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STR. SITE No. 14-339

HWY. No. 402

LOCATION BLACKWELL SIDEROAD

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS: 2 documents to be unfolded before microfilming

70-11049

# GEOPROBE ONTARIO

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GEOPROBE PRESSUREMETER TESTS

for

THE ONTARIO DEPARTMENT OF HIGHWAYS

at

BLACKWELL ROAD & HWY. 402

## 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

$$W = C_L \frac{\tau D}{2 E} \qquad (5)$$

or

$$\tau = \frac{2 E W}{C_1 D} \qquad (6)$$

where

$D$ = pile diameter in inches
$W$ = pile movement in inches
$E$ = pressiometric modulus of deformation, T.S.F.
$\tau$ = 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:

$$W = \frac{N}{4\ E_a} \quad \lambda_2 \quad D \qquad (7)$$

where

$N$ = Normal stress acting on soil at pile tip
$E_a$ = Alternated modulus
$\lambda_2$ = 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 $\tau$ 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 $S_s/\sigma$ 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 slatted 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 $P_L{}^1$ 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

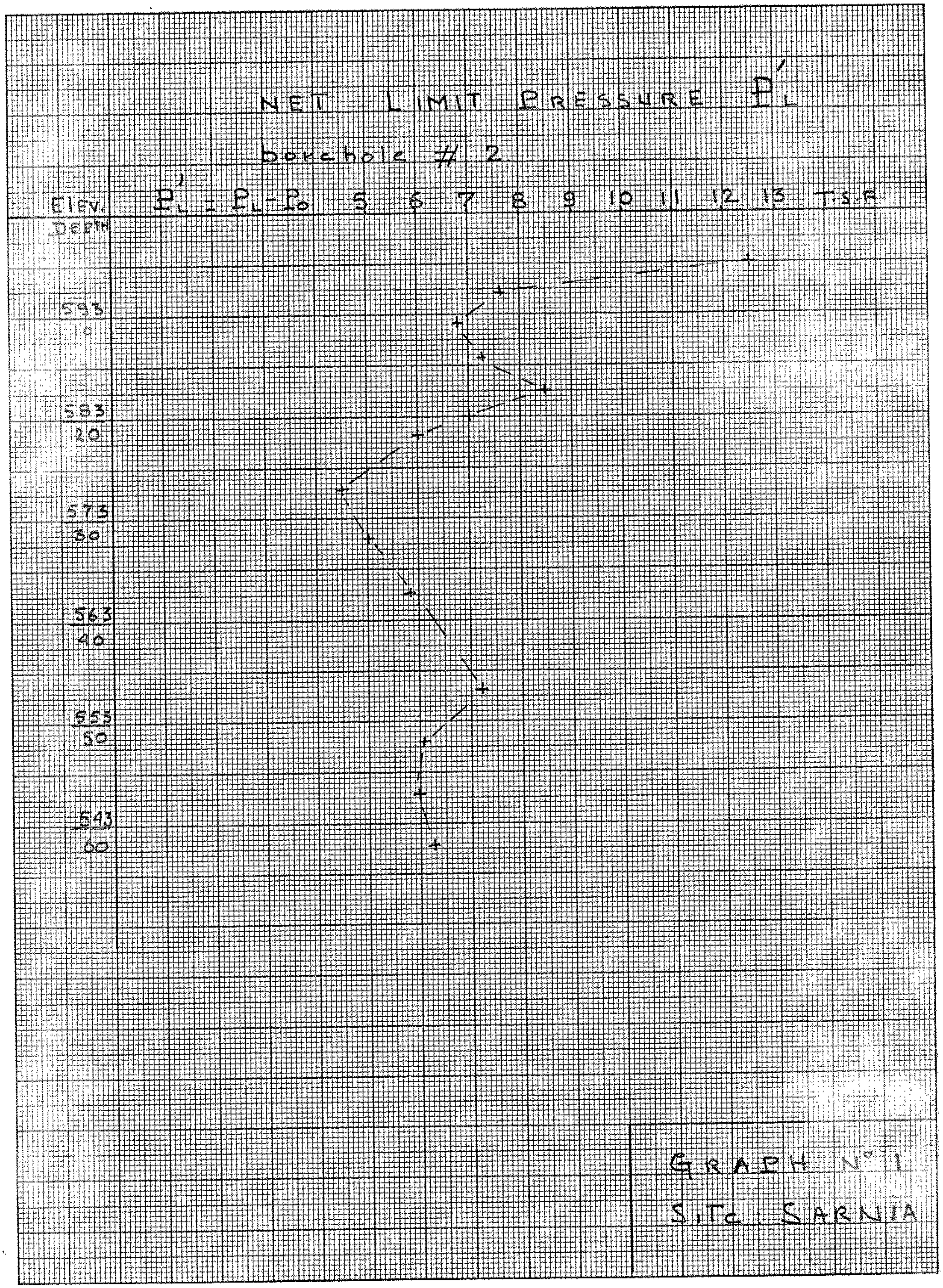
The anticipated behaviour of H, tubular steel and "Hercules" concrete piles driven at depths of 20, 40 and 60 ft. was studied utilizing the method described in a previous paragraph. The results are presented in the following pages under the form of graphs numbered 4-12; a summary showing the allowable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

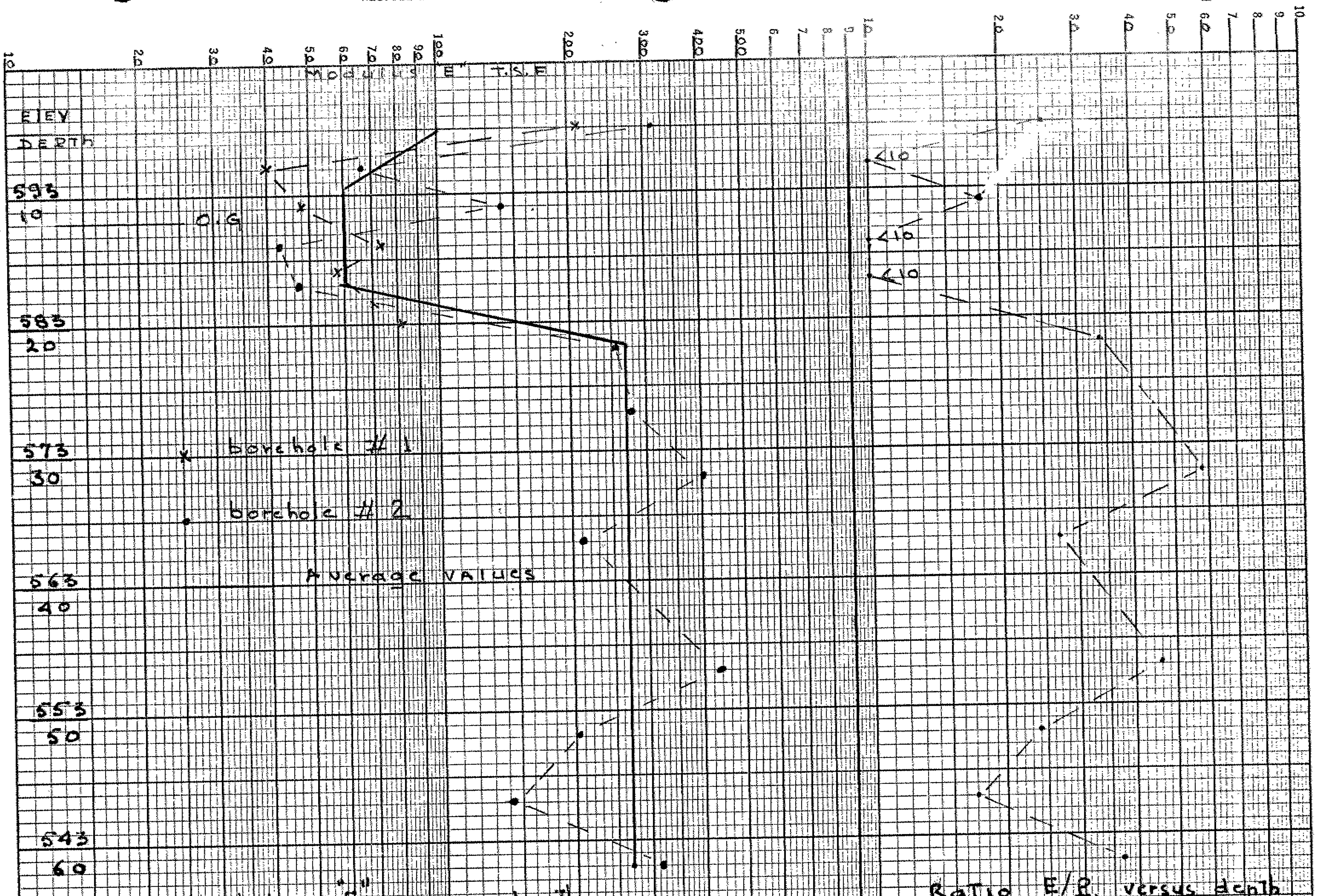
## 9.0 DISCUSSION

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

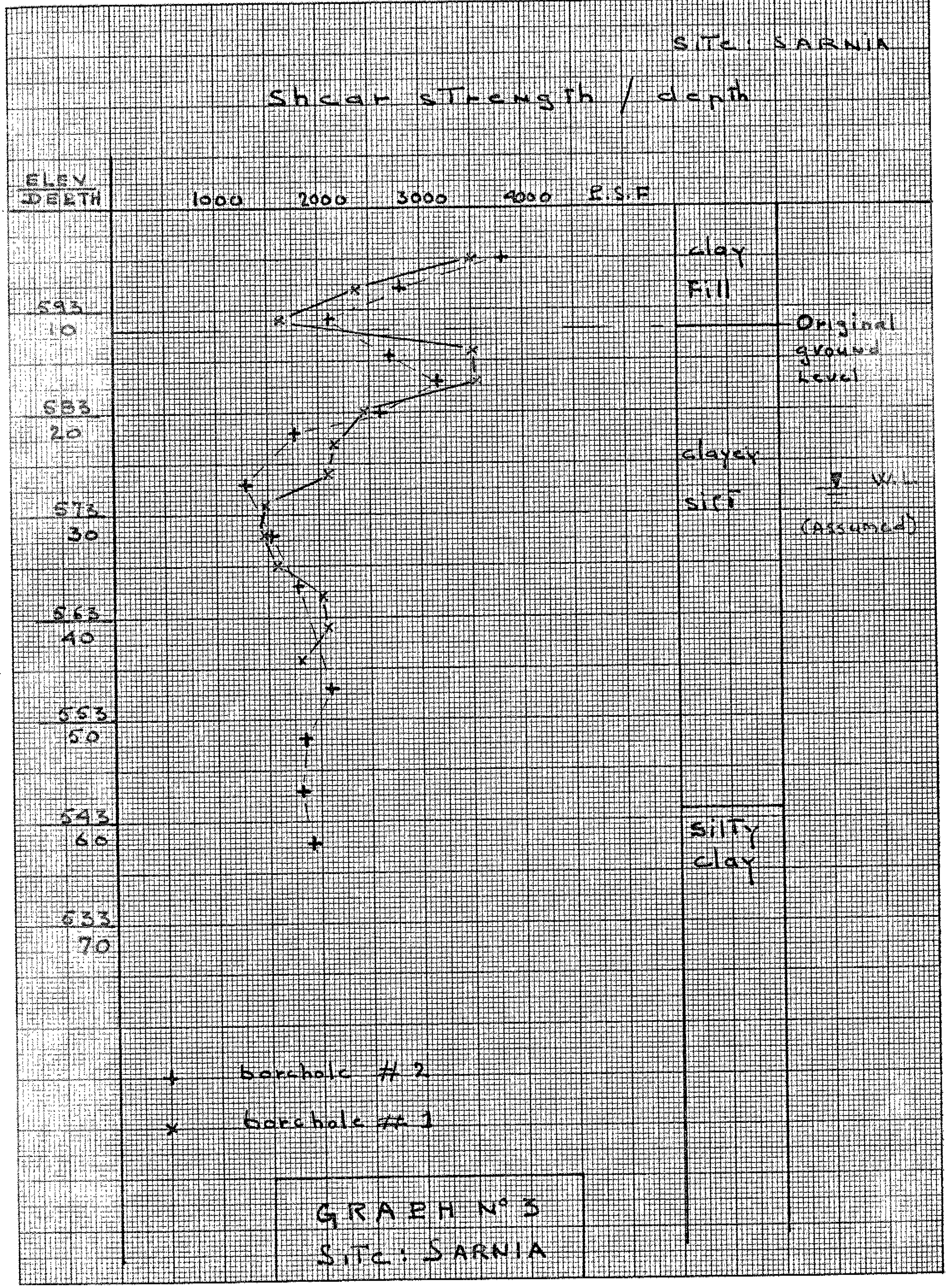

NET LIMIT PRESSURE P'L
borehole # 2
Elev.
Depth
P'L = PL - Po
5 6 7 8 9 10 11 12 13 T.S.F
593
10
583
20
573
30
563
40
553
50
543
60
GRAPH No 1
Site SARNIA




GRAPH N°2

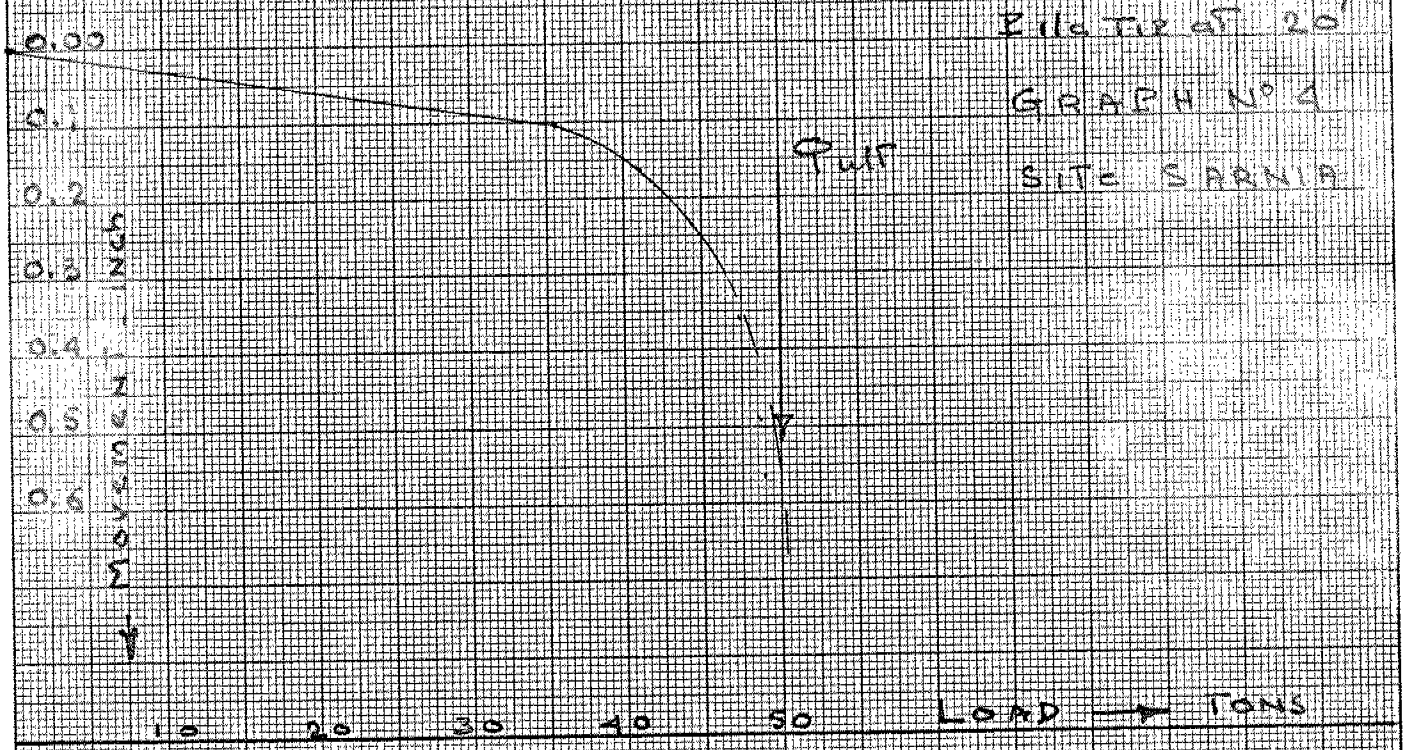
SITE: SARNIA . borehole #2

13


SITE: SARNIA
Shear strength / depth
ELEV
DEPTH
1000
2000
3000
4000
P.S.F
593
10
583
20
573
30
563
40
553
50
543
60
533
70
clay
Fill
Original
ground
Level
clayey
silt
W.L.
(Assumed)
silty
clay
borehole #2
borehole #1
GRAPH No 3
SITE: SARNIA

