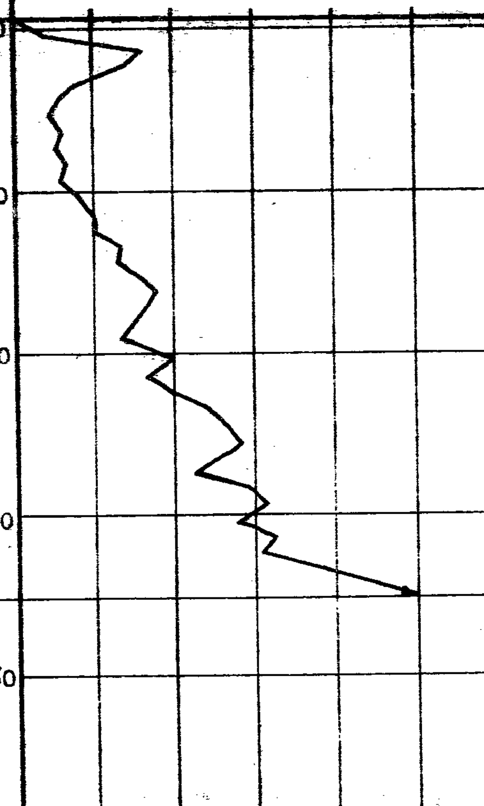
FOUNDATION SECTION

JOB 69-F-91 LOCATION Sta. 15+76; 18' Rt. ORIGINATED BY PP

W.P. 43-66-05 BORING DATE October 23, 1969 COMPILED BY PP

DATUM Geodetic BOREHOLE TYPE Cone Test Only CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT					
590.4	Ground Level					590				
0.0	Probably Clayey Silt					580 570 560				
554.5 35.9	End of Cone Test					550				



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 8

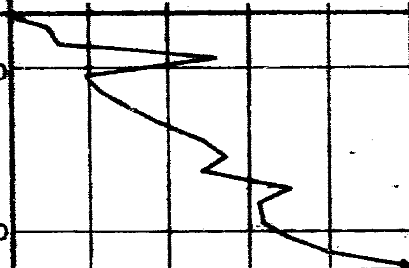
FOUNDATION SECTION

JOB 69-F-91 LOCATION Sta. 16+16; 23' Lt. ORIGINATED BY PP

W.P. 44-66-05 BORING DATE October 23, 1969 COMPILED BY PP

DATUM Geodetic BOREHOLE TYPE Cone Test Only CHECKED BY

SOIL PROFILE: ELEV. DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
593.4	Ground Level									
0.0	Probably									
589.5	Fill Material					590				
3.9	Probably Clayey Silt									
577.5						580				
15.9	End of Cone Test					570				



OFFICE REPORT ON SOIL EXAMINATION

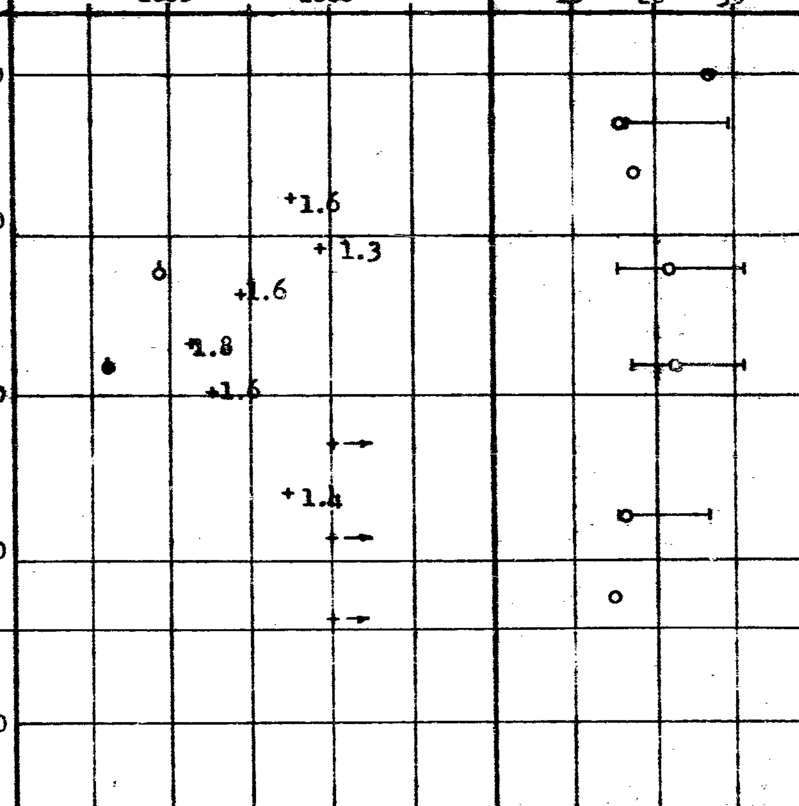
DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 9

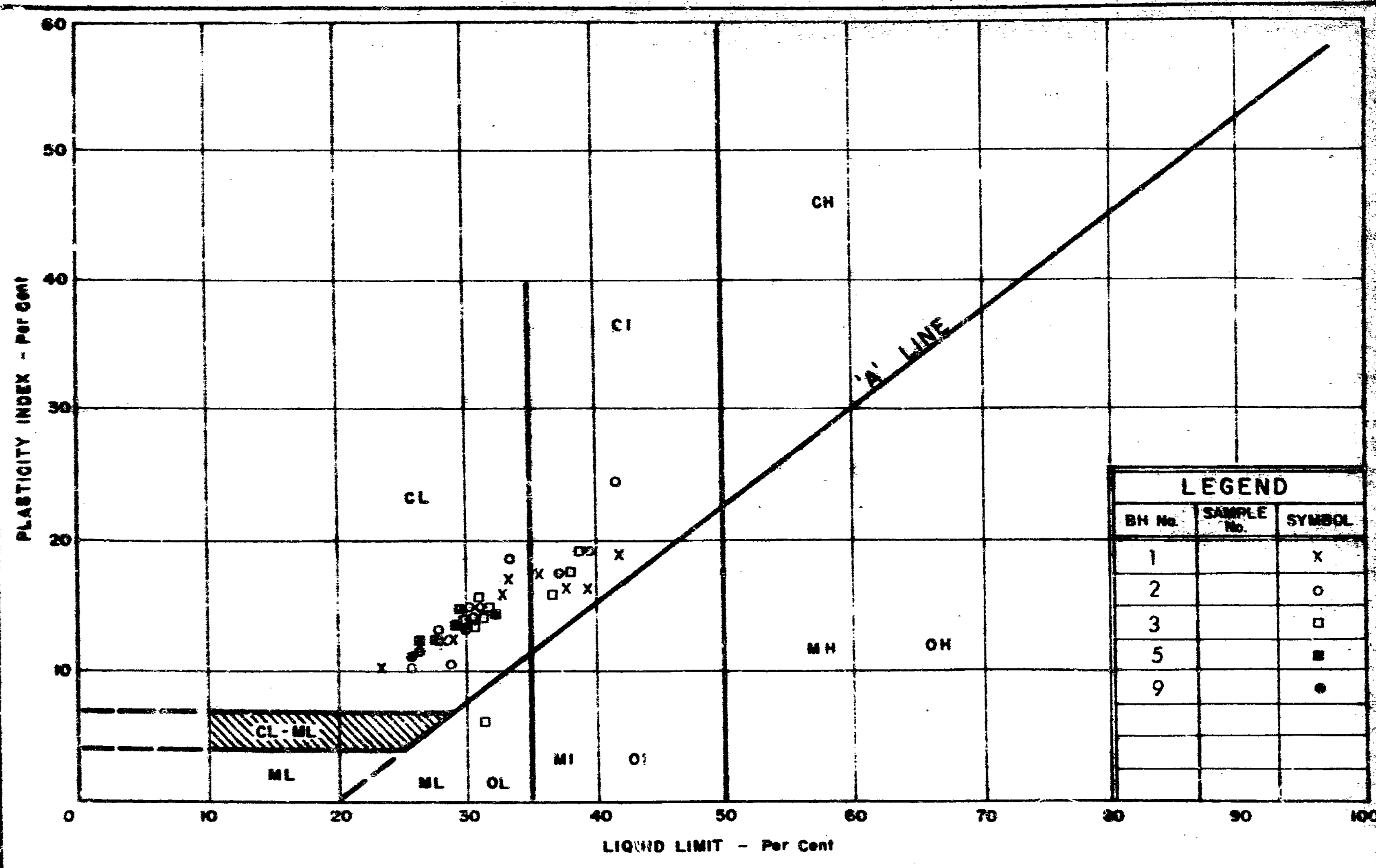
FOUNDATION SECTION

JOB	69-F-91	LOCATION	Sta. 15+76; 21' Lt.	ORIGINATED BY	PP
W.P.	43-66-05	BORING DATE	October 23, 1969	COMPILED BY	PP
DATUM	Geodetic	BOREHOLE TYPE	Cont. Flight Auger	CHECKED BY	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100
SHEAR STRENGTH P.S.F.: O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE — 1000 2000
LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w
WATER CONTENT %: 10 20 30



ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
593.6	Ground Level							
0.0	Fill Material Sand and Gravel Compact		1	SS	23	590		Water level not established
588.6 / 5.0	Clayey silt with some sand and traces of gravel Brown & Grey Firm to Hard		2	SS	35			
			3	SS	13			
			4	TW	PH	580		
			5	TW	PH		129	
			6	SS	10			
			7	TW	PH		129.5	2-16-48-34
			8	SS	16	570		
			9	TW	PH			
			10	SS	16			4-18-53-25
			11	SS	15	560		
555.6 / 38.0	End of Borehole					550		



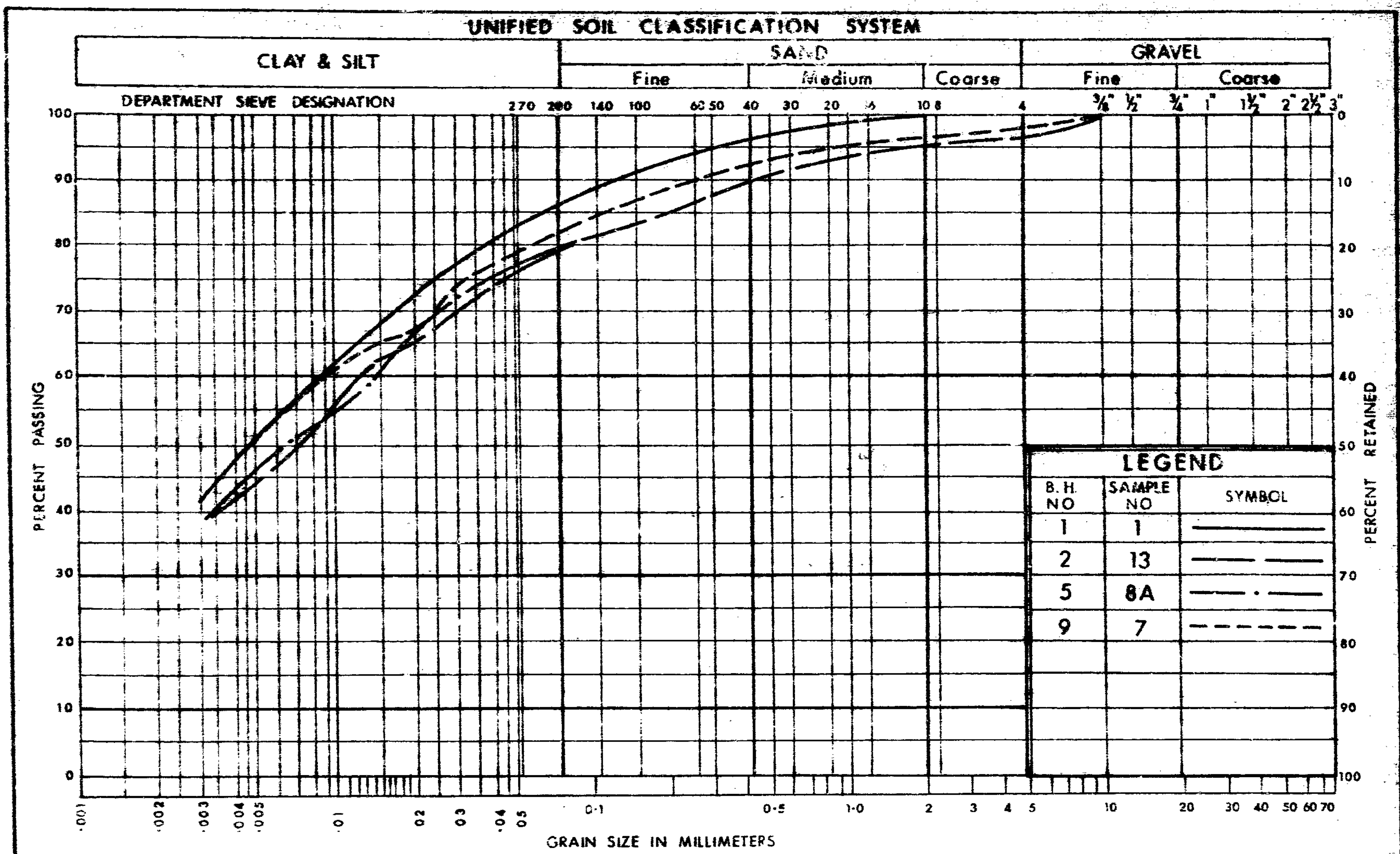
DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

PLASTICITY CHART

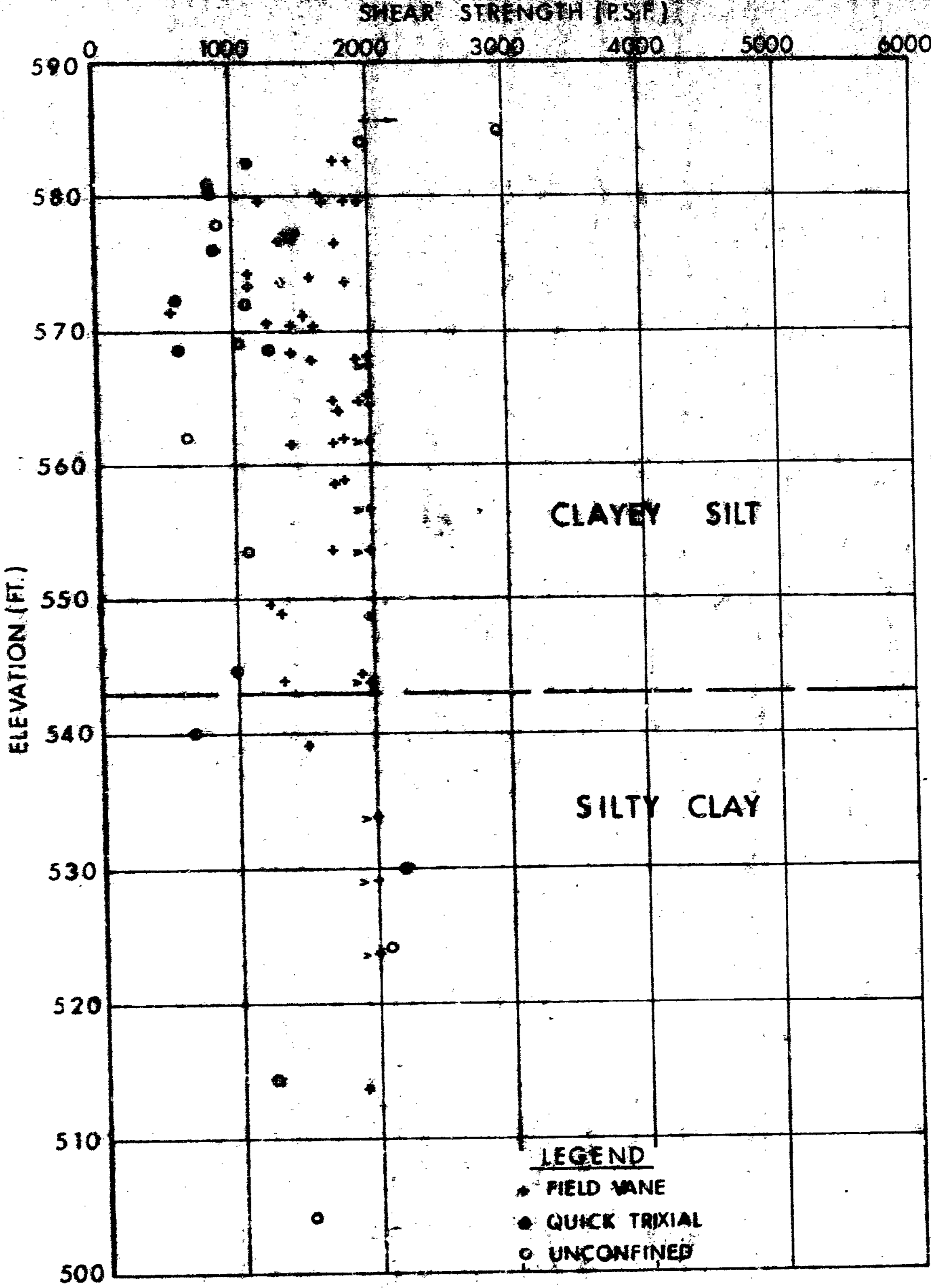
CLAYEY SILT & SILTY CLAY

W.P. No.	43 - 66 - 05
JOB No.	69 - F - 91
FIG. 1	

FORM OS MT-17
REV. APR. [illegible]



DEPARTMENT OF HIGHWAYS MATERIALS and TESTING DIVISION	GRAIN SIZE DISTRIBUTION CLAYEY SILT	W.P. No. 43-66-05 JOB No. 69-F-91 FIG. 2



ELEVATION VS SHEAR STRENGTH

FIG. 3

VOID RATIO - PRESSURE CURVES

JOB NO. 69 - F - 91

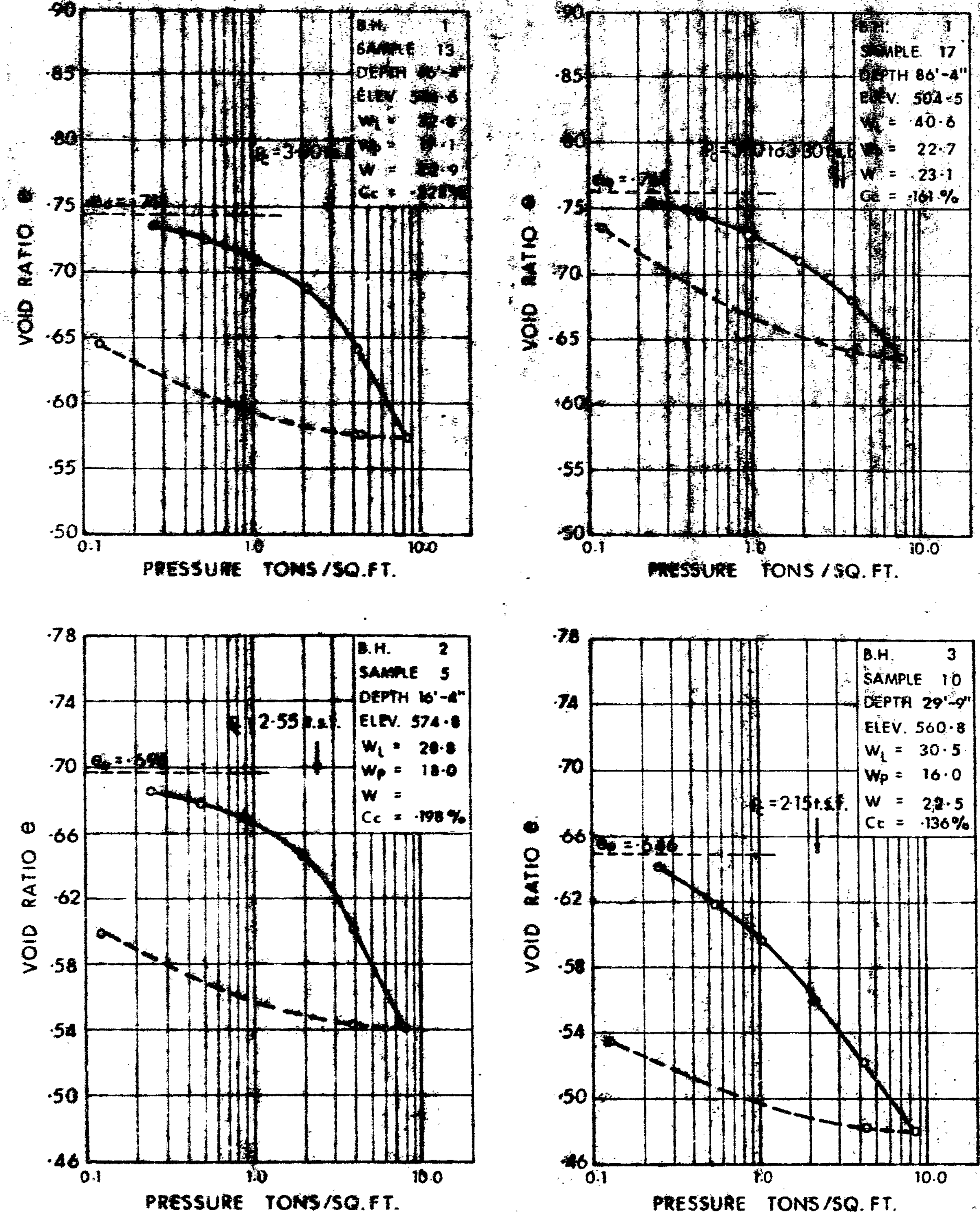
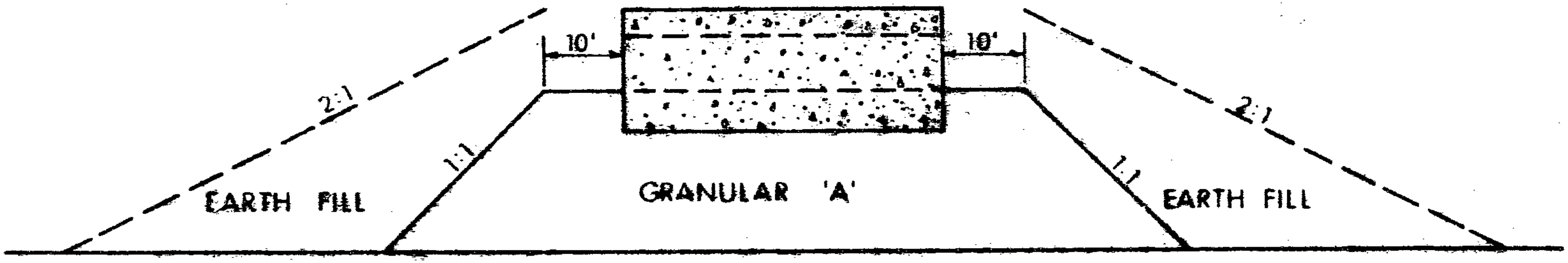
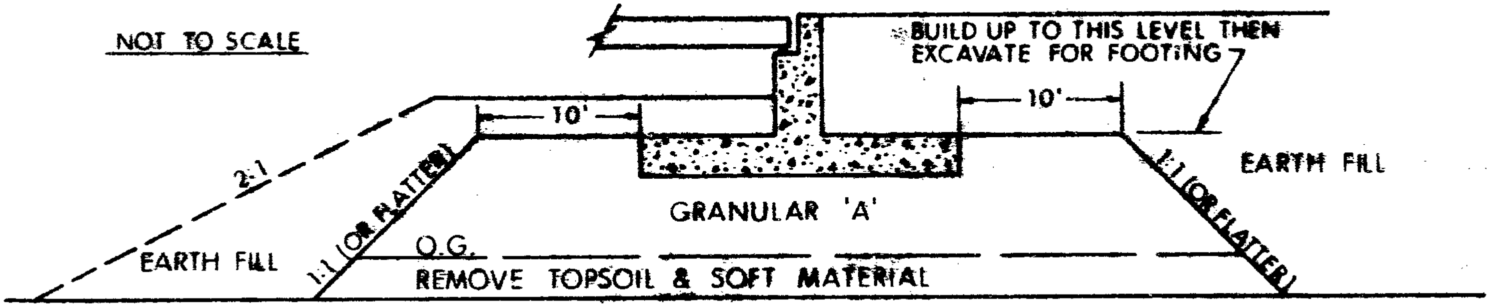


FIG. 4

ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE





LONGITUDINAL SECTION

NOTES

1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A'.

2 - PLACE GRANULAR 'A' TO TOP OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT D.H.O. STANDARDS.

3 - EXCAVATE COMPACTED GRANULAR 'A' MATERIAL FOR FOOTING.

FIG. 5

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 :-

CONSISTENCY	'N' BLOWS / FT.	c LB. / SQ. FT.	DENSENESS	'N' BLOWS / FT.
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

Q_u	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Q_{cu}	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q_d	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
(IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$)
c_u APPARENT COHESION
ϕ_u APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
(IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$)
μ COEFFICIENT OF FRICTION
S_t SENSITIVITY

GENERAL

π = 3·1416
e BASE OF NATURAL LOGARITHMS 2·7183
$\log_e a$ OR $\ln a$ NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$ LOGARITHM OF a 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 (YOUNGS MODULUS)
G MODULUS OF SHEAR DEFORMATION
K MODULUS OF COMPRESSIBILITY
η COEFFICIENT OF VISCOSITY

EARTH PRESSURE

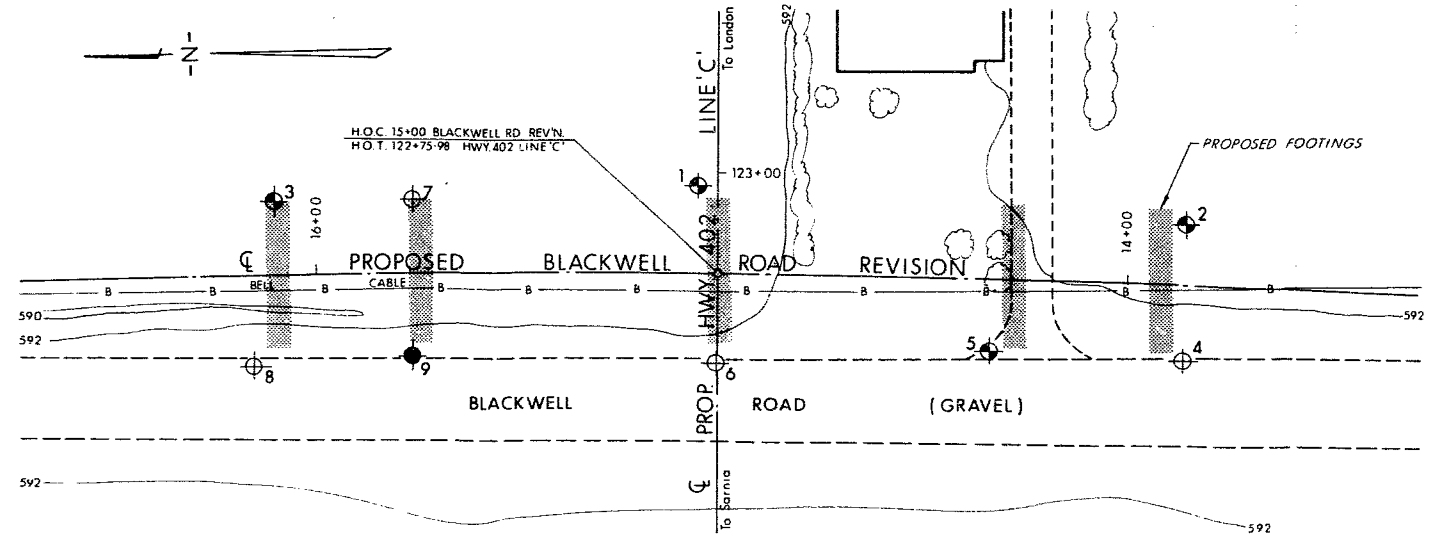
d DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ ANGLE OF WALL FRICTION
K DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o COEFFICIENT OF EARTH PRESSURE AT REST

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

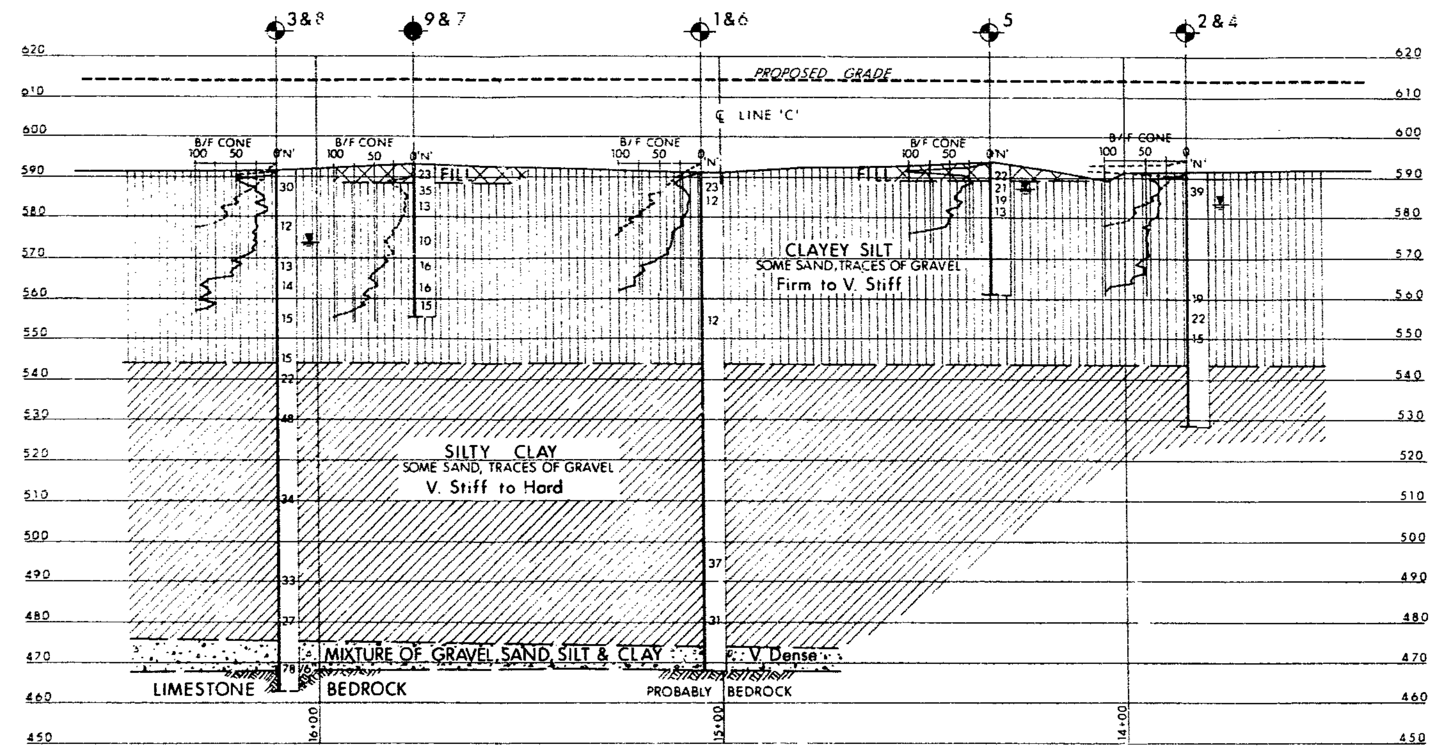
SLOPES

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



PLAN

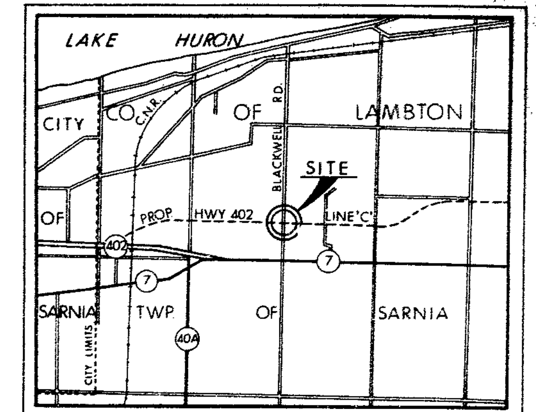
20 10 0 SCALE 20 40 FT



℄ PROFILE

SCALE

20 10 0 20 40 FT



KEY PLAN

SCALE IN MILES

1 0.5 0 1 2 3

LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation. OCT. 1969

NOTE - WaterLevels not established in Bore Holes 1&9 at time of field investigation

NO.	ELEVATION	STATION	OFFSET
1	590·9	15+06	22' RT
2	591·1	13+86	15' RT
3	590·6	16+10	18' RT
4	594·1	13+86	19' LT
5	593·9	14+34	18' LT
6	593·9	15+01	22' LT
7	590·4	15+76	18' RT
8	593·4	16+16	23' LT
9	593·6	15+76	21' LT.

