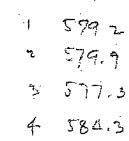
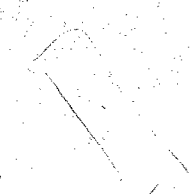
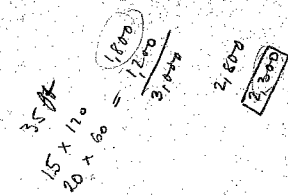


#61-F-247M

VIDAL ST. S.

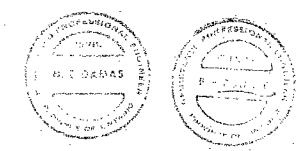
C.N.R. OVERPASS



NOTES:

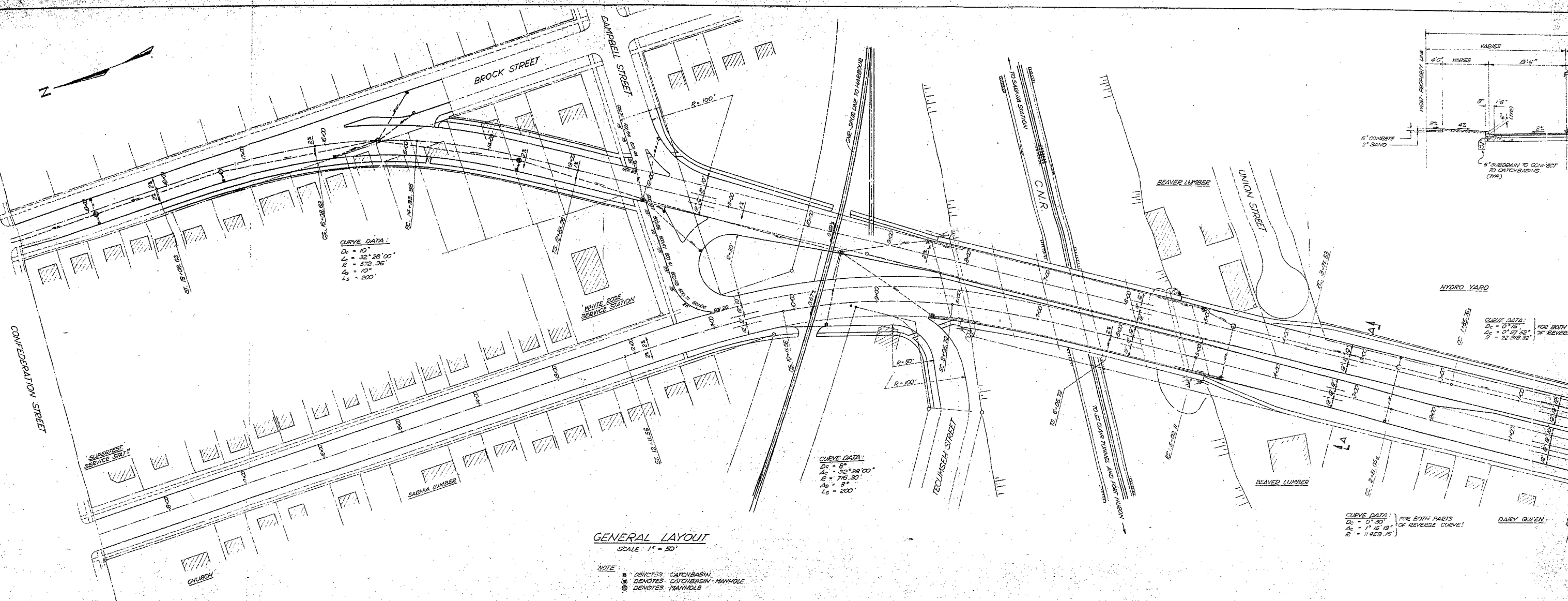
- WA 49671

• ANGLE OF SKEW -  $27^{\circ}26'40''$   
SIN - 0.4608883  
COS - 0.8874581  
TAN - 0.5193353  
SEC - 1.1268138



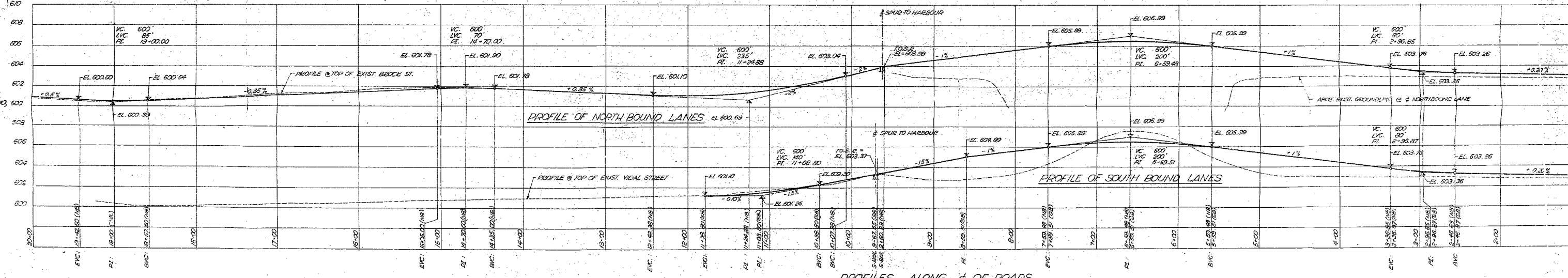
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TRACING	CHECKED			
DATE NOVEMBER 24 1961			691-P,	