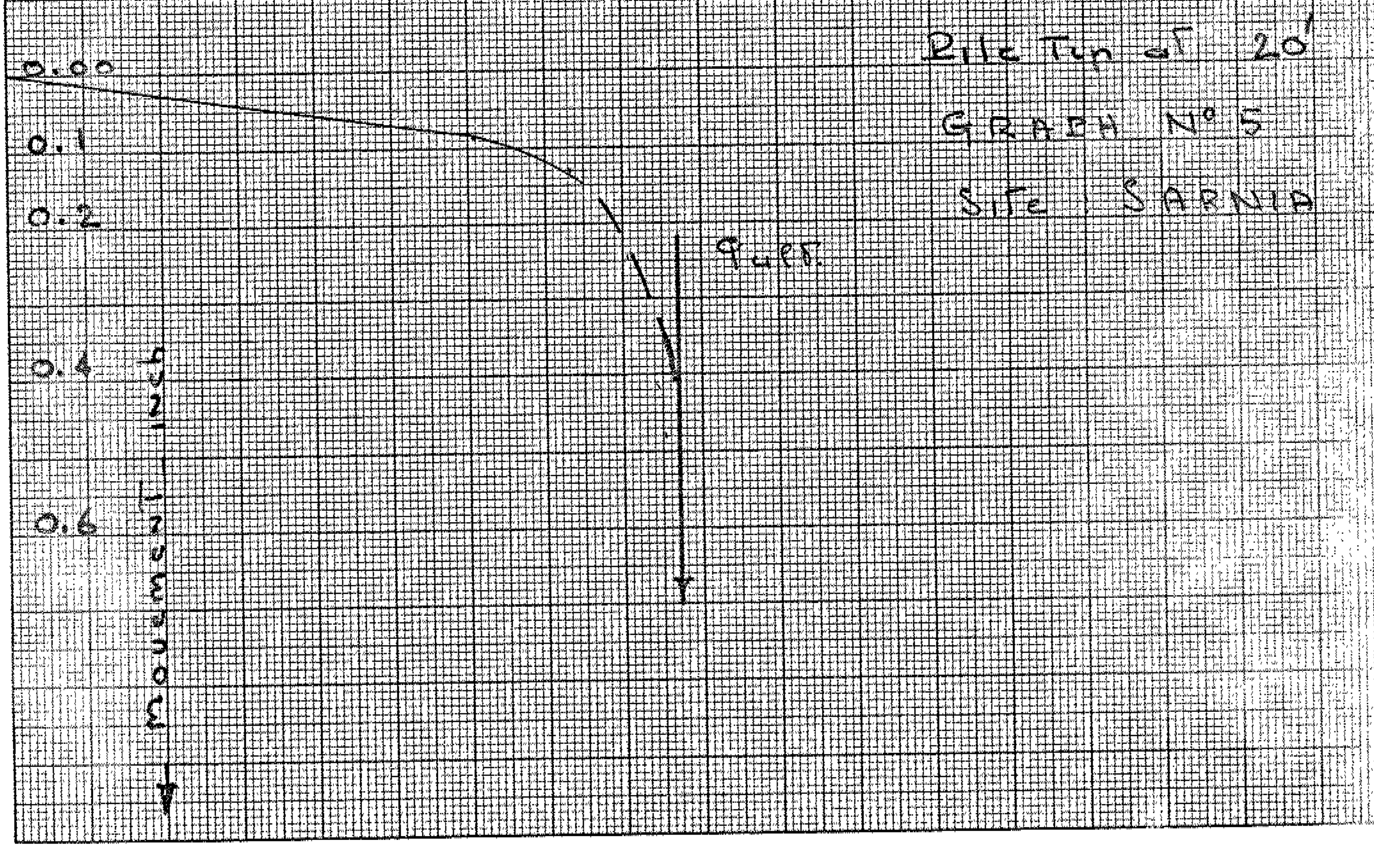
ANTICIPATED LOAD/SETTLEMENT

12" "Herculex" Pile



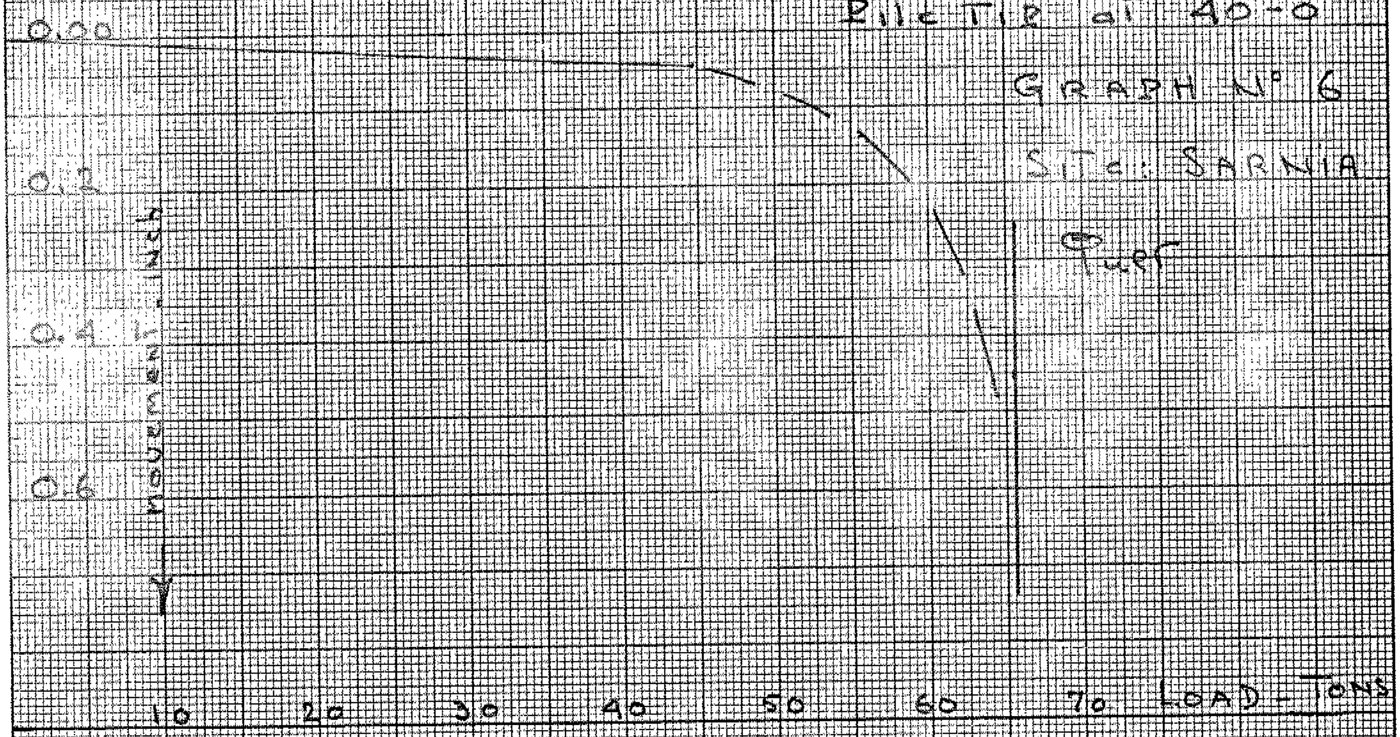

ANTICIPATED LOAD/SETTLEMENT

12".3/4 Tubular steel Pile



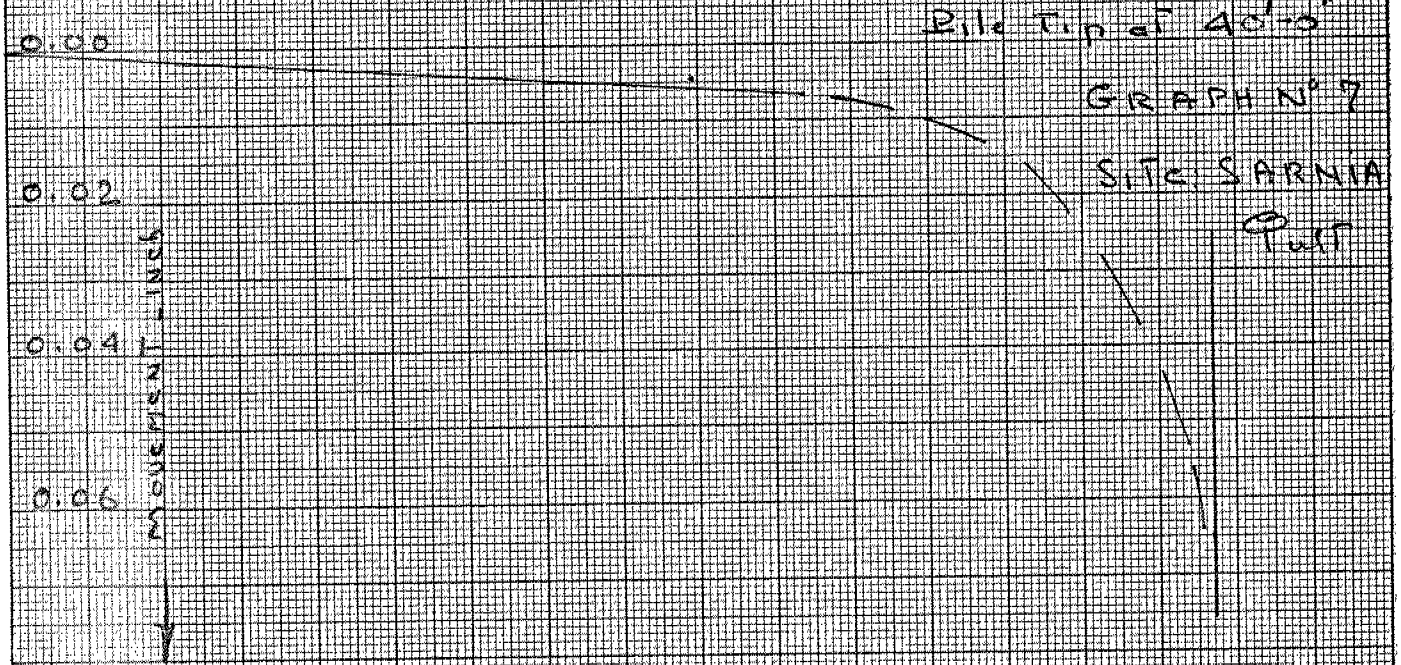
ANTICIPATED LOAD/Settlement

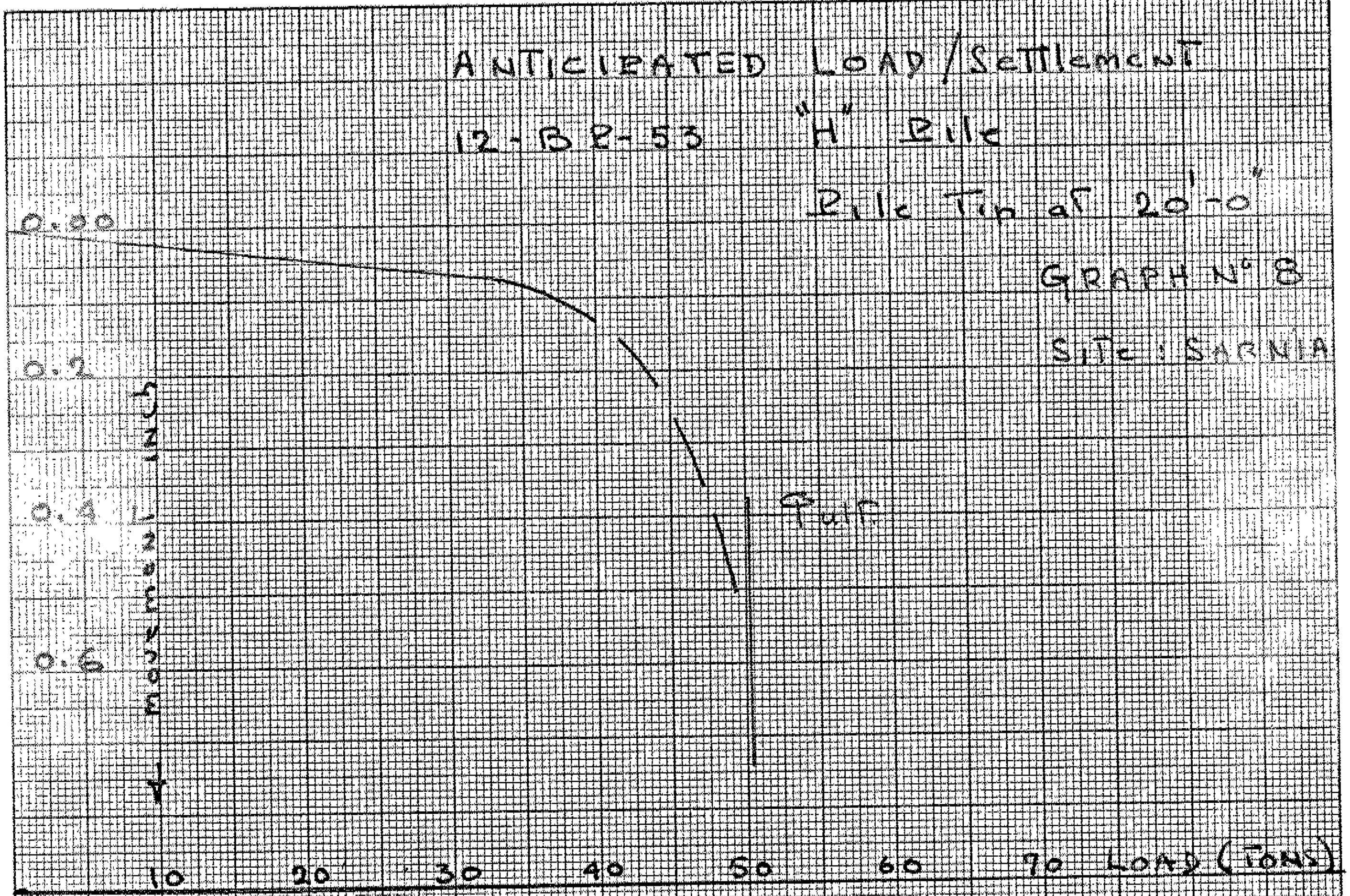
12" 3/4 Tubular Steel Pile

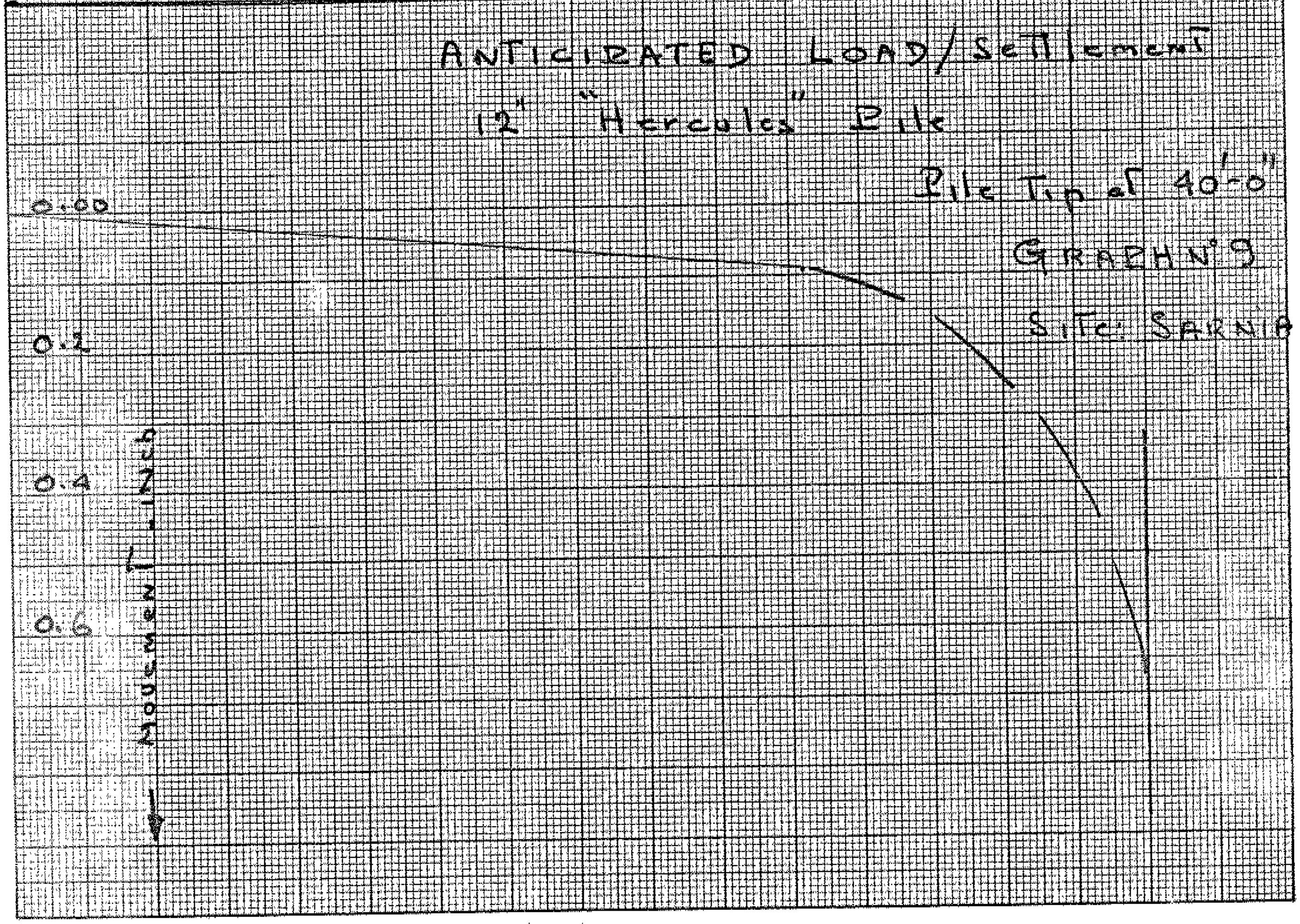


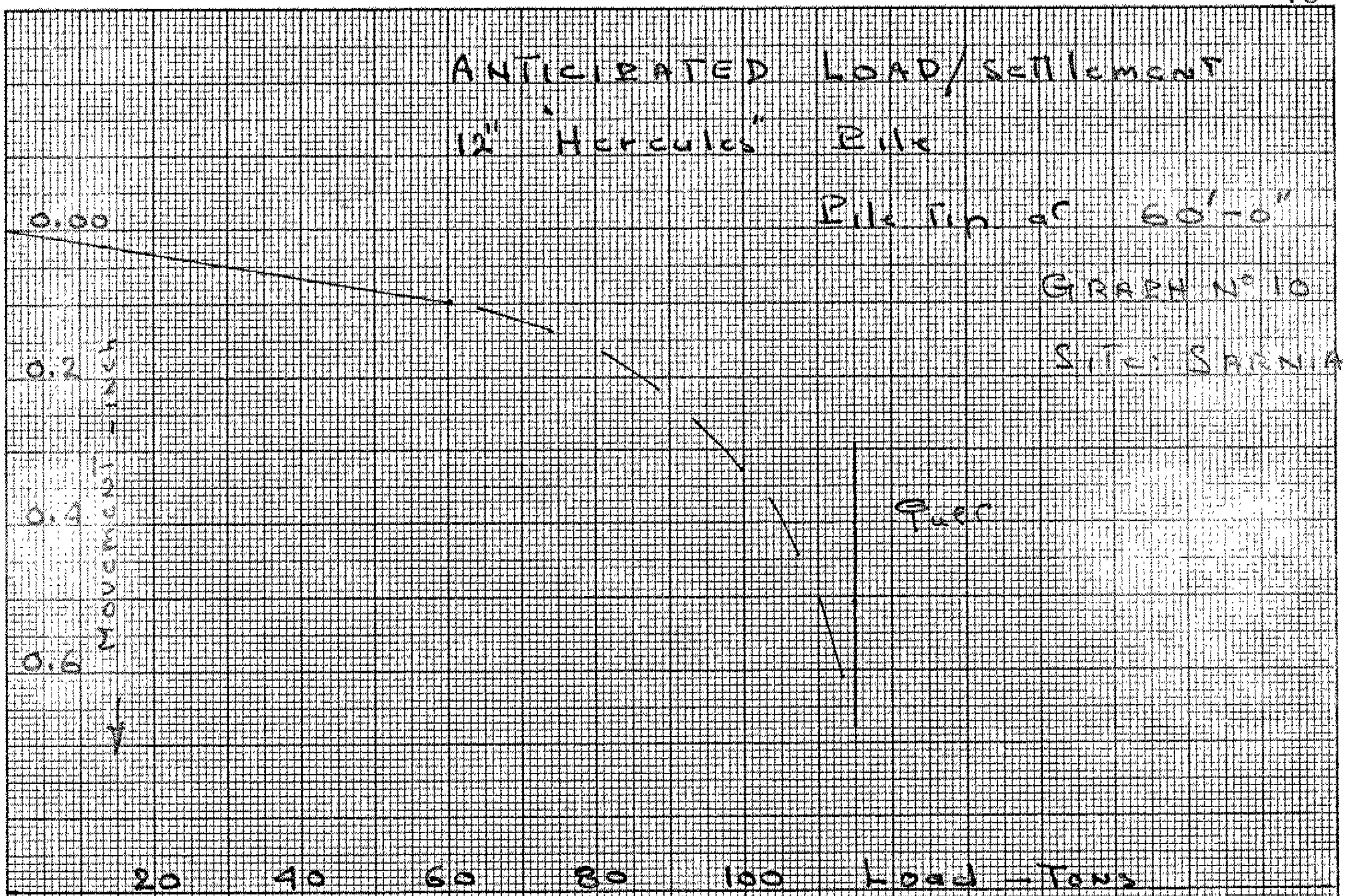

ANTICIPATED LOAD/Settlement

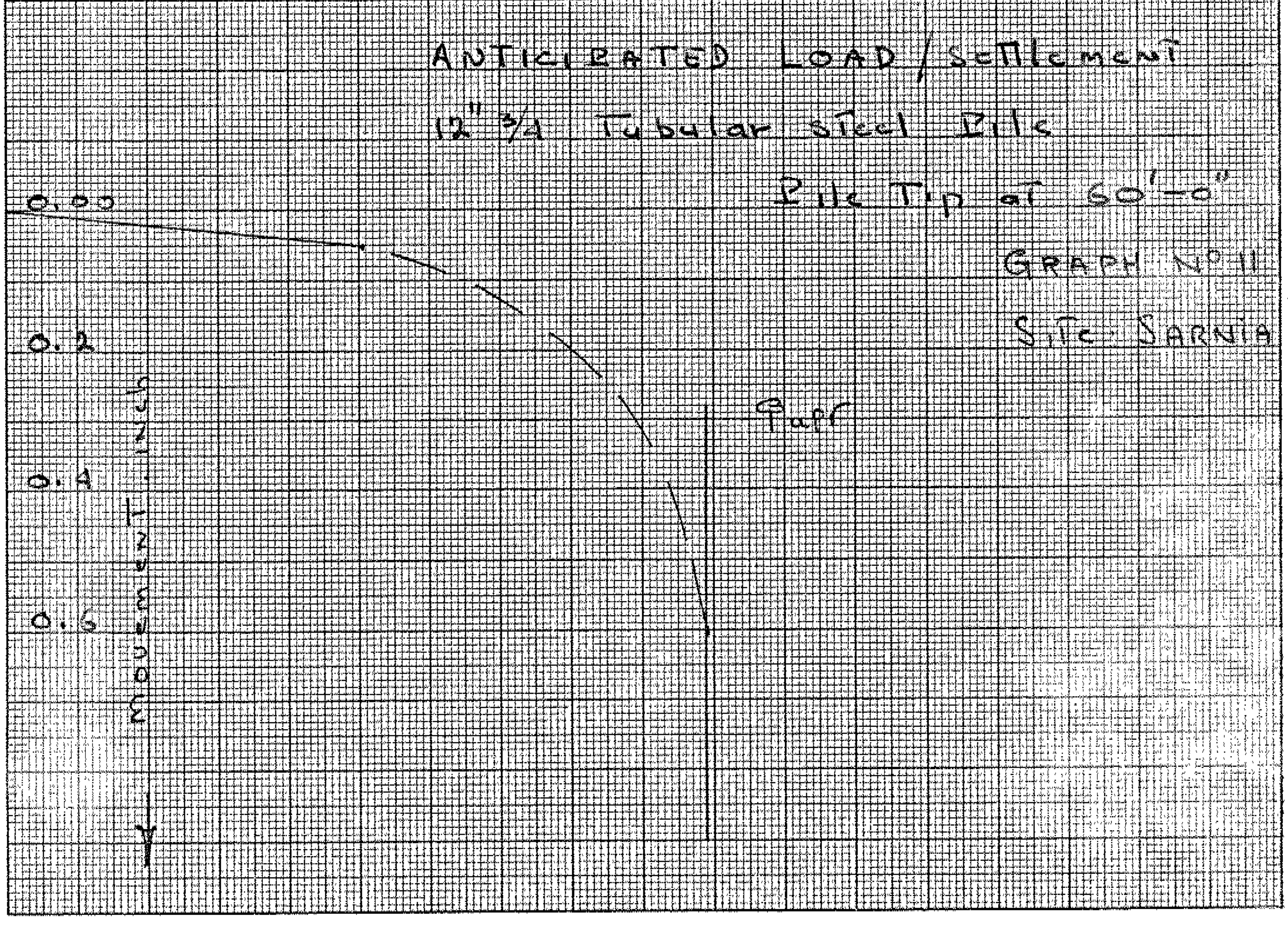
12-BP-53 "H" Pile




ANTICIPATED LOAD/SETTLEMENT
12-BP-53 "H" Pile
Pile Tip at 20'-0"
GRAPH N° 8
SITE: SARNIA
0.00
0.2
0.4
0.6
MOVEMENT INCH
Fail
10
20
30
40
50
60
70
LOAD (TONS)


ANTICIPATED LOAD/SETTLEMENT
12" "Hercules" Pile
Pile Tip at 40'-0"
GRAPH N° 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
0.00
0.2
0.4
0.6
MOVEMENT - INCH
20
40
60
80
100
Load - Tons


ANTICIPATED LOAD/SETTLEMENT
12" 3/4 Tubular steel Pile
Pile Tip at 60'-0"
GRAPH No 11
SITE SARNIA
0.00
0.2
0.4
0.6
MOVEMENT - INCH

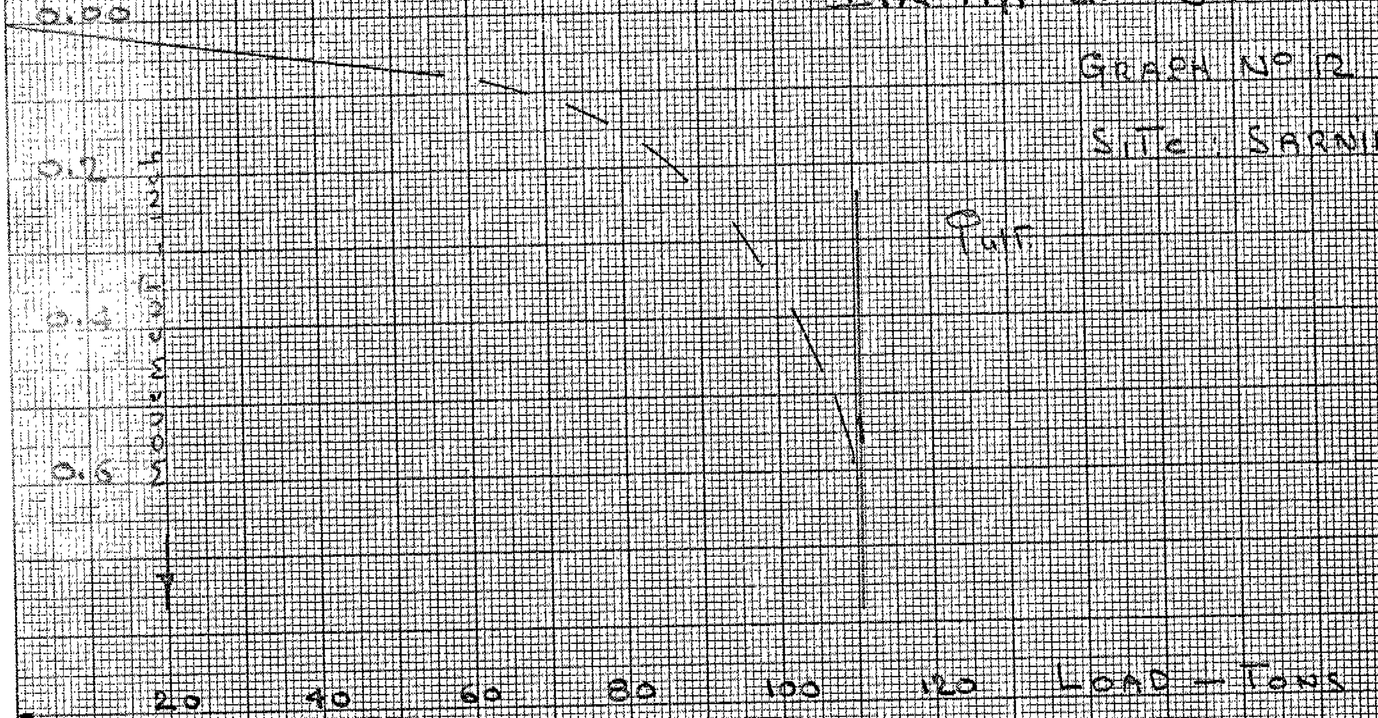
ANTICIPATED LOAD/SETTLEMENT

12-BP-53 "H" Pile

Pile Tip at 60'-0"