- 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.

REVISIONS		
DATE	BY	DESCRIPTION

PRINT RECORD

NO	FOR	DATE

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING OFFICE - FOUNDATION SECTION

BLACKWELL ROAD

KING'S HIGHWAY NO. 402 LINE 'C' DIST. NO. 1

CO. LAMBTON

TWP. SARNIA LOT 12 & 13 CON. VII

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. P.P.	CHECKED PP	W.P. NO. 43-66-05	M.&T. DRAWING NO. 69-F-91A
DRAWN S.O.	CHECKED	JOB NO. 69-F-91	
DATE 26 NOV. 1969		SITE NO	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

REF. NO. E-4845-1

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

TO: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

FROM: C.S. Greb ski,
Bridge Office

ATTENTION:

DATE: November 9, 1970

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 43-66-05, Site No. 14-339
Blackwell Sideroad Underpass
1.2 Mi. East of Modeland Avenue,
Highway 402, District No. 1

69-F-91

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.

C.S. Grebski,
Bridge Design Engineer

CSG:rd

Attach.

c.c. Foundation Office

(PILE TEST RESULT)

24/70.

26 Nov 70

~~70-11-0119~~
69-F-91

GEOPROBE ONTARIO

Suite 289
37 King St. East
Toronto 1, Ont.
Tel. 368-0760

GEOPROBE PRESSUREMETER TESTS

for

THE ONTARIO DEPARTMENT OF HIGHWAYS

at

BLACKWELL ROAD & HWY. 402

Dist. 1 - Chatham

1.0 INTRODUCTION

Geoprobe (Ontario) Ltd. was authorized by Mr. A. Rutka, Ontario Department of Highways, to carry out a foundation investigation using the Geoprobe pressuremeter equipment.

The site is located near the proposed underpass of Highway 402 and Blackwell Rd. The exact location is shown on D.H.O. drawing 70-11049A, copy of which is enclosed in this report.

The purpose of the investigation was to evaluate the performance of round steel, H and "Hercules" precast concrete piles driven to depths of 20, 40 and 60 ft.

2.0 GEOPROBE PRESSIOMETRIC TEST

2.1 General

A pressuremeter test is a test carried out in a previously drilled borehole in order to obtain in-situ stress-strain information. A radially expandable, cylindrical probe is inserted into a borehole and set at a test elevation; the probe is then expanded incrementally against the side of the borehole with a combination of gas and liquid pressure; each pressure increment, in a standard test, is held for a period of one minute. The recession of the side of the borehold under each pressure application is measured by a volume change in the central part of the probe only. The pressure increments are continued until failure of the soil is reached. By plotting volume versus pressure, a stress-strain curve is obtained. As mentioned previously, the expansion of the borehole is measured over the central part of the probe only, in order to approximate conditions of plane stress, plane strain; thus the soil deformation can be analysed as a two dimensional problem in a radial plane. From the stress-strain curve the following parameters are derived:

(a) The limit pressure denoted P_L, which corresponds to the pressure at which total failure occurs in the soil surrounding the pressuremeter probe. This parameter reflects directly the strength of the material under test and is used in the derivation of the shear strength.

(b) The creep pressure denoted P_f, which indicates the upper limit of the pseudo- elastic zone or, in other words, the pressure at which plasticity is initiated.

(c) The modulus of deformation E, which is derived from the slope of the curve in the pseudo-elastic range. It should be noted here that the Modulus E is derived from the parameter $\frac{E}{1+\theta}$ which is measured directly by the test. The parameter $\frac{E}{1+\theta}$ is related to the shear modulus G by the relation

$$2G = \frac{E}{1+\theta}$$

3.0 COMPUTATION OF SHEAR STRENGTH

The computation of shear strength in a cohesive material is derived from the Expression

$$C = \frac{P_L - P_o}{1 + \log_e \frac{E_a}{(1+\theta)2C}} \qquad (1)$$

Experience shows that the denominator of this expression varies within well-defined limits according to the values of E/P_L. Therefore expression (1) is usually presented under the form

$$C = \frac{P_L - P_o}{2K_B} \qquad (2)$$

where K_B takes the following values:

For	$E/P_L = 10$		$K_B = 2.7$
	$E/P_L = 15$		$K_B = 3.2$
	$E/P_L = 20$		$K_B = 3.3$

4.0 COMPUTATION OF BEARING CAPACITY OF PILES

4.1 The End bearing value is given by the expression

$$Q_{ultimate} = Q_o + K (P_L - P_o) \qquad (3)$$

where Q_o is the overburden pressure at the depth of the foundation.

$Q_{ultimate}$	is the ultimate pressure that the soil can sustain at failure at the depth of the foundation.
K	a coefficient varying with the depth and the type of soil, here K = 2.
P_L	is the limit pressure
P_o	the horizontal pressure at rest ($P_o \cong K_o \gamma z$) for clays

If we express Q_{ult} as the net surcharge that the soil can sustain, neglecting the difference between the density of the pile and the soil and writing $P_L' = P_L - P_o$, we may write

$$Q'_{ult} = K P_L' \qquad (4)$$

4.2 Skin friction

The peak values of the skin friction S_s which may be mobilized are given as a function of P_L' and will be found on graph No. 14 appendix "A". It should be noted that these values are peak values used in the computation of pile settlement and should be reduced by 80% (in clay) for estimating the load carried by skin friction at failure of the pile.

5.0 PILE SETTLEMENT

The pile settlements have been calculated utilizing a method proposed by P. Gambin (sols-soils No. 7, 1963). It involves the use of Rules T.2 and 3 described in the Geoprobe handbook.

Briefly Rule T.1 treats of the skin friction mobilization as a function of the relative movement between the pile shaft and the soil; in the present case it is expressed by the relation

$$W = C_L \frac{\tau D}{2E} \tag{5}$$

or

$$\tau = \frac{2EW}{C_1 D} \tag{6}$$

where

D = pile diameter in inches
W = pile movement in inches
E = pressiometric modulus of deformation, T.S.F.
τ = shearing stress around pile shaft
C_1 = a coefficient, here C_1 = 2.1

Rule T.2 treats of the mobilization of the point resistance in function of the movement at the pile tip. It is expressed here by the relation:

$$W = \frac{N}{4\ E_a} \quad \lambda_2 \quad D \tag{7}$$

where

N = Normal stress acting on soil at pile tip
E_a = Alternated modulus
λ_2 = Shape coefficient

D = pile diameter

The calculations are proceeded with as follows:

The pile is divided in a number of sections; the first section has a height of 3 pile diameters, and then each following section has a height of 5 feet. If a normal stress N_1 is allowed on the soil at the pile tip, this will result in a movement W_1 at the pile tip which is calculated from equa. (7). Adding the elastic shortening of Section (1) to W_1, the shear stress mobilized around the pile shaft is then calculated from equa. (6). Knowing the shear stress, the load carried by section (1) is then calculated. This operation is repeated for each section of the pile. The results are then tabled and the shear stress τ mobilized at each section is then compared with the maximum skin friction S_s as computed from graph No. 14 (see paragraph No. 4.2). From the ratio S_s/σ the maximum value of the normal stress N_o on the pile head, for which no overstressing will take place, can be calculated (as the relation is proportional for a range inferior to overstressing). For a value of N_o greater than the above plastic deformations will start to occur.

The complete calculations relating to three of the piles are given as examples and will be found in appendix "B".

6.0 FIELD TESTING PROGRAM

The field testing program was carried out from Nov. 10 to Nov. 16, 1970.

Two boreholes were advanced and tested as the drilling progressed. The location of these boreholes, denoted G1 and G2, has been superimposed on D.H.O. drawing 70-11049-B which will be found in appendix "C".

The exact elevation of the boreholes as well as the stratigraphy encountered are not recorded here since the responsibility for this work was assumed by the Department of Highways, Ontario.

Both boreholes were advanced from the top of a 12 foot high fill. Borehole G1 was advanced by driving a Bx casing, washing out the inside of the casing and then pulling the casing back in order to expose the soil; the probe was then inserted through the casing and the test carried out. This operation was repeated every 3 feet in borehole G1 till a depth of 42' when the hole had to be abandoned due to broken casing. However, this method did not prove satisfactory as the borehole had a tendency to "squeeze-in" as soon as the casing was pulled back; this phenomenon was probably due to the recent placing of the 12 foot fill. Borehole G2 was similarly advanced to a depth of 20 ft. Thereafter borehole G2 was advanced utilizing a special tool referred to as an "open end split tube". It consists of a 4 foot length of Bx casing the bottom end of which is shaped into a sharp driving shoe and incorporates 6 equidistant longitudinal slits along a 3 foot length. The tool was screwed to standard Bx casing and driven into the soil, the inside washed out at regular intervals to prevent the forming of a "plug" inside the tool. Whenever a test was required, the probe was lowered to the elevation of the slotted casing and the test performed at that elevation. The resistance to outward expansion of the "split tube" is quite small and in any event is measured by performing an "inertia test" prior to testing. This method permitted the performance of satisfactory tests.

7.0 GEOPROBE TEST RESULTS

A total of 28 tests were carried out; the curves and derivations relating to each test will be found in appendix "A ". The results have also been grouped and are presented on the following pages under the form of 3 graphs.

Graph No. 1 shows the values of $P_L{}'$ versus depth ($P_L{}' = P_L - P_o$).

Graph No. 2 shows the variations of the modulus of deformation "E" in function of depth. An average value line has been drawn through this graph and represents the values utilized in the calculation of settlements.

Graph No. 3 indicates the shear strength versus depth.

Examinations of the tests performed at 27'-0' and over in borehole 2 reveal a high ratio of E/P_L which is indicative of an overconsolidated deposit.

Examination of the curves reveal an extremely sharp break once the creep pressure P_f has been reached. The writer has never experienced as sharp a break except in leda clay. This was totally unexpected in a basically unsensitive clay (sensitivity index 1.4 to 2) and probably reflects a fragile structure. Any sample taken for laboratory testing would therefore require the utmost care in sampling technique as well as handling.

8.0 ANTICIPATED PILE BEHAVIOUR

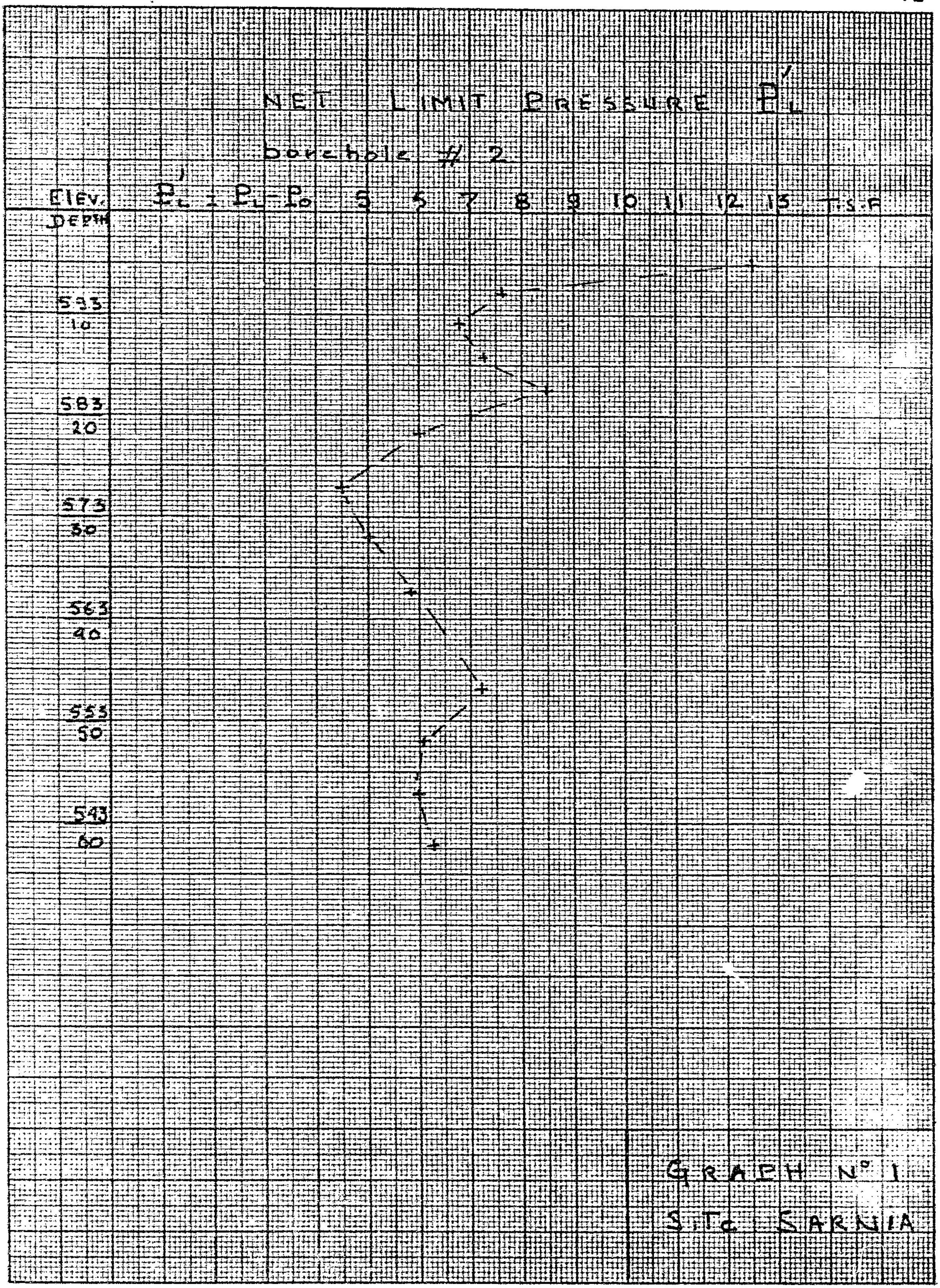
The anticipated behaviour of H, tubular steel and "Hercules" concrete piles driven at depths of 20, 40 and 60 ft. was studied utilizing the method described in a previous paragraph. The results are presented in the following pages under the form of graphs numbered 4-12; a summary showing the allowable bearing capacity, the limit of the pseudo-elastic range and the ultimate load at failure will be found on pages 20 to 22.

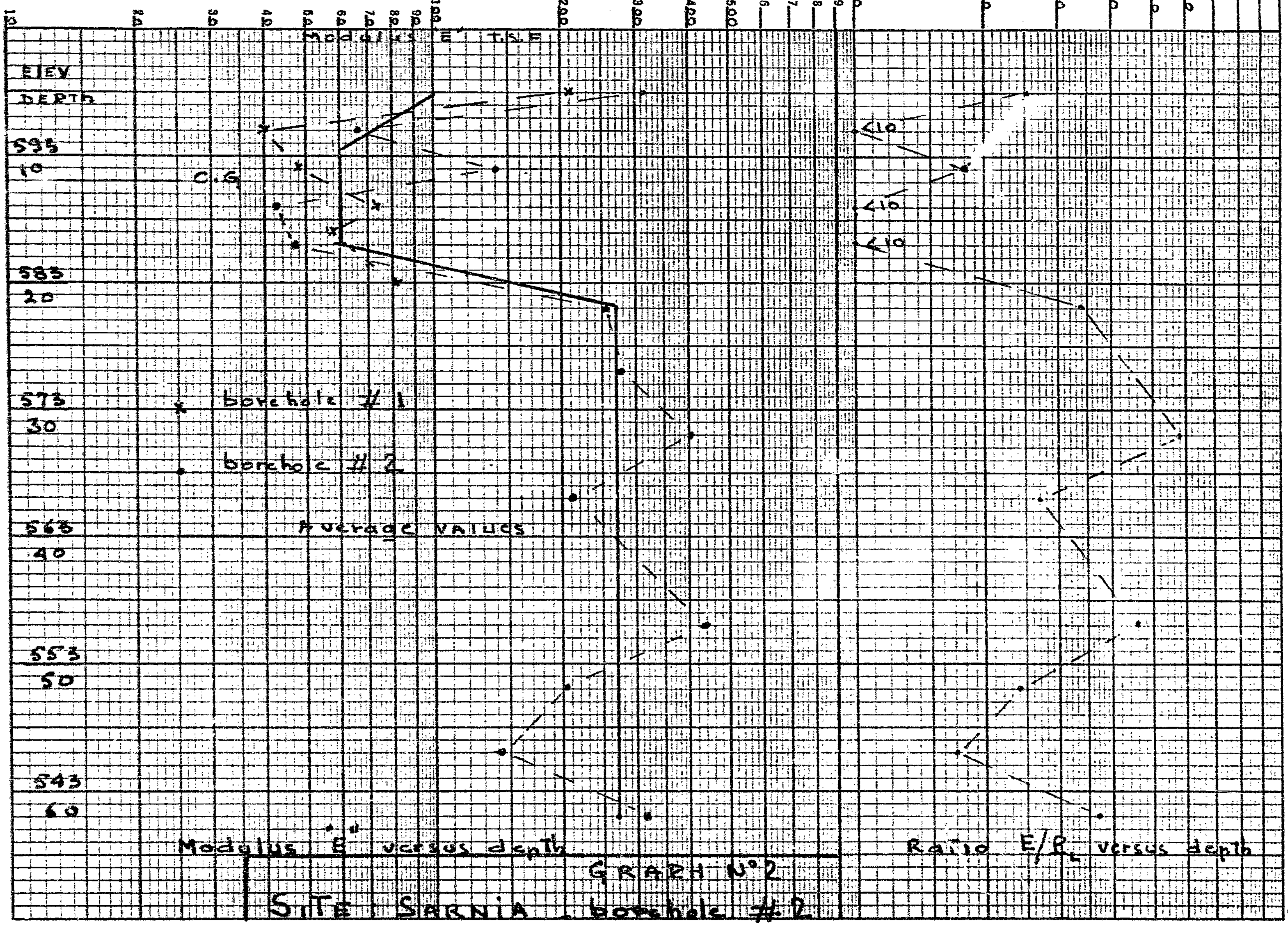
9.0 DISCUSSION

The effects of driving the piles into this soil which exhibits a fragile structure are unknown; the probable effect will be a reduction of the values of the modulus "E" which govern the settlement of the piles. One should therefore anticipate settlements somewhat larger than those calculated if the piles are tested shortly after driving.

GEOPROBE ONTARIO

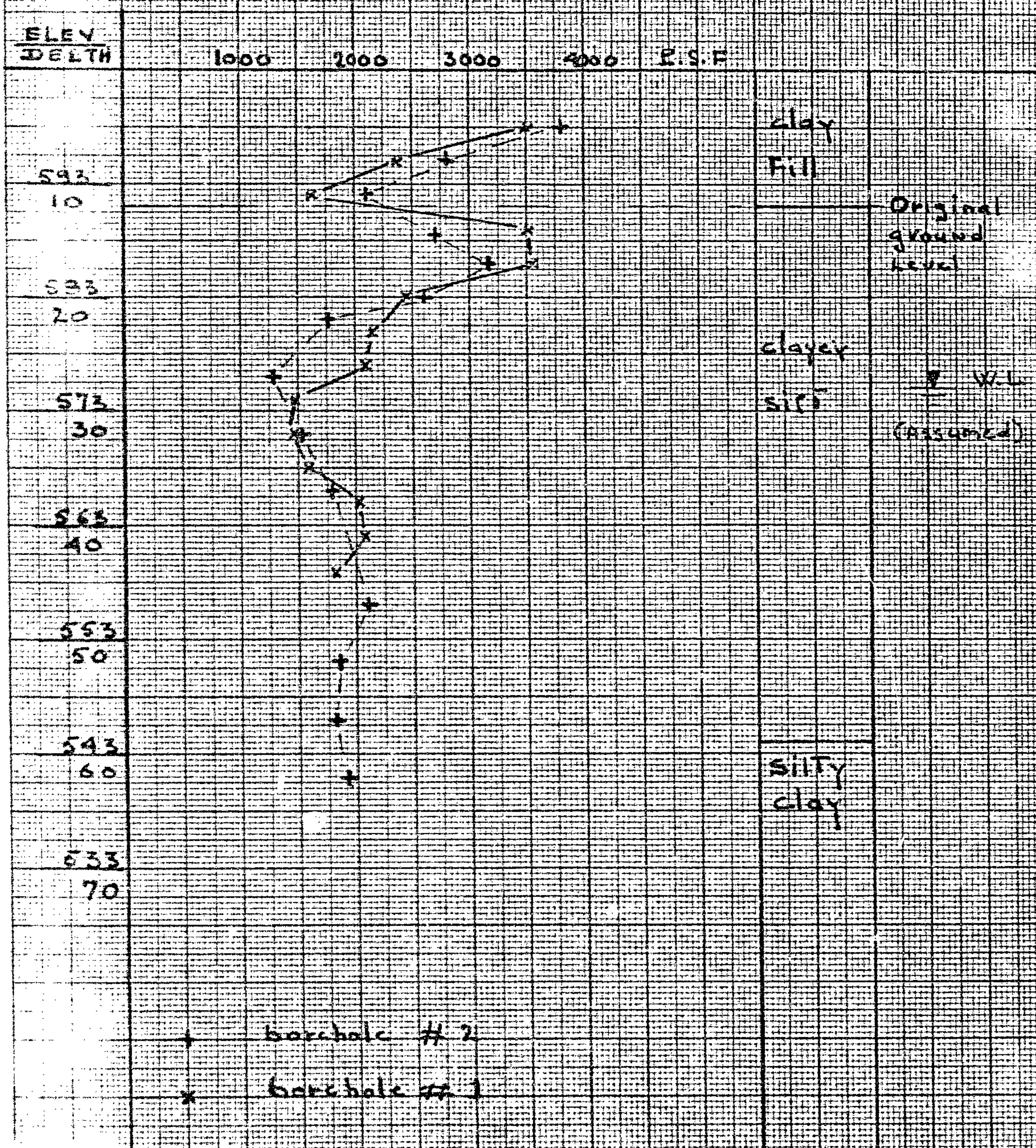
Y. BROISE


NET LIMIT PRESSURE P_L'
borehole # 2
Elev. Depth
$P_L' = P_L - P_o$
5 6 7 8 9 10 11 12 13 T.S.F
593
10
583
20
573
30
563
40
553
50
543
60
GRAPH N° 1
SITE SARNIA


Modulus E T.S.F
ELEV
DEPTH
595
10
C.G
585
20
575
30
565
40
555
50
545
60
borehole #1
borehole #2
Average values
<10
<10
<10
Modulus E versus depth
Ratio E/PL versus depth
GRAPH N°2
SITE : SARNIA . borehole #2