GRAPH No 12

SITE : SARNIA



## 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 |

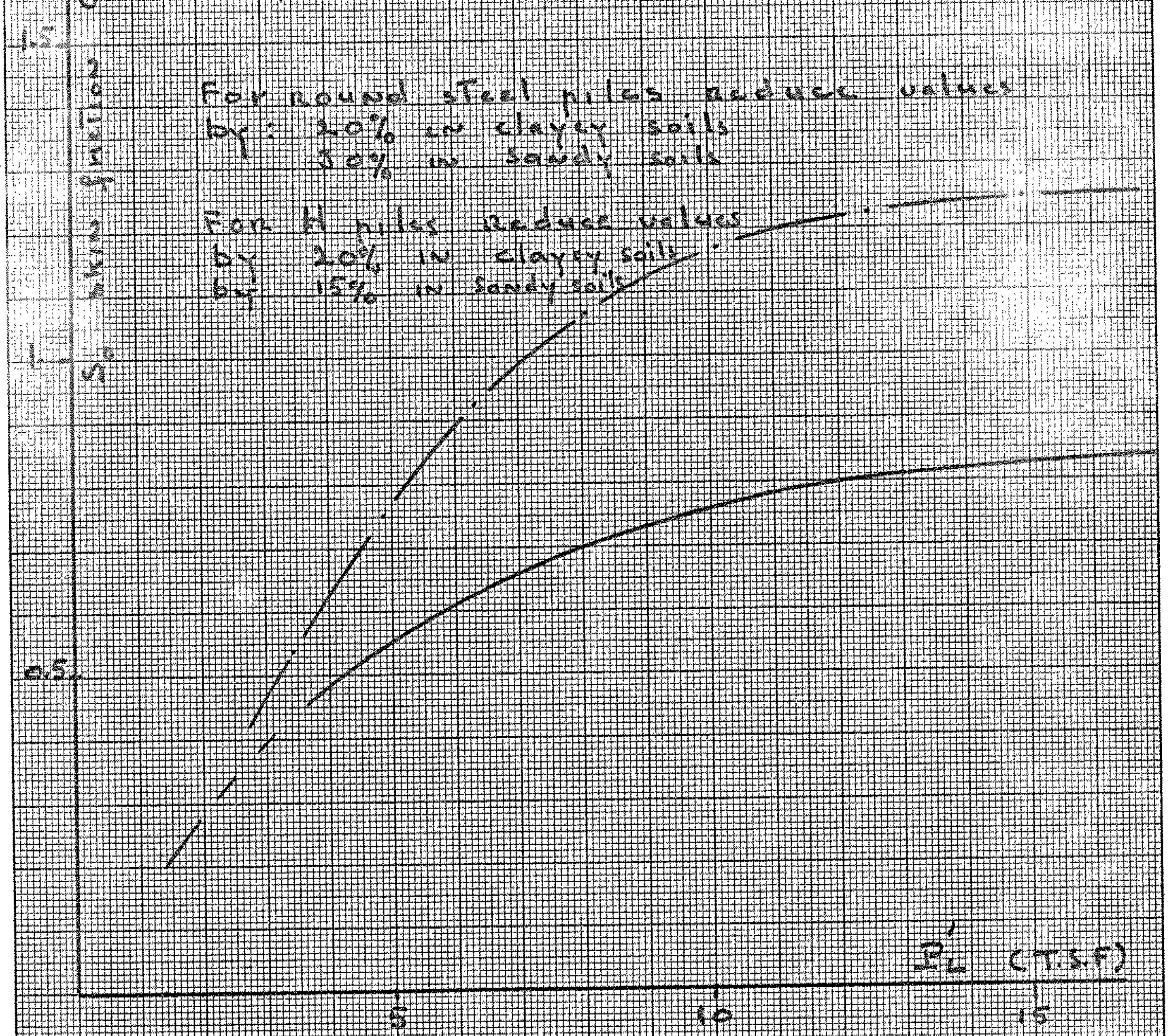
## SUMMARY OF RESULTS

| 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

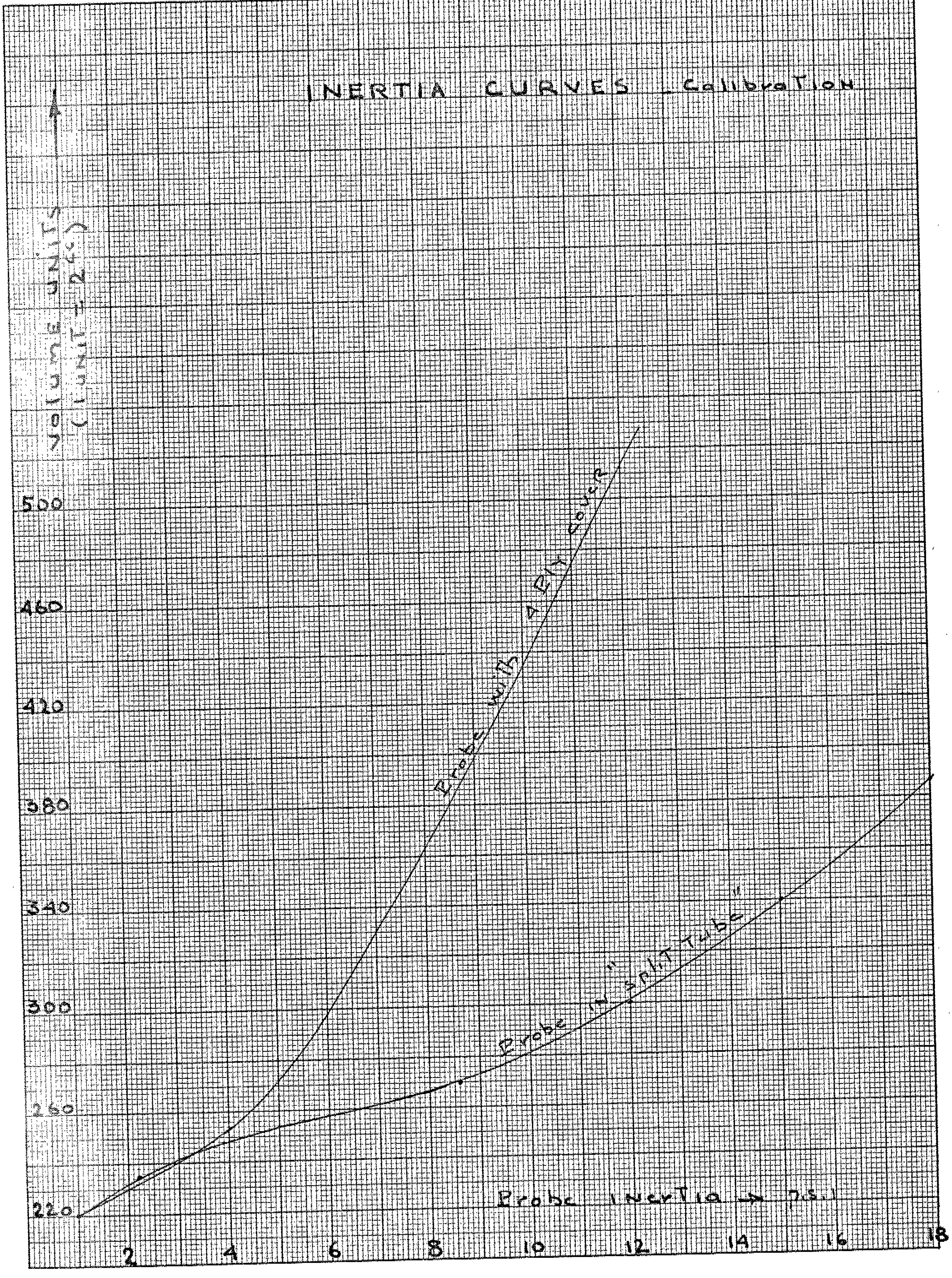
## for driven or drilled piles (concrete)

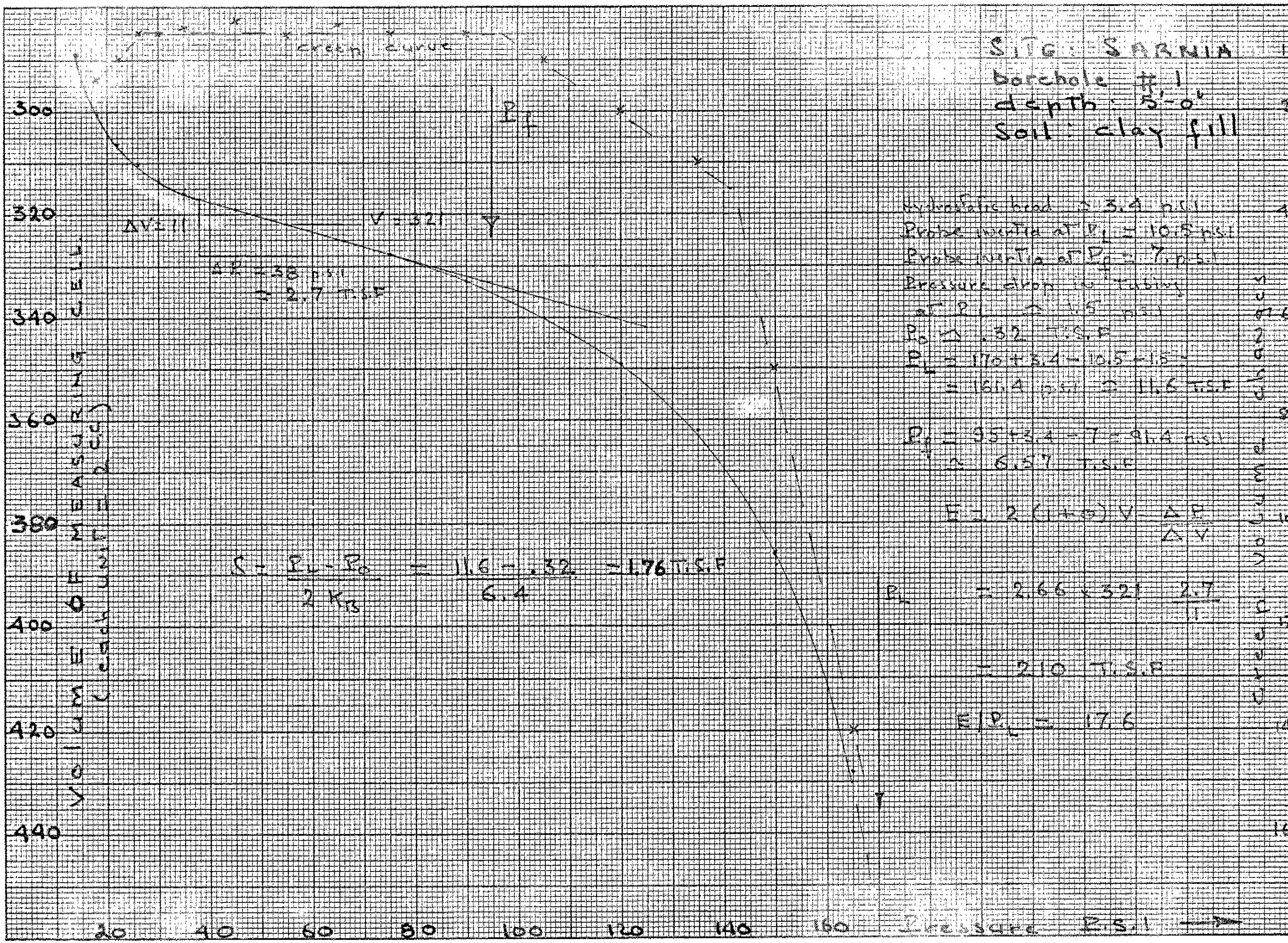



— — · — skin friction within 3 diam. of Tip

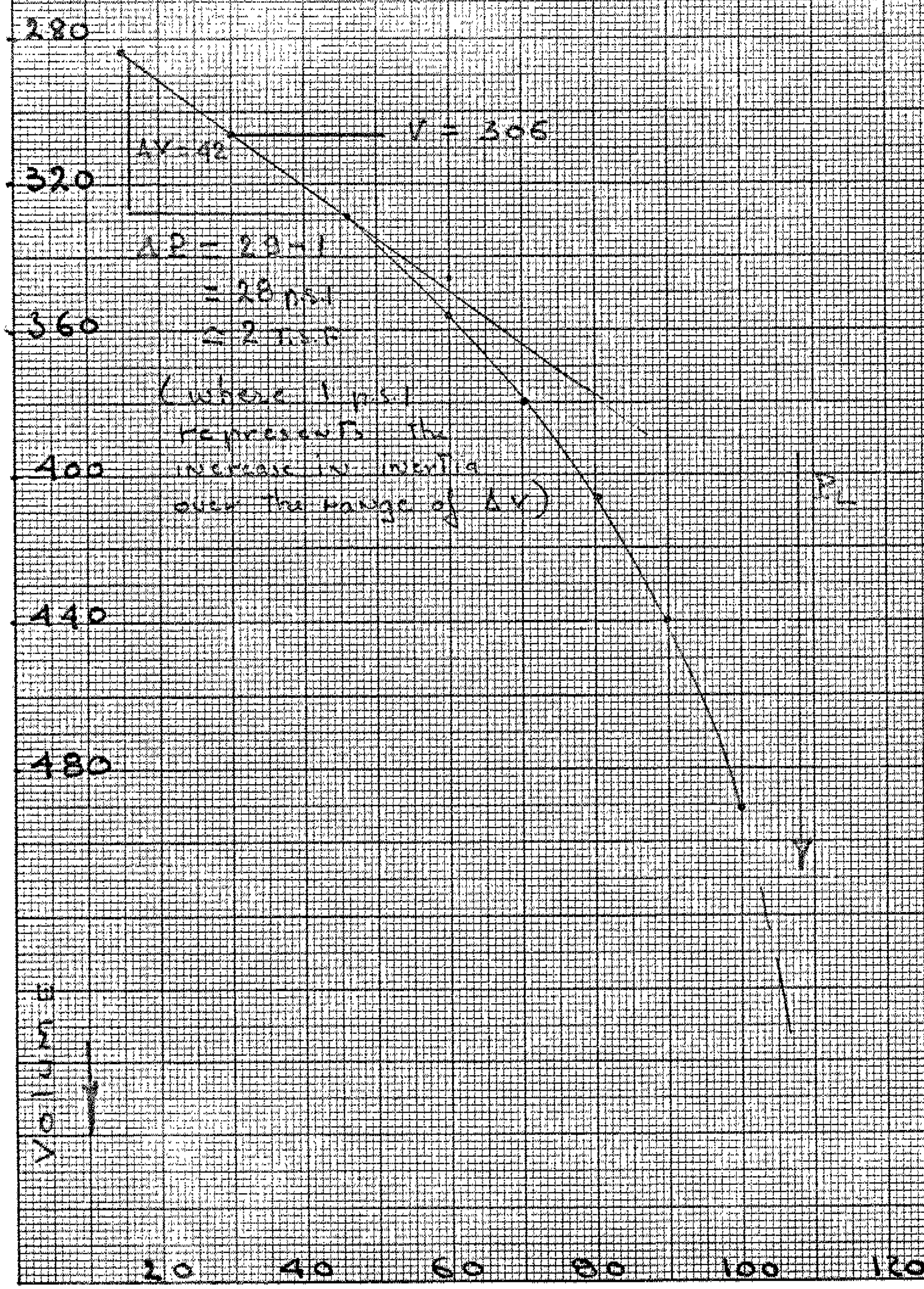
——— skin friction along shaft.

Fig 14


INERTIA CURVES _ Calibration
Volume units
(1 unit - 2 cc)
500
460
420
380
340
300
260
220
Probe with A PLY cover
Probe in "split tube"
Probe Inertia → T.S.I
2
4
6
8
10
12
14
16
18


creep curve
SITE : SARNIA
borehole #1
depth : 5'-0"
Soil : clay fill
$P_f$
ΔV = 11
V = 321
ΔP = 38 psi
= 2.7 T.S.F
Hydrostatic head = 3.4 psi
Probe inertia at $P_L$ = 10.5 psi
Probe inertia at $P_f$ = 7. psi
Pressure drop in tubing
at $P_L$ ≈ 1.5 psi
$P_o$ ≈ .32 T.S.F
$P_L$ = 170 + 3.4 − 10.5 − 1.5 =
= 161.4 psi ≈ 11.6 T.S.F
$P_f$ = 95 + 3.4 − 7 = 91.4 psi
≈ 6.57 T.S.F
E = 2 (1 + σ) V ΔP / ΔV
= 2.66 × 321 × 2.7 / 11
= 210 T.S.F
$E/P_L$ = 17.6
S = ($P_L$ − $P_o$) / 2 $K_B$ = (11.6 − .32) / 6.4 = 1.76 T.S.F
VOLUME OF MEASURING CELL (each unit = 2 cc)
300
320
340
360
380
400
420
440
volume change 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

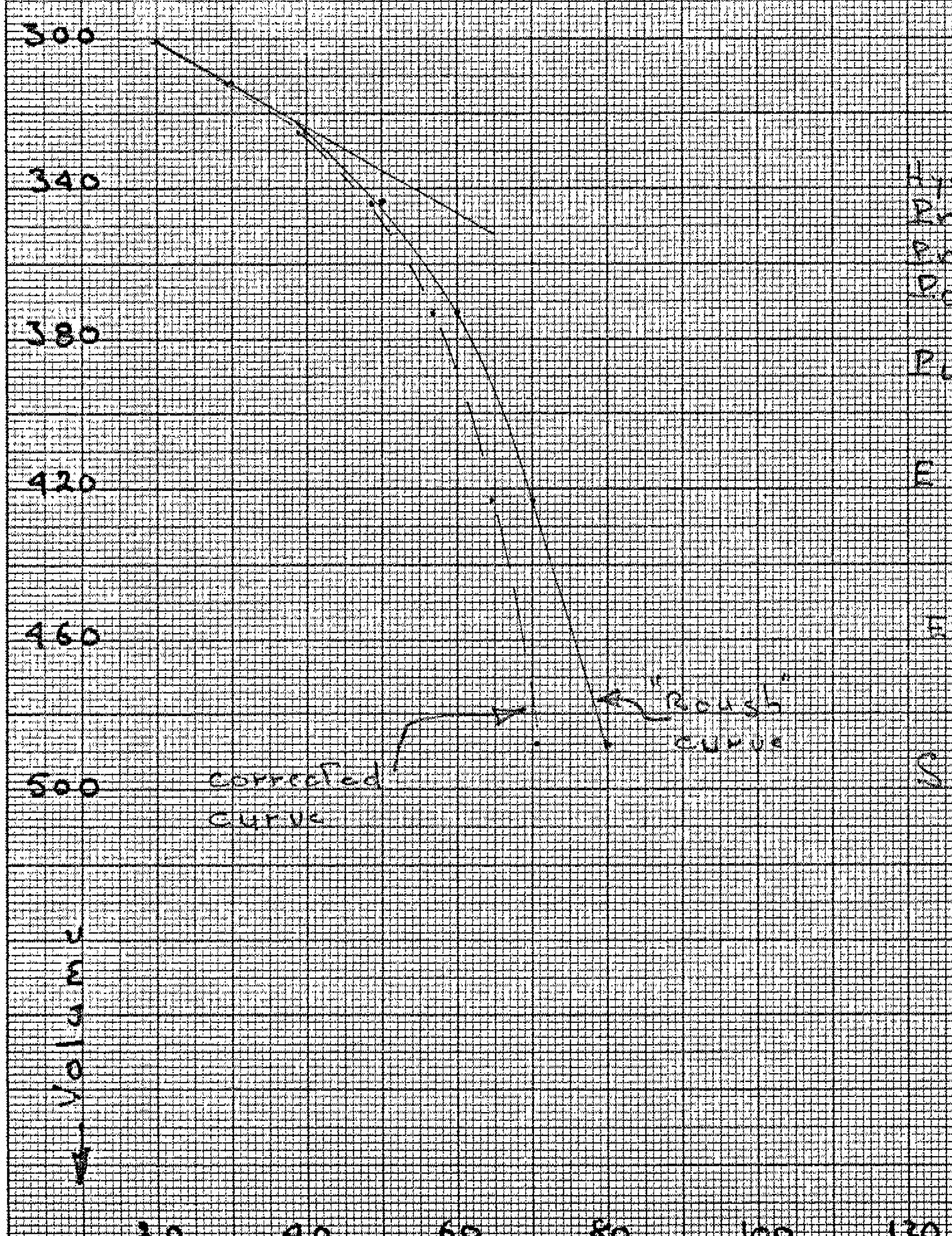
$P_L = 108 + 4.8 - 13 - 2 = 97.8$ p.s.i $= 6.95$ T.S.F

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

$E/P_L = 5.75$

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

SARNIA
borehole # 1
depth 11'-0"



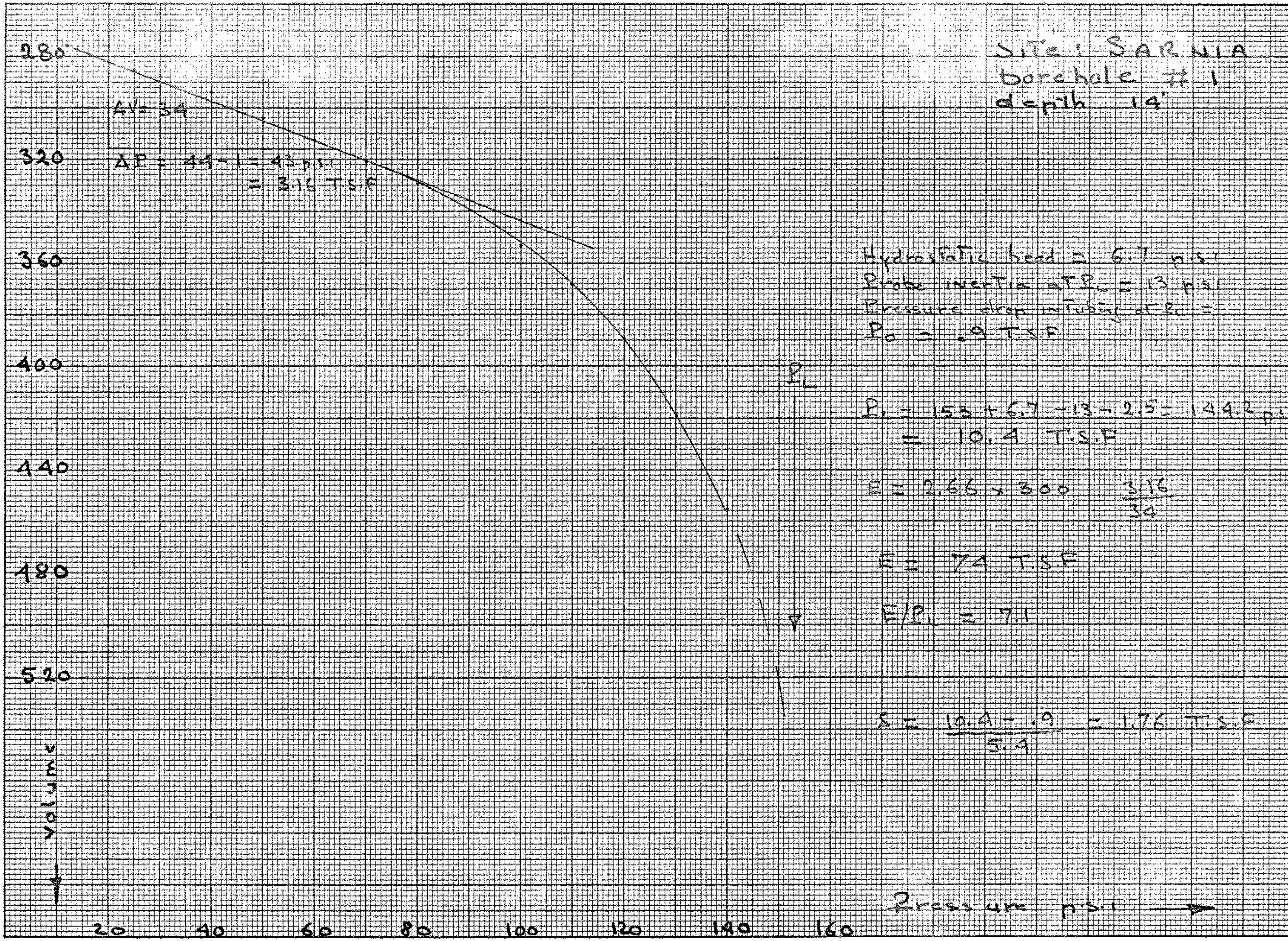

Hydrostatic head = 5.8 p.s.i
Probe inertia at $P_L$ = 11.3 p.s.i
Pressure drop in tubing at $P_L$ = 4 p.s.i
$P_o$ = .68 T.S.F

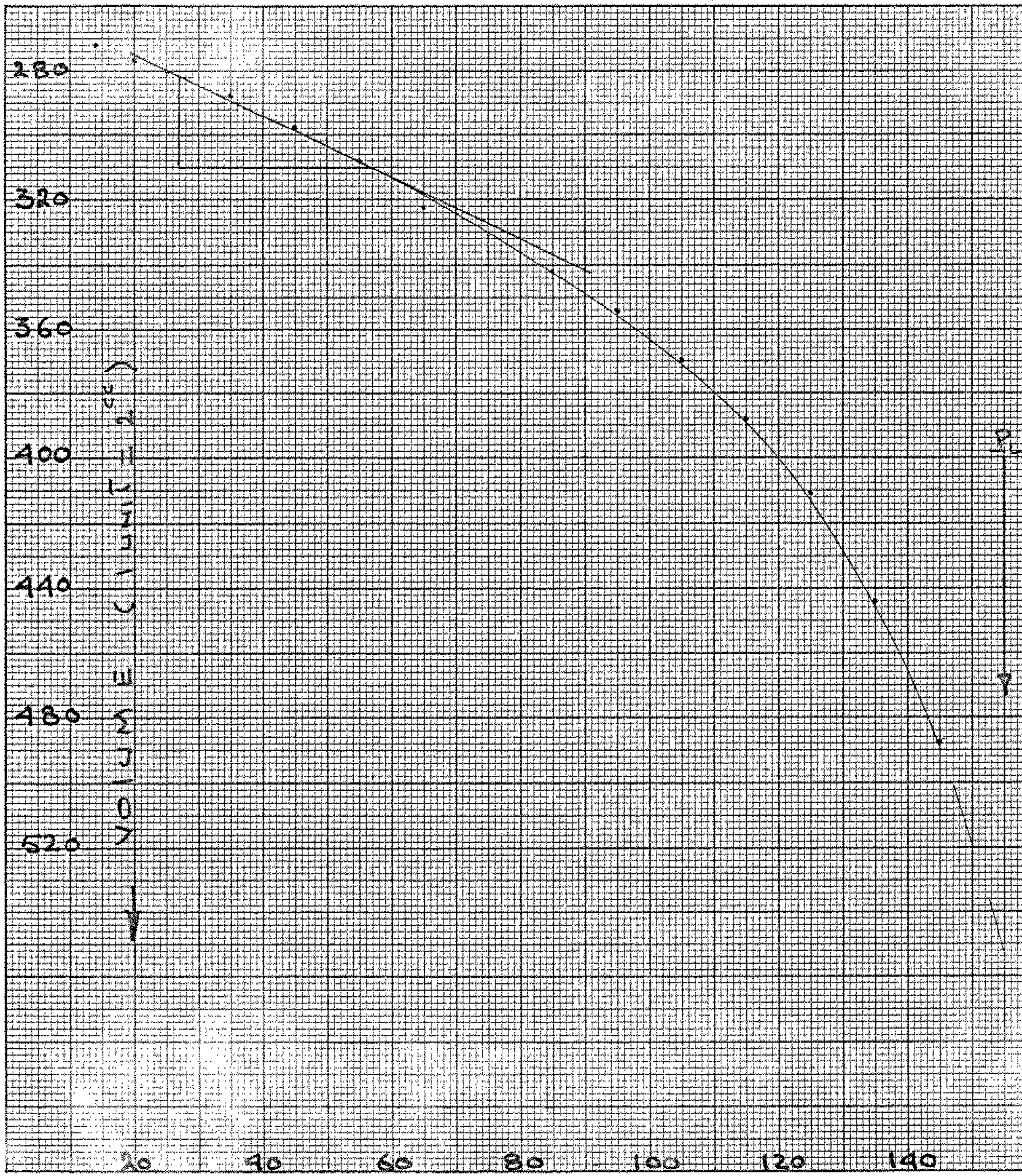
$P_L = 80 + 5.8 - 11.3 - 4 = 70.5$ p.s.i $= 5.07$ T.S.F

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

$E/P_L = 9.5$

$S = \dfrac{P_L - P_o}{2 K_B} = \dfrac{5.07 - .68}{5.4} = .815$ T.S.F


Site : SARNIA
borehole # 1
depth 14'
ΔV = 34
ΔP = 44 − 1 = 43 psi
= 3.16 T.S.F
Hydrostatic head = 6.7 psi
Probe inertia at $P_L$ = 13 psi
Pressure drop in Tubing at $P_L$ =
$P_o$ = .9 T.S.F
$P_L$
$P_L$ = 153 + 6.7 − 13 − 2.5 = 144.2 psi
= 10.4 T.S.F
E = 2.66 × 300 $\frac{3.16}{34}$
E = 74 T.S.F
E/$P_L$ = 7.1
S = $\frac{10.4 - .9}{5.9}$ = 1.76 T.S.F
280
320
360
400
440
480
520
Volume
20
40
60
80
100
120
140
160
Pressure p.s.i




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

Hydrostatic head = 8.6 p.s.i
Probe inertia of $P_L$ = 13 p.s.i
pressure drop in Tubing = 2.5 p.s.i
$P_o$ = 1.08 T.S.F

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

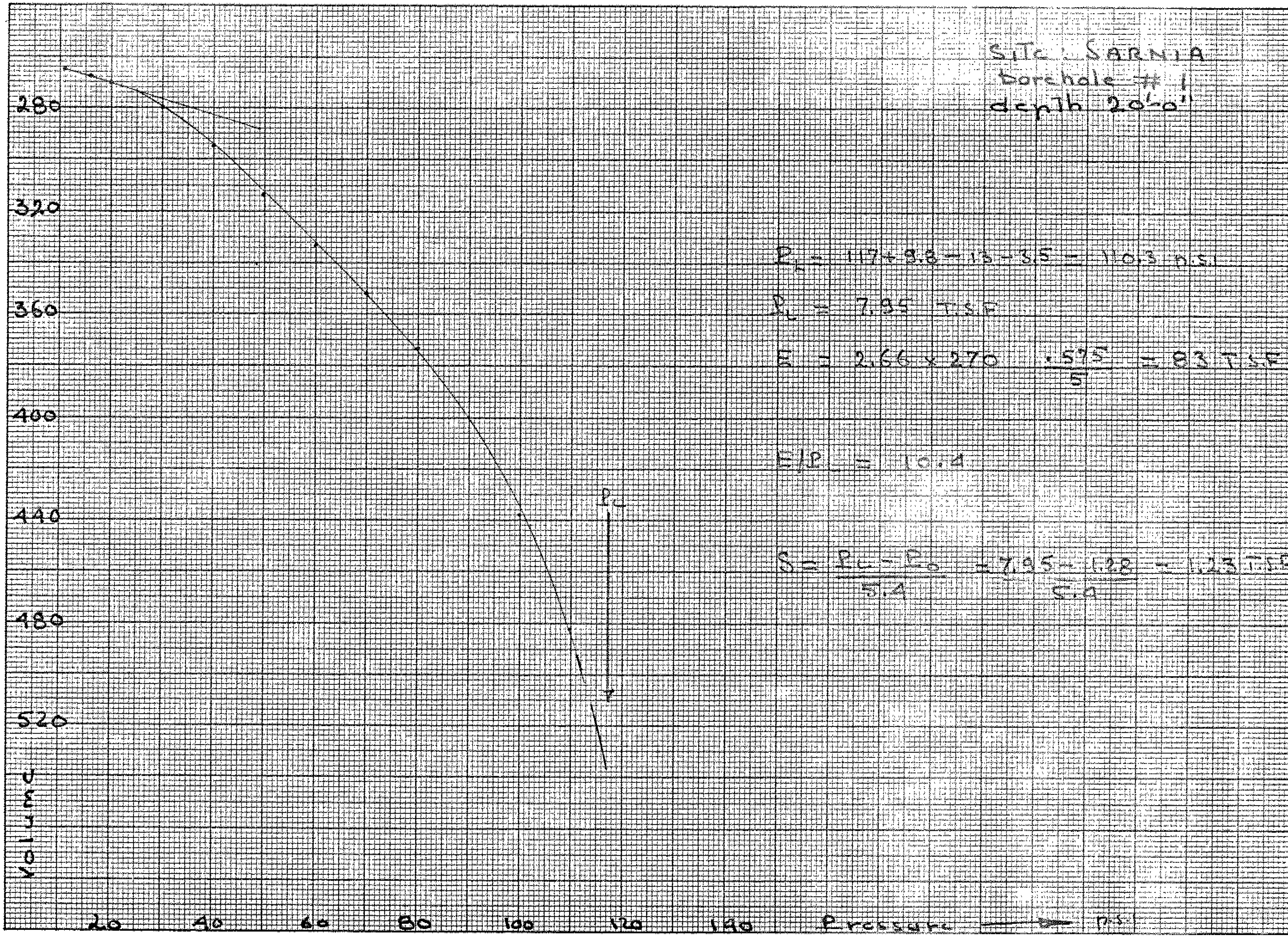
$P_L$ = 10.7 T.S.F

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

E = 58.5 T.S.F

$E/P_L$ = 5.5

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




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

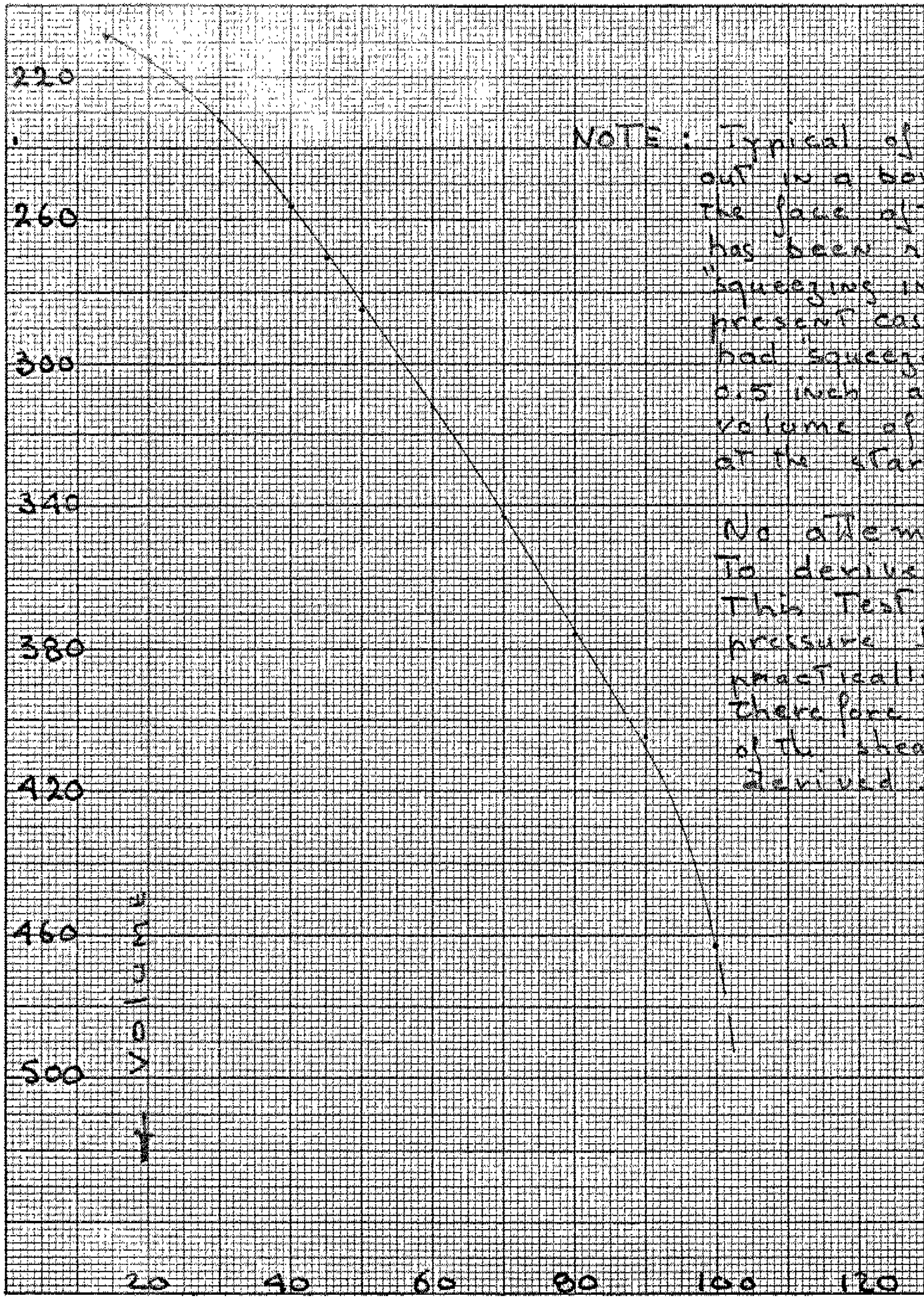
$E = 2.66 \times 270 \dfrac{.575}{5} = 83$ T.S.F

$E/P_L = 10.4$

$S = \dfrac{P_L - P_o}{5.4} = \dfrac{7.95 - 1.28}{5.4} = 1.23$ T.S.F

30

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.

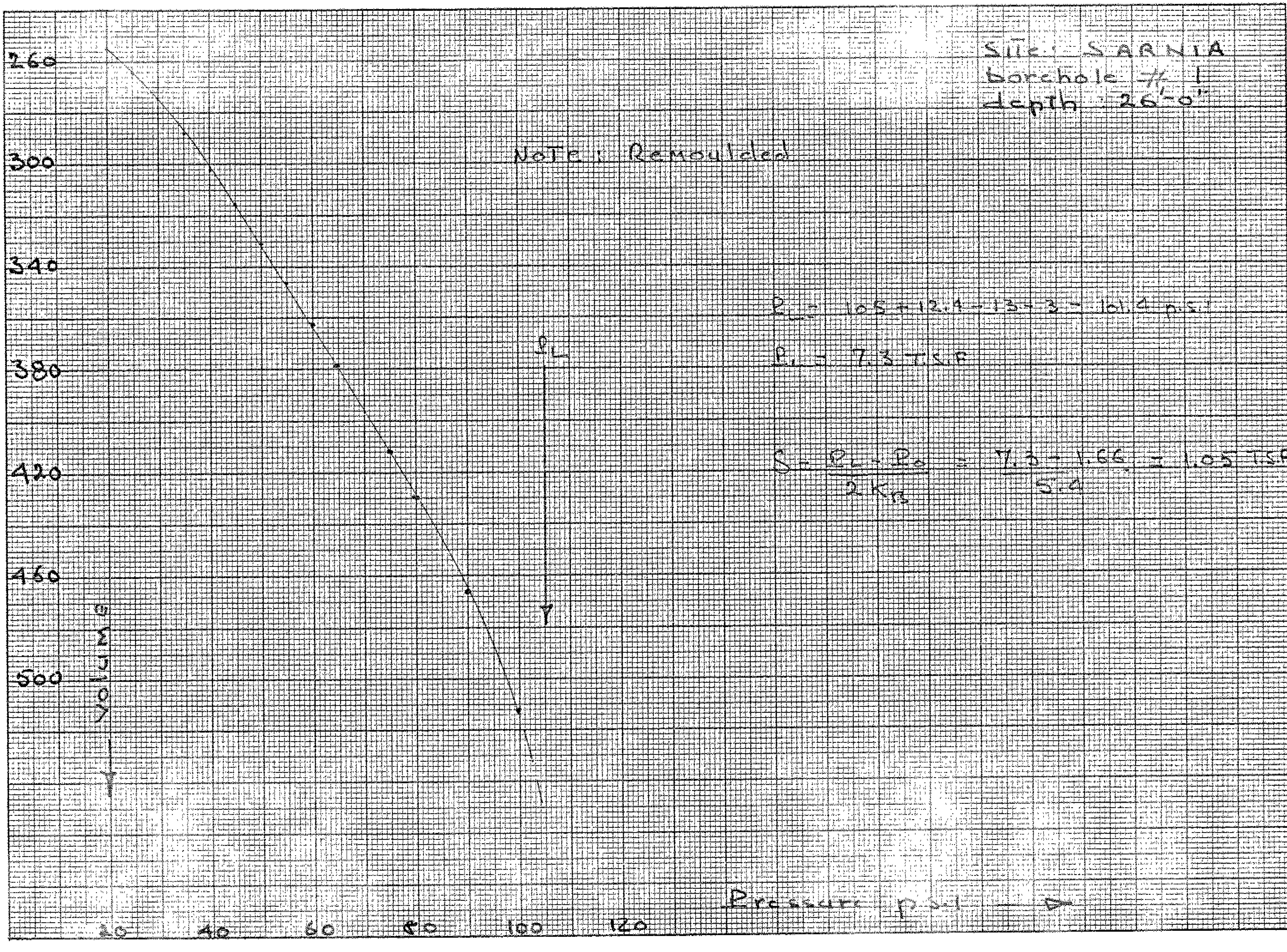
No attempt should be made to derive a Modulus from this Test, However the Limit pressure $P_L$ will remain practically unaffected and therefore an acceptable value of the shear strength can be derived.

$P_L = 104 + 11 - 11.6 - 2.5 = 100.9$ p.s.i

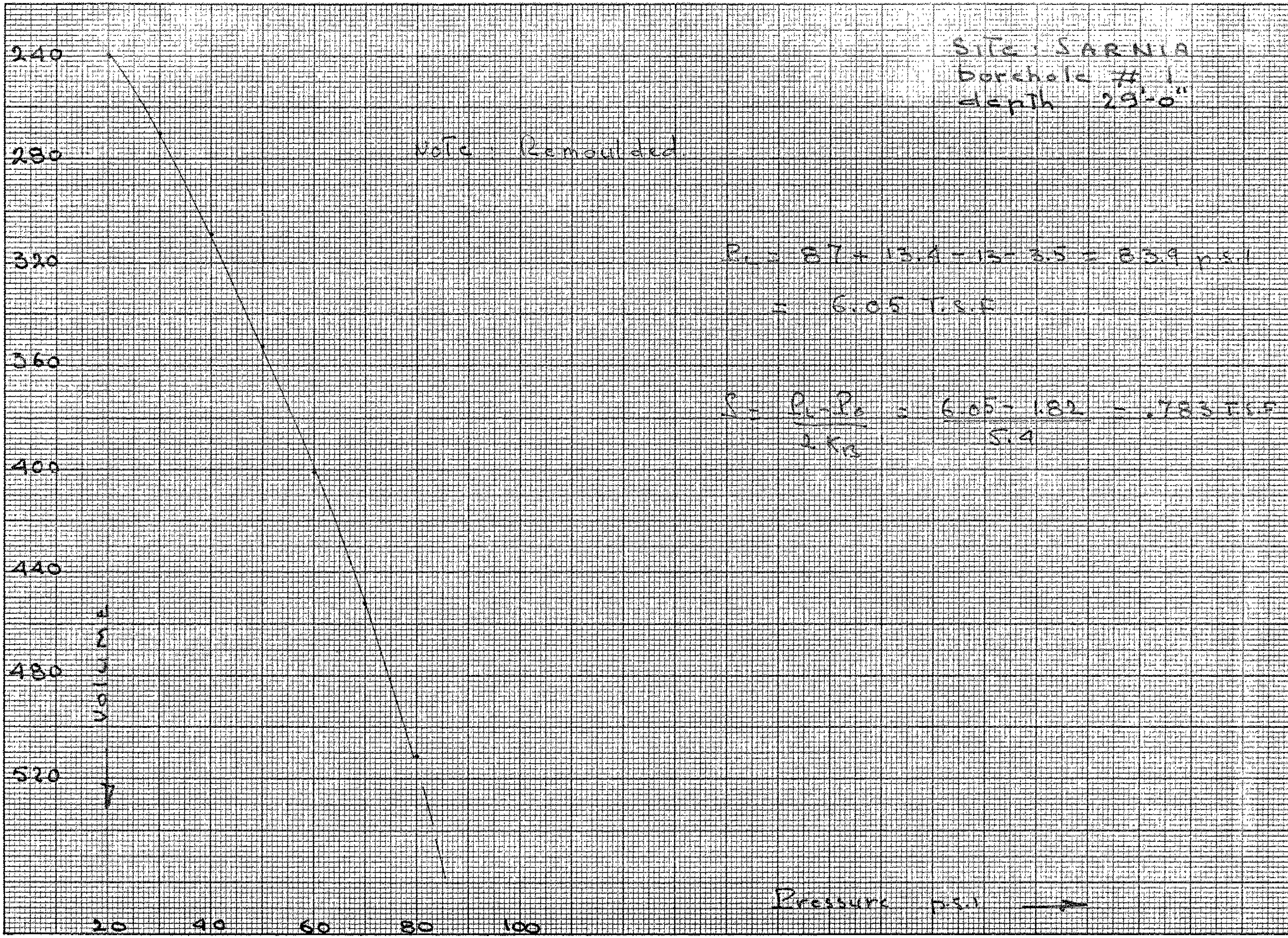
$= 7.25$ T.S.F

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

Pressure psi →

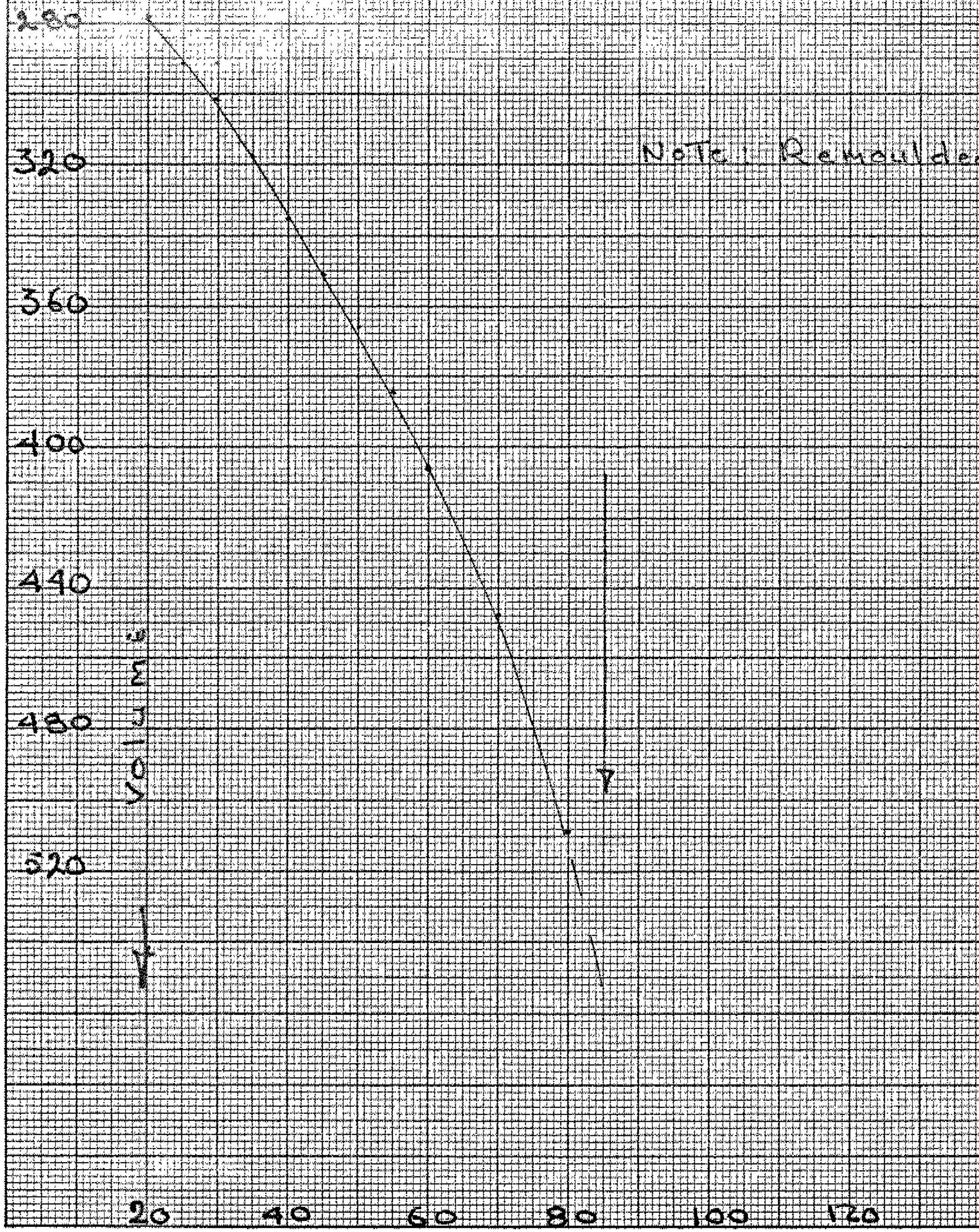

Site: SARNIA
borehole # 1
depth: 26'-0"
NOTE: Remoulded
$P_L = 105 + 12.4 - 13 - 3 = 101.4$ p.s.i
$P_L = 7.3$ T.S.F
$S = \frac{P_L - P_o}{2K_B} = \frac{7.3 - 1.66}{5.4} = 1.05$ T.S.F
$P_L$
260
300
340
380
420
460
500
Volume
20
40
60
80
100
120
Pressure p.s.i

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



Site: SARNIA
borehole # 1
depth 32'-0"