Site Sarnia

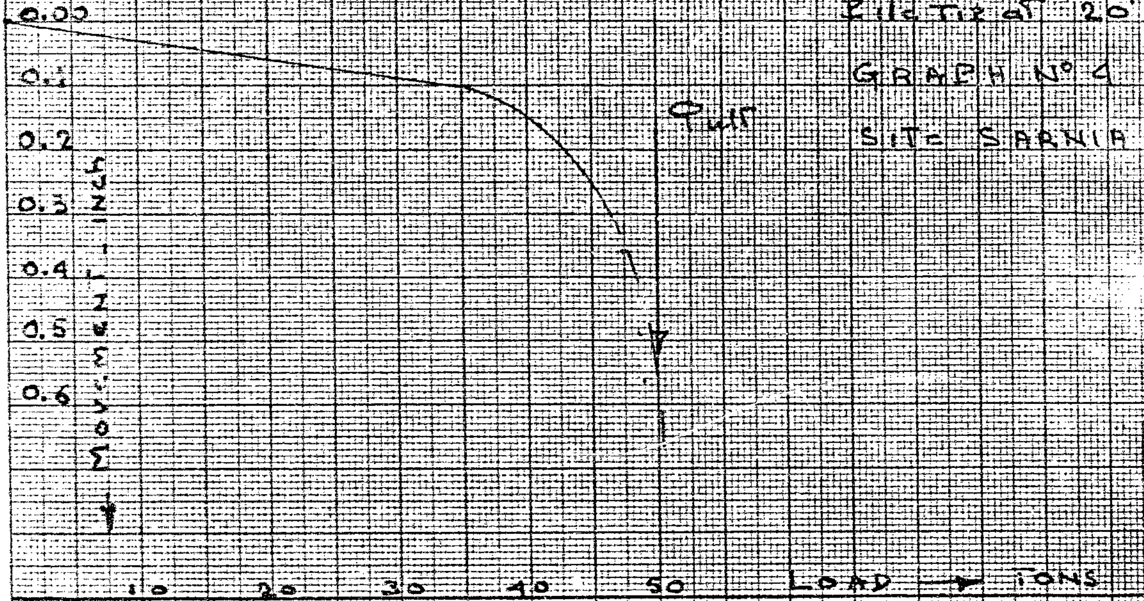
Shear strength / depth



GRAPH N° 3
Site: SARNIA

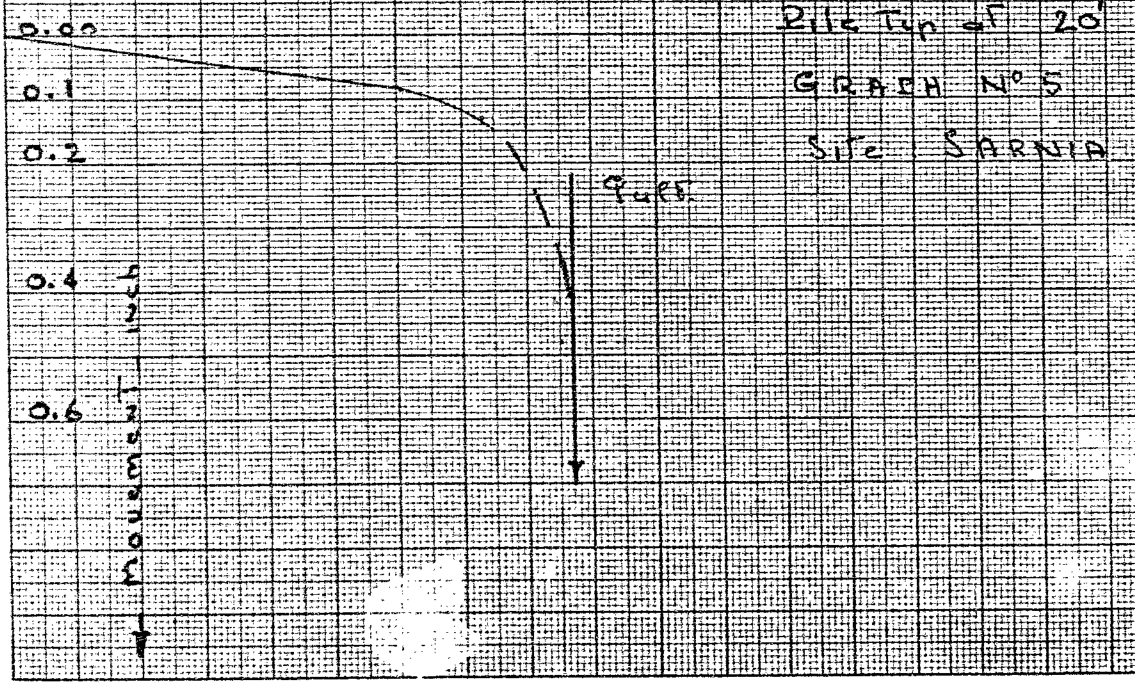
ANTICIPATED LOAD/SETTLEMENT

12" "Hercules" Pile



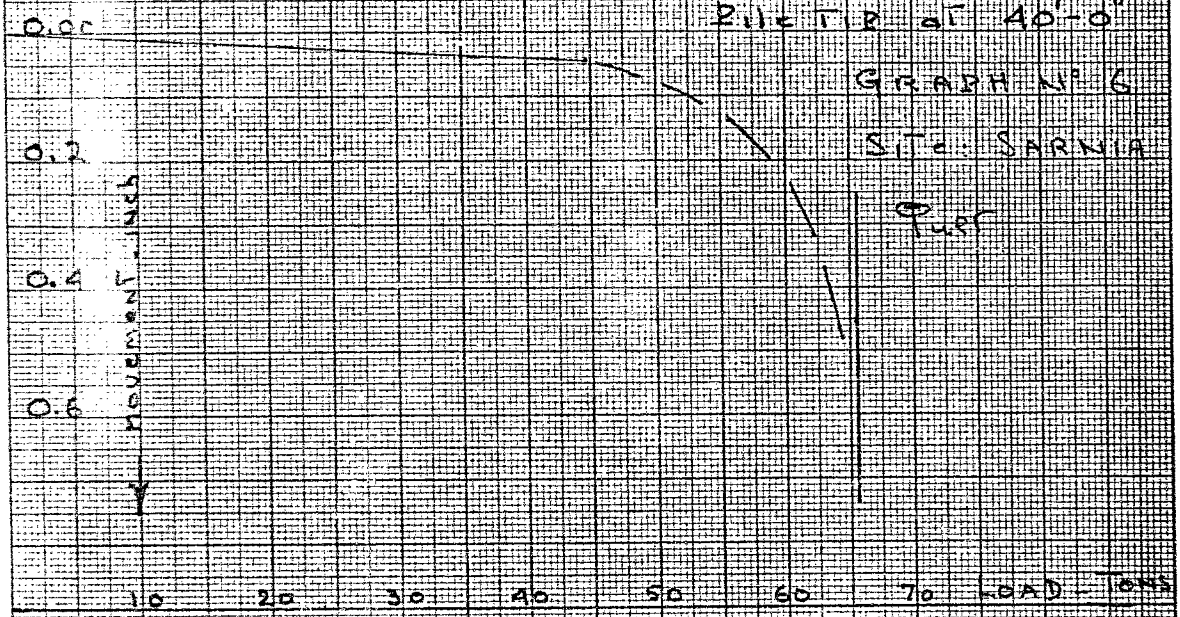
ANTICIPATED LOAD/SETTLEMENT

12".3/4 Tubular steel Pile



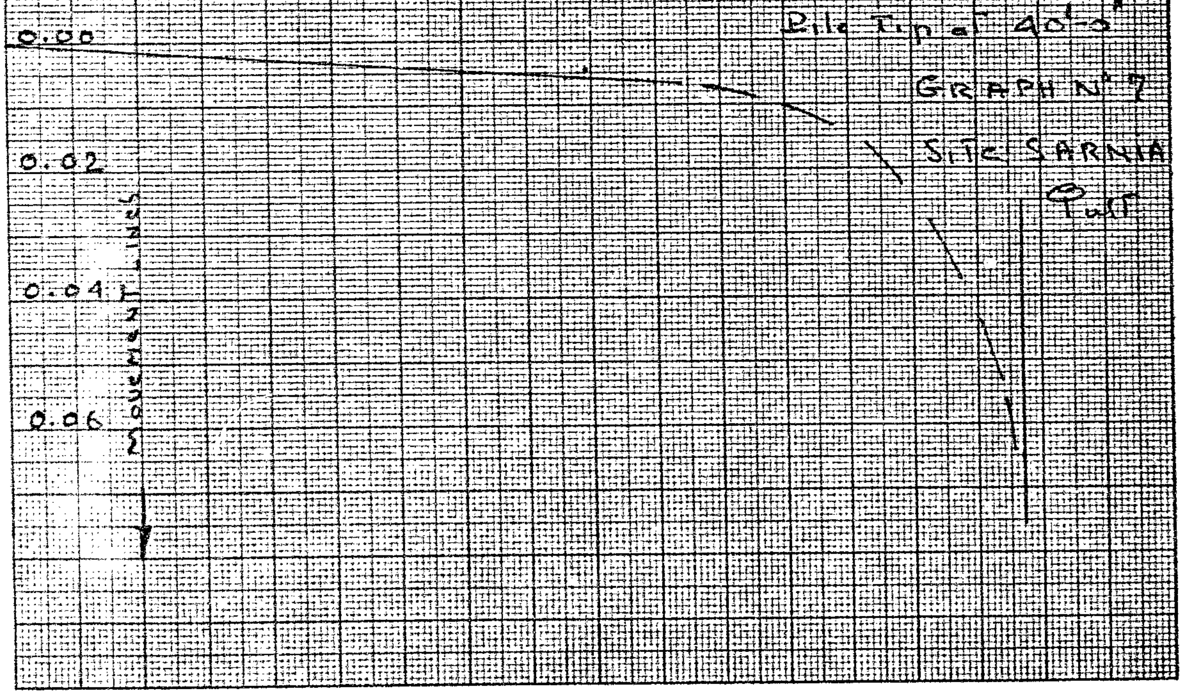
ANTICIPATED LOAD/SETTLEMENT

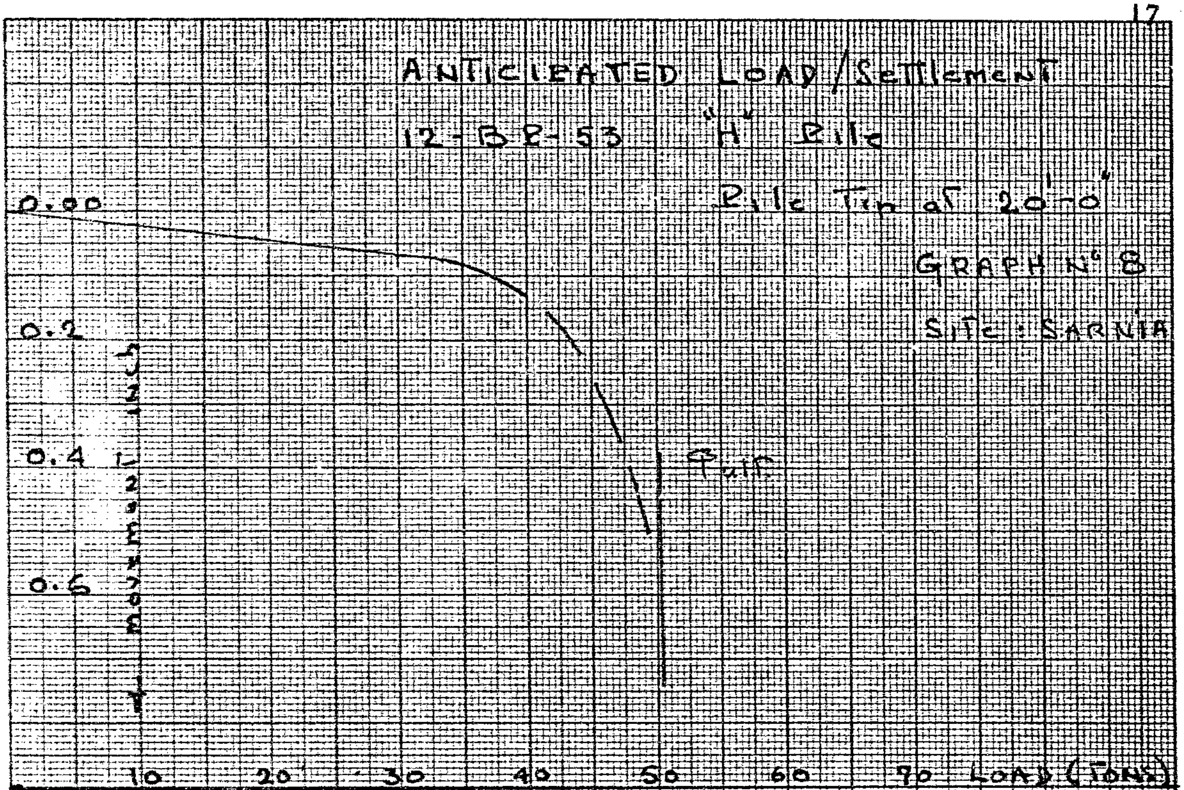
12" 3/4 Tubular Steel Pile

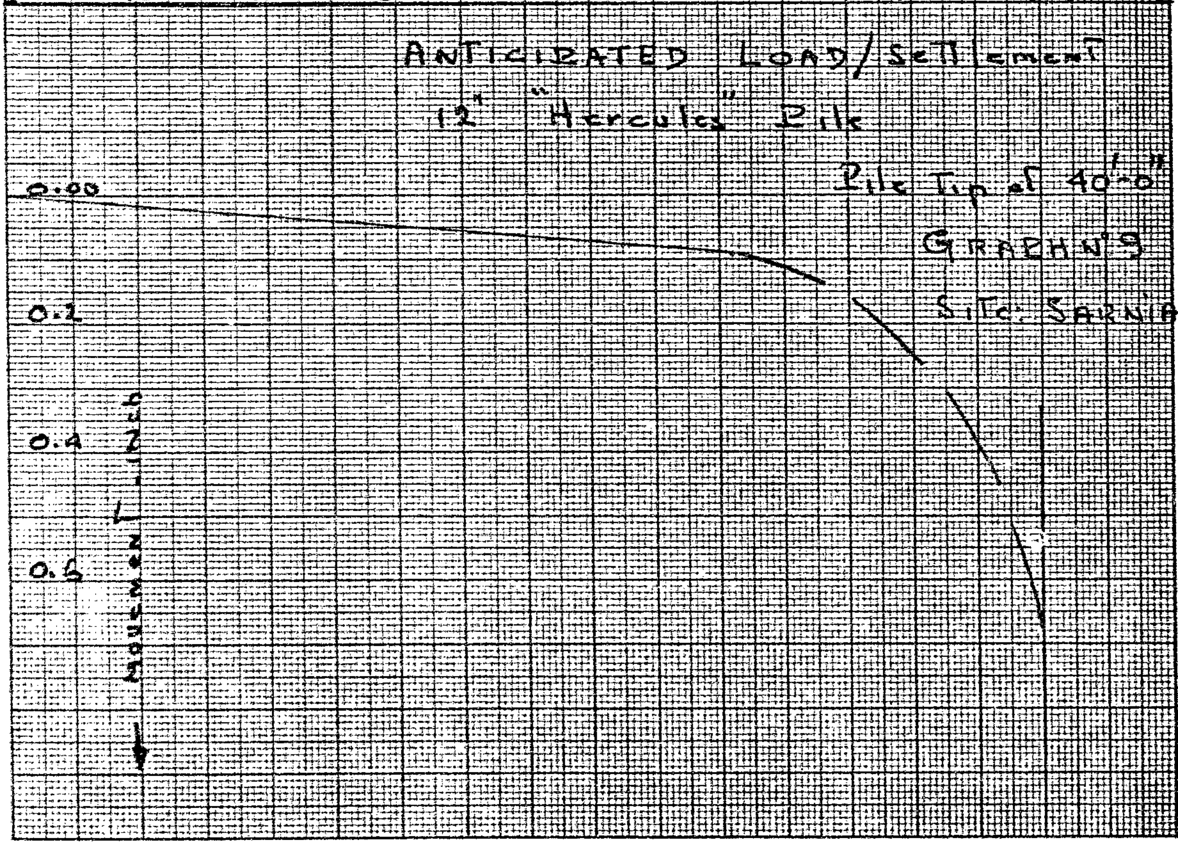


ANTICIPATED LOAD/SETTLEMENT

12-BP-53 "H" Pile




ANTICIPATED LOAD/SETTLEMENT
12-BP-53 "H" Pile
Pile Tip at 20'-0"
GRAPH No 8
SITE: SARNIA
0.00
0.2
0.4
0.6
MOVEMENT INCH
Pult
10
20
30
40
50
60
70
LOAD (TONS)


ANTICIPATED LOAD/SETTLEMENT
12" "Hercules" Pile
Pile Tip at 40'-0"
GRAPH No 9
SITE: SARNIA
0.00
0.2
0.4
0.6
MOVEMENT INCH

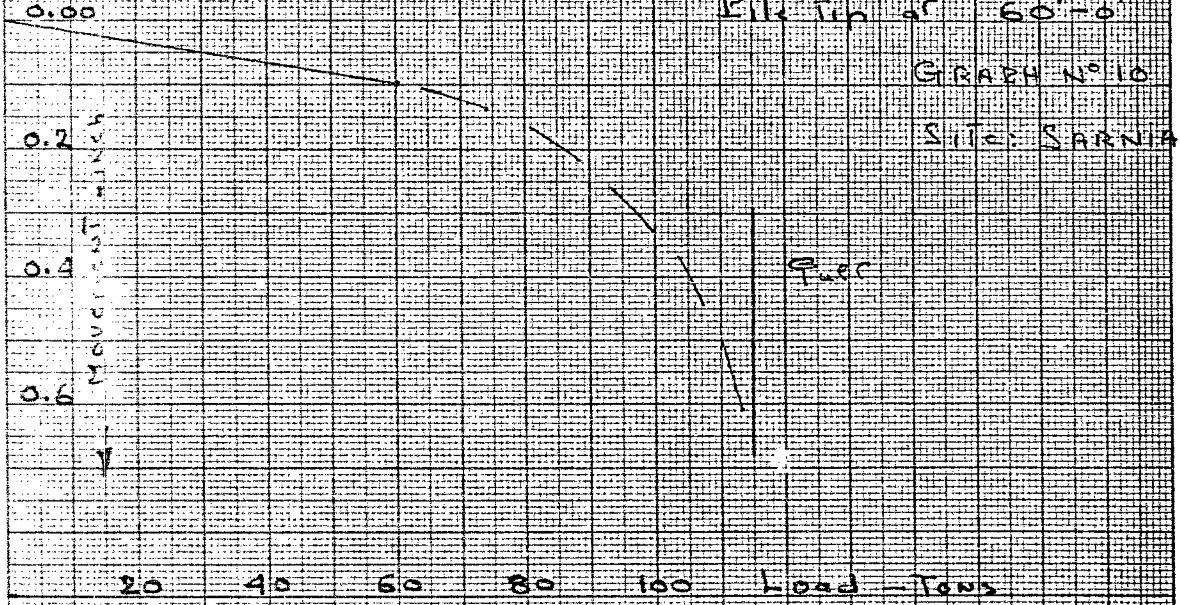
ANTICIPATED LOAD/SETTLEMENT

12" "Hercules" Pile

Pile Tip at 60'-0"

GRAPH No 10

Site: SARNIA



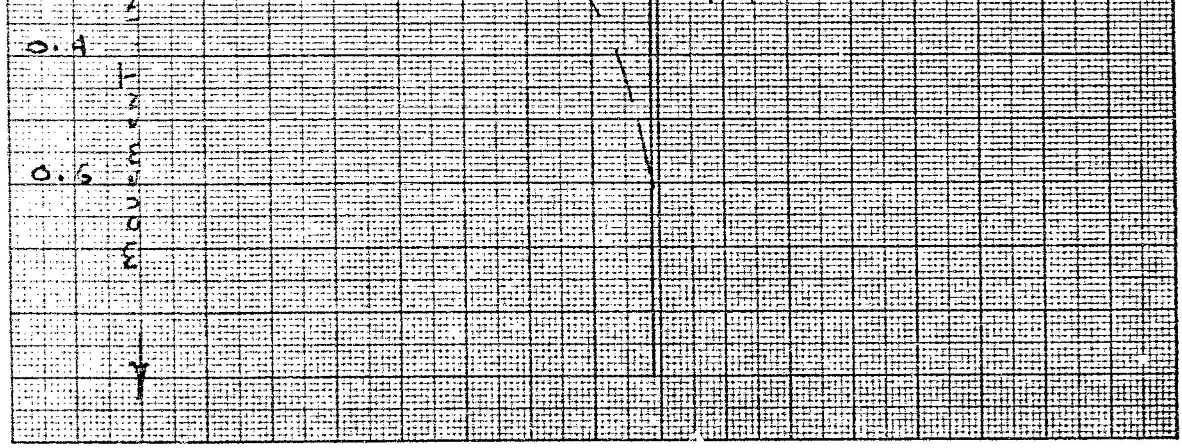
ANTICIPATED LOAD/SETTLEMENT

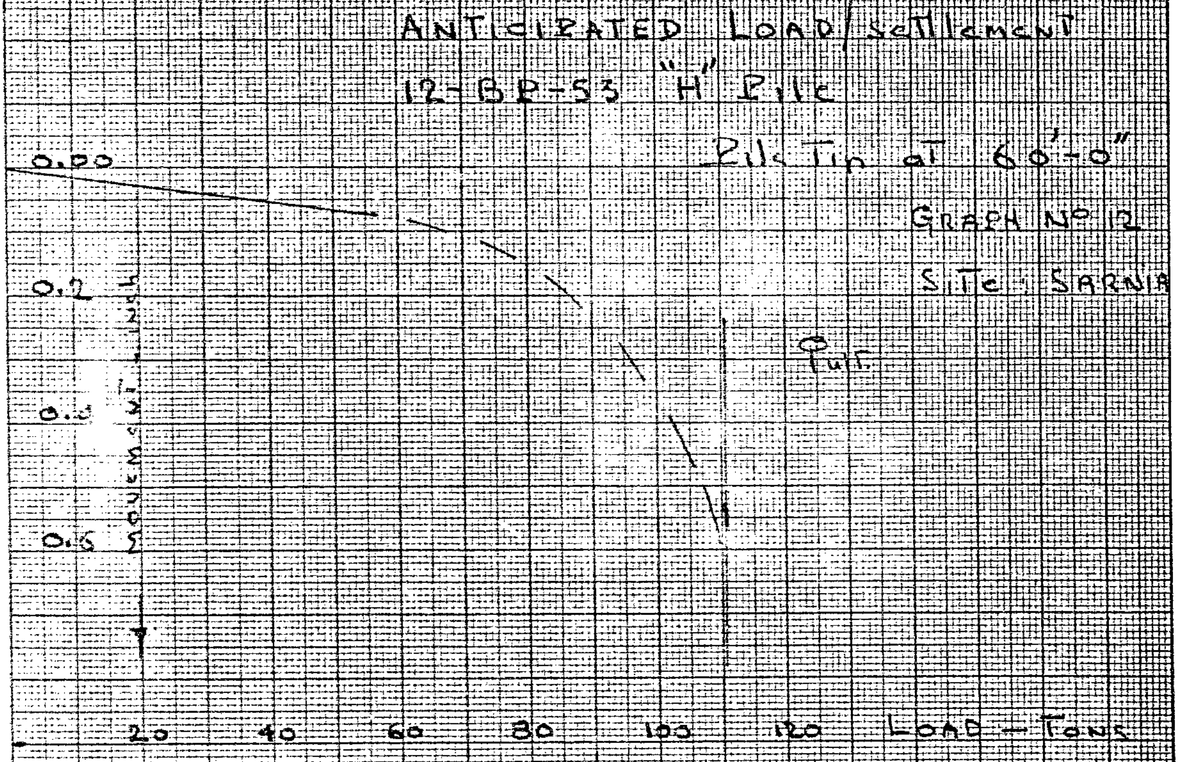
12" 3/4 Tubular Steel Pile

Pile Tip at 60'-0"

GRAPH No 11

Site: SARNIA




ANTICIPATED LOAD/SETTLEMENT
12-BP-53 "H" PILE
PILE TIP AT 60'-0"
GRAPH No 12
SITE: SARNIA
0.00
0.2
0.4
0.6
MOVEMENT - INCH
Pult.
20
40
60
80
100
120
LOAD - TONS

SUMMARY OF RESULTS

Depth of Pile Tip	20 FEET		
	Allow. Load Tons	Pseudo-elastic Limit Tons	Ultimate Load Tons
Tubular Steel Pile 12" 3/4	19.45	30.4	43.5
Concrete Pile "Hercules" 12"	23	35.8	50.
H Pile 12-BP-53	22.6	34	50.

SUMMARY OF RESULTS

Depth of Pile Tip	40 FEET		
	Allow. Load Tons	Pseudo-elastic Limit Tons	Ultimate Load Tons
Tubular Steel Pile 12" 3/4	30.7	44.8	65.7
Concrete Pile "Hercules" 12"	38.18	55.7	79.5
H Pile 12-BP-53	37.1	52	78.5

SUMMARY OF RESULTS

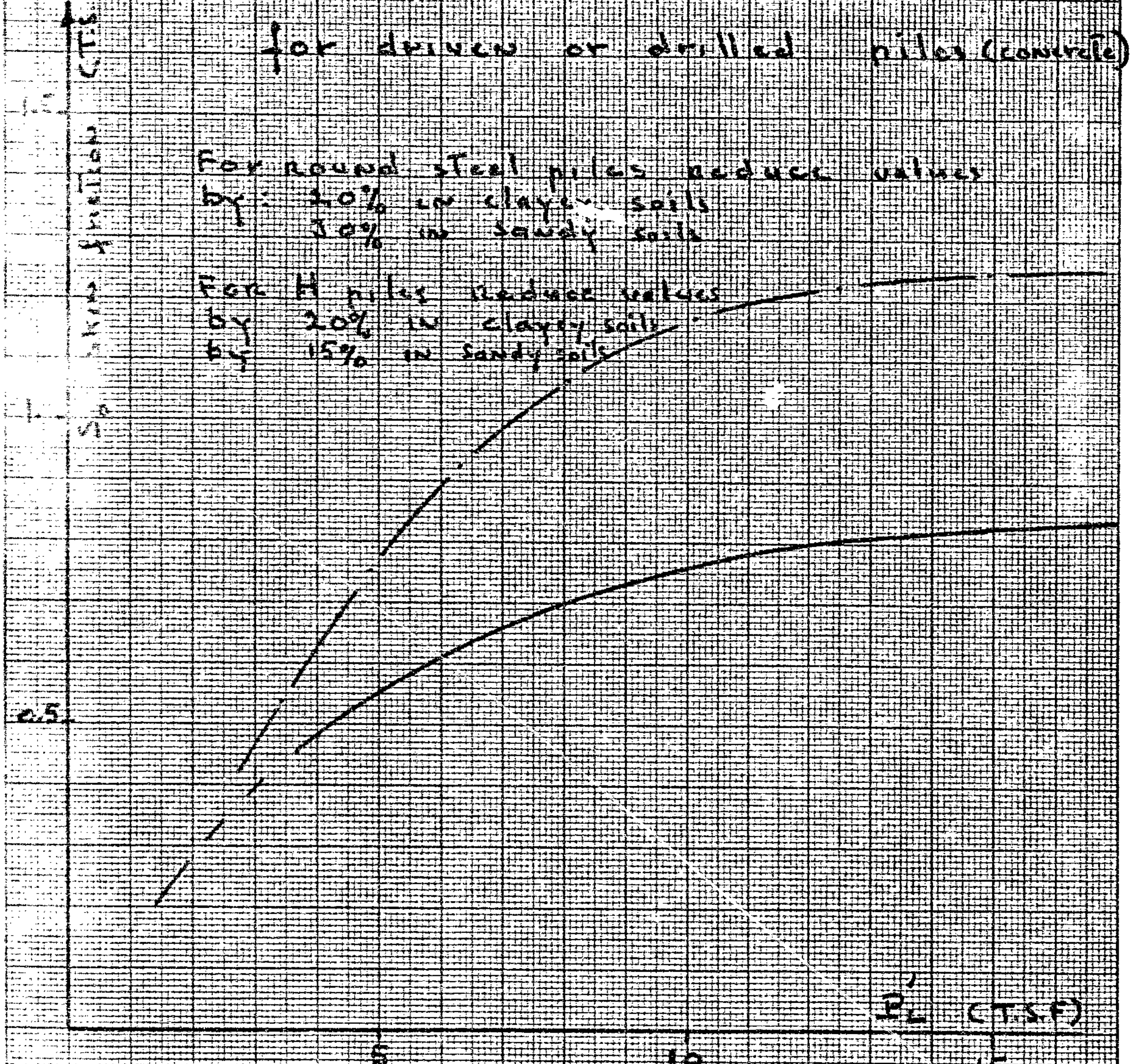
Depth of Pile Tip	60 FEET		
	Allow. Load Tons	Pseudo-elastic Limit Tons	Ultimate Load Tons
Tubular Steel Pile 12" 3/4	47.5	50.5	99
Concrete Pile "Hercules" 12"	55.8	60.4	115
H Pile 12-BP-53	53.1	56.5	110.5

APPENDIX "A"

Skin friction
for driven or drilled piles (concrete)