Note: Remoulded

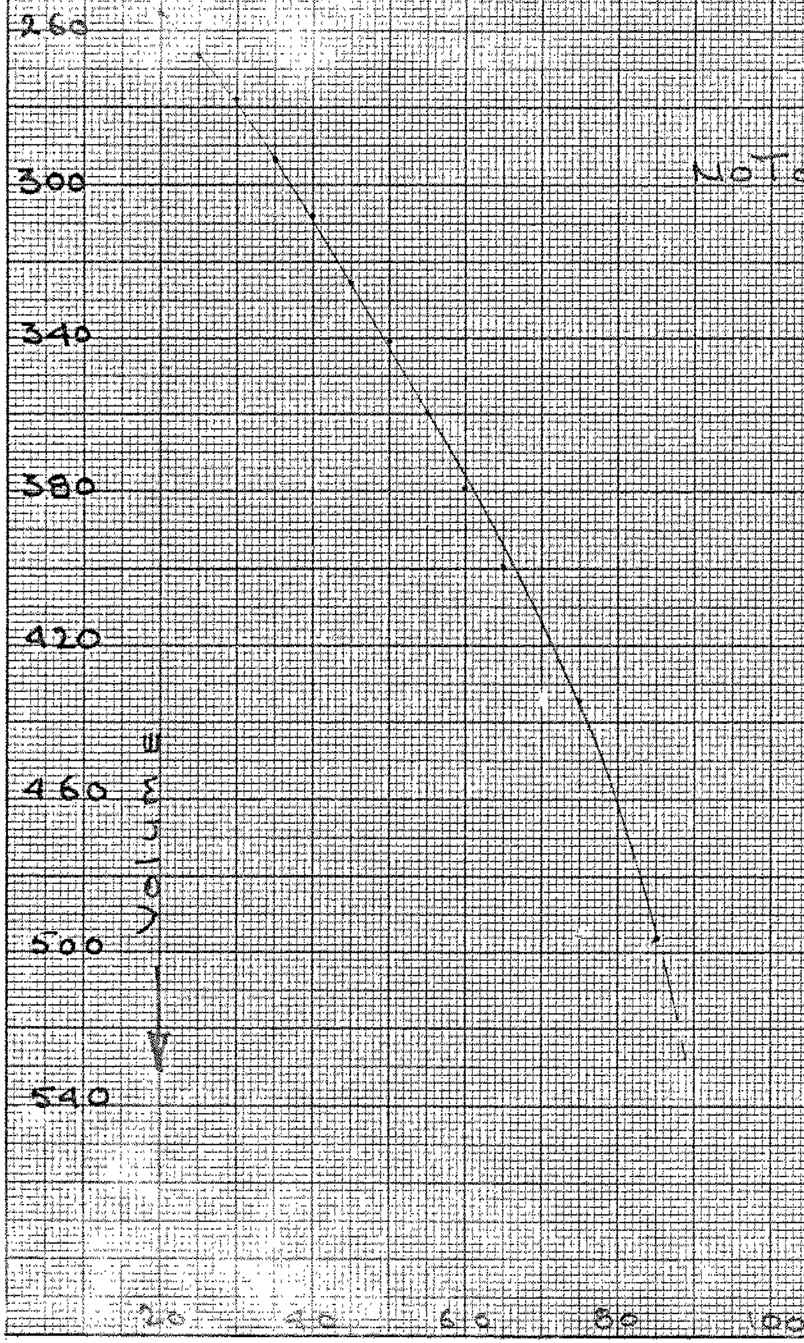



$$P_L = 85 + 13.4 - 13 - 3.5 = 81.9 \text{ p.s.i}$$
$$= 5.9 \text{ T.S.F}$$

$$S = \frac{P_L - P_o}{2K_b} = \frac{5.9 - 1.9}{5.4} = .74 \text{ T.S.F}$$

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

NOTE: Remoulded

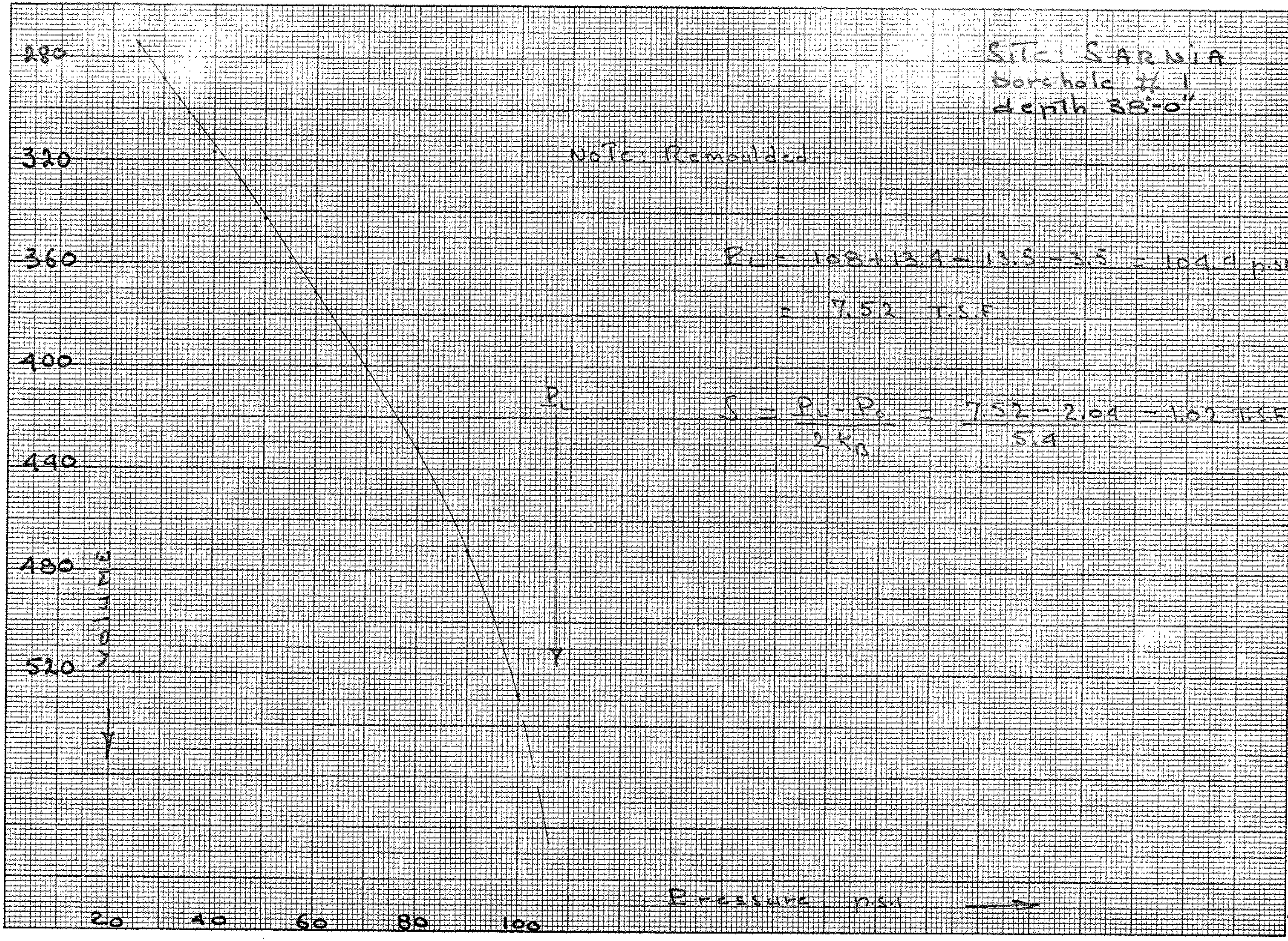



$P_L = 90 + 13.4 - 13 - 3.5 = 86.9$ p.s.i.

$= 6.25$ T.S.F

$S = \frac{P_L - P_o}{2K_B} = \frac{6.25 - 1.96}{5.4} = .795$ 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.

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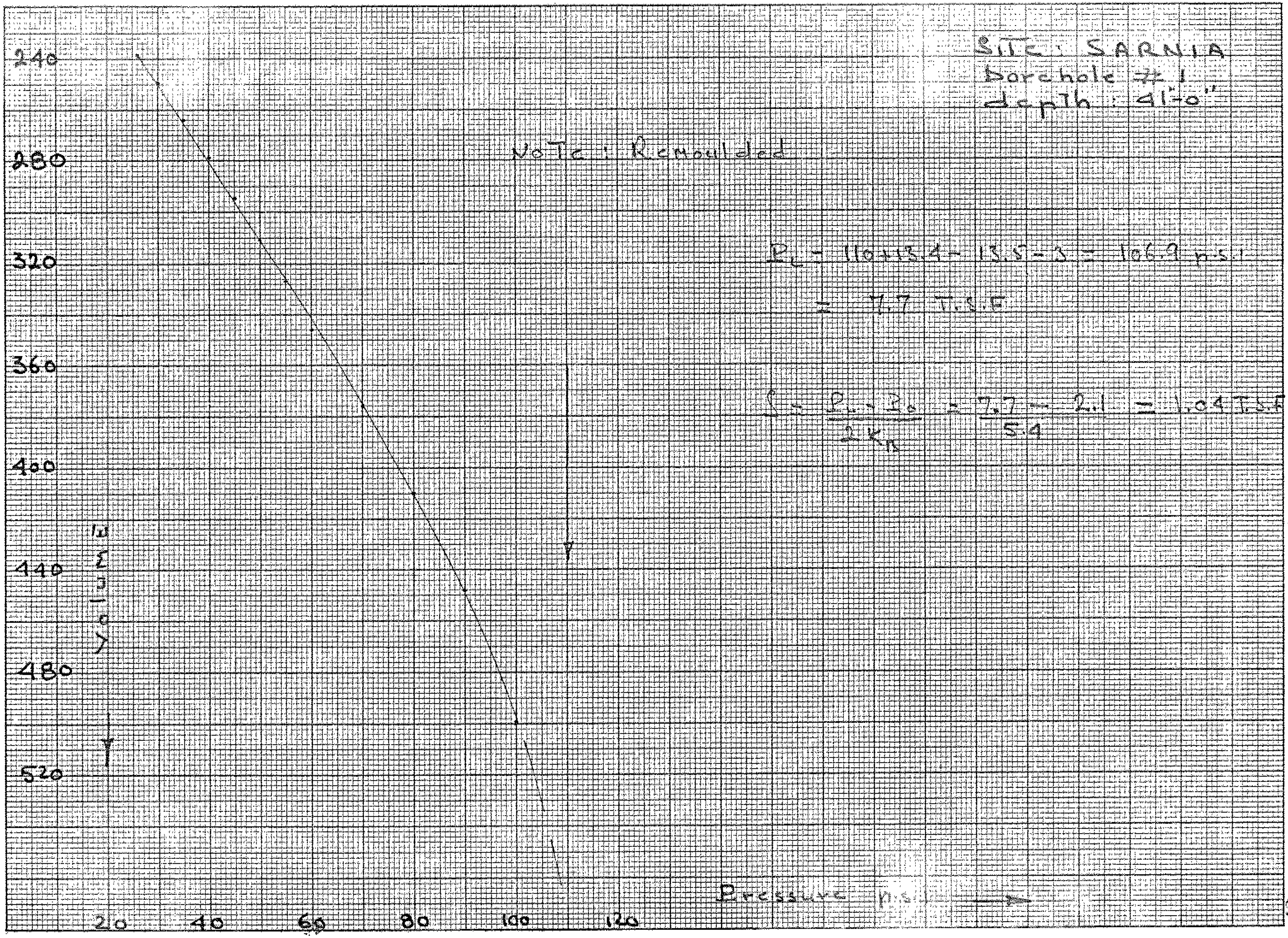
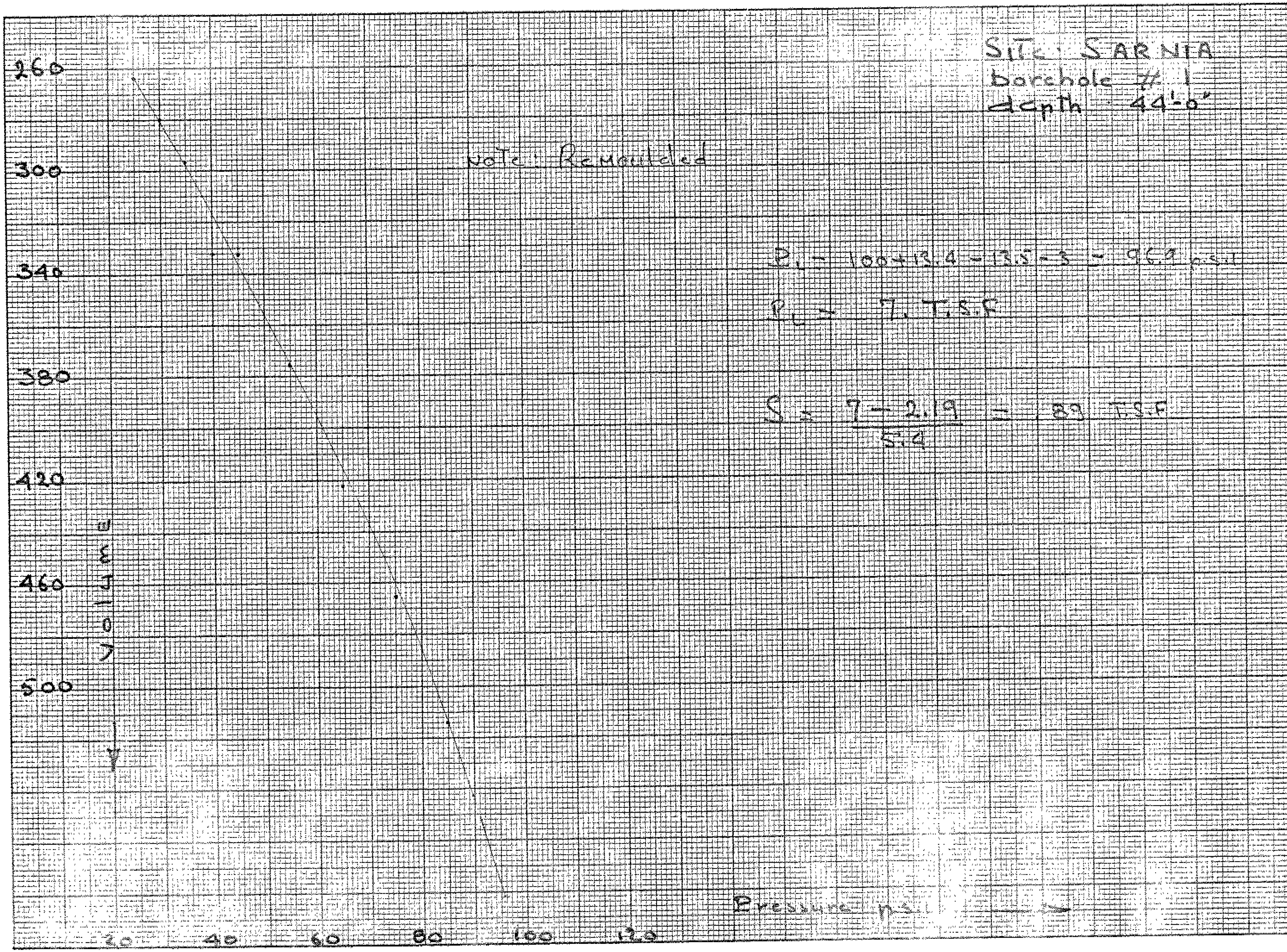
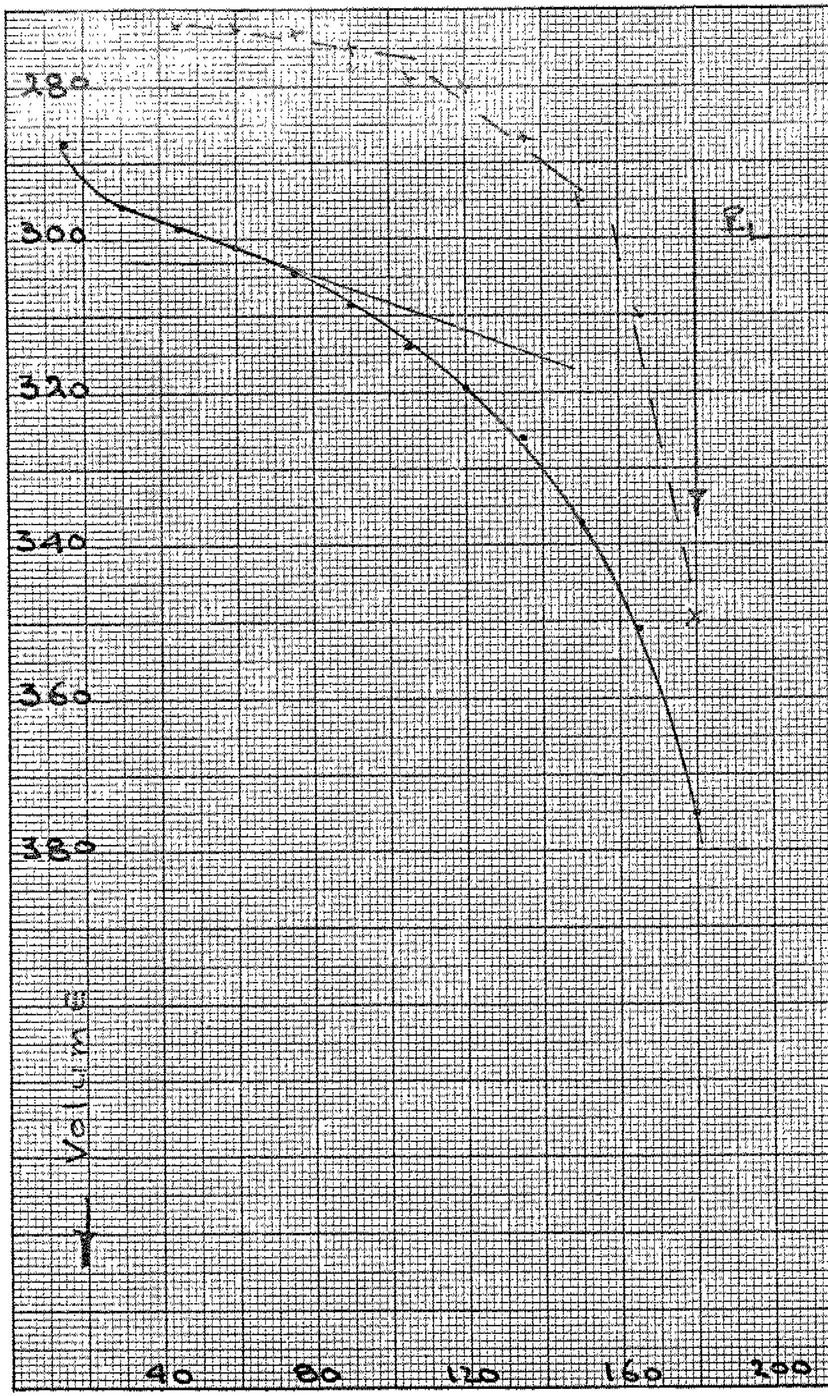

SITE: SARNIA
Borehole # 1
depth: 41'-0"
Note: Remoulded
P_L = 110+13.4 − 13.5 − 3 = 106.9 p.s.i
= 7.7 T.S.F
S = (P_L − P_0) / 2K_n = (7.7 − 2.1) / 5.4 = 1.04 T.S.F
240
280
320
360
400
440
480
520
Volume
20
40
60
80
100
120
Pressure p.s.i


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
Volume
20
40
60
80
100
120
Pressure psi




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
$P_o = \gamma z = 0.325$ T.S.F

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

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

$E = (1+\theta)\,2\,V\,\frac{\Delta P}{\Delta V}$

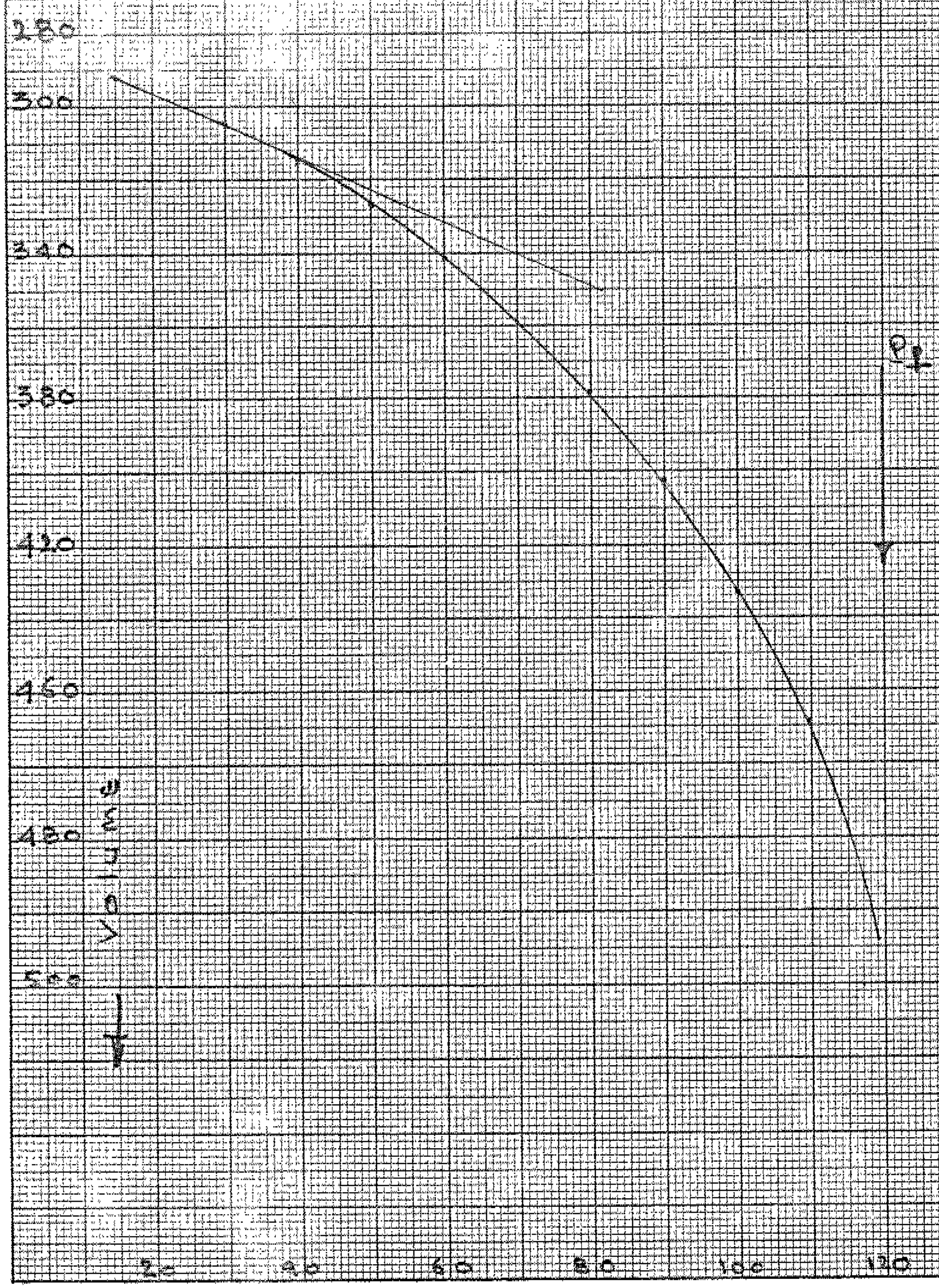
$= 2.66 \times 300\ \frac{2.88}{7} = 318$ T.S.F

$P_L / P_f = 1.64$

$E / P_L = 25.3$

$S = \frac{P_L - P_o}{2\,K_b} = \frac{12.55}{6.6} = 1.9$ T.S.F

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




Hydrostatic head = 5 psi
Probe inertia at $P_L$ = 12 psi
$P_0 = \gamma z = .5$ T.S.F

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

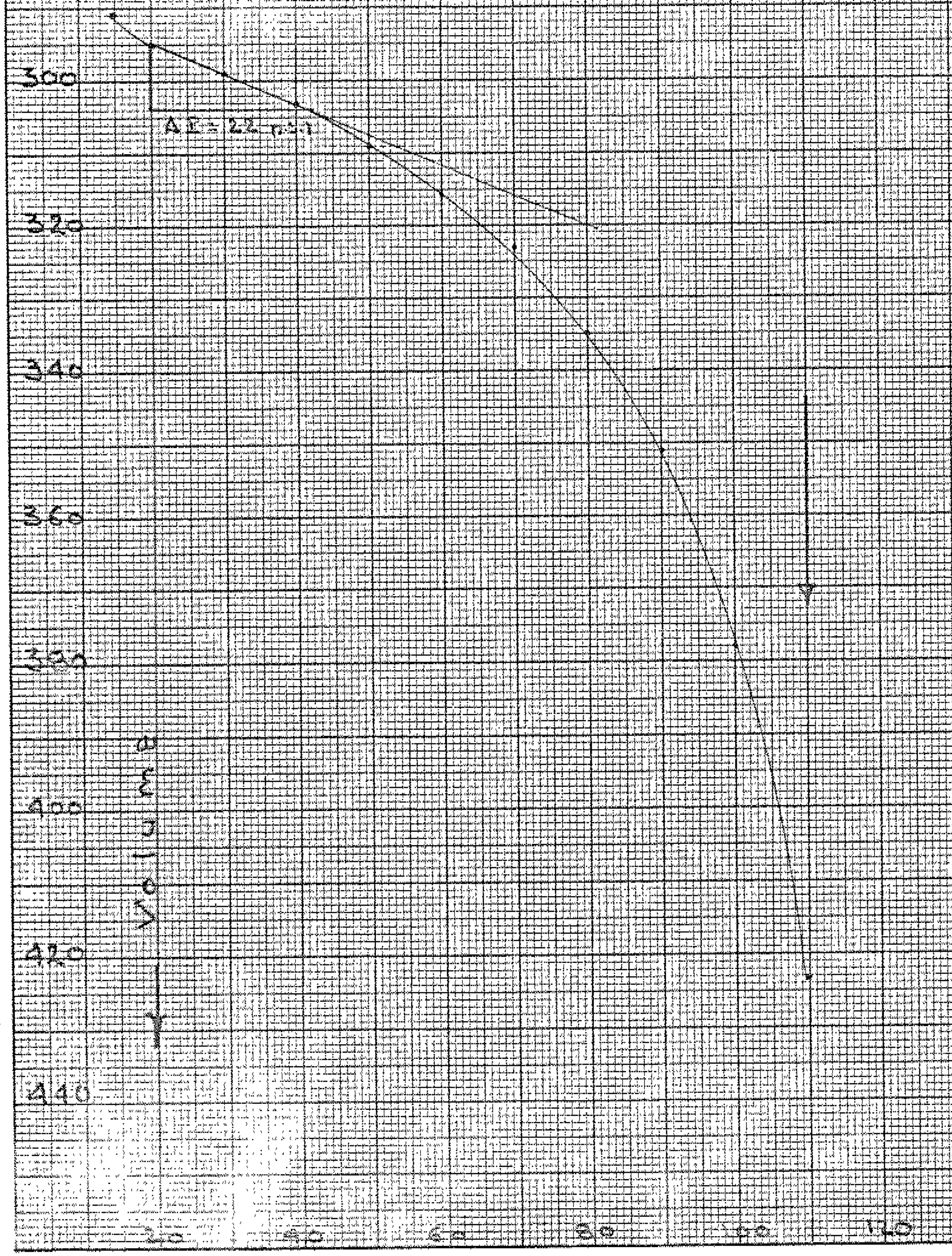
$E = (1+\nu)2V \dfrac{\Delta P}{\Delta V}$

$= 2.66 \times 304 \dfrac{1.8}{2.2} = 66$

$E/P_L = 8.15$

$S = \dfrac{P_L - P_0}{2K_b} = \dfrac{8.1 - .5}{5.4} = 1.41$ 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
$P_o = \gamma Z = .87$ T.S.F

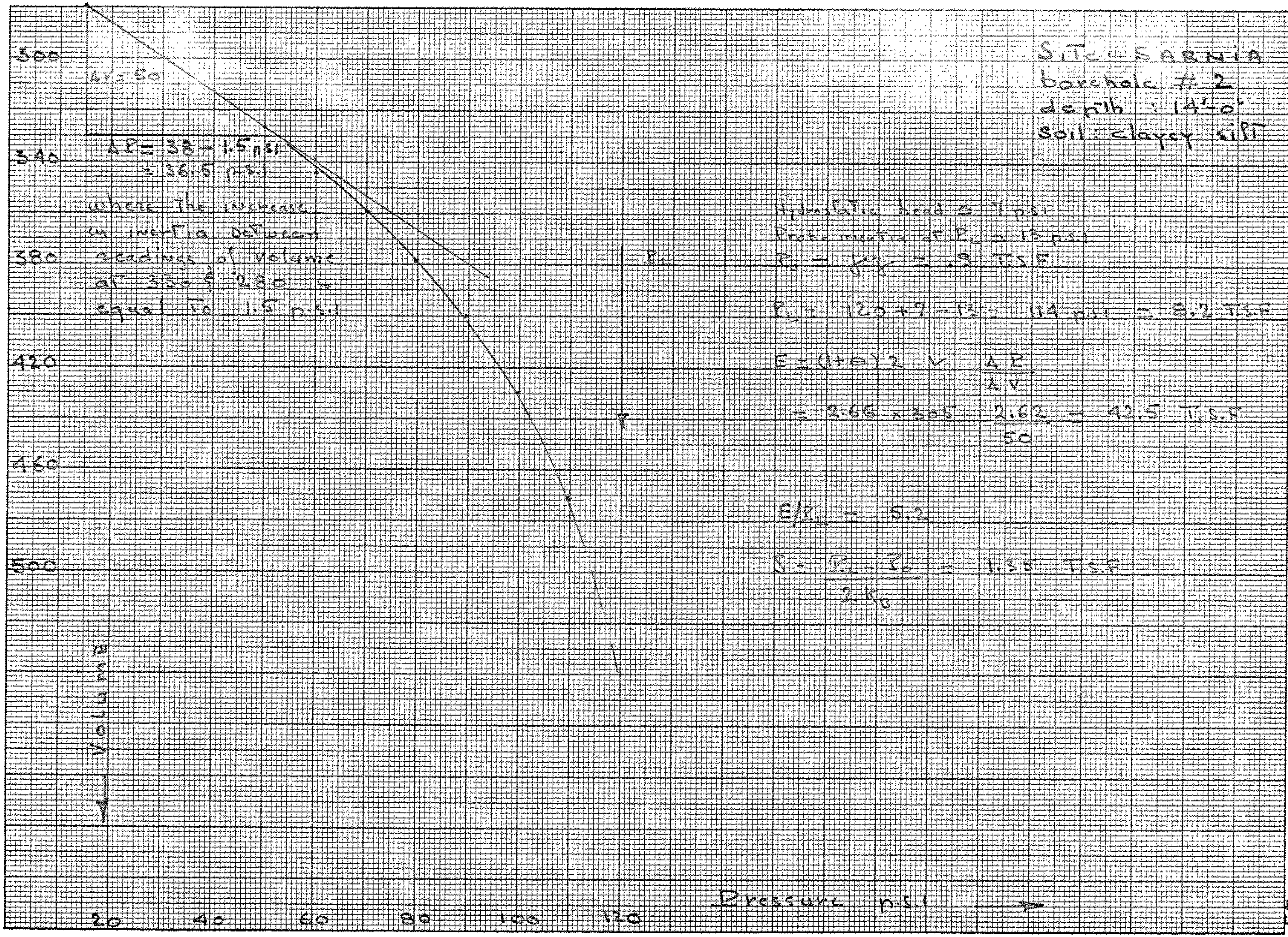
$P_L = 110 + 6 - 9.5 = 106.5$ p.s.i = 7.65 T.S.F

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

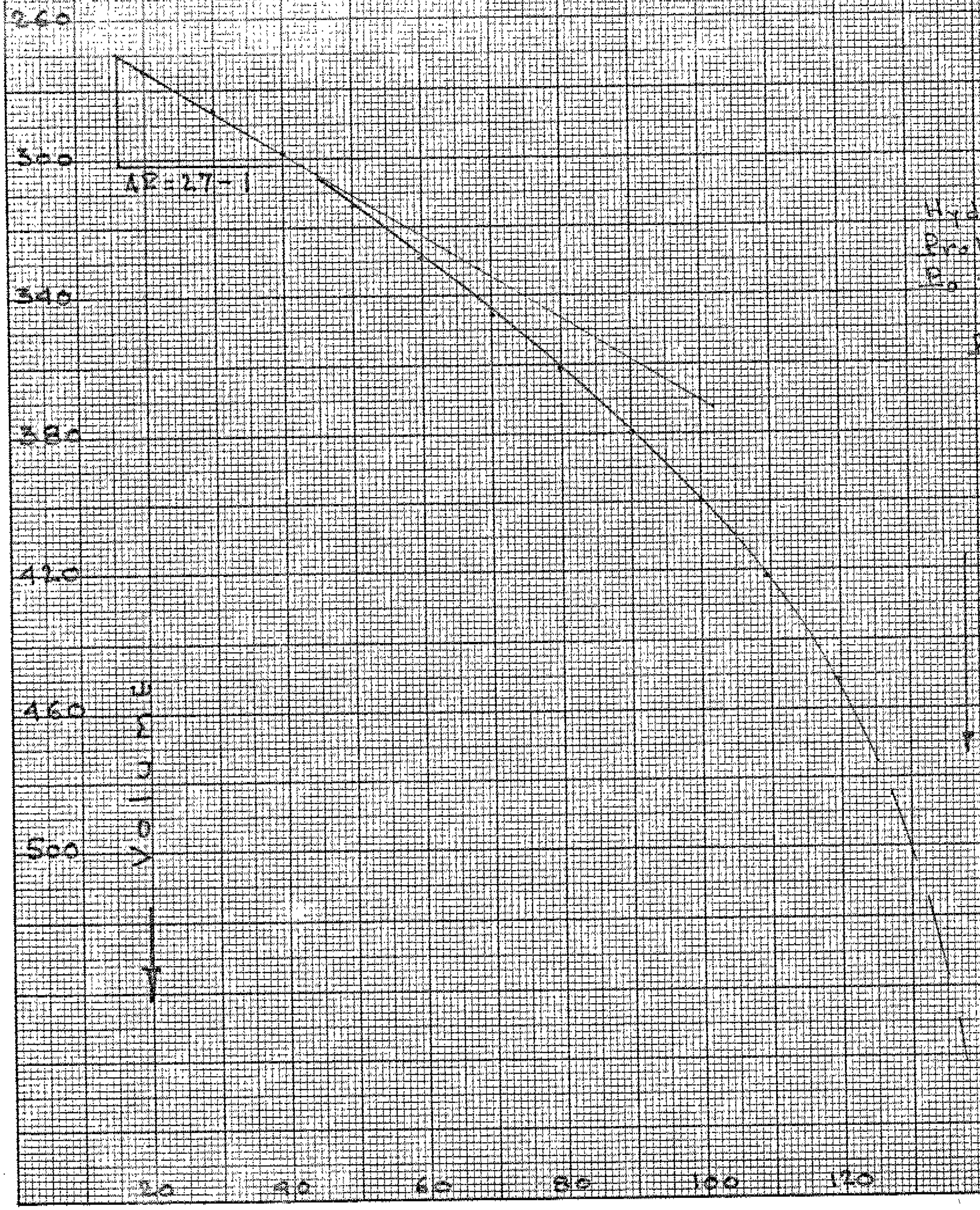
$E/P_L = 18.3$

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

Pressure p.s.i ⟶


Site: SARNIA
borehole # 2
depth : 14'-0"
soil: clayey silt
300
340
380
420
460
500
$\Delta V = 50$
$\Delta P = 38 - 1.5$ psi
$= 36.5$ p.s.i
where the increase
in inertia between
readings of volume
at 330 & 280 is
equal to 1.5 p.s.i
$P_L$
Hydrostatic head = 7 p.s.i
Probe inertia at $P_L$ = 13 p.s.i
$P_o = \gamma z = .9$ T.S.F
$P_L = 120 + 7 - 13 = 114$ p.s.i $= 8.2$ T.S.F
$E = (1+\nu)\, 2\, V \dfrac{\Delta P}{\Delta V}$
$= 2.66 \times 305 \dfrac{2.62}{50} = 42.5$ T.S.F
$E/P_L = 5.2$
$S = \dfrac{P_L - P_o}{2 K_b} = 1.35$ T.S.F
Volume
20
40
60
80
100
120
Pressure p.s.i

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
$P_o = K\gamma = 1.20$ T.S.F

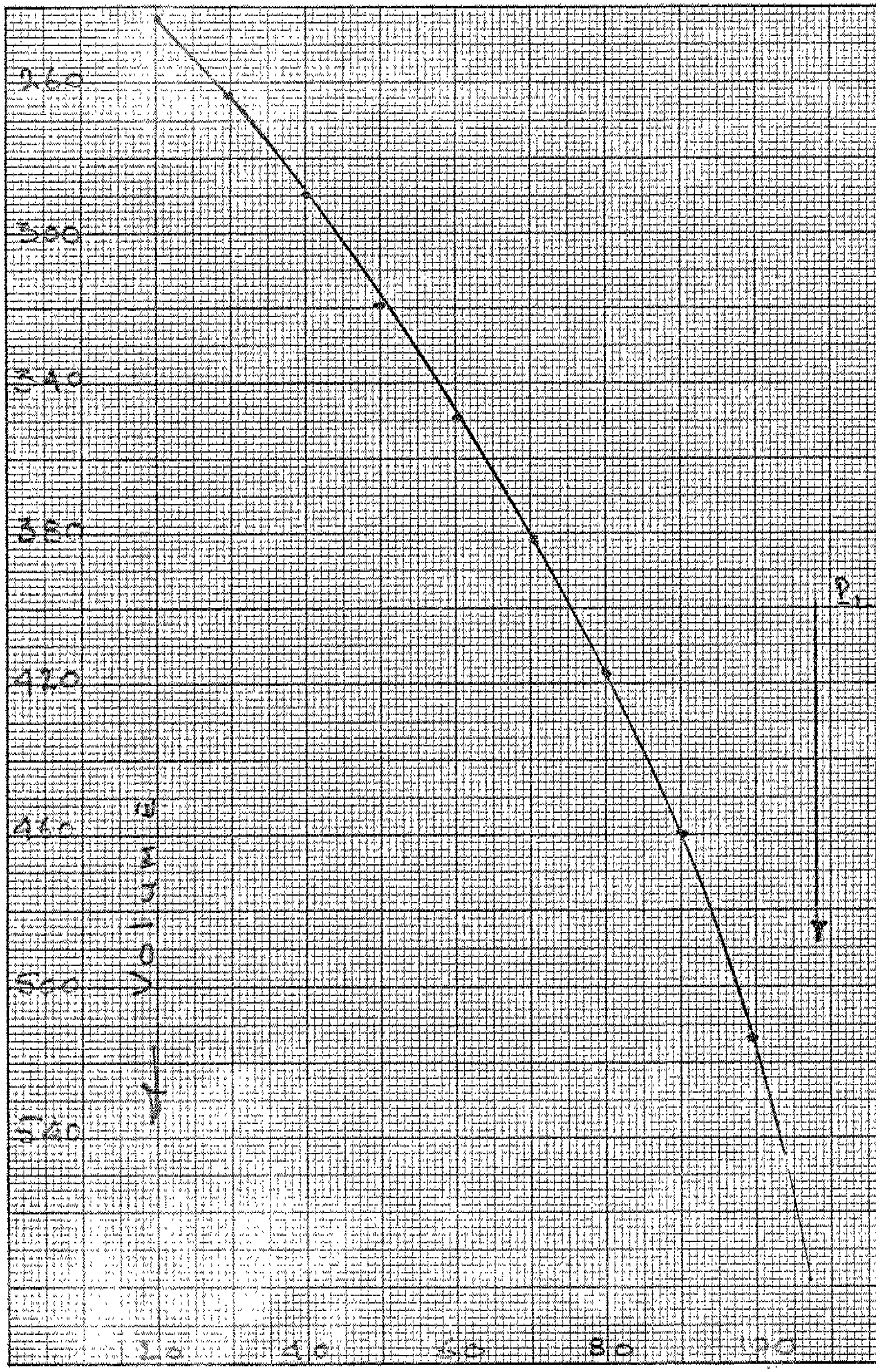
$P_L = 138 + 8.5 - 13 = 133.5$ p.s.i $= 9.6$ T.S.F

$E = 2.66 \times 285 \quad \frac{1.87}{32} = 47$ T.S.F

$E/P_L = 4.9$

$S = \frac{P_L - P_o}{2K_b} = \frac{9.6 - 1.20}{5.4} = 1.58$ T.S.F

Site : SARNIA
borehole # 2
depth : 20'-0"
soil : clayey silt.




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

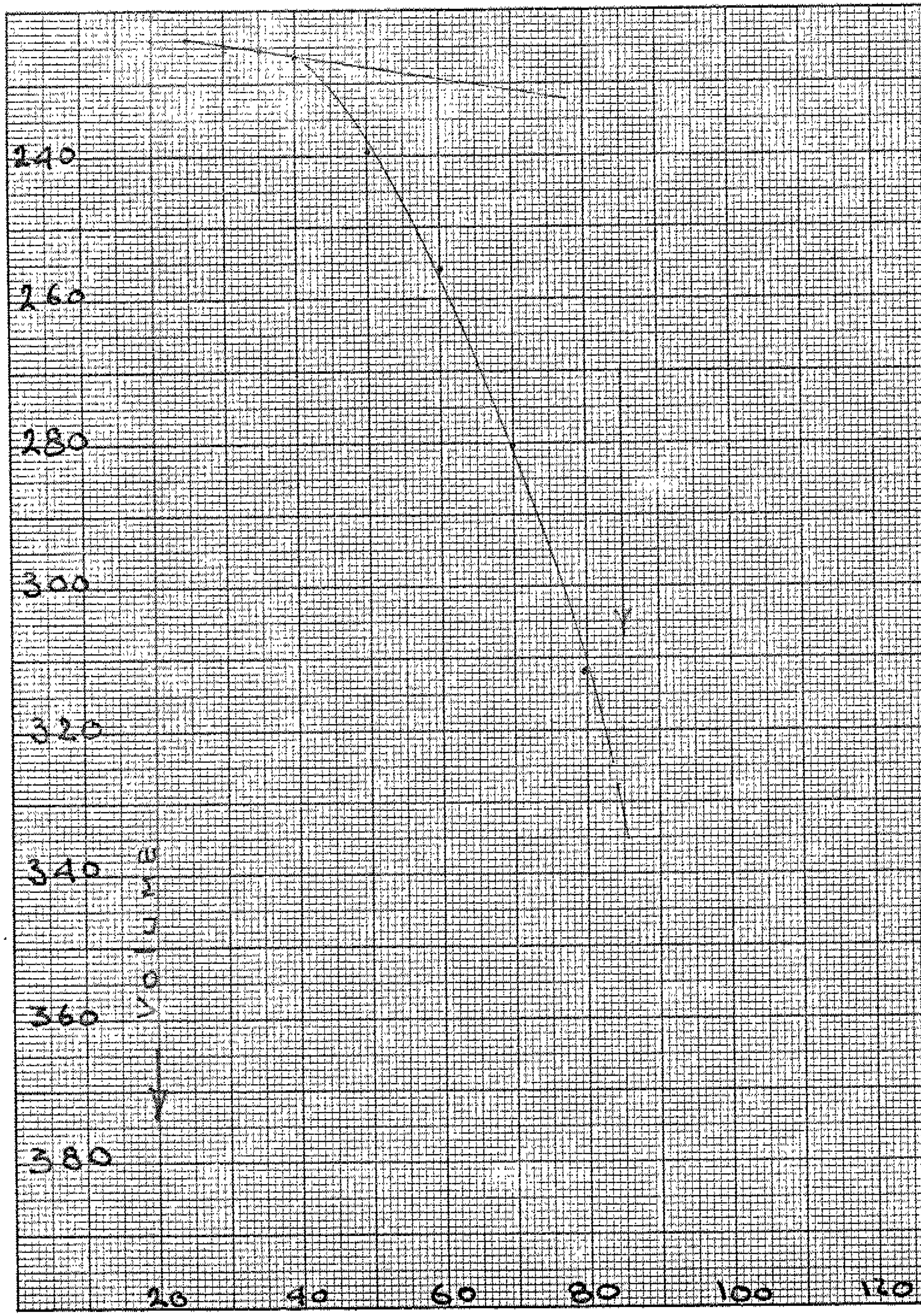
$P_L = 108 + 10 - 13 = 115$ p.s.i $= 8.3$ T.S.F

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

NOTE : soil remoulded – borehole "squeezed in"

Pressure p.s.i →

Site: SARNIA
borehole #2
depth 27'-0"




Hydrostatic head = 13 p.s.i
$P_o$ = 1.74
inertia at $P_L$ = 15 p.s.i

$P_L$ = 85 + 13 - 15 = 83 p.s.i = 5.97 T.S.F

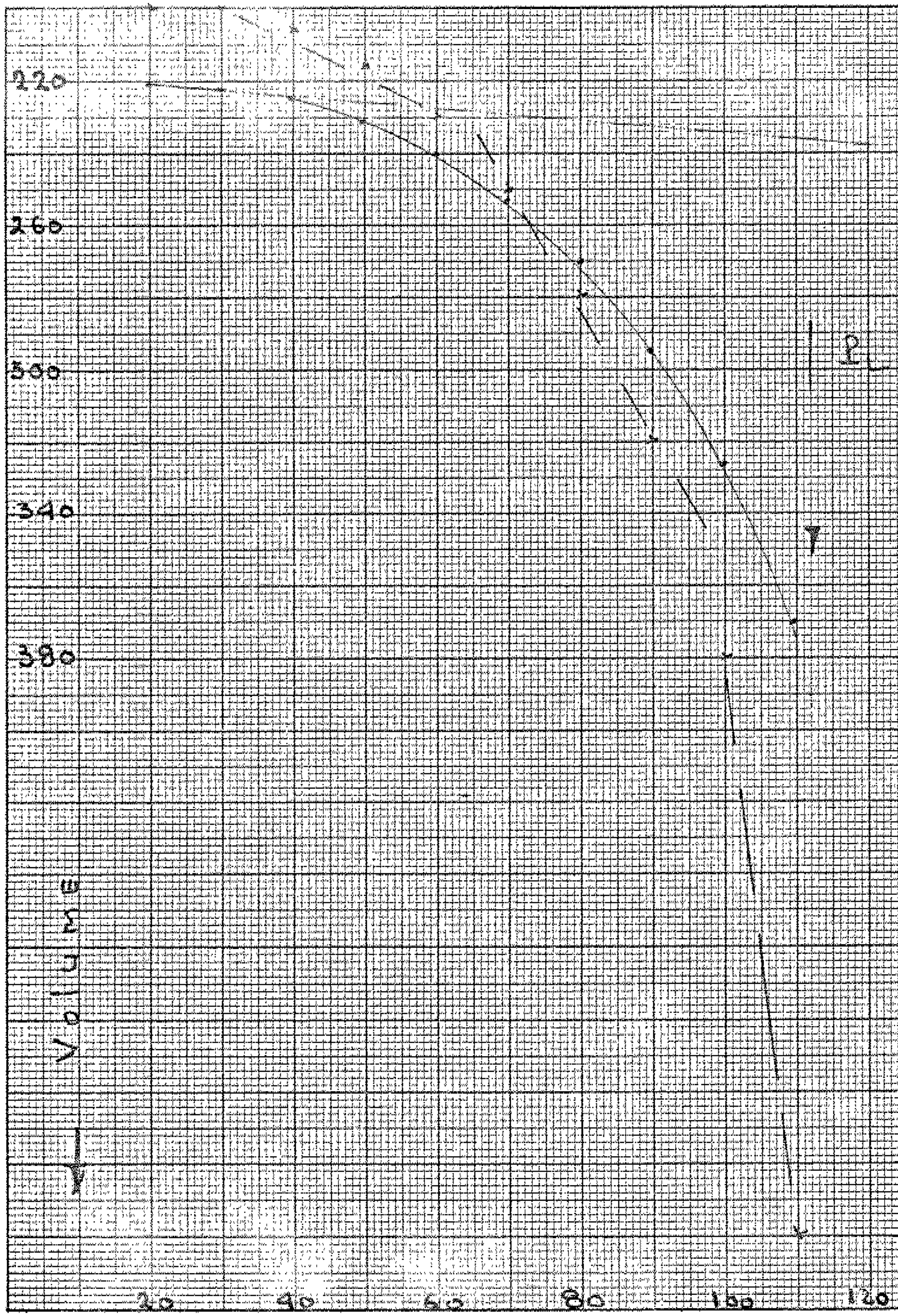
$E = 2.66 \times 225 \; \frac{1.08}{2.4} = 273$ T.S.F

$E/P_L$ = 45

$S = \frac{5.97 - 1.74}{6.6} = .64$ 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 # 2
depth 22'-0"
Soil : clayey silt




Hydrostatic head = 10.7

$P_o$ = 1.42 T.S.F

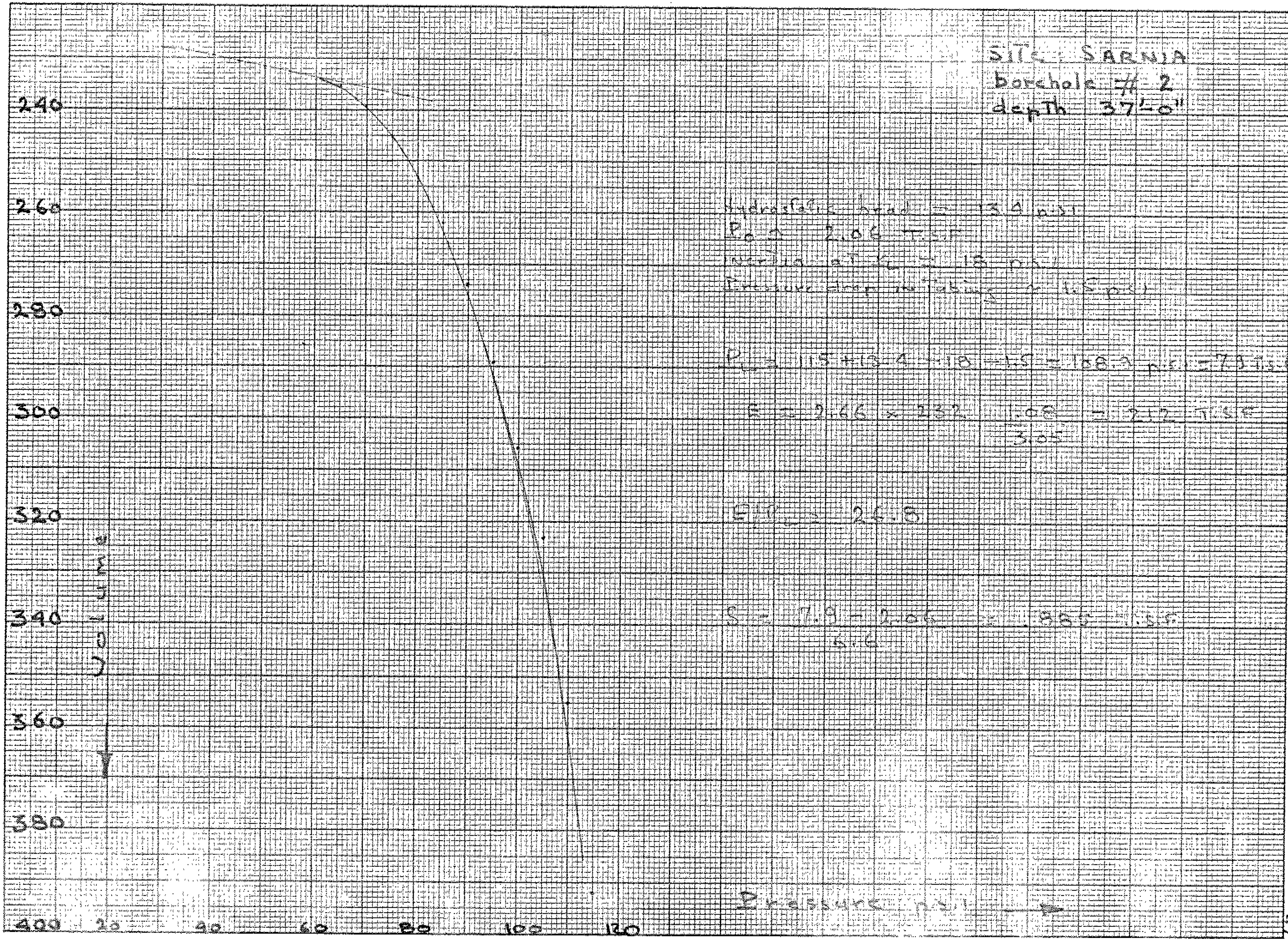
Inertia at $P_L$ = 18 p.s.i
Pressure drop in tubing = 1.5 p.s.i
$P_L$ = 112 + 10.7 − 18 − 1.5 = 103.2 p.s.i = 7.41 T.S.F