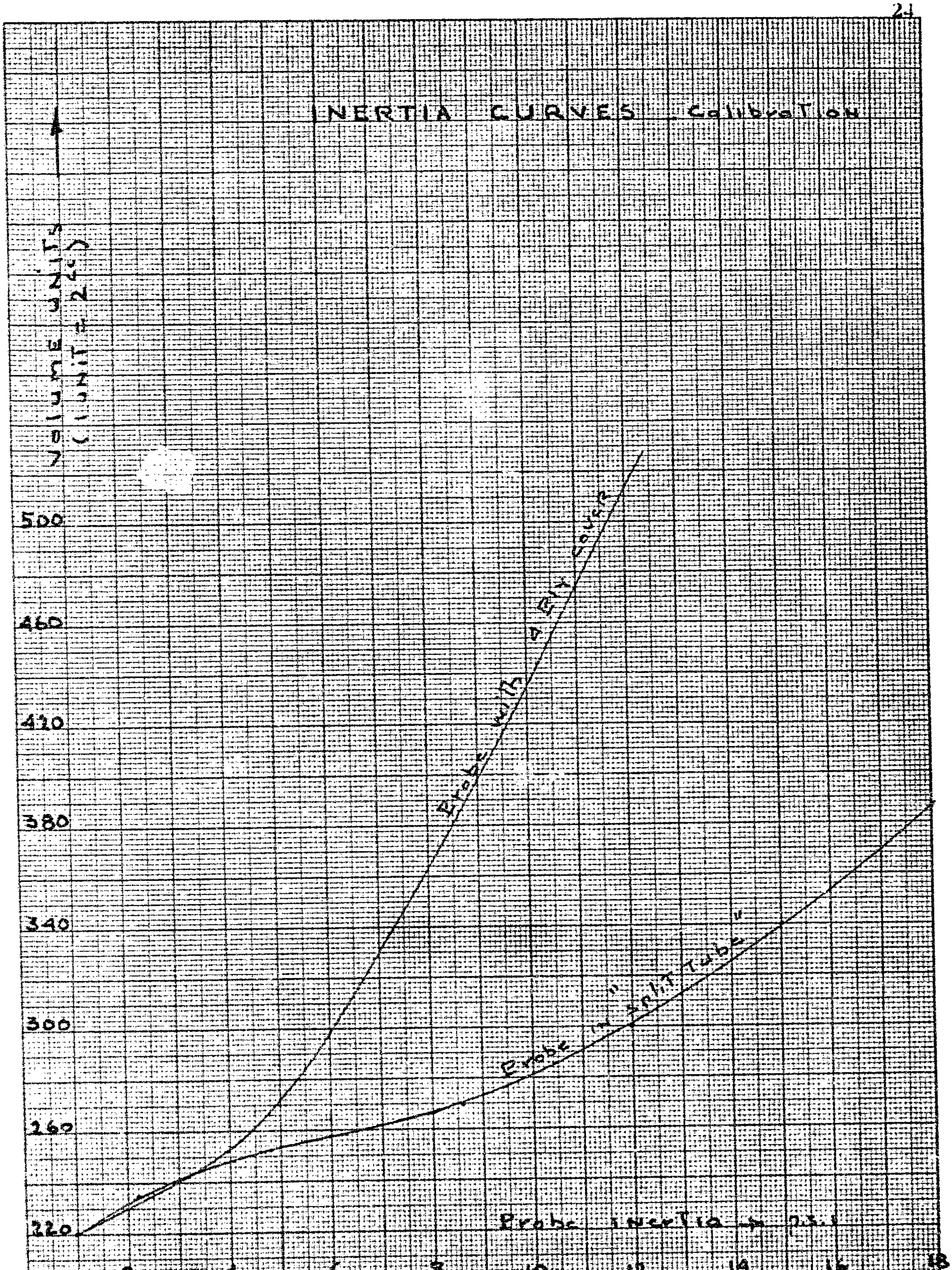
For round steel piles reduce values
by: 20% in clayey soils
30% in sandy soils

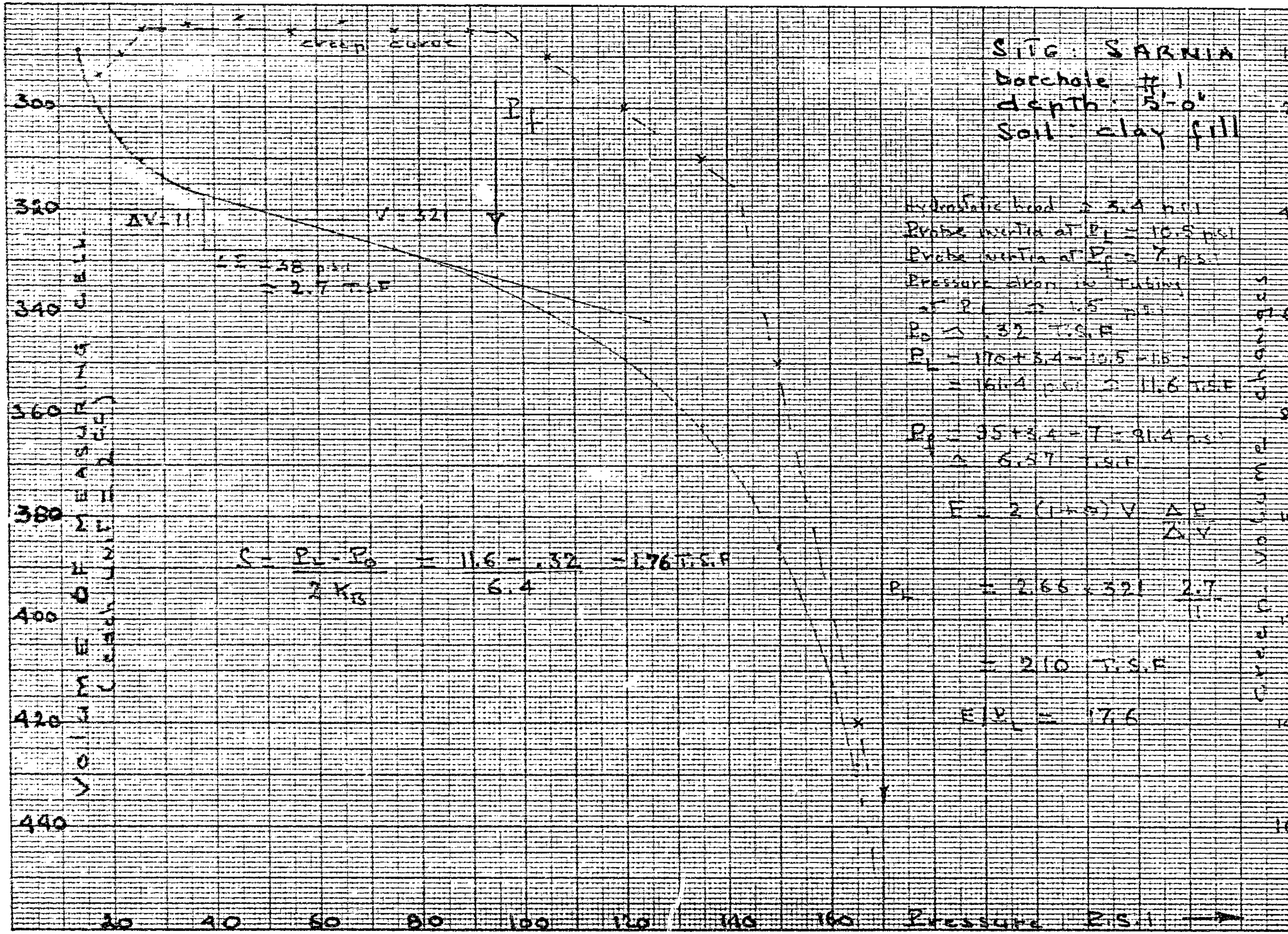
For H piles reduce values
by 20% in clayey soils
by 15% in sandy soils

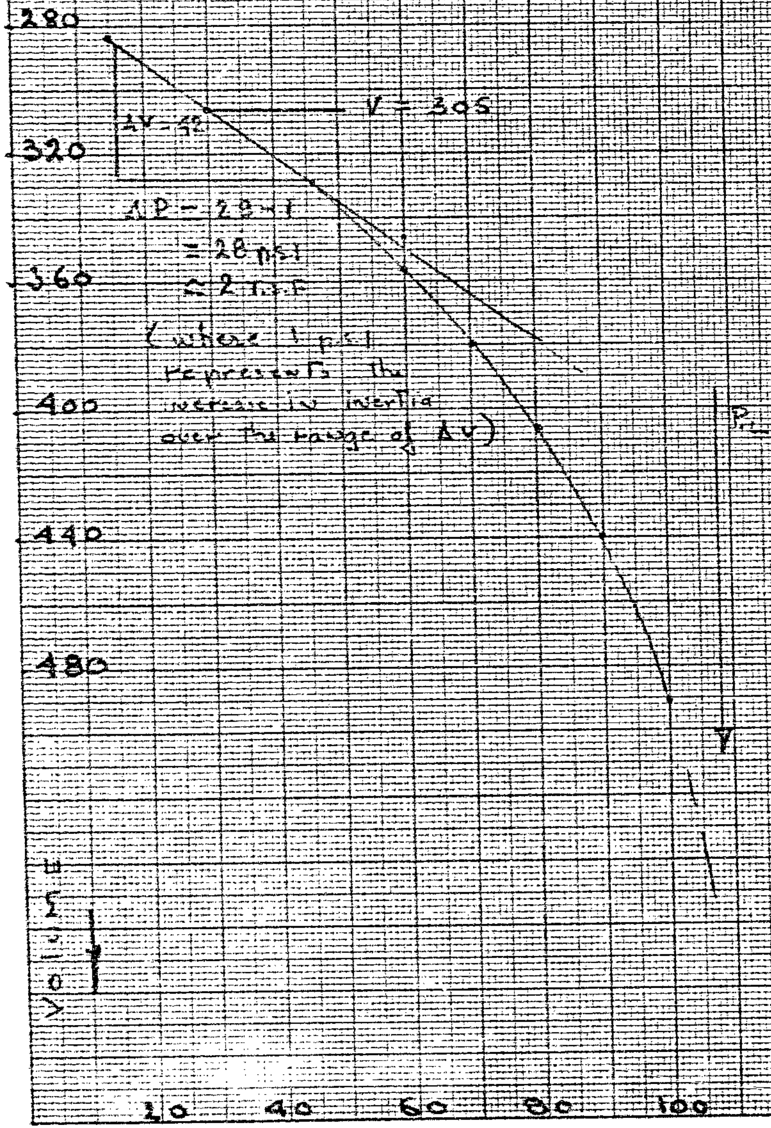


— — — skin friction within 3 diam. of Tip
——— skin friction along shaft

Fig 14


INERTIA CURVES Calibration
Volume Units
(1 unit = 2cc)
500
460
420
380
340
300
260
220
2
4
6
8
10
12
14
16
18
Probe with 4 ply cover
Probe in "split tube"
Probe inertia


Site: SARNIA
Borehole #1
depth: 5'-0"
Soil: clay fill
creep curve
P_f
ΔV = 11
V = 321
ΔP = 38 p.s.i
= 2.7 T.S.F
Hydrostatic head = 3.4 p.s.i
Probe inertia at P_L = 10.5 p.s.i
Probe inertia at P_f = 7 p.s.i
Pressure drop in Tubing
at P_L = 1.5 p.s.i
P_o = .32 T.S.F
P_L = 170 + 3.4 − 10.5 − 1.5
= 161.4 p.s.i = 11.6 T.S.F
P_f = 95 + 3.4 − 7 = 91.4 p.s.i
= 6.57 T.S.F
$E = 2(1+\nu)V\frac{\Delta P}{\Delta V}$
P_L = 2.66 × 321 $\frac{2.7}{11}$
= 210 T.S.F
E/P_L = 17.6
$C = \frac{P_L - P_o}{2K_B} = \frac{11.6 - .32}{6.4} = 1.76$ T.S.F
300
320
340
360
380
400
420
440
VOLUME OF MEASURING CELL (cc)
20
40
60
80
100
120
140
160
Pressure P.S.I



Site: SARNIA
Borehole # 1
depth: 8'-0"
Soil: clay fill

Hydrostatic head = 4.8 p.s.i
Probe inertia at P_L = 13 p.s.i
Pressure drop in tubing = 2 p.s.i
P_o = .52 T.S.F

P_L = 108 + 4.8 - 13 - 2 = 97.8 p.s.i = 6.95 T.S.F

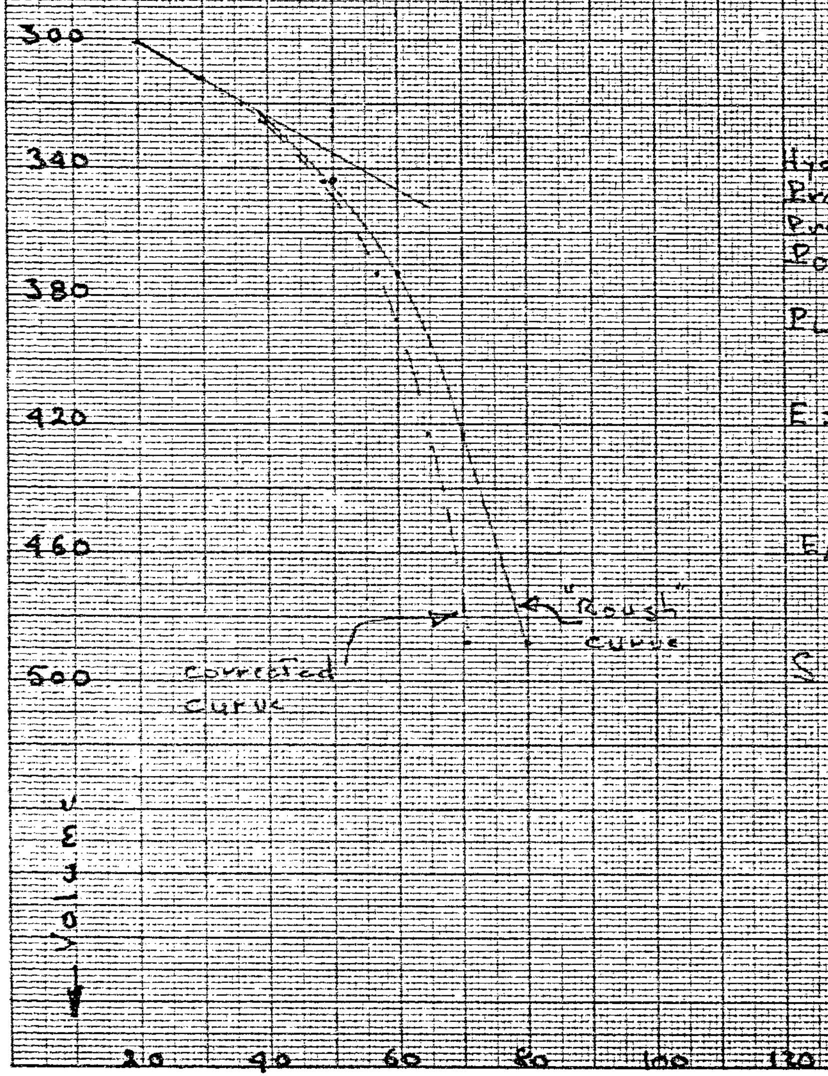
$= 2.66 \times 306 \; \frac{2}{42} = 40$ T.S.F

$E/P_L = 5.75$

$S = \frac{P_L - P_o}{2 K_B} = \frac{6.95 - .52}{5.4} = 1.19$ T.S.F

Pressure (p.s.i) ⟶

SARNIA
borehole # 1
depth 11'-0"



Hydrostatic head = 5.8 p.s.i
Probe inertia at P_L = 11.3 p.s.i
Pressure drop in tubing at P_L = 4 p.s.i
P_0 = .68 T.S.F

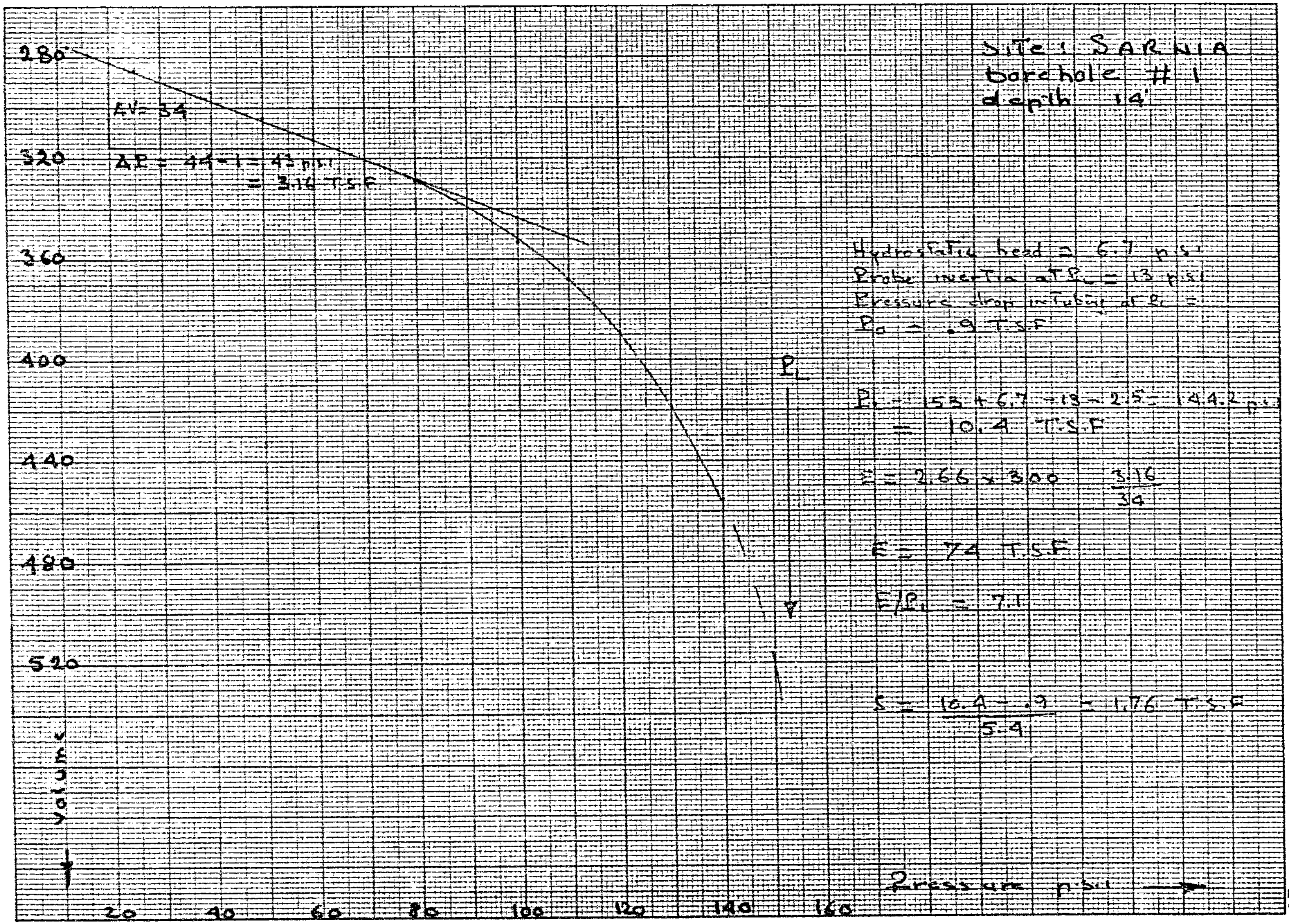
$P_L = 80 + 5.8 - 11.3 - 4 = 70.5$ p.s.i = 5.07 T.S.F

$E = 2.66 \times 310 \frac{1}{17} = 48.5$ T.S.F

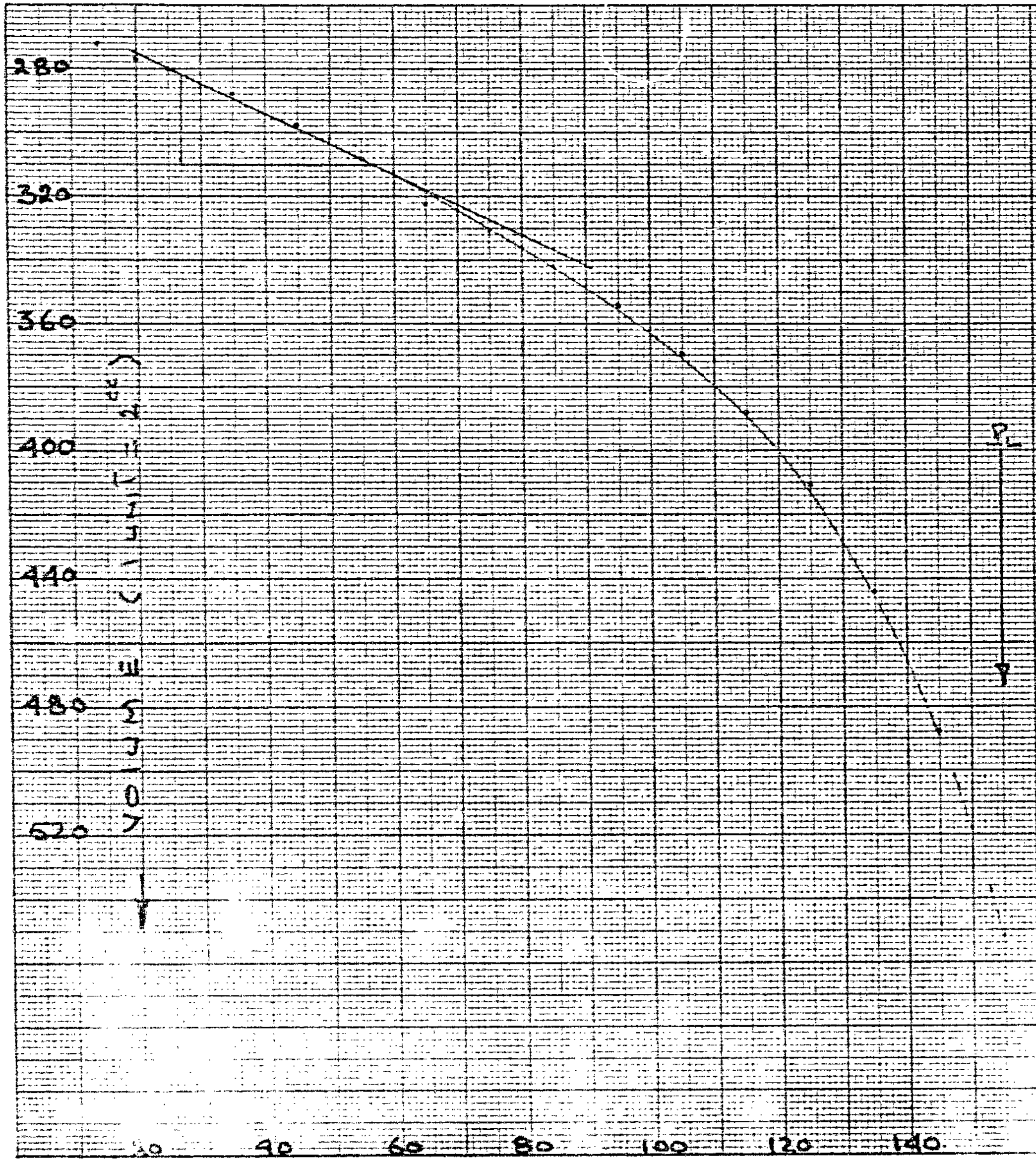
$E/P_L = 9.5$

$S = \frac{P_L - P_0}{2K_n} = \frac{5.07 - .68}{5.4} = .815$ T.S.F

K+E 10 X 10 TO THE CENTIMETER 18 X 25 CM. 46 1513 MADE IN U.S.A.
KEUFFEL & ESSER CO.



SITE: SARNIA
borehole # 1
depth: 17'-0"



Hydrostatic head = 8.6 p.s.i
Probe inertia at P_L = 13 p.s.i
pressure drop in tubing = 2.5 p.s.i
P_0 = 1.08 T.S.F

P_L = 155 + 8.6 − 13 − 2.5 = 148.1 p.s.i

P_L = 10.7 T.S.F

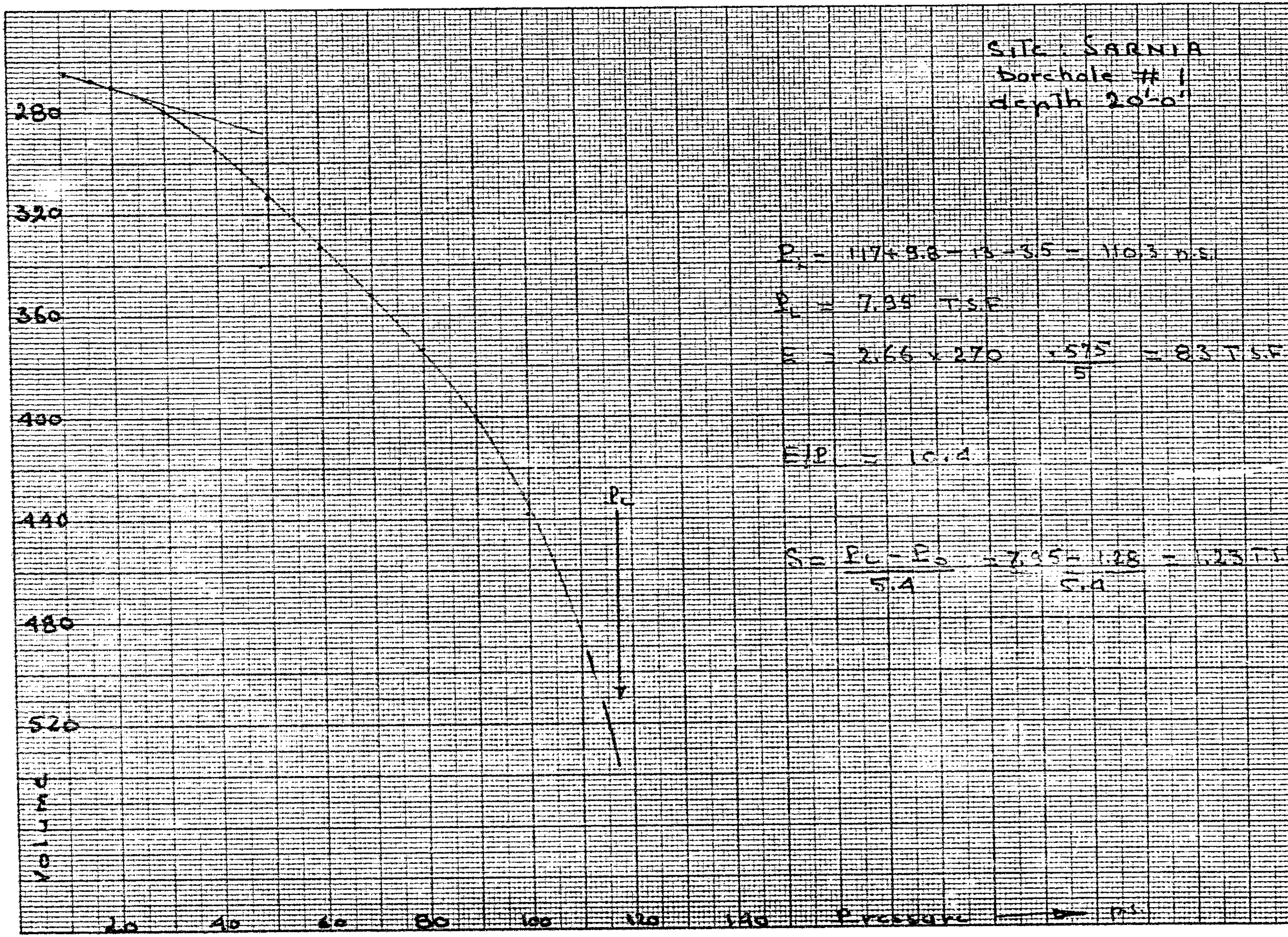
$E = 2.66 \times 296 \; \frac{2.08}{28}$

E = 58.5 T.S.F

E/P_L = 5.5

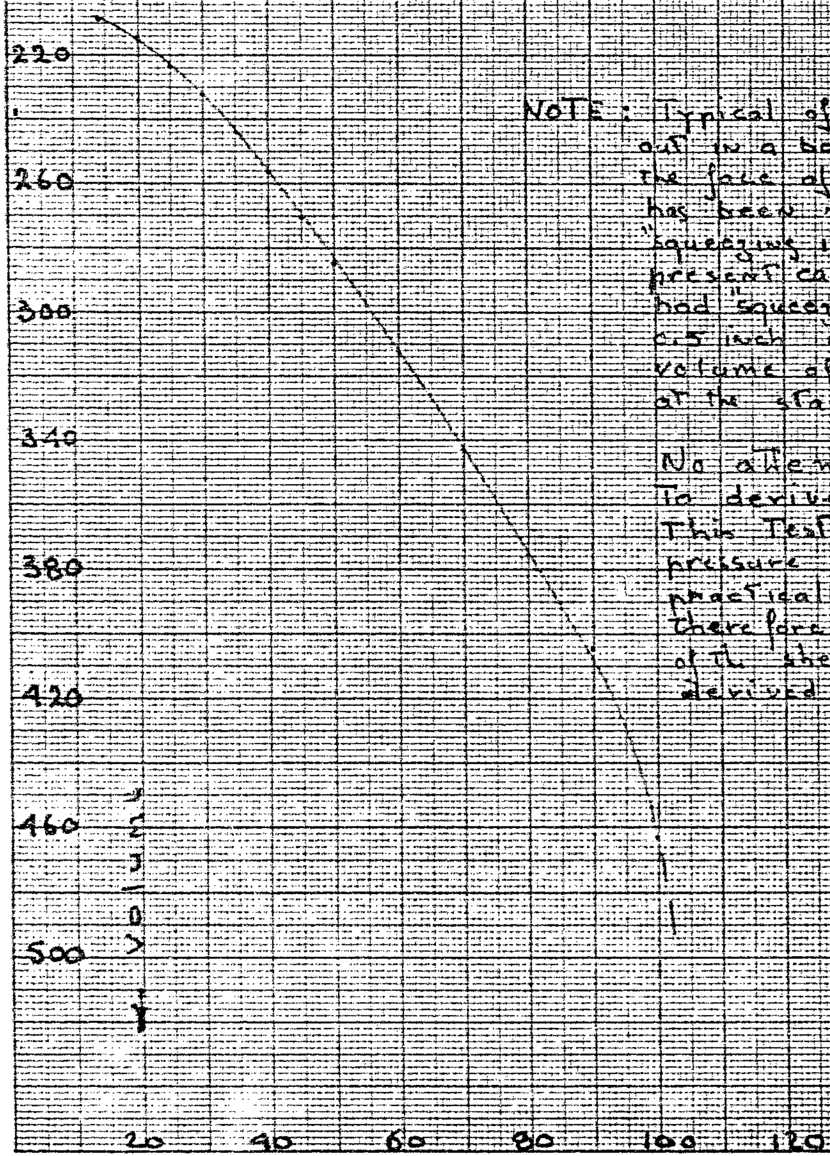
$S = \frac{10.7 - 1.08}{5.4}$ = 1.78 T.S.F

Pressure p.s.i →


Site : SARNIA
Borehole # 1
depth 20'-0"
$P_L = 117 + 9.8 - 13 - 3.5 = 110.3$ p.s.i.
$P_L = 7.95$ T.S.F
$E = 2.66 \times 270 \frac{575}{5} = 83$ T.S.F
$E/P_L = 10.4$
$S = \frac{P_L - P_o}{5.4} = \frac{7.95 - 1.28}{5.4} = 1.23$ T.S.F
P_L
280
320
360
400
440
480
520
VOLUME
20
40
60
80
100
120
140
Pressure p.s.i.