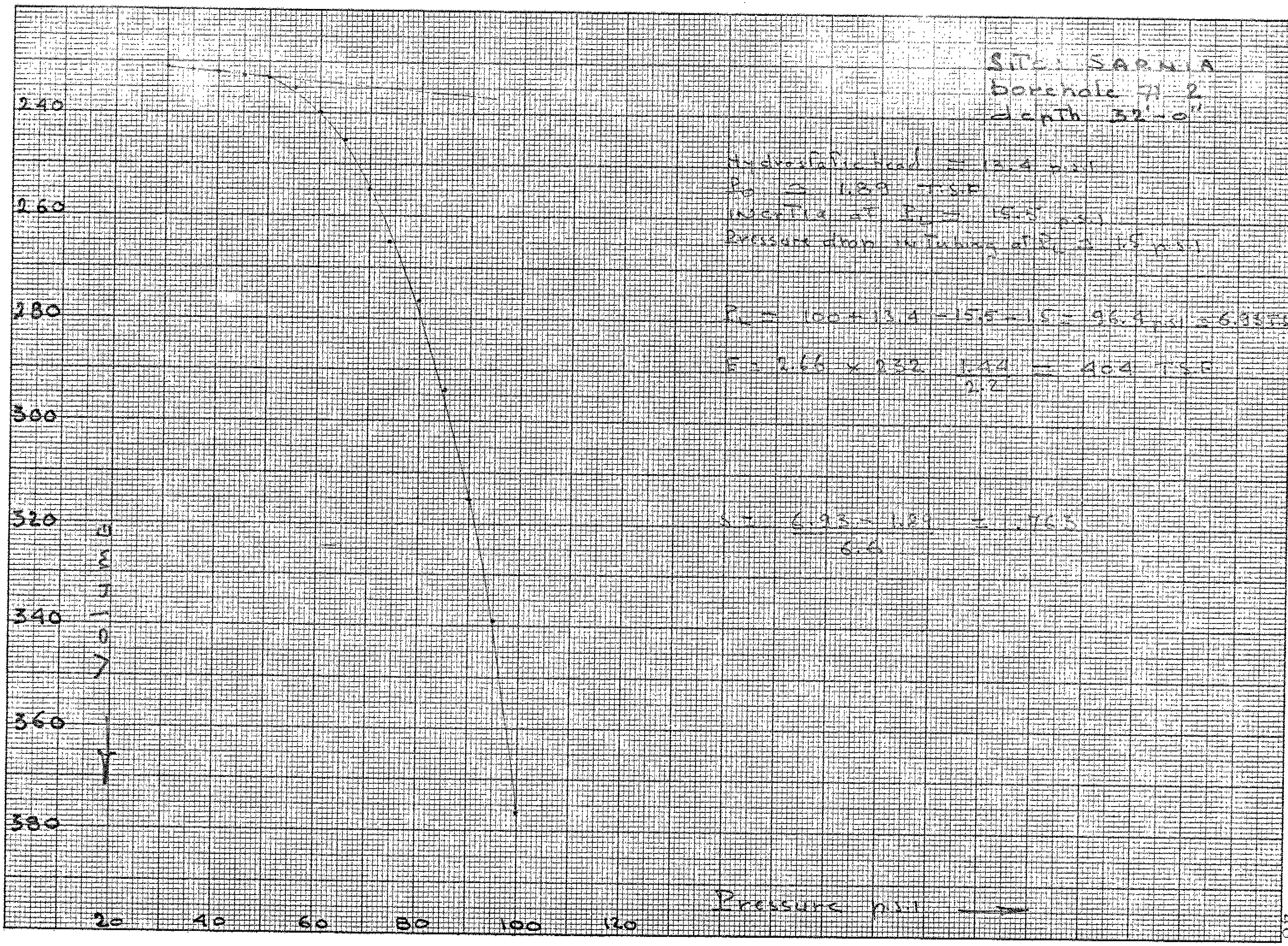
$E = 2.66 \times 221 \times \frac{1.44}{3.3} = 256$ T.S.F

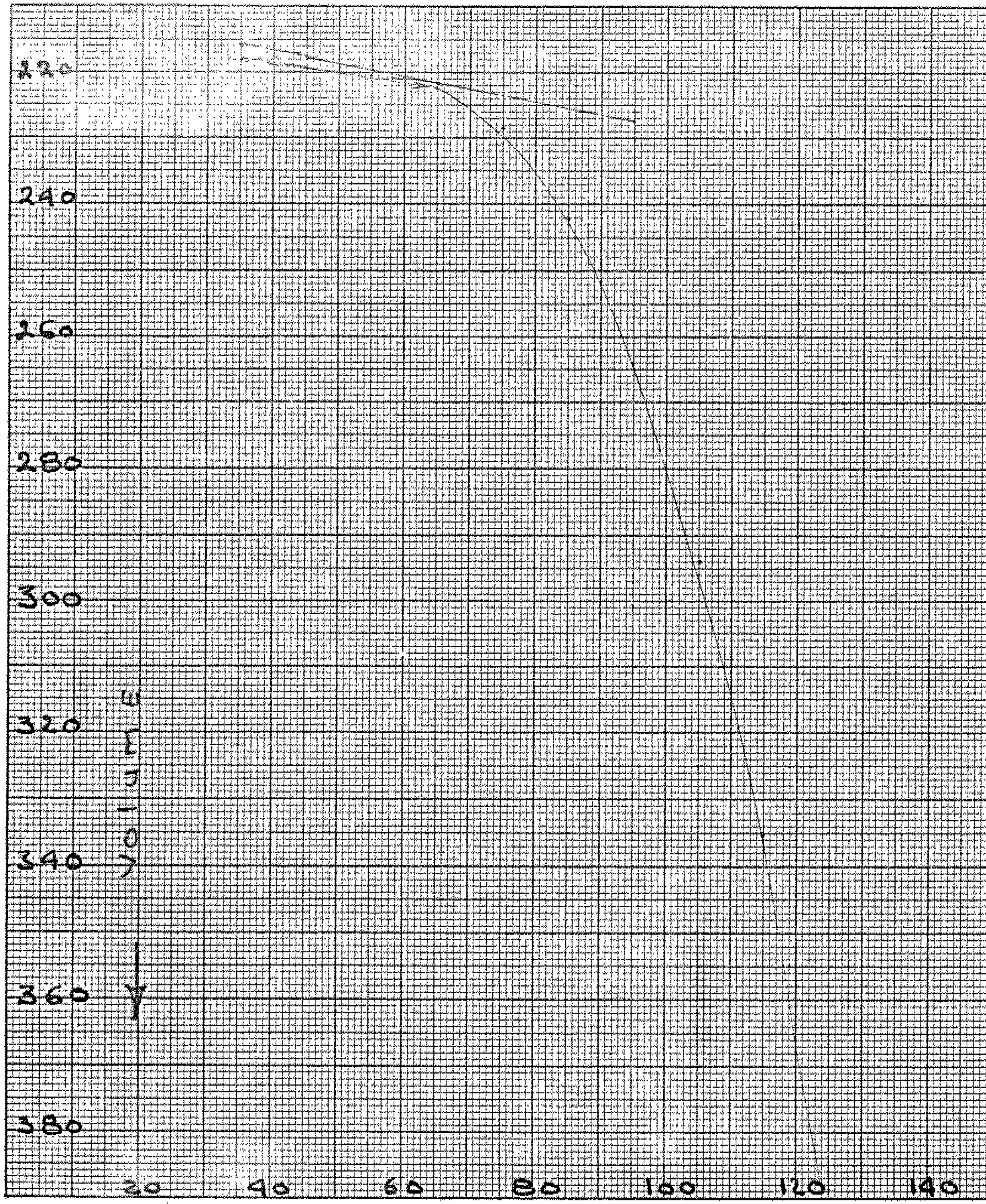
$S = \frac{7.41 - 1.42}{6.6} = .91$ T.S.F

Pressure p.s.i

6


SITE: SARNIA
borehole # 2
depth 37'-0"
Hydrostatic head = 13.4 psi
$P_o$ = 2.06 T.s.f
Inertia at 6 = 18 psi
Pressure drop in tubing = 1.5 psi
$P_L$ = 115 + 13.4 − 18 − 1.5 = 108.9 psi = 7.9 T.s.f
E = 2.66 × 232 × 1.08 / 3.05 = 212 T.s.f
$E/P_L$ = 26.8
S = (7.9 − 2.06) / 6.6 = .885 T.s.f
240
260
280
300
320
340
360
380
400
Volume
20
40
60
80
100
120
Pressure psi


SITE : SARNIA
borehole # 2
depth 32'-0"
Hydrostatic head = 13.4 p.s.i
Po = 1.89 T.S.F
Inertia at PL = 15.5 p.s.i
Pressure drop in Tubing at PL = 1.5 p.s.i
PL = 100 + 13.4 − 15.5 − 1.5 = 96.4 p.s.i = 6.93 T.S.F
E = 2.66 × 232 1.44 / 2.2 = 404 T.S.F
S.I = 6.93 − 1.89 / 6.6 = .765
240
260
280
300
320
340
360
380
Volume
20
40
60
80
100
120
Pressure p.s.i




Site: SARNIA
Borehole # 2
depth: 52

Hydrostatic head = 13.4 p.s.i.
$P_0$ = 2.38 T.S.F.
inertia at $P_L$ = 18 p.s.i.
Pressure drop = 7.5 × .332 = 2.5 p.s.i.

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

$E = 2.66 \times 220 \frac{1.44}{4.18} = 202$ T.S.F

$E_a \simeq E$ (Normal for overconsolidated clay)

$E/P_L$ = 23.8

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

Pressure p.s.i. →

67

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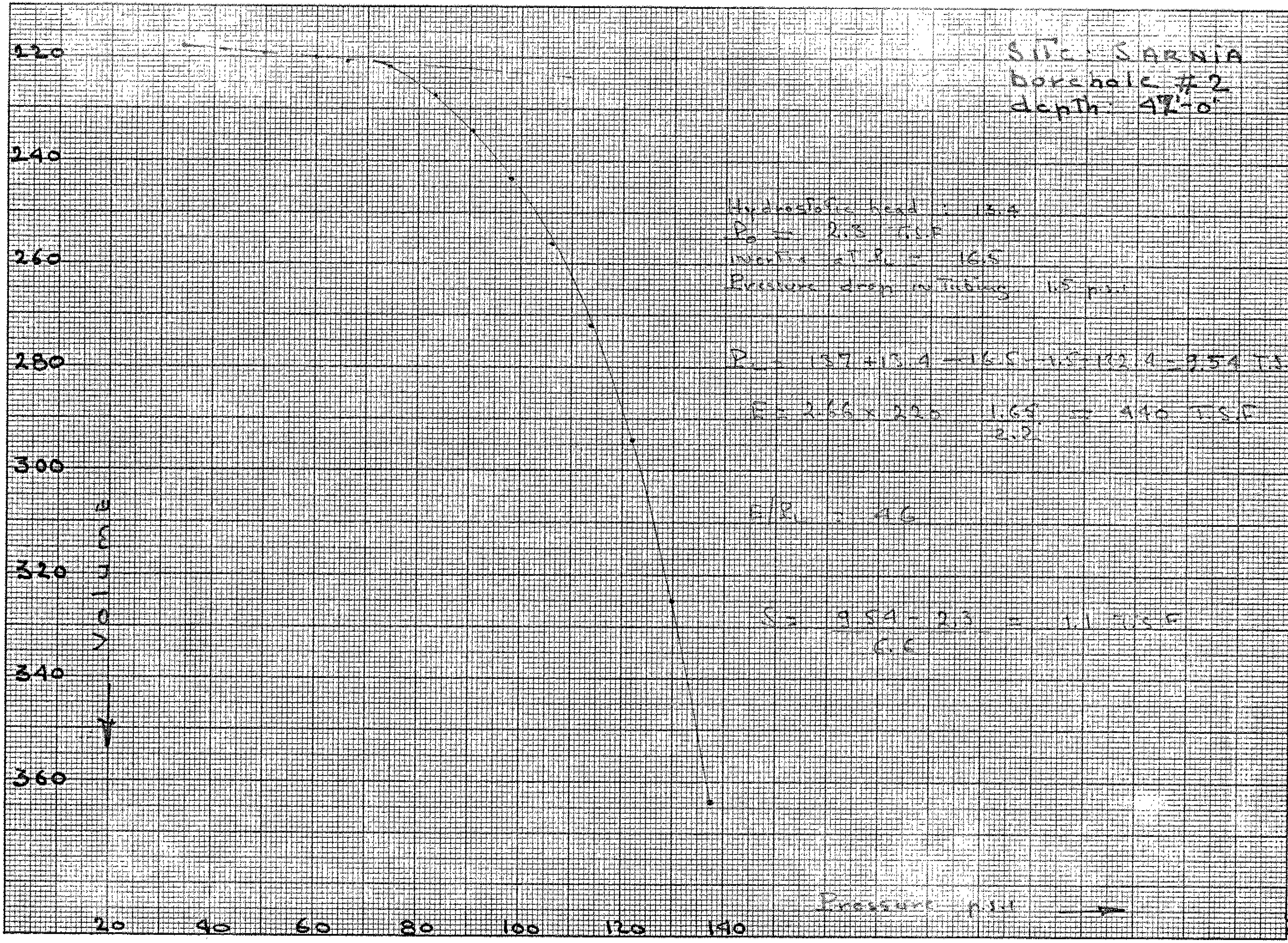
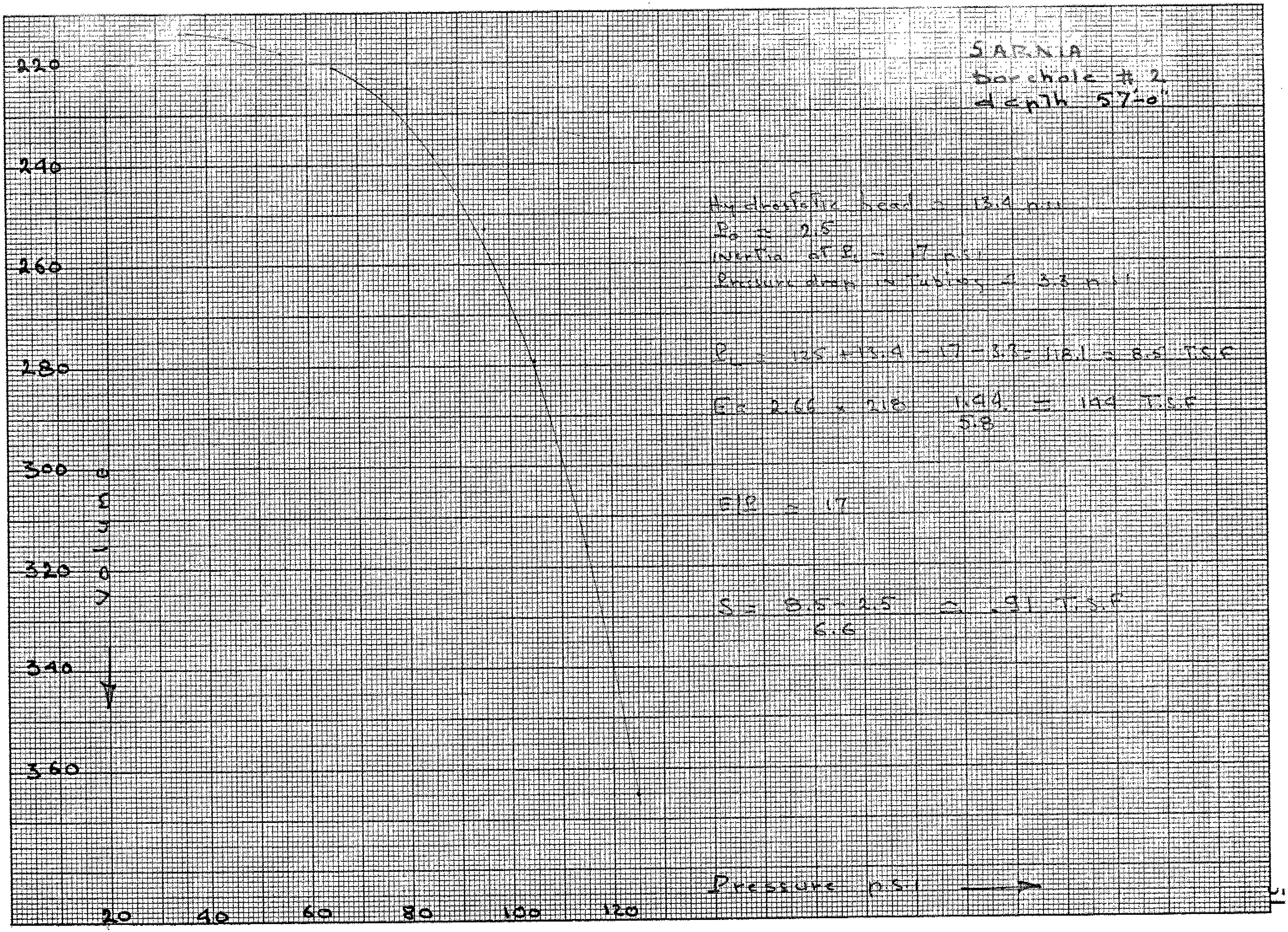


K+E 10 X 10 TO THE CENTIMETER 18 X 25 CM. 46 1513 MADE IN U.S.A. KEUFFEL & ESSER CO.




SARNIA
borehole # 2
depth 57'-0"

Hydrostatic head = 13.4 psi
$P_o$ = 2.5
inertia at $P_L$ = 17 psi
Pressure drop in tubing = 3.3 psi

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

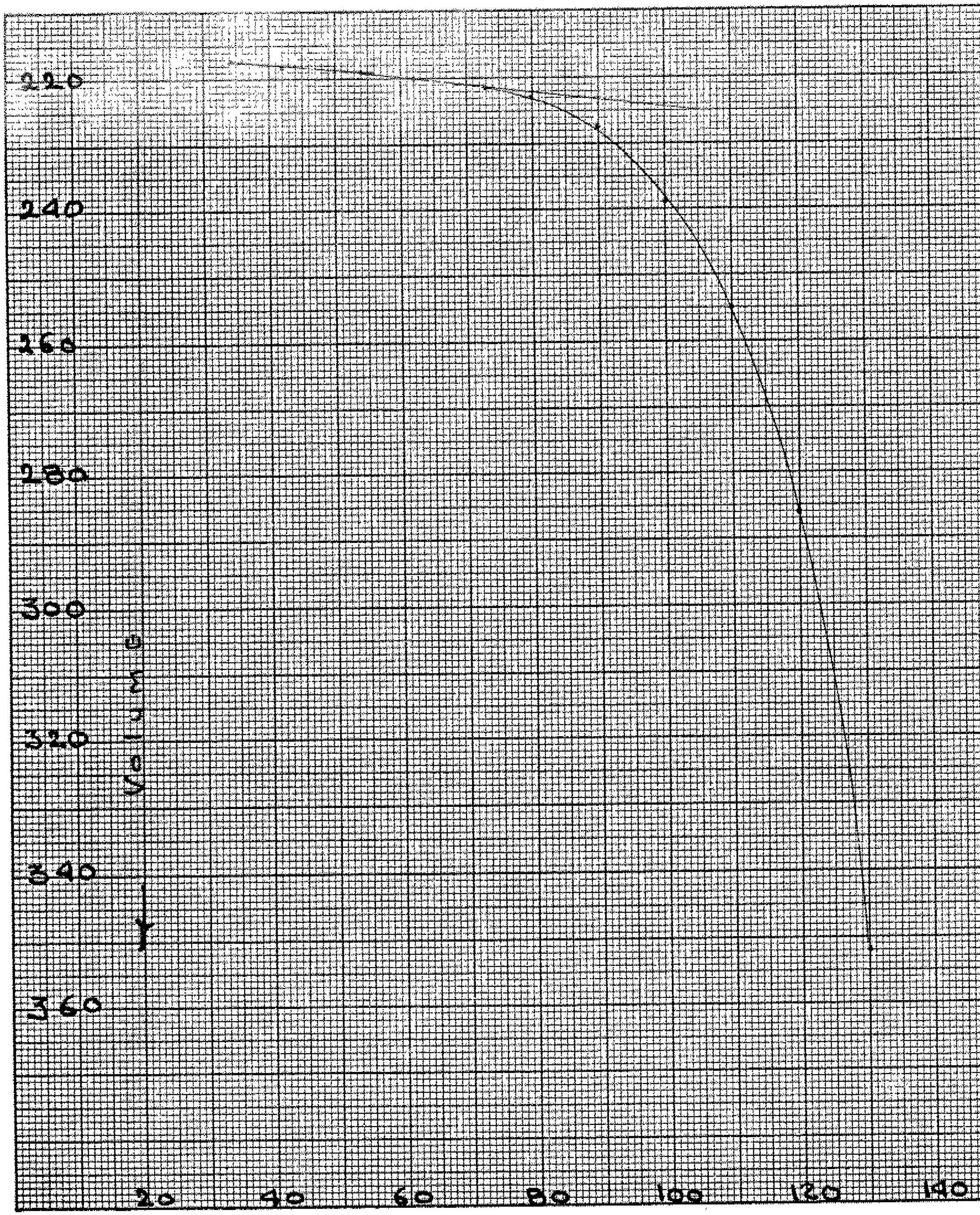
E = 2.66 × 218 $\frac{1.44}{5.8}$ = 144 T.S.F

E/P = 17

$S = \frac{8.5 - 2.5}{6.6}$ = .91 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 # 2
depth 62'-0"




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

$P_L$ = 130 + 13.4 − 16 − 4 = 123.4 psi = 8.9 T.S.F

$E = 2.66 \cdot 220 \; \frac{1.37}{2.4} = 334$ 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 | $E_L'$ T.S.F | $S_s$ T.S.F | N T.S.F | W inch | σ T.S.F | $S_s/σ$ |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | 21.75 | .05332 | | |
| 2 | 5 | | | | 21.75 | .05315 | | |
| | 4 | 100 | 9.5 | .625 | | | .396 | 1.57 |
| 7 | | | | | 14.52 | .05106 | | |
| | 3 | 60 | 7.5 | .545 | | | .228 | 2.39 |
| 12 | | | | | 10.04 | .0496 | | |
| | 2 | 60 | 7.6 | .545 | | | .222 | 2.45 |
| 17 | | | | | 5.87 | .04846 | | |
| 20 | 1 | 70 | 7.6 | .825 | 3. | .0482 | .255 | 3.23 |
| | | $E_q$=210 | 7. | | | | | |

TABLE N° 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

$$E_b = 4.1 \times 10^5 \text{ T.S.F.}$$

## 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

$Q_{ult} = 14$ 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

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

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

## 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 allowable = 14.75 + 4.7 = 19.45 Tons

### Ultimate Load at failure

P ultimate = 29.5 + 14 = 43.5 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 $W_o$ 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:

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

where $Q$ = contact pressure = 3 T.S.F.
$\lambda_2$ = shape coefficient = 1
$D$ = diameter = 13.5 ins.
$E_a = 3E$ = 210 T.S.F

Then

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

<u>Settlement at Tip of Section</u> (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:

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

$$\tau_1 = \frac{2 \times 70 \times .04874}{2.1 \times 12.75} = .255 \text{ T.S.F.}$$

The mobilisation of the skin friction $\tau_1$ = .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)

$$W_3 = W_2 + \frac{60 \times 5.87}{4.1 \times 10^5} = .04874 + .00086 \cong .04960 \text{ inch}$$

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

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

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, $\tau$ 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 $21.75 \times 1.57 = 34.1$ T.S.F. we will reach the maximum stress $\tau_4 = .625$ 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

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

# 12" "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 | $\sigma$ T.S.F | $S_s/\sigma$ |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | 24.08 | .05942 | | |
| 2 | 5 | | | | 24.08 | .05662 | | |
| | 4 | 100 | 9.5 | .74 | | | .413 | 1.79 |
| 7 | | | | | 15.38 | .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 | 1.03 | 3. | .0465 | .241 | 4.27 |
| | | $E_q$ = 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

$Q_{ult} = K\ P_L{}^{l} = 2 \times 7 = 14$ 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, $P_L{}^{l}$, .

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)

$$W_2 = W_1 + \frac{36 \times 3}{2 \times 10^5} = .0465 + .00054 = 04704 \text{ inch}$$

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

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

### Settlement at Tip of Section (3)

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

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

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

Settlement at Tip of Section (4)

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

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

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

Settlement at Tip of Section (5)

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

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

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

Settlement at pile head

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

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

Listing the values of N, W, $\sigma$ on Table No.16 we see that the ratio $S_s/\sigma$ 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 $S_s/\sigma$ 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" PILE
## AT 20'-0"

| depth | | E T.S.F | $E'_L$ T.S.F | $S_s$ T.S.F | N T.S.F | W inch | $\sigma$ T.S.F | $S_s/\sigma$ |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | 25.85 | .0620 | | |
| 2 | 5 | | | | 25.85 | .05889 | | |
| | | 100 | 9.5 | .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

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

## 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 $E_p = 2 \times 10^5$ T.S.F

### Bearing capacity

1) End bearing

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

in terms of load

$$Q_{ult} = 14 \times 1.04 = 14.5 \text{ Tons}$$

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

2) Skin friction

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

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

### Capacity of pile

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

at failure

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

### Settlement

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

$N_1 = 3$ T.S.F.

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

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

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

$$\tau_1 = \frac{2 \times 70 \times .04854}{2.1 \times 12} = .270 \text{ T.S.F.}$$

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

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

$$\tau_2 = 2 \times \frac{60 \times .05039}{2.1 \times 12} = .240 \text{ T.S.F}$$

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

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

$$\tau_3 = \frac{2 \times 60 \times .05389}{2.1 \times 12} = .256 \text{ T.S.F.}$$

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

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

$$\tau_4 = \frac{2 \times 100 \times .0589}{2.1 \times 12} = .467 \text{ T.S.F}$$

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

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

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

Examining the ratio $S_3/\gamma$ it is seen that overstressing will first occur at section (4) for a value of

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

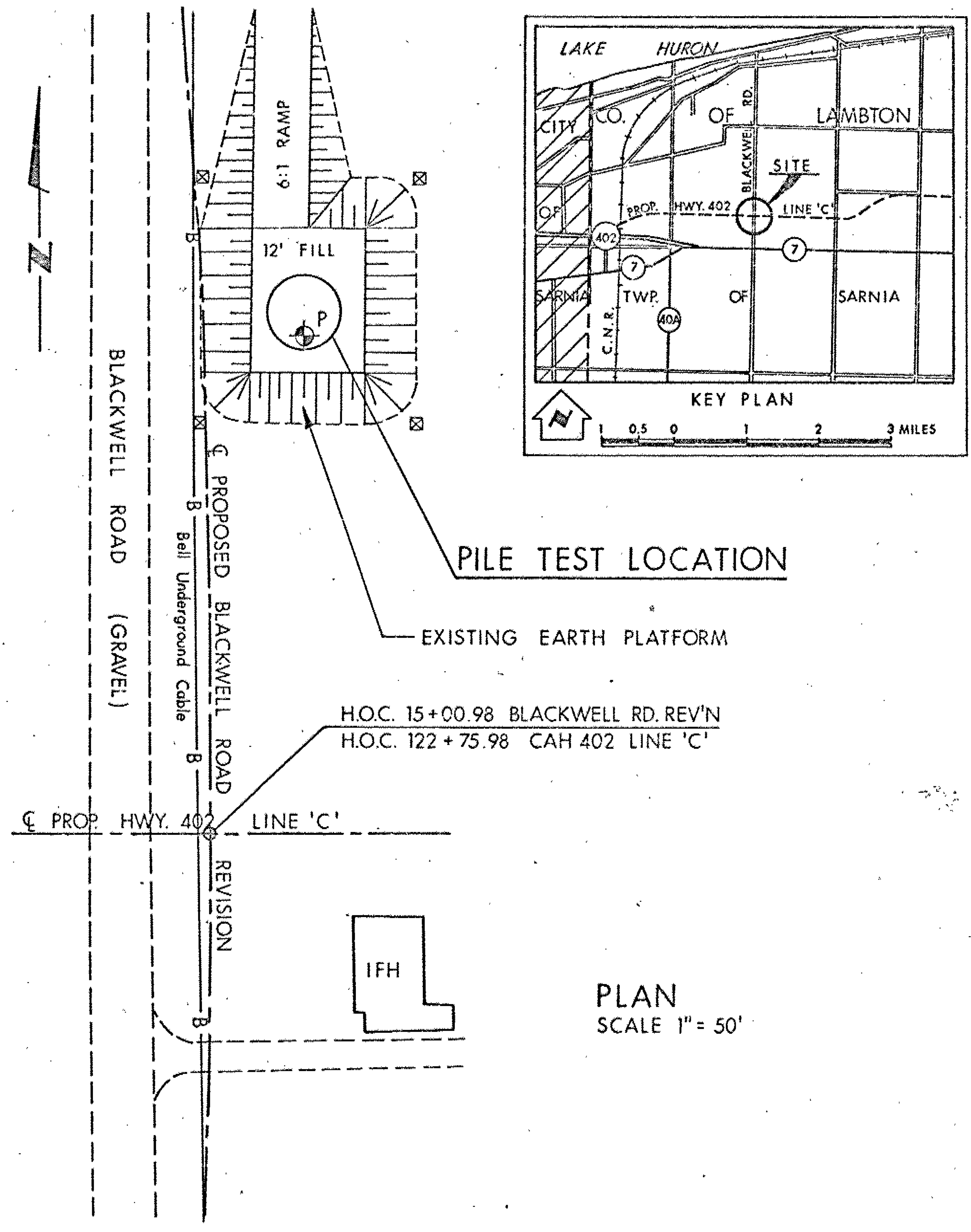
in terms of load this will occur at

$$P_t = 32.6 \times 1.04 = 34 \text{ Tons}$$

The corresponding settlement will be

$$W = W_o \times 1.26 = .078 \text{ inch}$$

APPENDIX "C"



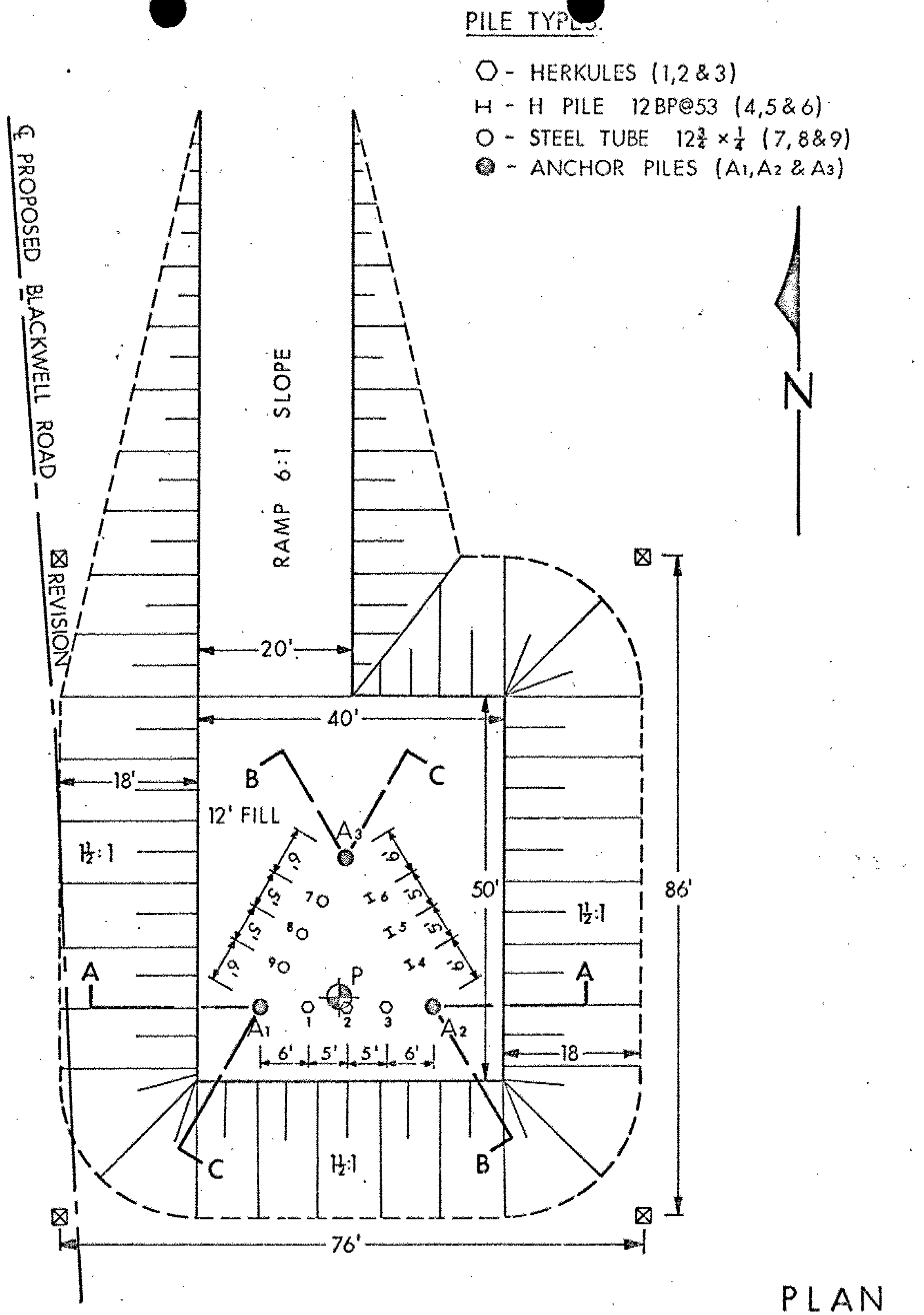

DEPARTMENT OF HIGHWAYS
**MATERIALS and TESTING DIVISION**
ONTARIO

BLACKWELL ROAD & HWY. 402

# PILE TEST LOCATION

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

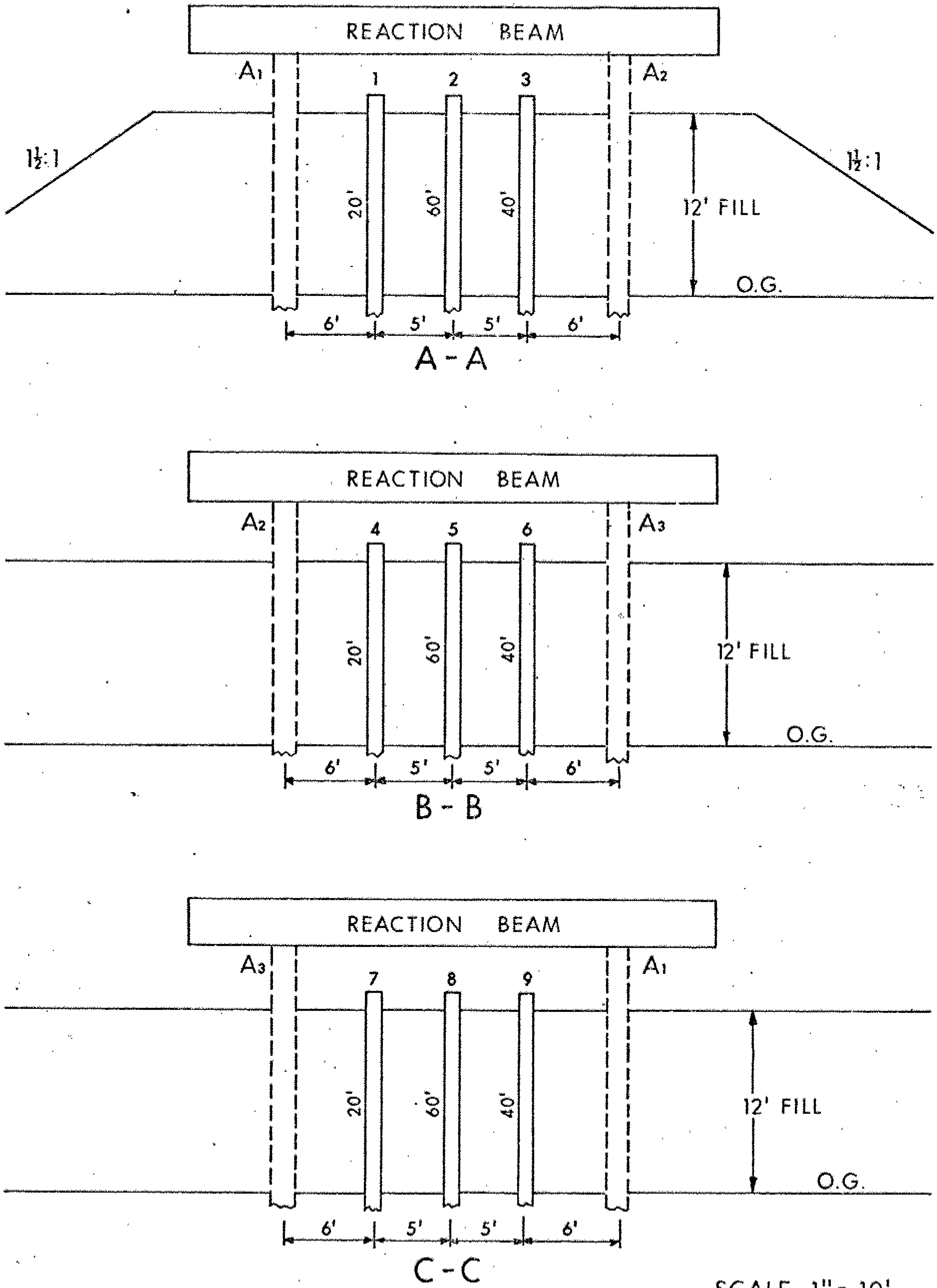
DATE 21 SEPT. 1970 | APPROVED | DRAWING NO. 70 - 11049 A




PLAN

SCALE 1"= 20'

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




SCALE 1" = 10'

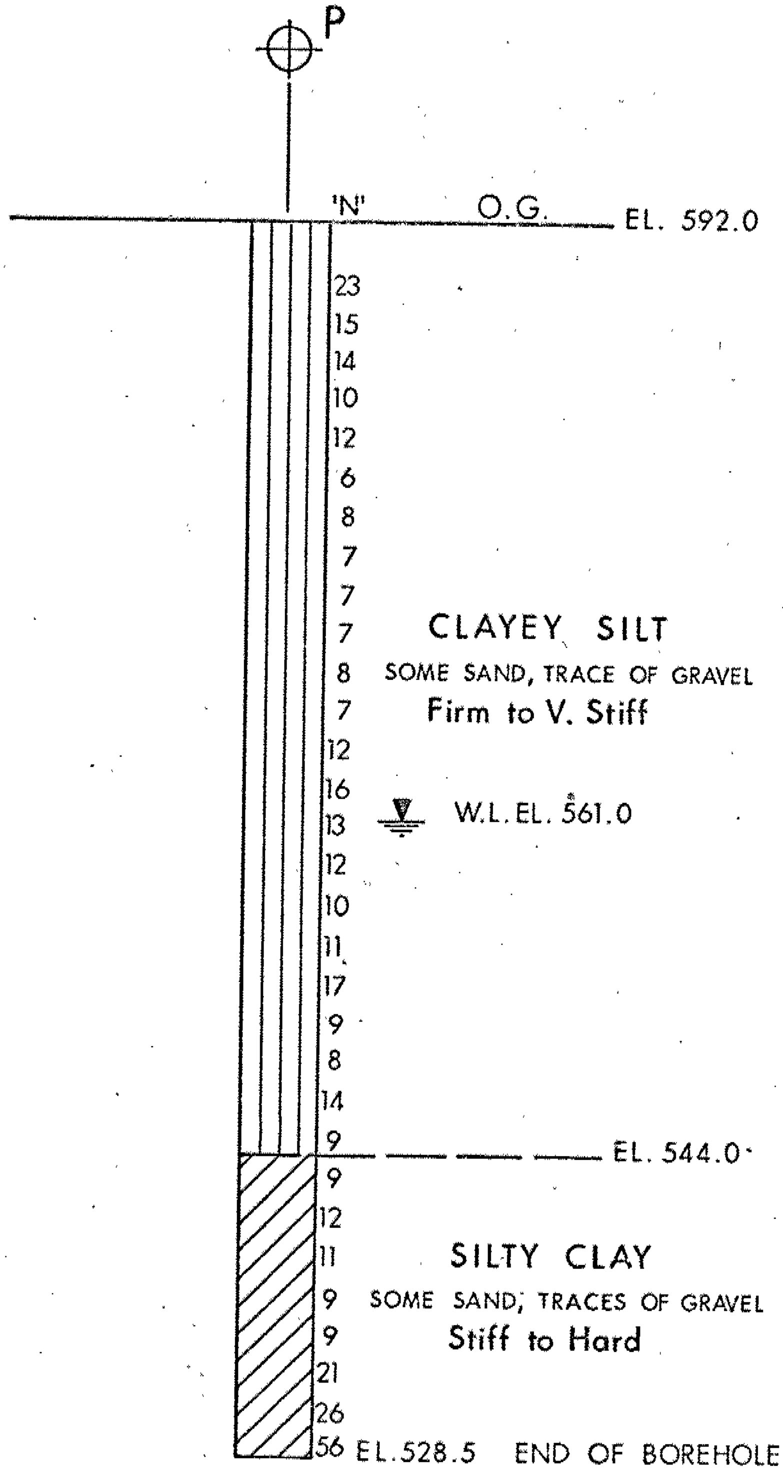
DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION
ONTARIO

BLACKWELL ROAD & HWY. 402

# PILE TEST ARRANGEMENT - ELEVATION

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

DATE 24 SEPT. 1970 | APPROVED | DRAWING NO. 70 - 11049 C

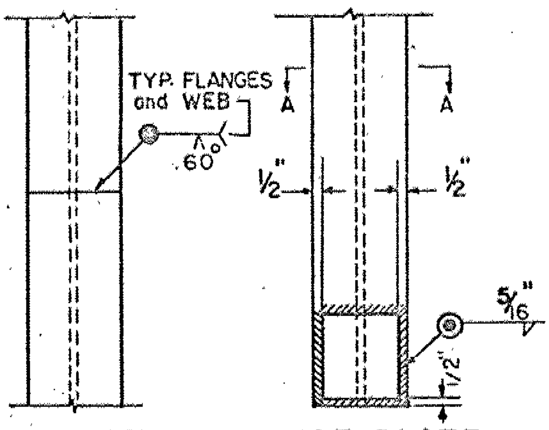



SCALE 1" = 10'

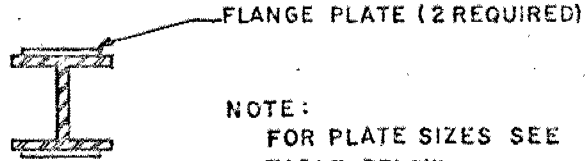
| | |
|---|---|
| DEPARTMENT OF HIGHWAYS<br>MATERIALS and TESTING DIVISION<br>ONTARIO | BLACKWELL ROAD & HWY. 402<br>**BORE HOLE DETAILS**<br>W.P. 43 - 66 - 05 DIST. 1 JOB 70 - 11049 |
| DATE 24 SEPT. 1970 | APPROVED / DRAWING NO. 70 - 11049 D |

# PILE SPLICES AND SHOES

## STEEL H PILES




SPLICE SHOE PLATE

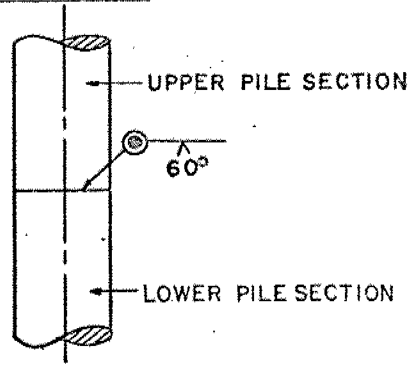



SECTION A–A

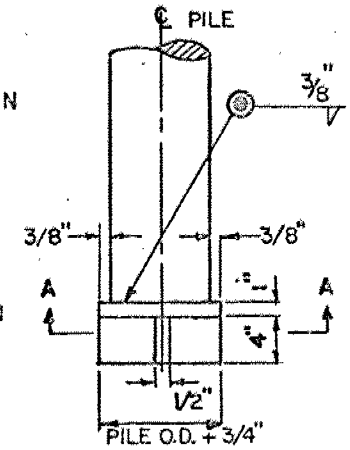
NOTE:
FOR PLATE SIZES SEE TABLE BELOW.

| PILE | 10 B.P. 42 | 12 B.P. 53 | 14 B.P. 73 |
|---|---|---|---|
| FLANGE PLATES | 9"X 1/2"X 12" | 11"X 1/2"X 12" | 13"X 1/2"X 12" |

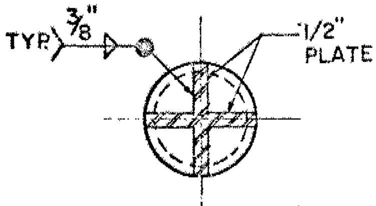
## TUBE PILE




SPLICE



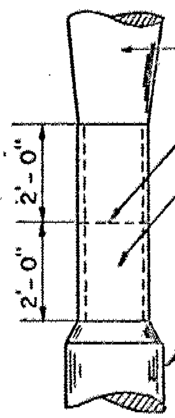

SHOE




SECTION A–A

THIS SHOE FOR ANY DIAMETER TUBE PILE.

## TIMBER PILES



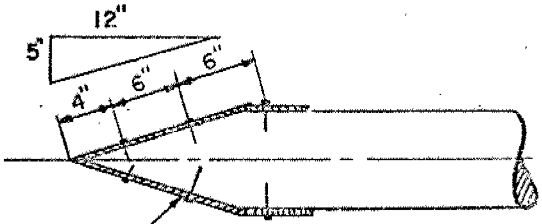

UPPER PILE SECTION: CUT TIP TO FIT SNUGLY INTO PIPE.

PILE END MUST BEAR ON SQUARE ENDS.

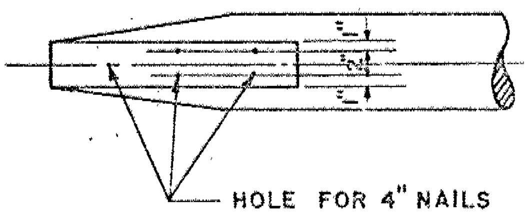
PIPE -- 8"⌀ FOR SIZE 12 PILES.
10"⌀ FOR SIZE 14 PILES.

PIPE TO BE BLACK IRON WITH 0.279" WALL. THICKNESS OR STEEL WITH 0.188" WALL THICKNESS.

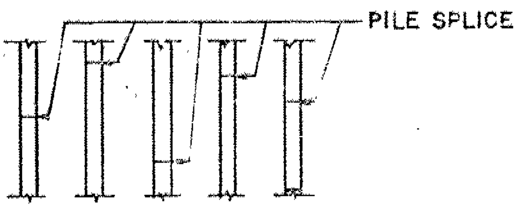
CUT BUTT TO FIT SNUGLY INTO PIPE.




1/4" X 4" X 34" STEEL PLATE BENT TO SUIT.




HOLE FOR 4" NAILS




DIAGRAMATIC SKETCH SHOWING SPLICE STAGGERING.