K+E 10 X 10 TO THE CENTIMETER 46 1513
18 X 25 CM. MADE IN U.S.A.
KEUFFEL & ESSER CO.

Site: SARNIA
Borehole # 1
depth 23'-0"



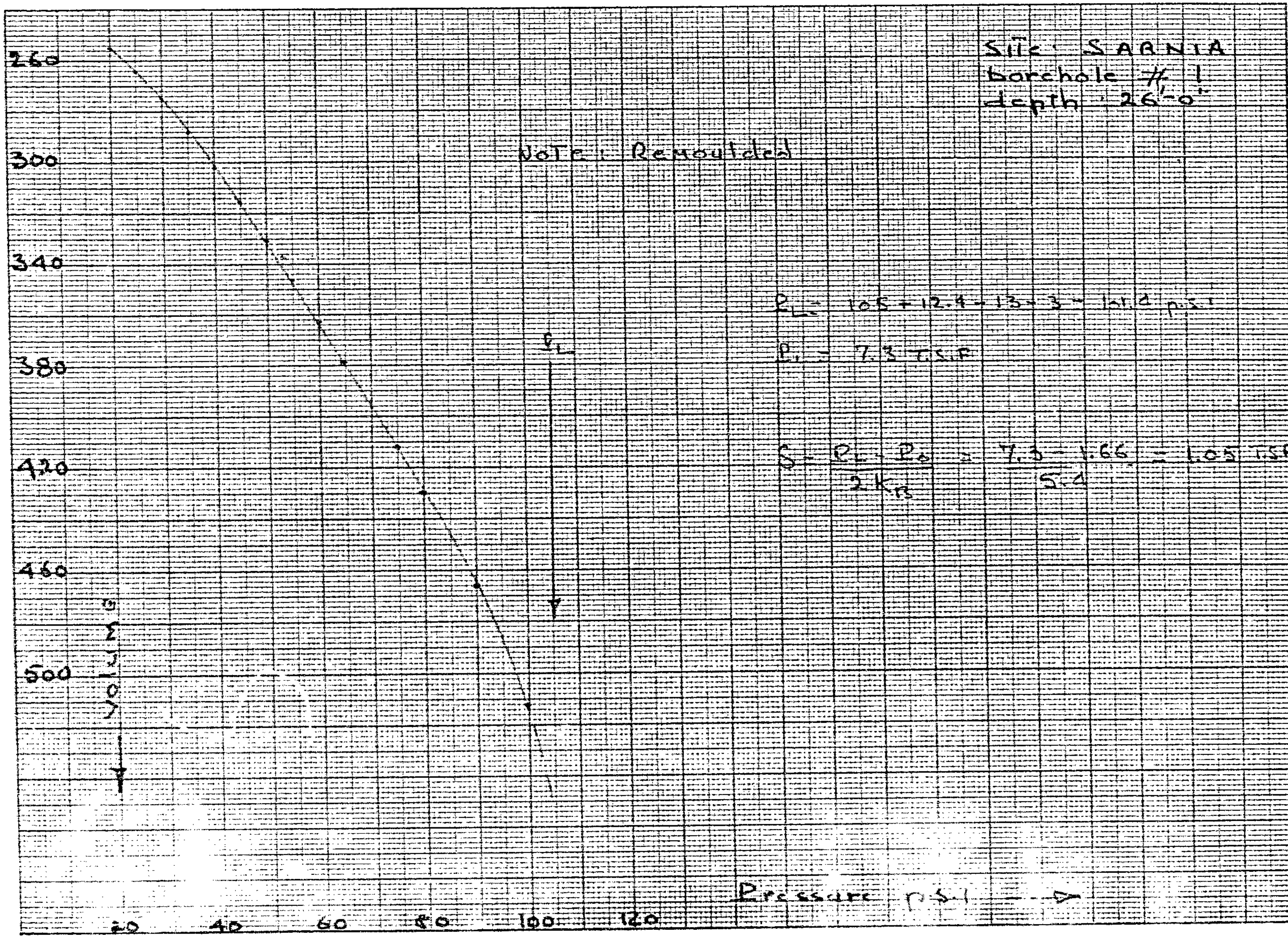
NOTE: Typical of a test carried out in a borehole where the face of the borehole has been remoulded by "squeezing in". In the present case the borehole had "squeezed in" approximately 0.5 inch as indicated by the volume of the measuring cell at the start of the test.

No attempt should be made to derive a Modulus from this test. However the limit pressure P_L will remain practically unaffected and therefore an acceptable value of the shear strength can be derived.

$$P_L = 104 + 11 - 11.6 - 2.5 = 100.9 \text{ psi}$$

$$= 7.25 \text{ T.S.F}$$

$$S = \frac{P_L - P_o}{2K_B} = \frac{7.25 - 1.48}{5.4} = 1.07 \text{ T.S.F}$$


Site: SARNIA
borehole # 1
depth 26'-0"
NOTE: Remoulded
$P_L = 105 + 12.4 - 13 - 3 = 101.4$ p.s.i
$P_L = 7.3$ T.S.F
$S = \frac{P_L - P_o}{2K_B} = \frac{7.3 - 1.66}{5.4} = 1.05$ T.S.F
P_L
260
300
340
380
420
460
500
Volume
20
40
60
80
100
120
Pressure p.s.i

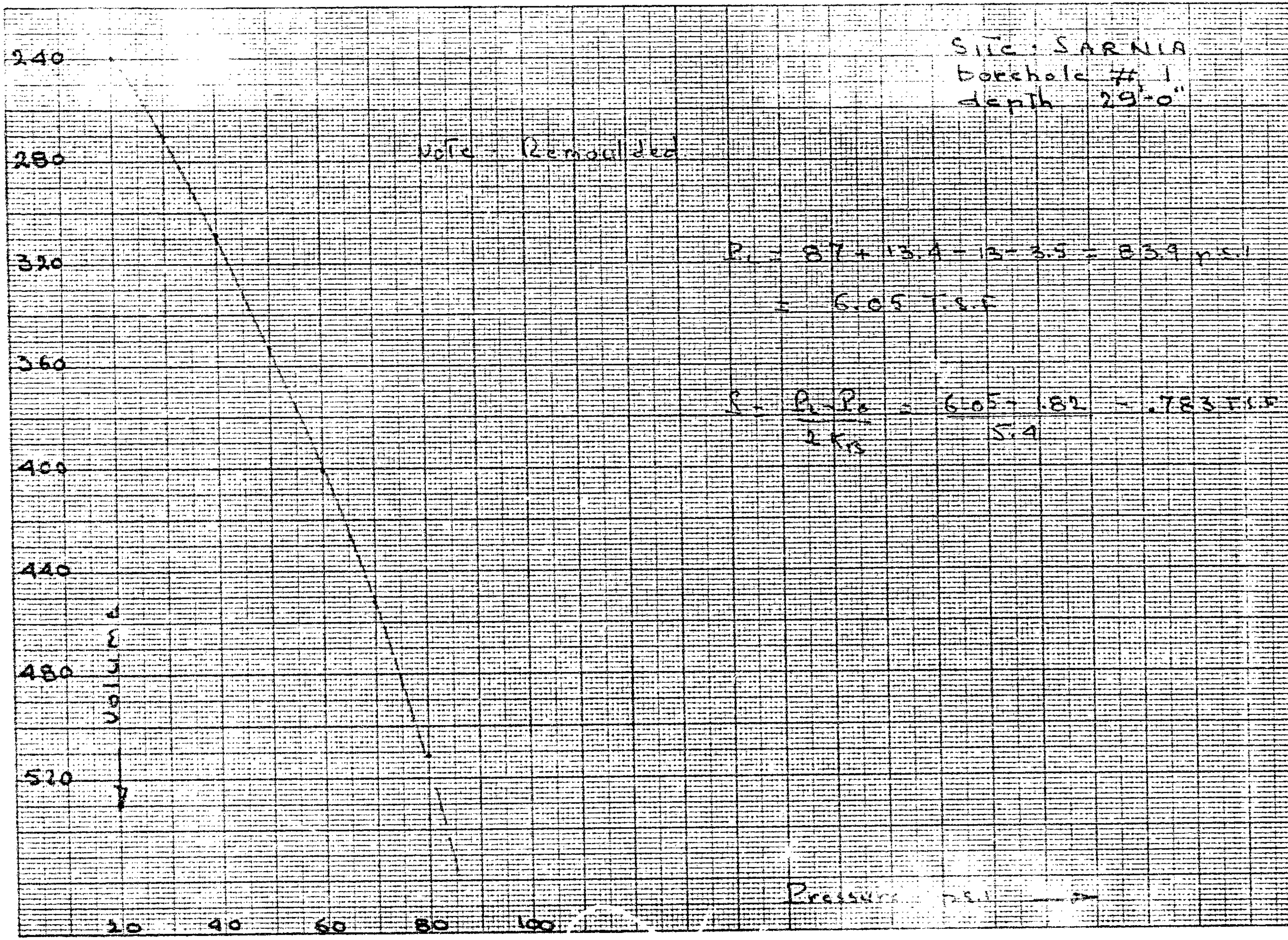
K+E 10 X 10 TO THE CENTIMETER 18 X 25 CM. 46 1513 MADE IN U.S.A. KEUFFEL & ESSER CO.

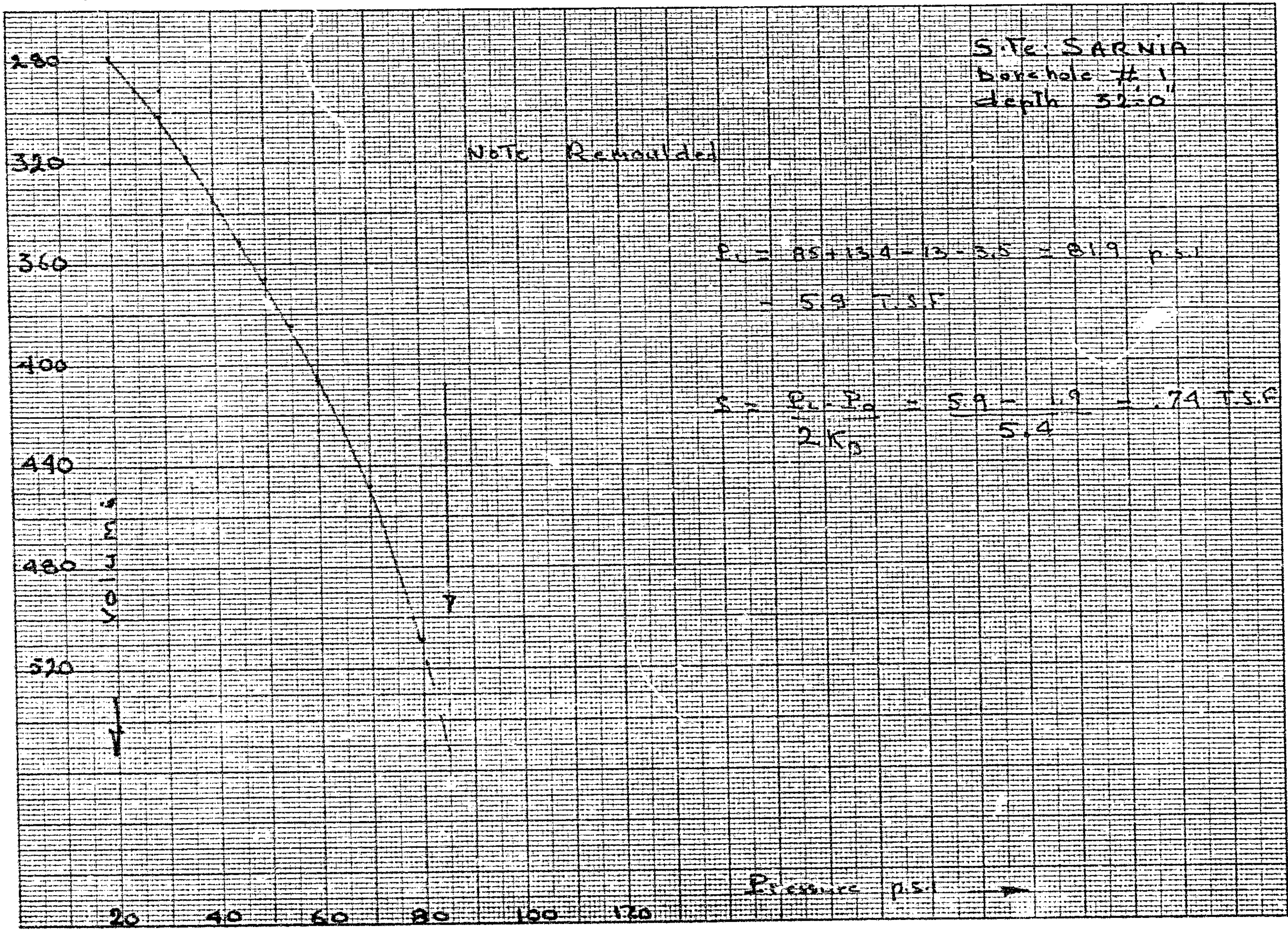
Site: SARNIA
borehole #1
depth 29'-0"

Note: Remoulded

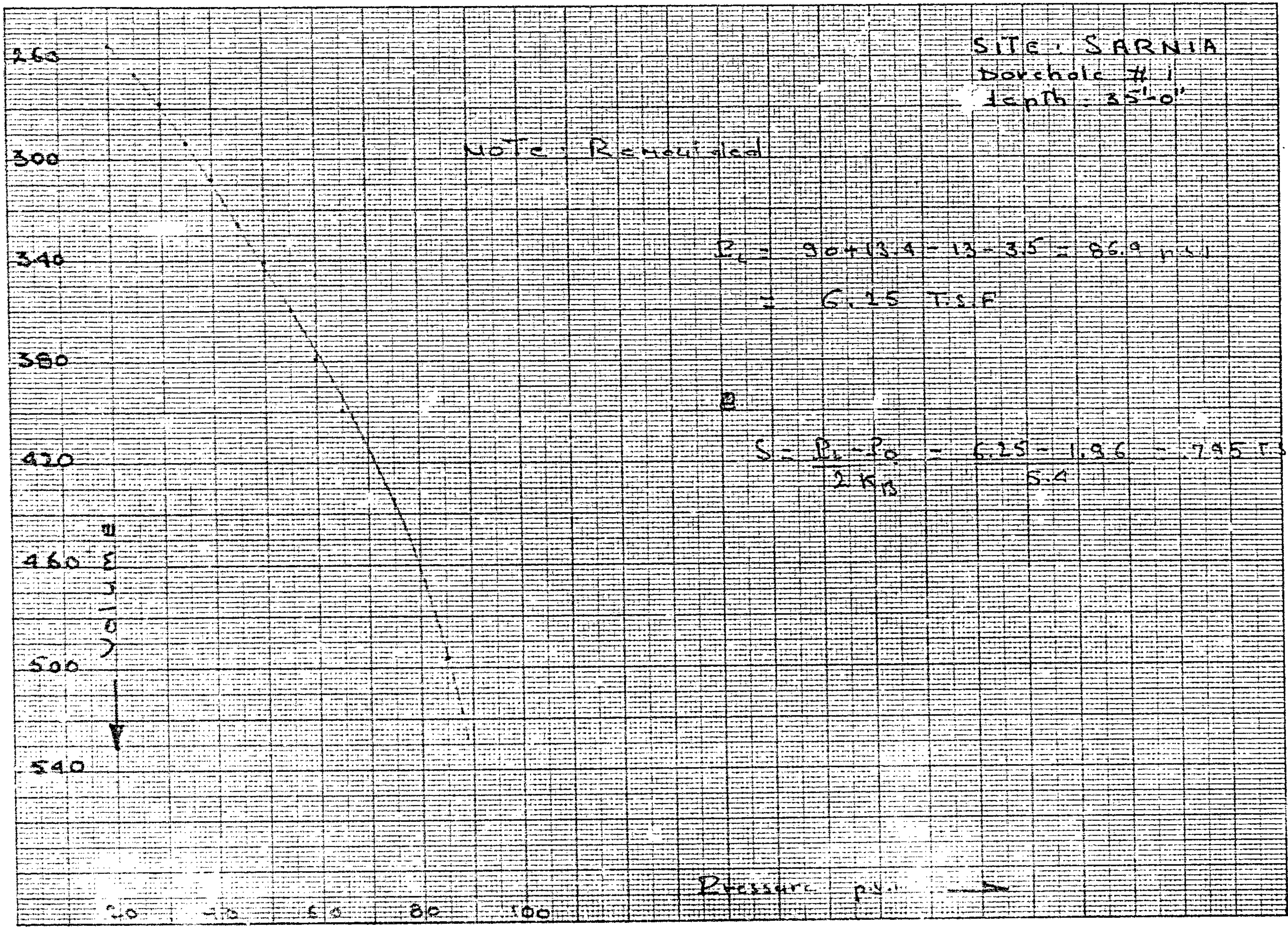
$P_L = 87 + 13.4 - 13 - 3.5 = 83.9$ p.s.i
$= 6.05$ T.S.F

$S = \frac{P_L - P_o}{2 K_b} = \frac{6.05 - 1.82}{5.4} = .783$ T.S.F




Site: SARNIA
Borehole # 1
depth 32'-0"
Note Remoulded
P_L = 85+13.4-13-3.5 = 81.9 p.s.i
= 5.9 T.S.F
S = (P_L - P_o) / 2K_D = (5.9 - 1.9) / 5.4 = .74 T.S.F
280
320
360
400
440
480
520
20
40
60
80
100
120
Pressure p.s.i

34



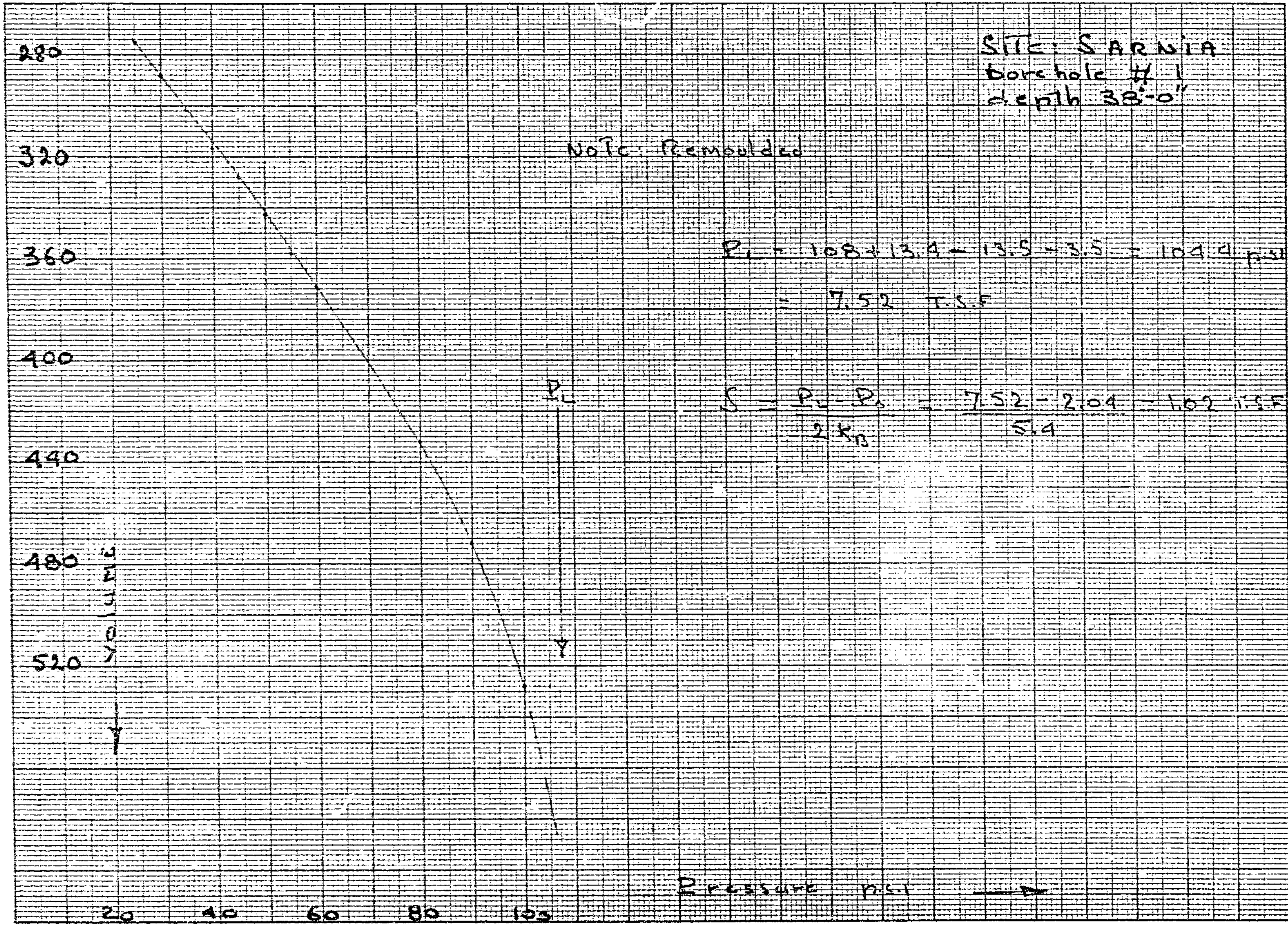
SITE : SARNIA
Borehole # 1
depth : 35'-0"

NOTE : Remoulded

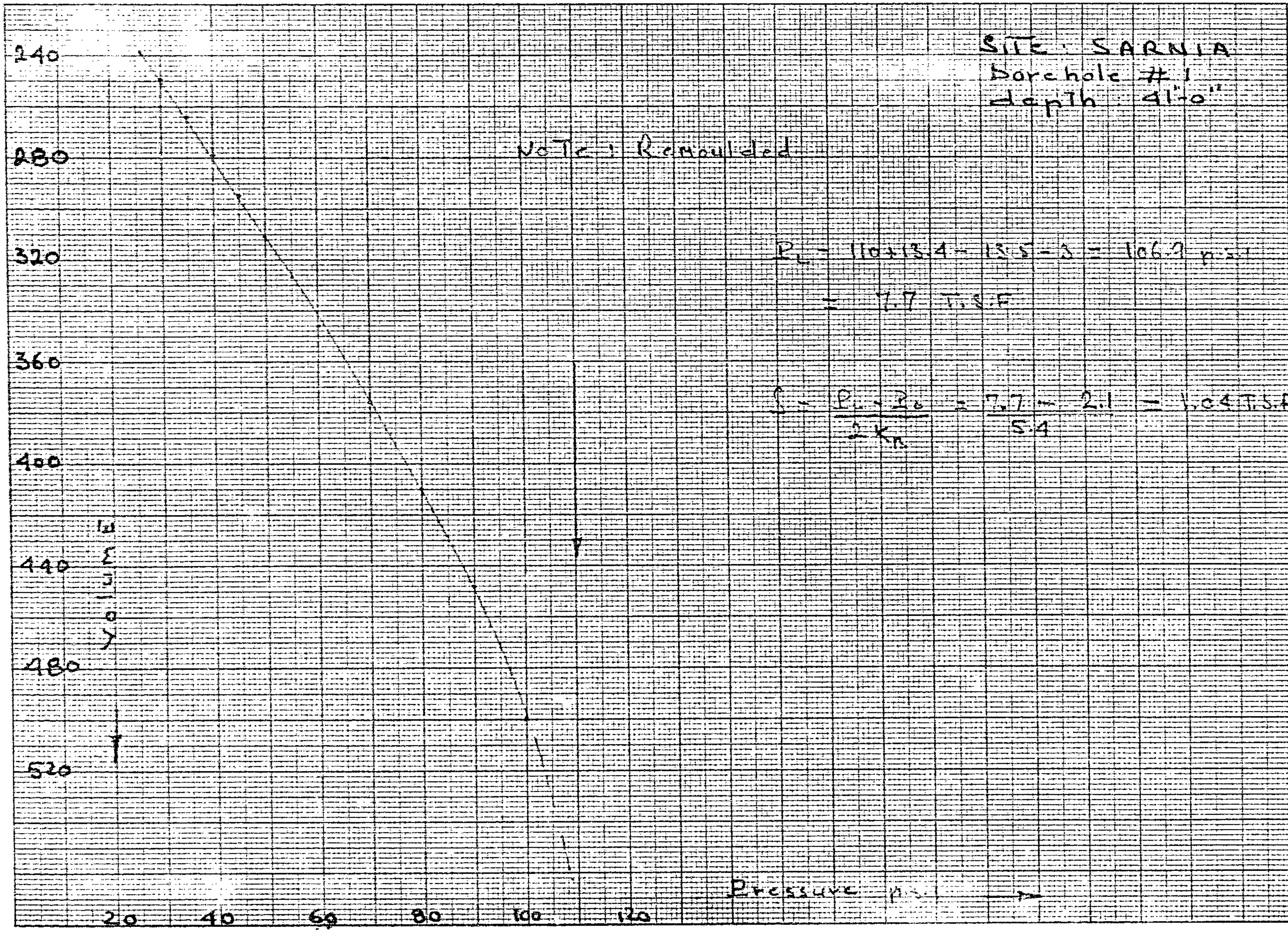
$P_L = 90 + 13.4 - 13 - 3.5 = 86.9$ p.s.i.

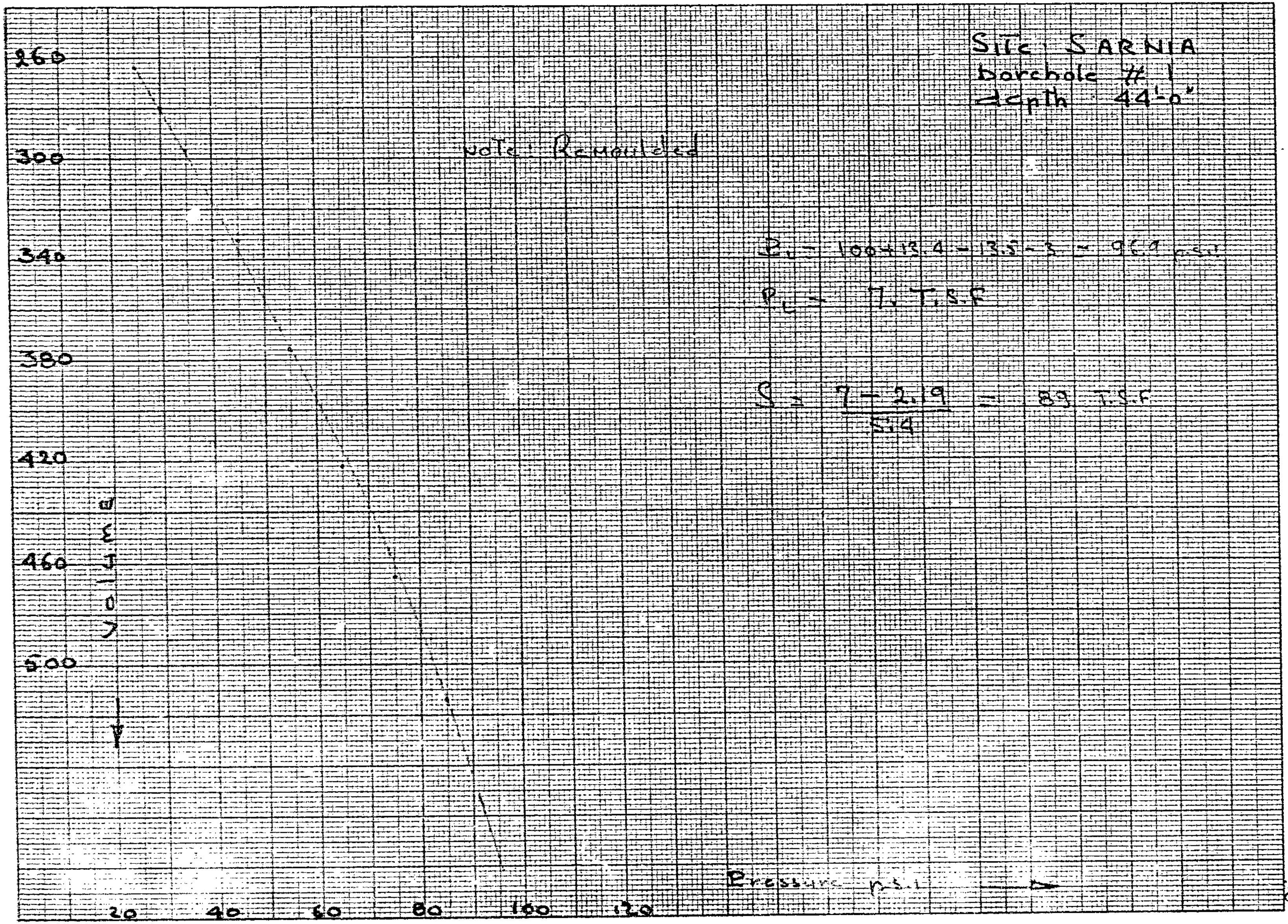
$= 6.25$ T.S.F

$$S = \frac{P_L - P_0}{2K_B} = \frac{6.25 - 1.96}{5.4} = .795 \text{ T.S.F}$$

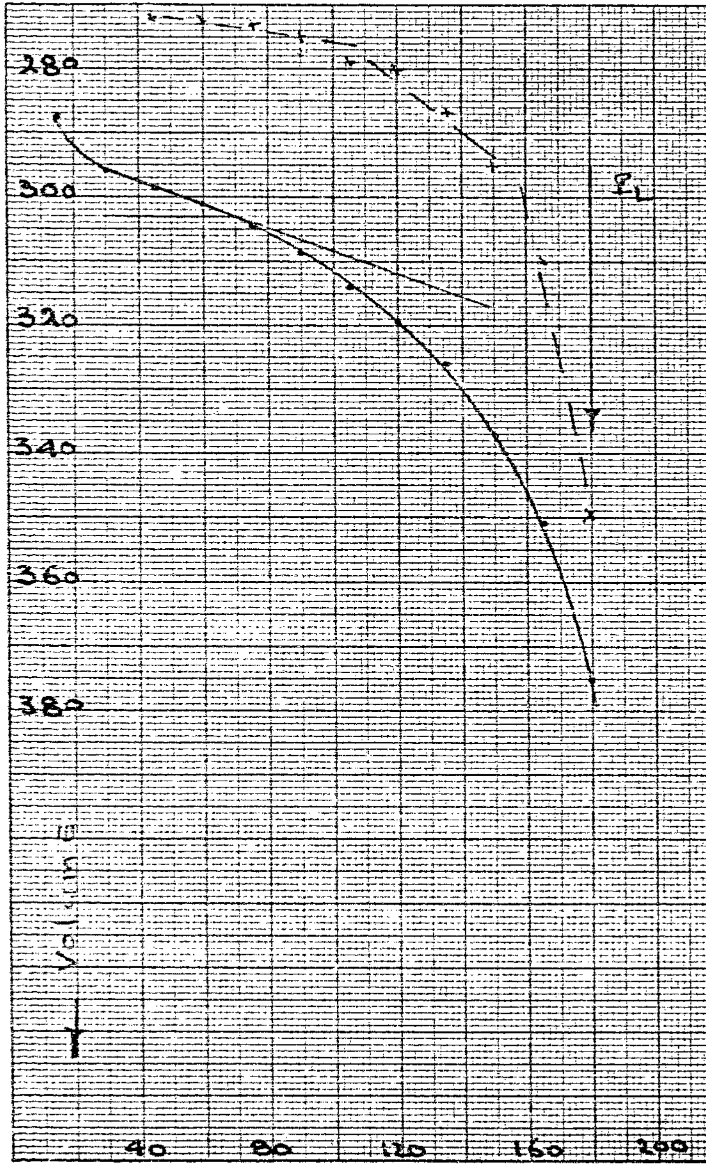

SITE: SARNIA
borehole # 1
depth 38'-0"
Note: Remoulded
$P_L = 108 + 13.4 - 13.5 - 3.5 = 104.4$ p.si
$= 7.52$ T.S.F
$S = \frac{P_L - P_o}{2K_B} = \frac{7.52 - 2.04}{5.4} = 1.02$ T.S.F
P_L
280
320
360
400
440
480
520
VOLUME
20
40
60
80
105
Pressure p.si

36


SITE: SARNIA
Borehole #1
depth: 41'-0"
Note: Remoulded
P_L = 110+13.4 - 13.5 - 3 = 106.9 p.s.i
= 7.7 T.S.F
S = (P_L - P_o) / 2K_n = (7.7 - 2.1) / 5.4 = 1.04 T.S.F
240
280
320
360
400
440
480
520
Volume
20
40
60
80
100
120
Pressure p.s.i


Site: SARNIA
borehole # 1
depth 44'-0"
note: Remoulded
$P_L = 100 + 13.4 - 13.5 - 3 = 96.9$ p.s.i
$P_L = 7.$ T.S.F
$S = \frac{7 - 2.19}{5.4} = .89$ T.S.F
260
300
340
380
420
460
500
c.c. Volume
20
40
60
80
100
120
Pressure p.s.i

SITE: SARNIA
borehole # 2
depth: 5'-0"
Soil: clay. Fill



Hydrostatic head = 3. p.s.i
Probe inertia at P_f = 6.5 p.s.i
Probe inertia at P_L = 8.5 p.s.i
$P_o \simeq \gamma z \simeq 0.325$ T.S.F

$P_L = 180 + 3 - 8.5 = 174.5$ p.s.i $\simeq 12.55$ T.S.F

$P_f = 110 + 3 - 6.5 = 106.5$ p.s.i $\simeq 7.75$ T.S.F

$E = (1+\nu)\,2\,V\,\dfrac{\Delta P}{\Delta V}$

$= 2.66 \times 300 \; \dfrac{2.88}{7} \simeq 318$ T.S.F

$P_L / P_f \simeq 1.64$

$E / P_L = 25.3$

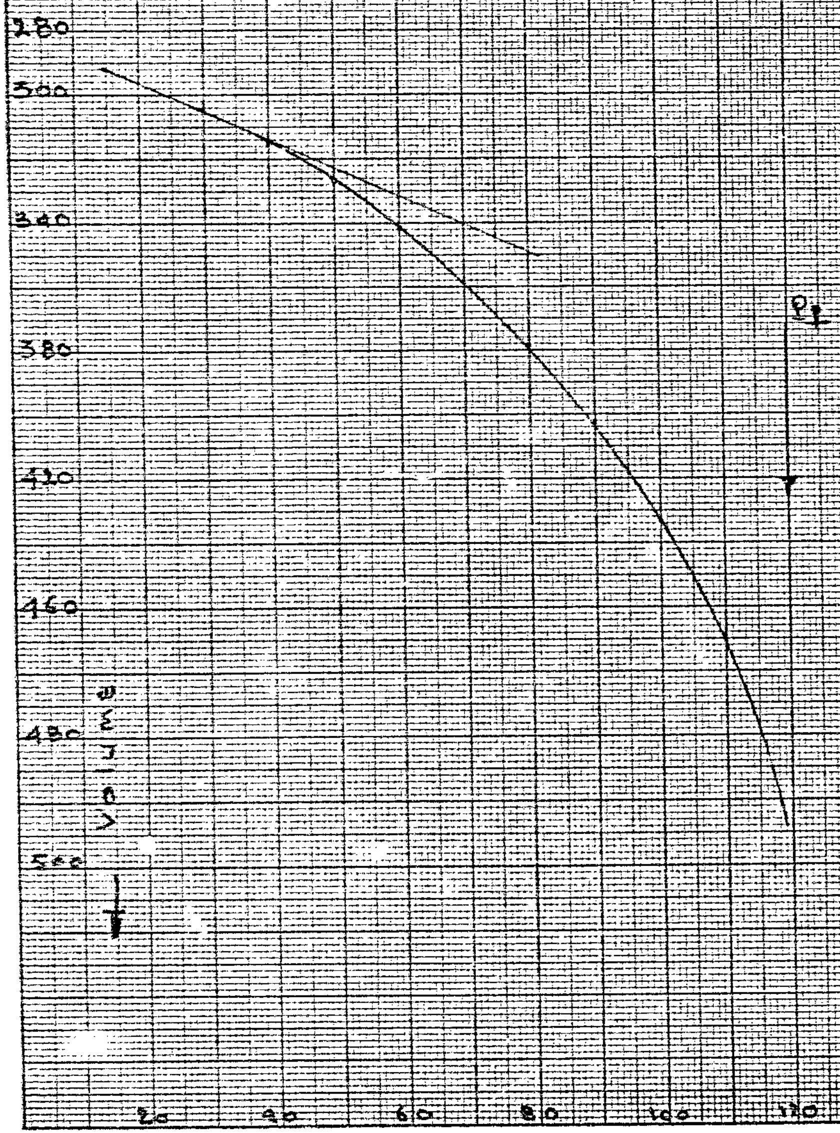
$S = \dfrac{P_L - P_o}{2 K_B} = \dfrac{12.55}{6.6} \simeq 1.9$ T.S.F

Pressure p.s.i →

39

K&E 10 X 10 TO THE CENTIMETER 18 X 25 CM. 46 1513 MADE IN U.S.A. KEUFFEL & ESSER CO.

Site: SARNIA
borehole # 2
depth: 8'0"
Soil: clay Fill



Pressure psi →

Hydrostatic head = 5 psi
Probe inertia at P_L = 12 psi
$P_o = \gamma z = .5$ T.S.F

$P_L = 120 + 5 - 12 = 113 = 8.1$ T.S.F

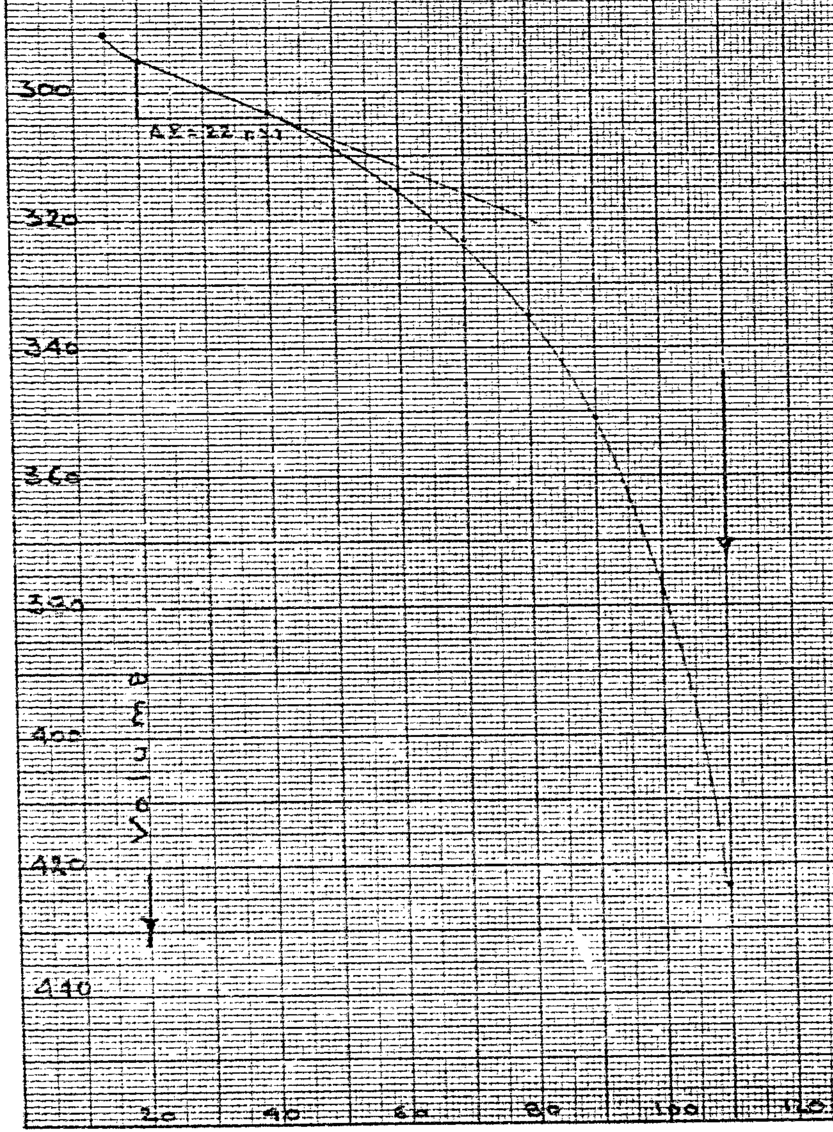
$E = (1+\sigma)\,2\,V\,\dfrac{\Delta P}{\Delta V}$

$= 2.66 \times 304 \quad \dfrac{1.8}{22} = 66$

$E/P_L = 8.15$

$S = \dfrac{P_L - P_o}{2K_b} = \dfrac{8.1 - .5}{5.4} = 1.41$ T.S.F

Site: SARNIA
borehole # 2
depth 11'-0"
Soil: clay fill



hydrostatic head ≈ 6 p.s.i
Probe inertia at P_L ≈ 9.5 p.s.i
P_o ≈ γz ≈ .87 T.S.F

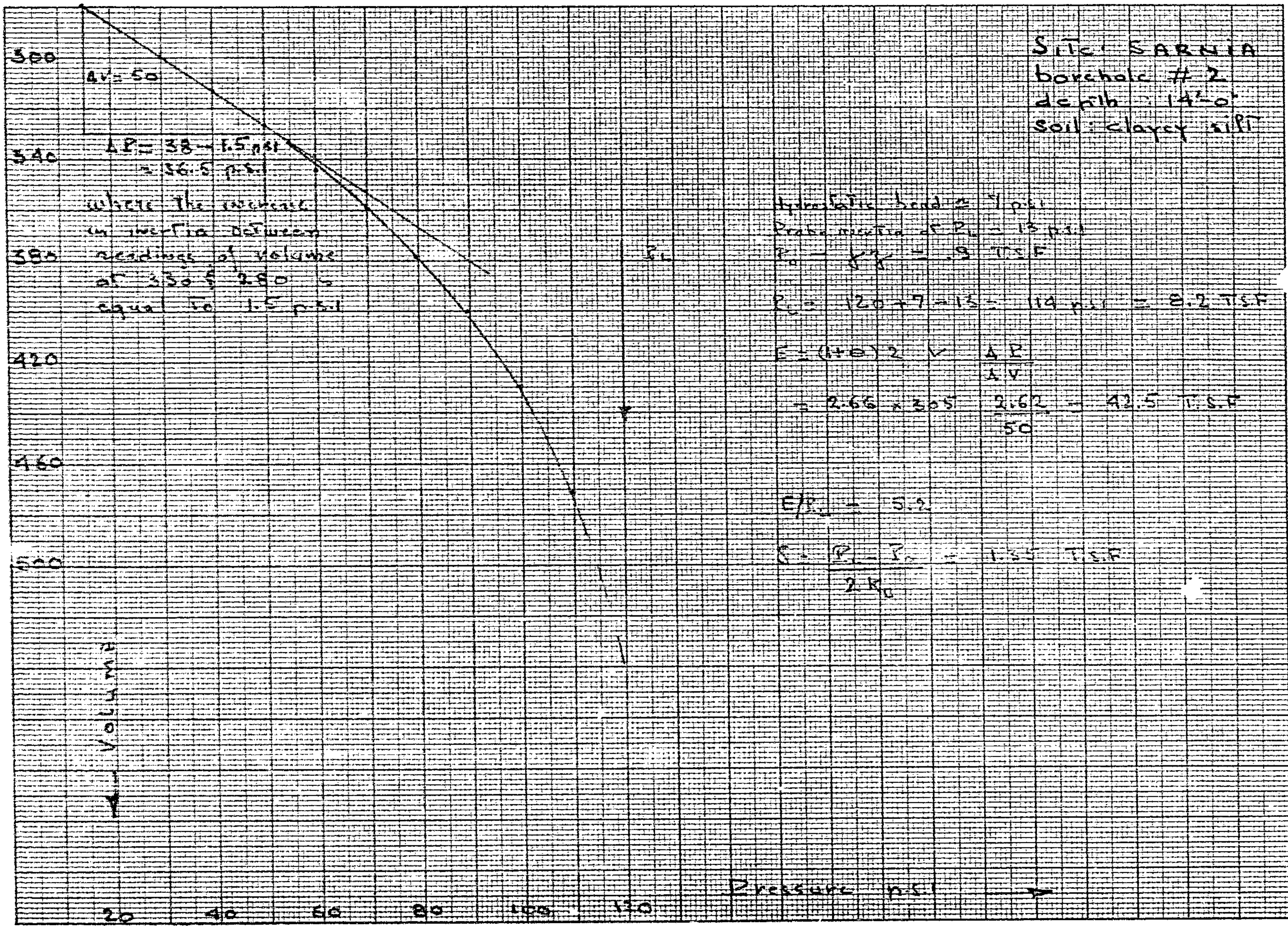
P_L = 110 + 6 − 9.5 = 106.5 p.s.i = 7.65 T.S.F

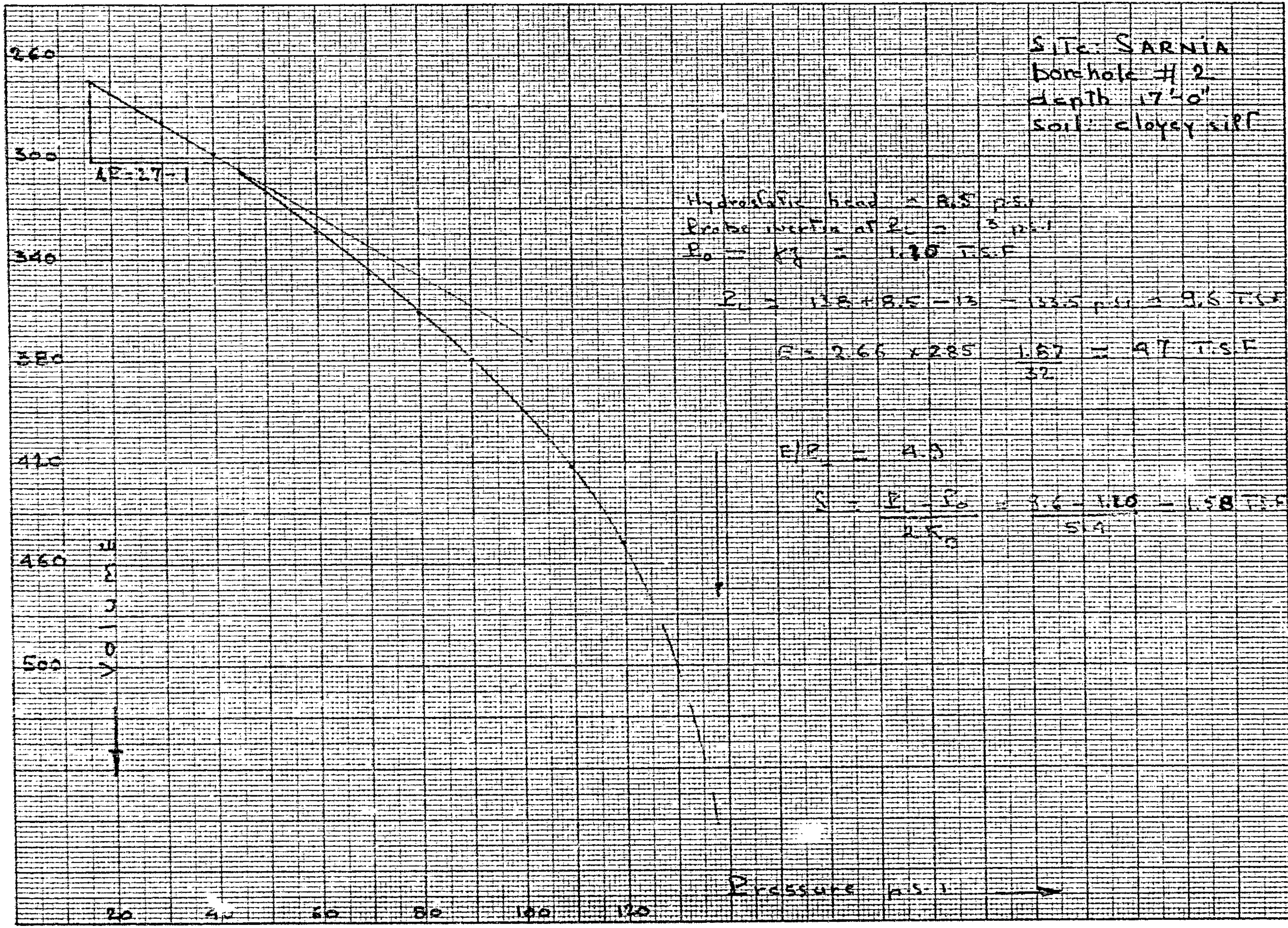
$E = 2.66 \times 300 \; \frac{1.58}{9} = 140$ T.S.F

E/P_L = 18.3

$S = \frac{P_L - P_o}{2k_s} = \frac{7.65 - .87}{6.5} = 1.06$ T.S.F

K+E 10 X 10 TO THE CENTIMETER 46 1513
18 X 25 CM. MADE IN U.S.A.
KEUFFEL & ESSER CO.





Site: SARNIA
borehole # 2
depth 17'-0"
soil: clayey silt

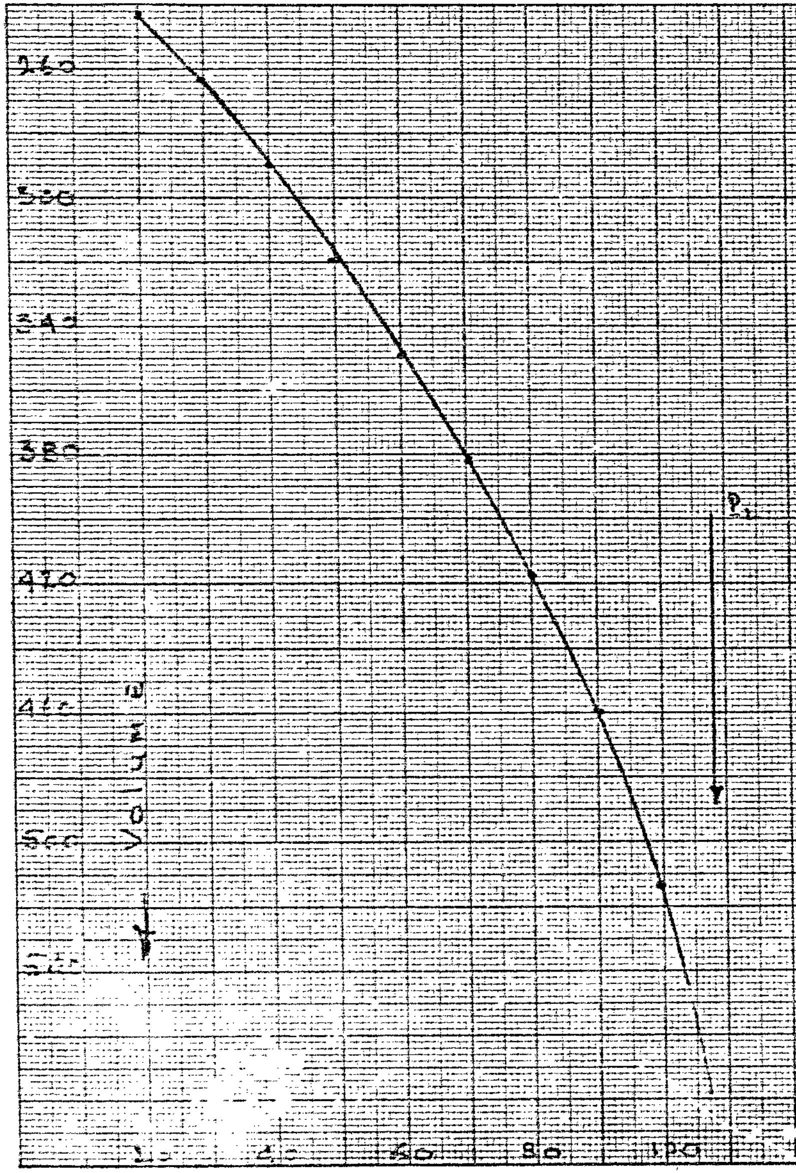
Hydrostatic head = 8.5 p.s.i
Probe inertia at P_L = 13 p.s.i
$P_o = K\gamma = 1.10$ T.S.F

$P_L = 138 + 8.5 - 13 = 133.5$ p.s.i $= 9.6$ T.S.F

$E = 2.66 \times 285 \; \frac{1.57}{32} = 47$ T.S.F

$E/P_L = 4.9$

$S = \frac{P_L - P_o}{2K_b} = \frac{9.6 - 1.10}{5.4} = 1.58$ T.S.F



Site: SARNIA
borehole # 1
depth: 20'-0"
Soil: clayey silt

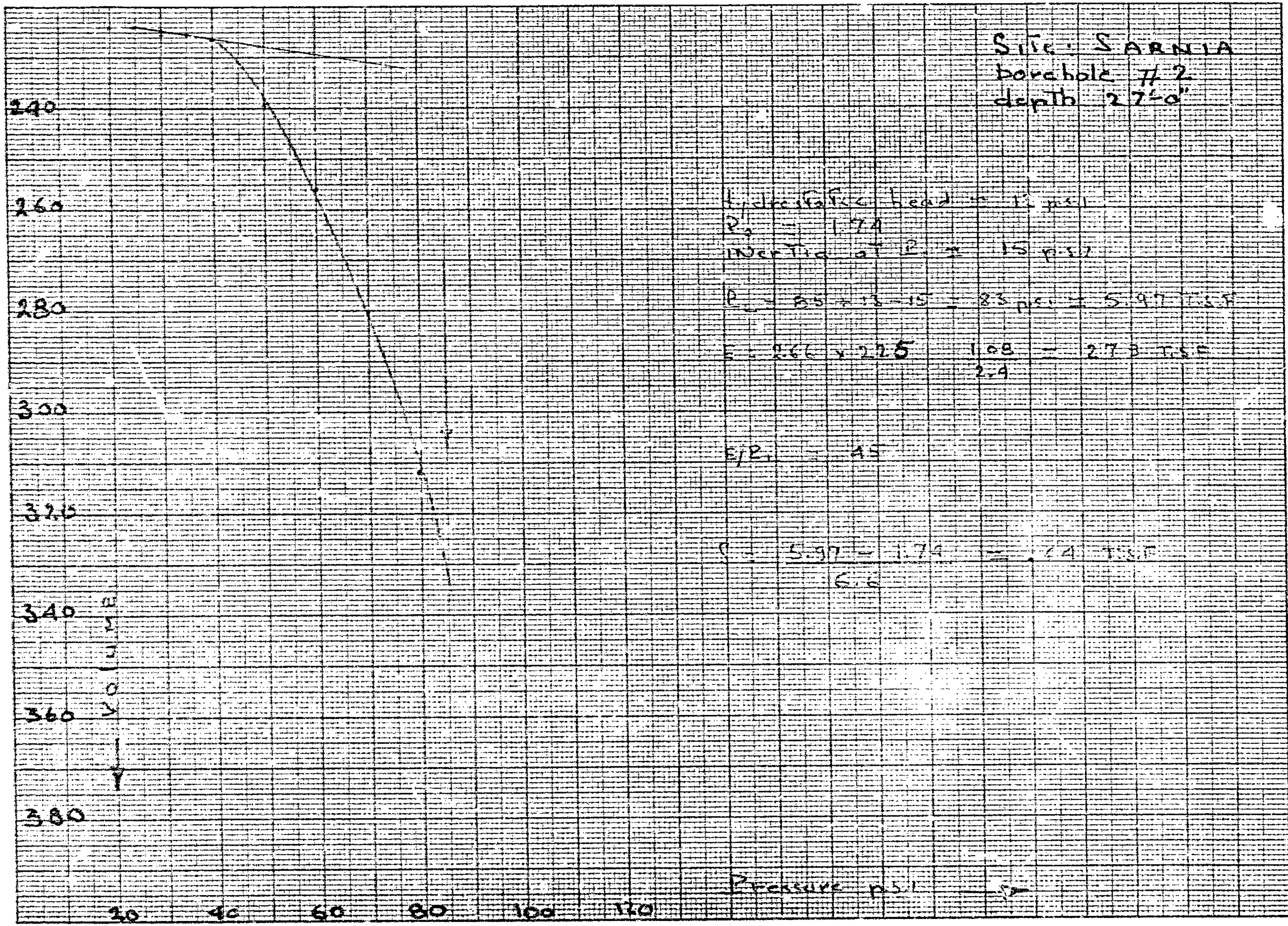
Hydrostatic head = 10 p.s.i
Probe inertia at P_L =
$P_o \simeq \gamma z$ = 1.25

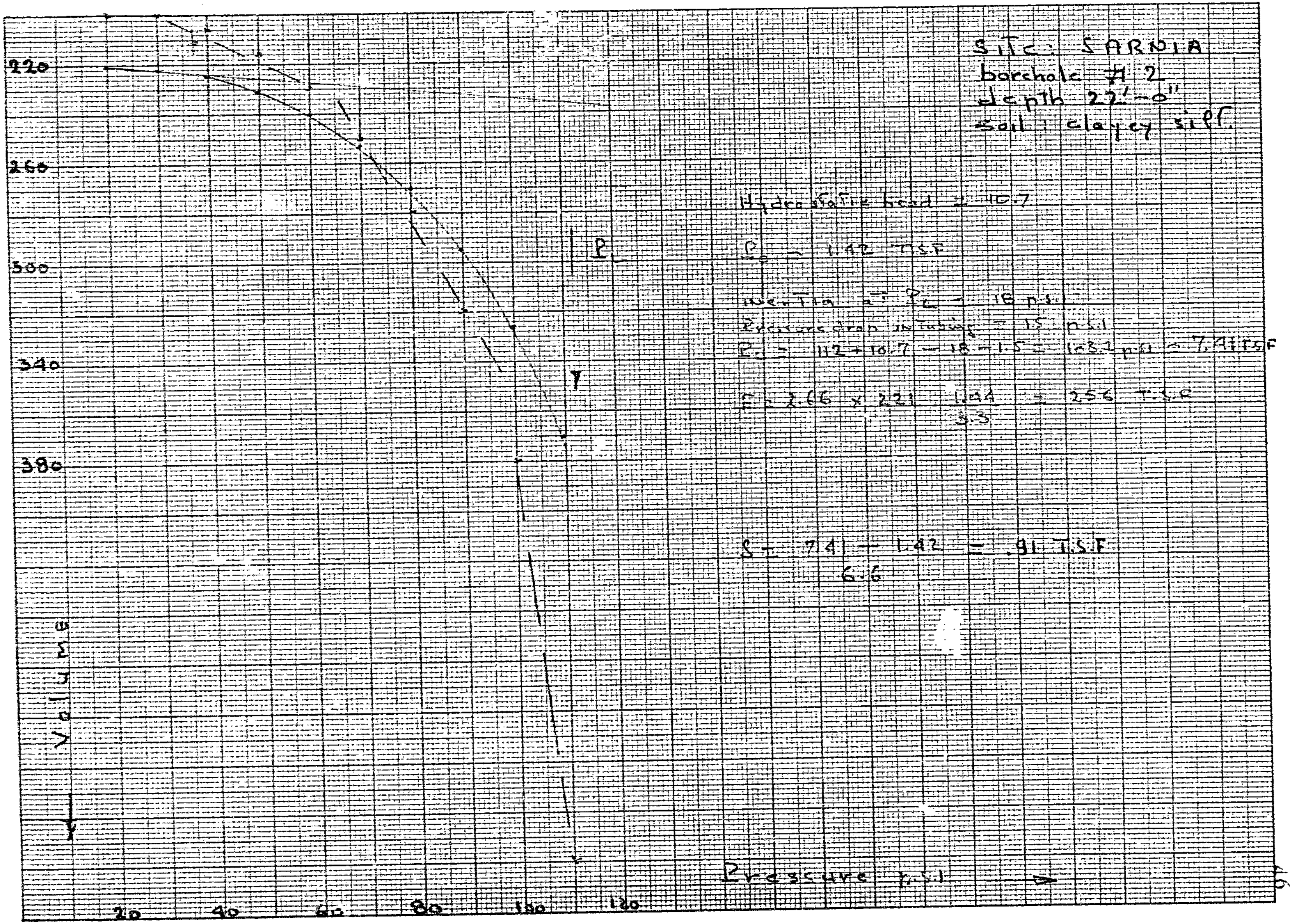
P_L = 108 + 10 − 13 = 115 p.s.i = 8.3 T.S.F

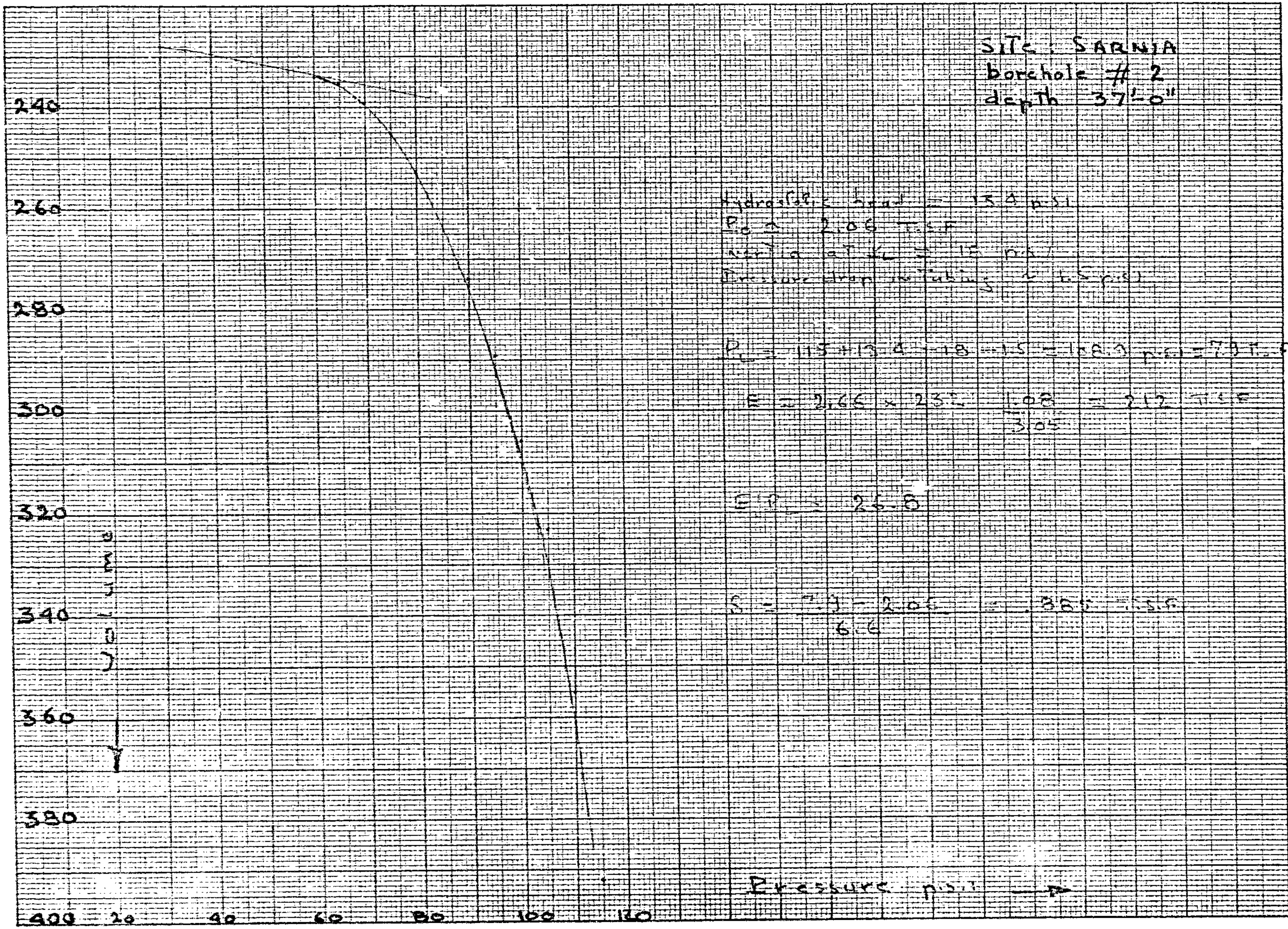
$$S = \frac{P_L - P_o}{2K_o} = \frac{8.3 - 1.25}{5.4} = 1.3 \text{ T.S.F}$$

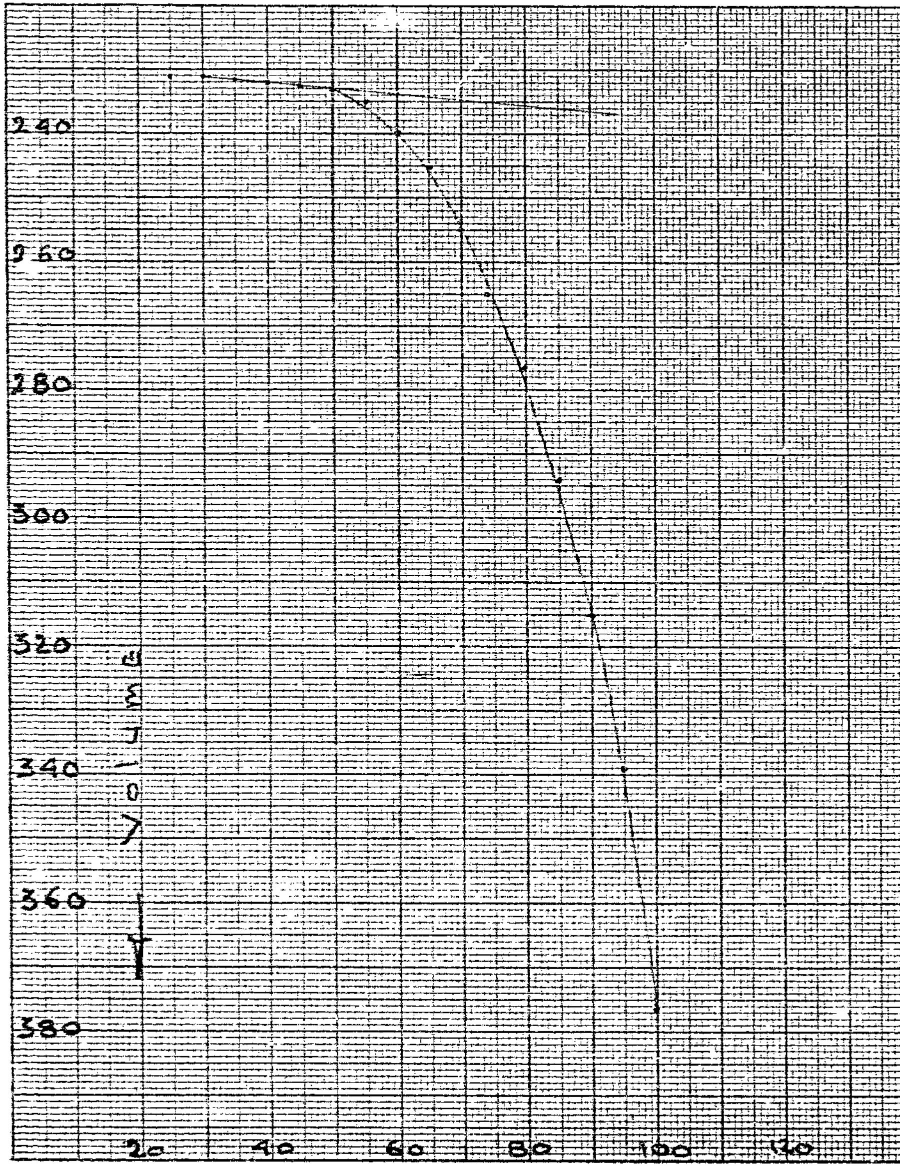
Note: soil remoulded − borehole "squeezed in"

Pressure p.s.i →


Site: SARNIA
borehole #2
depth 27'-0"
Hydrostatic head = 13 psi
P_o = 1.74
inertia at P_o = 15 psi
P_L = 85 + 13 − 15 = 83 psi = 5.97 T.S.F
E = 266 x 225 $\frac{1.08}{2.4}$ = 27.3 T.S.F
E/P_L = 45
S = $\frac{5.97 - 1.74}{6.6}$ = .64 T.S.F
240
260
280
300
320
340
360
380
Volume
20
40
60
80
100
120
Pressure psi


SITE: SARNIA
borehole # 2
depth 22'-0"
soil: clayey silt
Hydrostatic head = 10.7
P_o = 1.42 T.S.F
Inertia at P_L = 18 p.s.i
Pressure drop in tubing = 1.5 p.s.i
P_L = 112 + 10.7 − 18 − 1.5 = 103.2 p.s.i = 7.41 TSF
E = 2.66 x 221 1.44/3.3 = 256 T.S.F
S = 7.41 − 1.42/6.6 = .91 T.S.F
P_L
220
260
300
340
380
Volume
Pressure p.s.i
20
40
60
80
100
120


SITE : SARNIA
borehole # 2
depth 37'-0"
Hydrostatic head = 13.4 p.s.i
P_o = 2.06 T.S.F
P_L = 115 + 13.4 - 18 - 1.5 = 108.9 p.s.i = 7.9 T.S.F
E = 2.66 × 232 $\frac{1.08}{3.05}$ = 212 T.S.F
E/P_L = 26.8
S = $\frac{7.9 - 2.06}{6.6}$ = .885 T.S.F
240
260
280
300
320
340
360
380
400
20
40
60
80
100
120
Volume
Pressure p.s.i



SITE: SARNIA
borehole # 2
depth 32'-0"

Hydrostatic head = 13.4 p.s.i
P_o = 1.89 T.S.F
Inertia at P_L = 15.5 p.s.i
Pressure drop in tubing at P_L = 1.5 p.s.i

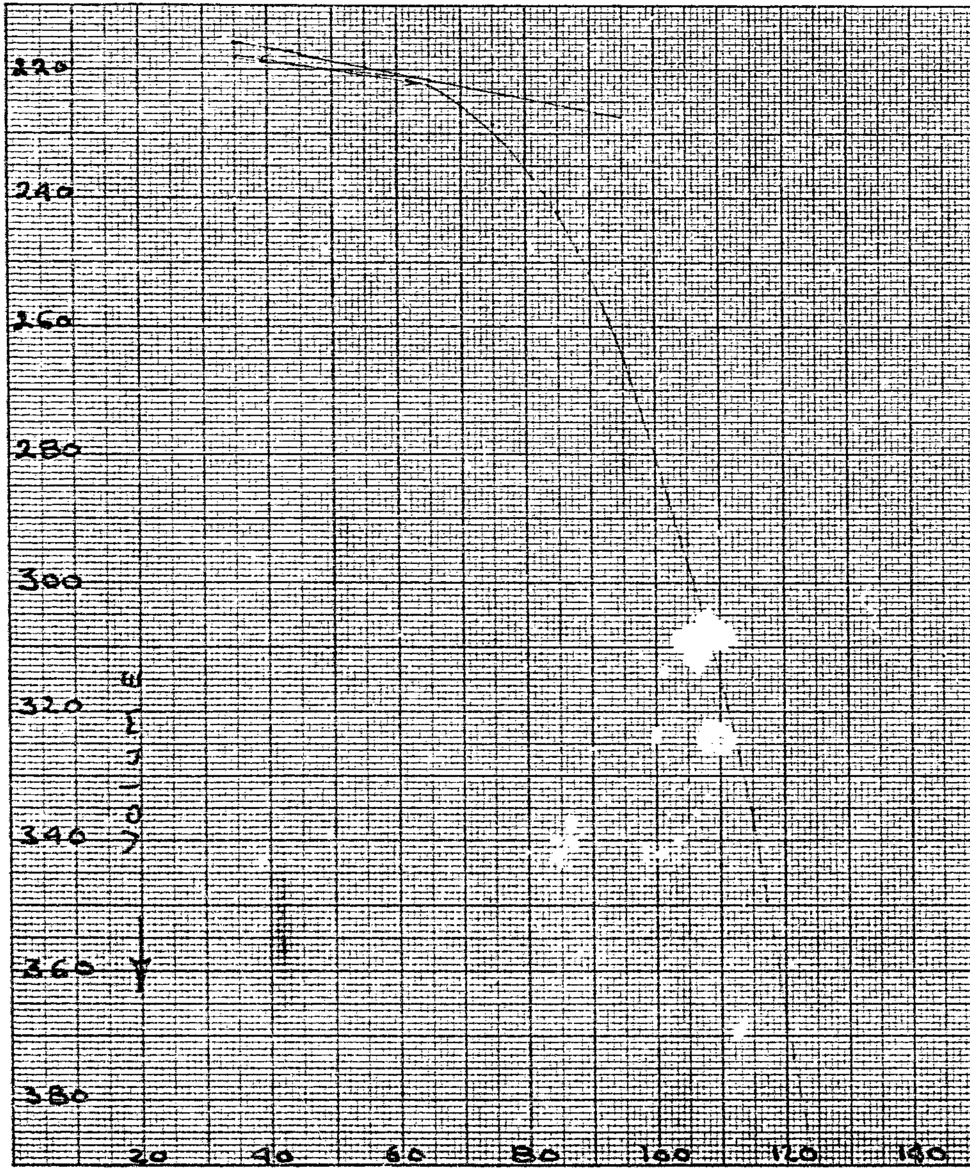
P_L = 100 + 13.4 − 15.5 − 1.5 = 96.4 p.s.i = 5.93 T.S.F

$E = 2.66 \times 232 \; \frac{1.44}{2.2} = 404$ T.S.F

$s = \frac{1.93 - 1.29}{6.4} = .165$

Pressure p.s.i →

SITE : SARNIA
borehole # 2
depth : 52



Hydrostatic head = 13.4 p.s.i
P_o = 2.38 T.S.F
Inertia of P_L = 18 p.s.i
Pressure drop in tubing at P_L = 2.5 p.s.i

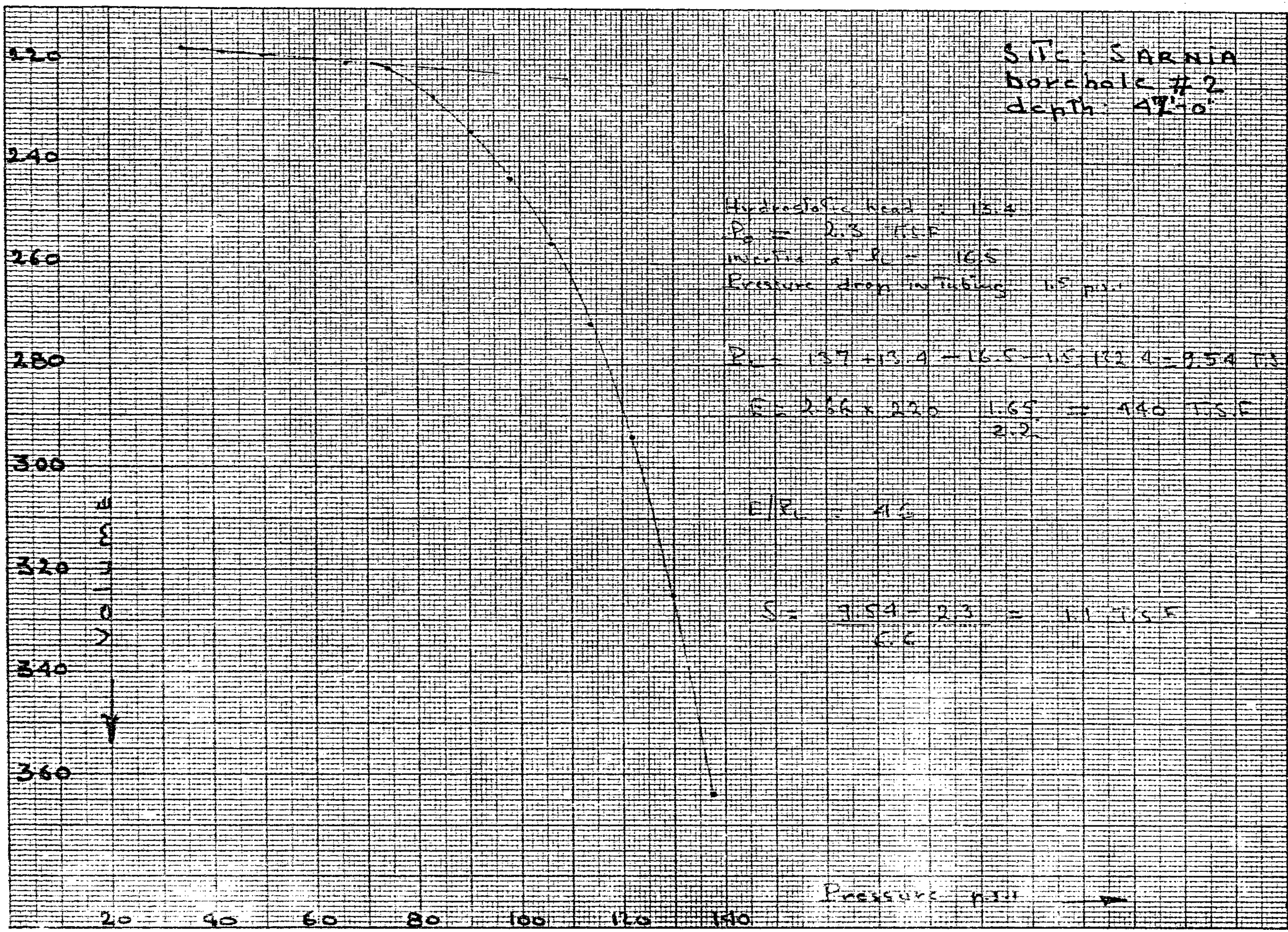
P_L = 125 + 13.4 − 18 − 2.5 = 117.9 p.s.i = 8.5 T.S.F

E = 2.66 × 220 $\frac{1.74}{4.18}$ = 202 T.S.F

E_d ≃ E (Normal for overconsolidated clay)

E/P = 23.8

S = $\frac{8.5 - 2.38}{6.6}$ = $\frac{6.12}{6.6}$ = .927 T.S.F



SITE: SARNIA
borehole #2
depth: 47'-0"

Hydrostatic head : 13.4
P_o = 2.3 T.S.F
inertia at P_L = 16.5
Pressure drop in tubing 1.5 psi

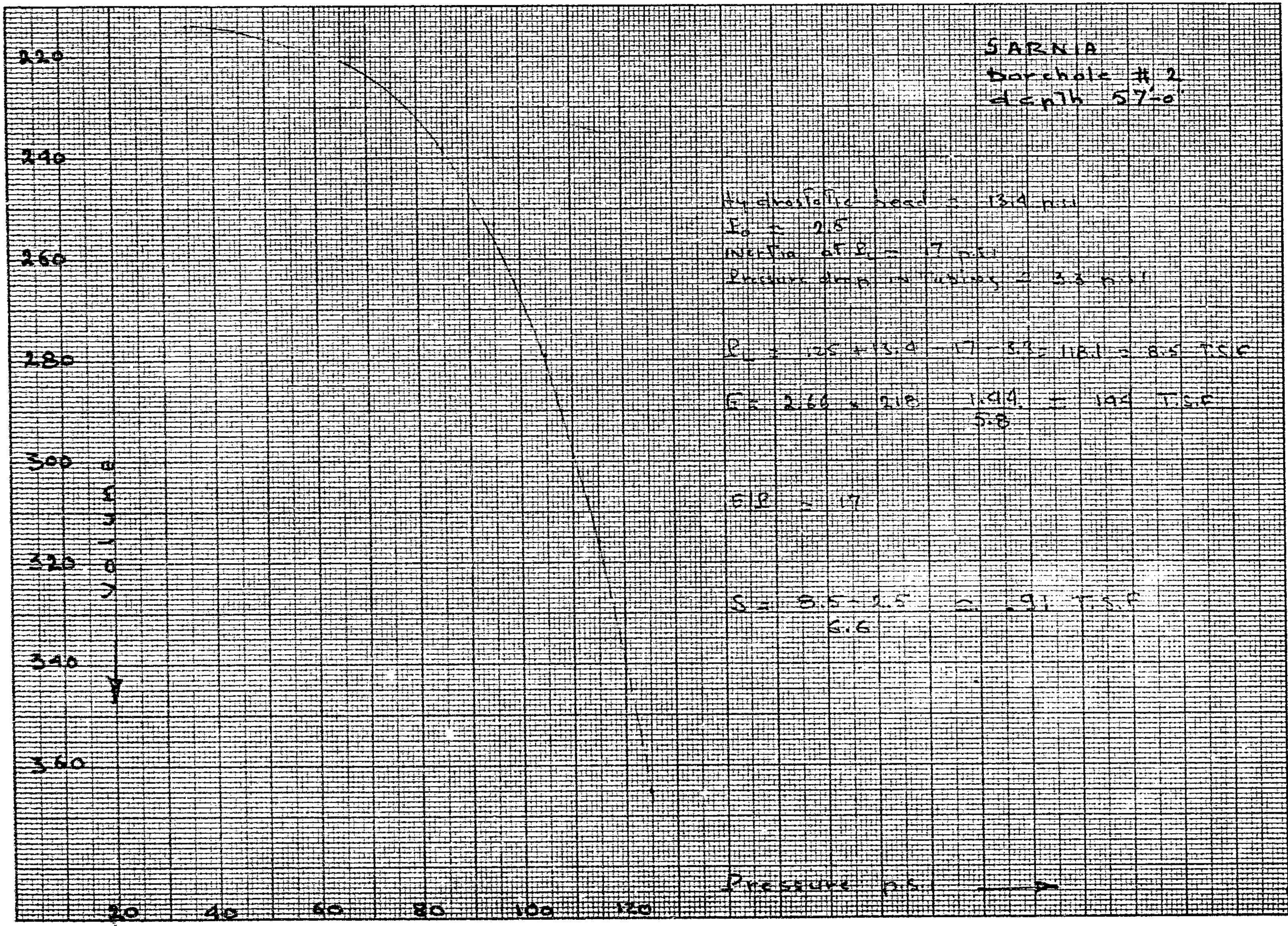
P_L = 137 + 13.4 − 16.5 − 1.5 = 132.4 = 9.54 T.S.F

$E = 2.66 \times 220 \times \frac{1.65}{2.2}$ = 440 T.S.F

E/P_L = 46

$S = \frac{9.54 - 2.3}{6.6}$ = 1.1 T.S.F

SARNIA
borehole # 2
depth 57'-0"



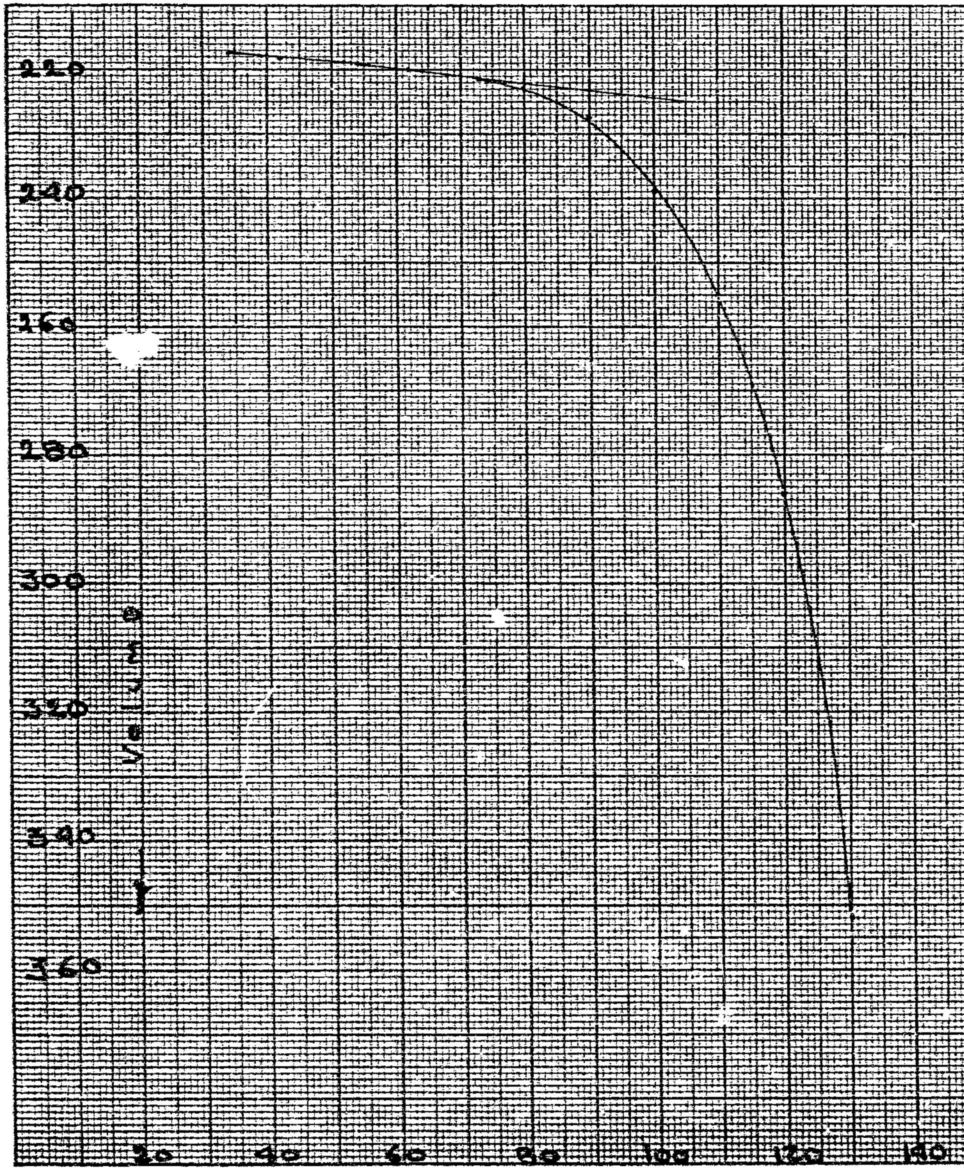
Hydrostatic head = 13.4 p.s.i
P_0 = 2.5
Inertia at P_L = 17 p.s.i
Pressure drop in tubing = 3.3 p.s.i

P_L = 125 + 13.4 − 17 − 3.3 = 118.1 = 8.5 T.S.F

$E = 2.66 \times 218 \times \frac{1.44}{5.8} = 144$ T.S.F

δP = 17

$S = \frac{8.5 - 2.5}{6.6} = .91$ T.S.F



Site: SARNIA
borehole # 2
depth 62'-0"

Hydrostatic head = 13.4
P_o = 2.6 T.S.F
inertia of P_L = 16 p.s.i
Pressure drop in Tubing = 4 p.s.i

$P_L = 130 + 13.4 - 16 - 4 = 123.4$ p.s.i $= 8.9$ T.S.F

$E = 2.66 \cdot 220 \cdot \frac{1.37}{2.4} = 334$ T.S.F

$E/P_L = 37.5$

$S = \frac{8.9 - 2.6}{6.6} = .955$

APPENDIX "B"

12" 3/4 Tubular steel Pile at 20'-0"

depth		E T.S.F	E_1 T.S.F	S_s T.S.F	N T.S.F	W inch	σ T.S.F	S_s/σ
0					21.75	.05432		
2	5				21.75	.05315		
	4	100	9.5	.625			.396	1.57
7					14.32	.05106		
	3	60	7.5	.545			.228	2.39
12					10.04	.0496		
	[illegible]	60	7.6	.545			.222	2.45
17					5.87	.04846		
	1	70	7.6	.825			.255	3.23
20					3.	.0482		
		E_q = 210	7.					

TABLE Nº 15

SITE: SARNIA

Round steel pile - concrete filled

driven to a depth of 20 ft.

Specifications of pile:

12.75 inch diameter, .25 inch wall
area of pile shoe .995 sq. ft.
area of pile shaft section: .89 sq. ft.
circumference of pile shaft = 3.34 ft.

Equivalent section modulus for calculation of pile shortening

$E_b = 4.\,[illegible] \times 10^5$ T.S.F.

Calculation of Bearing Capacity

End bearing capacity

$$Q_{ult} - Q_o = K (P_L - P_o)$$

Where Q_o represents the weight of the soil removed;

As the soil is being replaced here by a concrete pile of nearly the same density we will write

$$Q_{ult} = K (P_L - P_o)$$

and writing $P_L - P_o = P_L{}'$

$$Q_{ult} = K P_L{}'$$

From graph $K = 2$ (soil category I, $\frac{Hc}{D} > 2$)

As $P_L{}' = 7$ T.S.F.

Then

$$Q_{ult} = 14 \text{ T.S.F.}$$

with a safety factor of 3 $Q_a = 4.7$ T.S.F.

and in terms of load not stress $Q_a = 4.7 \times .995 \simeq 4.7$ Tons

Skin Friction

We will divide the pile in 5 sections and number them 1 to 5. (See Table No. 15)

Section No. 1 is 3 ft. long (3 pile diameters)
Section Nos. 2,3,4 are 5 ft. long each.
Section No. 5 is 2 ft. long (1 ft. + 1 pile diameter)

Against each section we will list the corresponding average values of the Modulus E, the limit pressure $P_L{}'$, and the skin friction S_S. (The values of the skin friction being given by graph No. 14). These values of skin friction are the peak values of skin friction which occur when relative movement between the pile shaft and the soil are small. As soon as movement becomes large, in the order of .5 to 1 inch, such as at failure, the skin friction will drop to a lower value which we will refer to as residual skin friction S_{SR}.

This would appear to be mainly true in clays and the writer has adapted a coefficient of 0.8 to obtain the value of the residual skin friction. (If dealing with sand no drop in skin friction will be allowed).

The contact area of the pile being 3.34 sq. ft. per foot length, then the total residual skin friction (at failure) that may be mobilised is

$$\sum_{H=20}^{H=2} S_{SR} = 0.8 \left[3.34 \ (3 \times .825 + 10 \times .545 + 5 \times .625) \right]$$

$$= 29.5 \text{ Tons}$$ (The skin friction on section 5 being disregarded)

Bearing capacity of pile

It is customary to adapt a safety factor of 2 with respect to skin friction, then

Allowable Load on pile

$$P \text{ allowable} = 14.75 + 4.7 = 19.45 \text{ Tons}$$

Ultimate Load at failure

$$P \text{ ultimate} = 29.5 + 14 = 43.5 \text{ Tons}$$

Settlement of pile

When a pile is subjected to a load P, the load induces normal stresses on the soil at the pile tip and shear stresses on the sides. The resulting settlement W_0 at the pile head originates from 3 phenomena:

1) the settlement at the pile tip.
2) the deformation of the soil along the shaft due to the shear stresses.
3) the shortening of the pile itself.

The pile has been divided in five sections, numbered 1 to 5, starting from the bottom. (Each section should be 5 - 7 ft. long.)

Let us denote N_1, N_2, N_3, N_4, N_5 the normal stresses in the pile at the bottom of sections 1, 2, 3, 4, 5; the corresponding settlement of the pile at each of these elevations will be W_1, W_2, W_3, W_4, W_5.

Then N_1 is the contact pressure at the pile tip.

Let us consider what occurs when we apply a contact pressure of 3 T.S.F:

$$N_1 = 3 \text{ T.S.F.}$$

The settlement will be given by the expression:

$$W = \frac{Q}{4E_a} \lambda_2 \quad D$$

where Q = contact pressure = 3 T.S.F.
λ_2 = shape coefficient = 1
D = diameter = 13.5 ins.
$E_a = 3E$ = 210 T.S.F

Then

$$W_1 = \frac{3 \times 13.5}{4 \times 210} = .0482 \text{ inch}$$

Settlement at Tip of Section (2)

To the settlement of the pile tip we will add the elastic shortening of section (1) applying Hooke's Law.

$$W_2 = W_1 + \frac{(\text{height} \times \text{stress}) \text{ of section } (1)}{\text{Modulus of pile material}} = W_1 + \frac{H\,N_1}{E_p}$$

$$W_2 = W_1 + \frac{36 \times 3.00}{4.1 \times 10^5} = .0482 + .00026 = .04846 \text{ inch}$$

The mobilisation of the skin friction in function of relative movement between the pile shaft and the soil is given by the expression

$$\tau = \frac{2\,E\,W}{C_L D}$$

where here
C_L = 2.1
D = 12.75 inch (shaft. diam)

Then we can write:

$$\tau_1 = \frac{2\ E\ W_2}{C_L D}$$

$$\tau_1 = \frac{2 \times 70 \times .04846}{2.1 \times 12.75} = .255 \text{ T.S.F.}$$

The mobilisation of the skin friction τ_1 = .255 T.S.F. around the circumference of section (1) means that this section is now carrying a load. Expressing this load in terms of stress (Not Load):

$$N_2 = N_1 + \frac{\text{height} \times \text{circumference}}{\text{section of pile}} (\text{of section } (1)) \times \tau_1$$

$$N_2 = 3 + \frac{3 \times 3.34}{.89} \times .255 = 3 + 2.87 = 5.87 \text{ T.S.F.}$$

Settlement of Tip of Section (3)

$$W_3 = W_2 + \frac{60 \times 5.87}{4.1 \times 10^5} = .04846 + .00086 \cong .04960 \text{ inch}$$

$$\tau_2 = \frac{2 \times 60 \times .0496}{2.1 \times 12.75} = .222 \text{ T.S.F.}$$

$$N_3 = N_2 + \frac{5 \times 3.34}{.89} \times .222 = 5.87 + 4.17 = 10.04 \text{ T.S.F.}$$

Settlement of Section (4)

$$W_4 = W_3 + \frac{60 \times 10.04}{4.1 \times 10^5} = .04960 + .00146 = .05106 \text{ inch}$$

$$\tau_3 = \frac{2 \times 60 \times .05106}{2.1 \times 12.75} = .228 \text{ T.S.F.}$$

$$N_4 = N_3 + \frac{5 \times 3.34}{.89} \times .228 = 10.04 + 4.28 = 14.32 \text{ T.S.F.}$$

Settlement of Section (5)

$$W_5 = W_4 + \frac{60 \times 14.32}{4.1 \times 10^5} = .05106 + .00209 = .05315 \text{ inch}$$

$$\tau_4 = \frac{2 \times 100 \times .05315}{2.1 \times 12.75} = .396 \text{ T.S.F.}$$

$$N_5 = N_4 + \frac{5 \times 3.34}{.89} \times .396 = 14.32 + 7.43 = 21.75 \text{ T.S.F.}$$

We now add the elastic shortening of section (5) -

$$W_o = W_5 + \frac{24 \times 21.75}{4.1 \times 10^5} = .05315 + .00127 = .05432 \text{ inch}$$

$$N_o = N_5 = 21.75 \text{ T.S.F.}$$

Listing the values of N, W, τ on Table No. 15, we see that the ratio $\frac{S_s}{\tau}$ is lowest (1.57) at Section (4), i.e. if we increase the normal stress N_o to a value of 21.75 X 1.57 = 34.1 T.S.F. we will reach the maximum stress τ_4 = .625 T.S.F.. For any value of N_o greater than 34.1 T.S.F. plastic deformations will start to occur; in terms of load on the pile head this will occur at

$$P = 34.1 \times .89 = 30.4 \text{ Tons}$$

The corresponding settlement at the pile head will be

$$W_o = .05432 \times 1.57 = .085 \text{ inch}$$

2" HERCULES CONCRETE PILE

at 20'-0"

depth		E T.S.F	P'_L T.S.F	S_s T.S.F	N T.S.F	W INCH	σ T.S.F	S_s/σ
0					24.08	.05942		
2	5				24.08	.05662		
	4	100	9.5	.74			.413	1.79
7					15.98	.05202		
	3	60	7.5	.68			.228	2.98
12					10.58	.04885		
	2	60	7.6	.68			.215	3.16
17					6.04	.04764		
20	1	70	7.6	1.05	3.	.0465	.241	4.27
		E_s 210	7.					

TABLE N° 16

SITE: SARNIA

Hercules concrete pile

driven to a depth of 20 ft.

Specifications of pile:

Hexagonal concrete pile, precast, 12" across flats
area of cross section .83 sq. ft.
circumference 3.5 ft.
Modulus E_b = 200.000 T.S.F. (of concrete)
Average diameter of shaft 13"

Calculation of bearing capacity

1) End bearing capacity

$Q_{ult} = K \, P_L{}^{1} = 2 \times 7 = 14$ T.S.F.

in terms of Load

$Q_{ult} = 14 \times .83 \simeq 12$ Tons

Taking a safety factor of 3

Q allow = 4 Tons

2) Mobilisation of skin friction

Divide the pile in 5 sections, and prepare Table No. 16 listing against each section the corresponding values of E, $P_L{}^{1}$, .

Then the total skin friction mobilised at failure will be (in terms of Load)

$$\sum_{H=20}^{H=2} S_{SR} = 0.8 \left[3.5(3 \times 1.03 + 10 \times .68 + 5 \times .74) \right]$$
$$= 38 \text{ Tons}$$

Taking a safety factor of 2, the permissible load carried by skin friction will be 19 tons.

3) Bearing capacity of pile

Allowable Load on pile

P allowable = 4 + 19 = 23 Tons

Ultimate Load at failure

$$P \text{ ultimate} = 12 + 38 = 50 \text{ Tons}$$

Settlement of pile

Settlement at Tip of Pile

Let us apply a contact pressure of 3 T.S.F. at the pile tip, then

$$N_1 = 3 \text{ T.S.F.}$$

$$W_1 = \frac{N_1}{4 E_a} \lambda_2 D$$

where $D = 13$ inches
$E_a = 210$ T.S.F.
$\lambda_2 = 1$

$$W_1 = \frac{3 \times 13}{4 \times 210} = .0465 \text{ inch}$$

Settlement at Tip of Section (2)

$$W_2 = W_1 + \frac{36 \times 3}{2 \times 105} = .0465 + .00054 = 04704 \text{ inch}$$

$$\sigma_1 = \frac{2 \times 70 \times .04704}{2.1 \times 13} = .241 \text{ T.S.F.}$$

$$N_2 = N_1 + \frac{3 \times 3.5}{.83} \times .241 = 3 + 3.04 = 6.04 \text{ T.S.F.}$$

Settlement at Tip of Section (3)

$$W_3 = W_2 + \frac{60 \times 6.04}{2 \times 105} = .04704 + .00181 = .04885 \text{ inch}$$

$$\sigma_2 = \frac{2 \times 60 \times .04885}{2.1 \times 13} = .215 \text{ T.S.F.}$$

$$N_3 = N_2 + \frac{5 \times 3.5}{.83} \times .215 = 6.04 + 4.54 = 10.58 \text{ T.S.F.}$$

<u>Settlement at Tip of Section (4)</u>

$$W_4 = W_3 + \frac{60 \times 10.58}{2 \times 10^5} = .04885 + .00317 = .05202 \text{ inch}$$

$$\sigma_3 = \frac{2 \times 60 \times .05202}{2.1 \times 13} = .228 \text{ T.S.F.}$$

$$N_4 = N_3 + \frac{5 \times 3.5}{.83} \times .228 = 10.58 + 4.8 = 15.38 \text{ T.S.F.}$$

<u>Settlement at Tip of Section (5)</u>

$$W_5 = W_4 + \frac{60 \times 15.38}{2 \times 10^5} = .05202 + 0046 = .05662 \text{ inch}$$

$$\sigma_4 = \frac{2 \times 100 \times .05662}{2.1 \times 13} = .413 \text{ T.S.F.}$$

$$N_5 = N_4 + \frac{5 \times 3.5}{.83} \times .413 = 15.38 + 8.7 = 24.08 \text{ T.S.F.}$$

<u>Settlement at pile head</u>

$$W_o = W_5 + \frac{24 \times 24.08}{2 \times 10^5} = .05662 + .0028 = .05942 \text{ inch}$$

$N_o = N_5 = 24.08$ (Section 5 at pile head being disregarded.)

Listing the values of N, W, σ on Table No.16 we see that the ratio S_s/σ is lowest at section (4) where it is equal to 1.79. Therefore the normal stress N_o can be increased on the pile head to a value of 24.08 X 1.79 = 43 T.S.F. approximately, before section (4) is overstressed.

This corresponds to a load on the pile of

$$P = 43 \times .83 = 35.8 \text{ Tons}$$

The corresponding settlement will be:

$$W_o = .05942 \times 1.79 = .106 \text{ inch}$$

A visual examination of the ratio S_s/σ indicates that after section (4) has been overstressed most of the load on the pile will be carried by sections 1, 2, and 3. Sections 2 and 3 will become overstressed at practically the same time and rapid degeneration of settlement should then occur. This degeneration should occur when settlement has reached a value of .15 to .2 inch.

12-BP-53 "H" BIIE

aT 20'-0"

depth		E T.S.F	E'_u T.S.F	S_s T.S.F	N T.S.F	W inch	σ T.S.F	S_s/σ
0					25.85	.0620		
2	5				25.85	.05889		
		100	3.5	.59			.467	1.26
7					16.65	.05389		
		60	7.5	.545			.256	2.12
					11.6	.05039 [redacted]		
		60	7.6	.545			.240	2.26
					6.2	.04854		
		70	7.6	.825			.270	3.05
					3	.048		
		E_a = 210	7.					

TABLE N° 17

SITE : SARNIA

H Pile

driven to a depth of 20 ft.

Specifications of pile

Area of pile section (within perimeter) 1.04 sq.ft.
circumference of pile 4.1 ft.
equivalent modulus of pile section $E_p = 2X10^5$ T.S.F

Bearing capacity

1) End bearing

$Q_{ult} = K\, P_L' = 2 X 7 = 14$ T.S.F.

in terms of load

$Q_{ult} = 14 X 1.04 = 14.5$ Tons

$Q_{allowable} = \frac{14.5}{3} \simeq 4.8$ Tons

2) Skin friction

The residual skin friction mobilized at failure will be (in terms of load)

$$\sum_{H=20}^{H=2} S_{sR} = 0.8 \left[4.1(5 X .59 + 10 X .59 + 3 X .825) \right]$$
$$= 35.6 \text{ Tons}$$

Capacity of pile

$Q_{allowable} = 17.8 + 4.8 = 22.6$ Tons

at failure

$Q_{ultimate} = 35.6 + 14.5 = 50.1$ Tons

Settlement

Allow contact pressure at tip of 3 T.S.F.

$N_1 = 3$ T.S.F.

$$W_1 = \frac{N_1 \lambda_2 D}{4 E_a} \qquad \text{where } D = 12 \text{ inches}, \ \lambda_2 = 1.12, \ E_a = 210 \text{ T.S.F.}$$

$$W_1 = \frac{3 \times 1.12 \times 12}{4 \times 210} = .048 \text{ inch}$$

$$W_2 = .048 + \frac{36 \times 3}{2 \times 10^5} = .048 + .00054 = .04854$$

$$\sigma_1 = \frac{2 \times 70 \times .04854}{2.1 \times 12} = .270 \text{ T.S.F.}$$

$$N_2 = 3 + \frac{3 \times 4.1}{1.04} .270 = 3 + 3.2 = 6.2 \text{ T.S.F.}$$

$$W_3 = .04854 + \frac{60 \times 6.2}{2 \times 10^5} = .04854 + .00185 = .05039$$

$$\sigma_2 = 2 \times \frac{60 \times .05039}{2.1 \times 12} = .240 \text{ T.S.F}$$

$$N_3 = 6.9 + \frac{5 \times 4.1}{1.04} .240 = 6.9 + 4.7 = 11.6 \text{ T.S.F}$$

$$W_4 = .05039 + \frac{60 \times 11.6}{2 \times 10^5} = .05039 + .0035 = .05389$$

$$\sigma_3 = \frac{2 \times 60 \times .05389}{2.1 \times 12} = .256 \text{ T.S.F.}$$

$$N_4 = 11.6 + \frac{5 \times 4.1}{1.04} .256 = 16.65 \text{ T.S.F}$$

$$W_5 = .05389 + \frac{60 \times 16.65}{2 \times 10^5} = .05389 + .005 = .05889$$

$$\sigma_4 = \frac{2 \times 100 \times .0589}{2.1 \times 12} = .467 \text{ T.S.F}$$

$$N_5 = 16.65 + \frac{5 \times 4.1}{1.04} .467 = 16.65 + 9.2 = 25.85 \text{ T.S.F.}$$

$$W_o = .0589 + \frac{24 \times 25.85}{2 \times 10^5} = .0589 + .0031 = .0620 \text{ inch.}$$

$$N_o = N_5 = 25.85 \text{ T.S.F}$$

Examining the ratio $^{S}/\gamma$ it is seen that overstressing will first occur at section (4) for a value of

$N = N_o \times 1.26 = 32.6$ T.S.F.

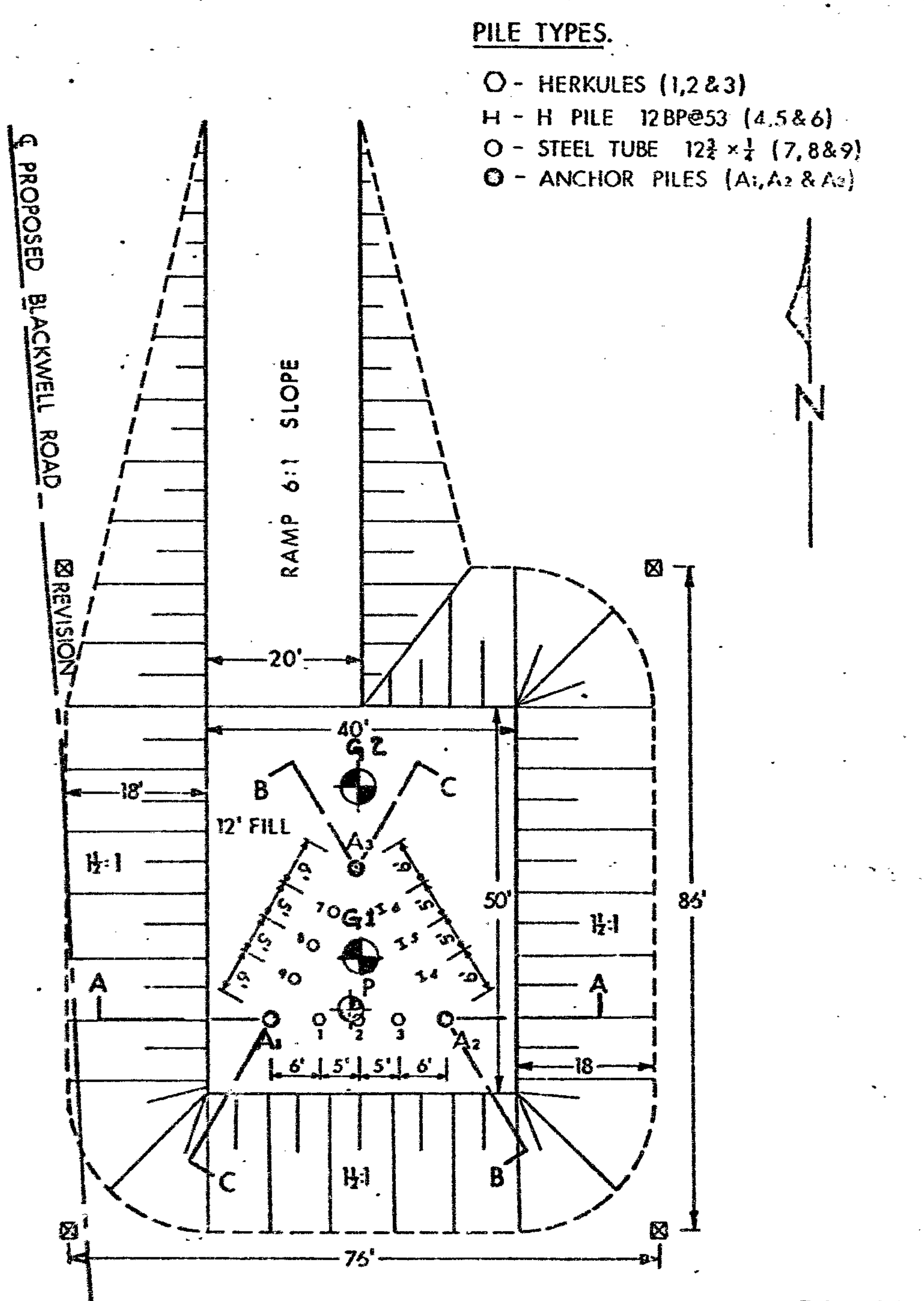
in terms of load this will occur at

$P_{\ell} = 32.6 \times 1.04 = 34$ Tons

The corresponding settlement will be

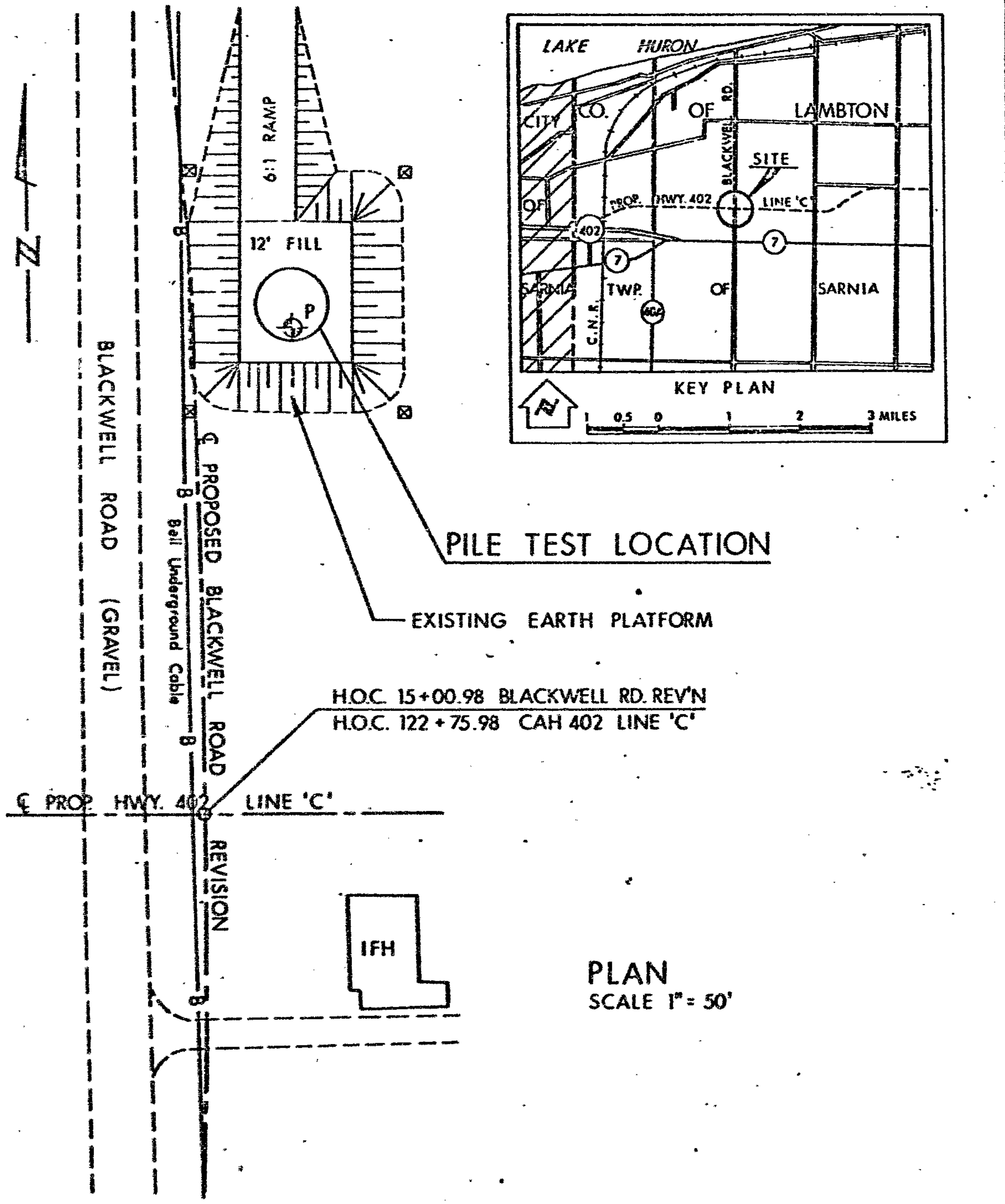
$W = W_o \times 1.26 = .078$ inch

APPENDIX "C"



PLAN
SCALE 1" = 20'

DEPARTMENT OF HIGHWAYS . MATERIALS and TESTING DIVISION ONTARIO	BLACKWELL ROAD & HWY. 402 PILE TEST ARRANGEMENT W.P. 43 - 66 - 05 DIST. 1 JOB 70 - 11049	
DATE 22 SEPT. 1970	APPROVED	DRAWING NO. 70 - 11049 B



DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION
ONTARIO

BLACKWELL ROAD & HWY. 402

PILE TEST LOCATION

W.P. 43 - 66 - 05 DIST. 1 JOB 70 - 11049

DATE 21 SEPT. 1970	APPROVED	DRAWING NO. 70 - 11049 A