					DESIGN <i>P</i>	CHECKED <i>NED</i>	APPROVED BY	DRAWING NO.	LATEST REVISION
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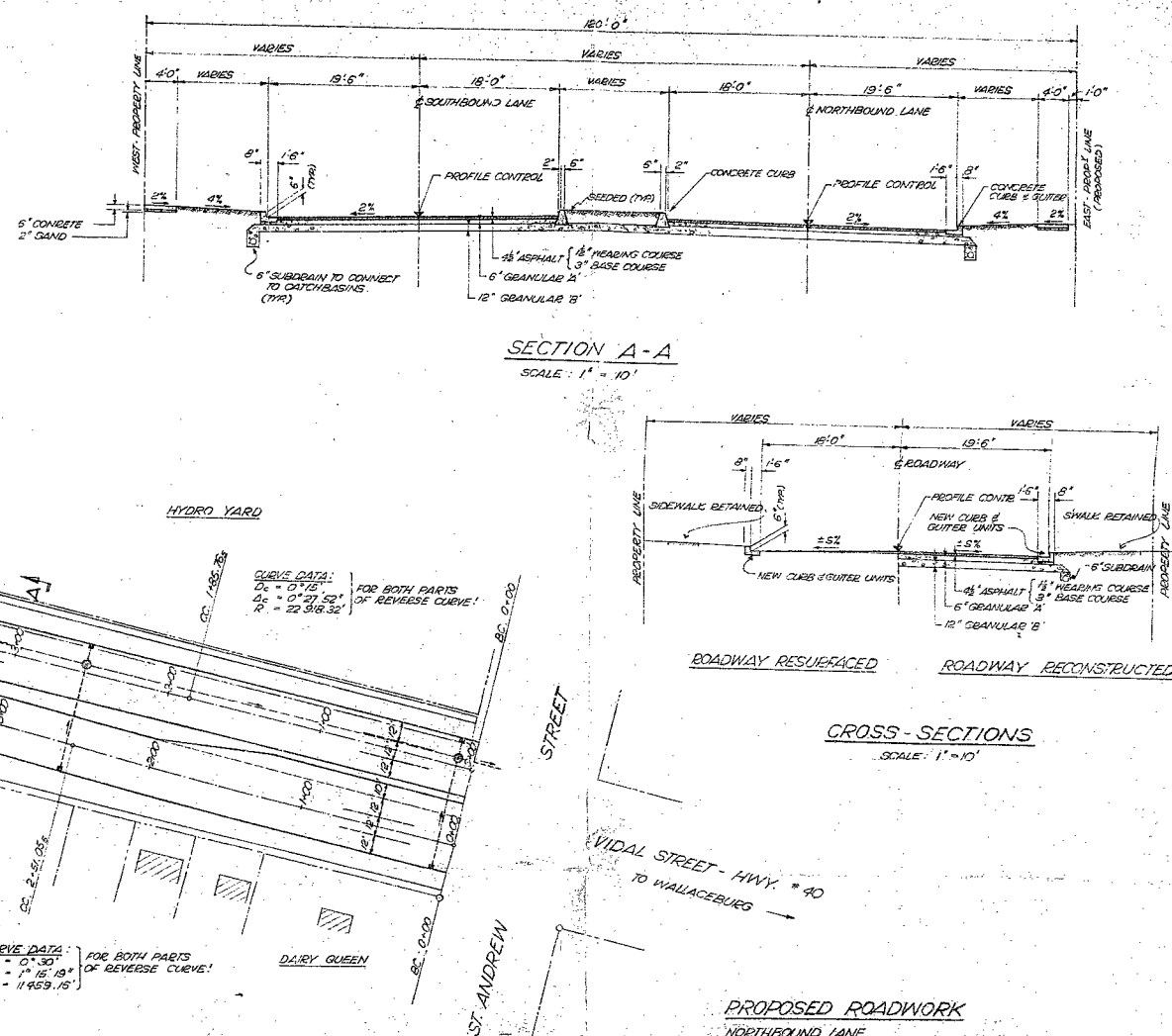


**GENERAL LAYOUT**  
 SCALE: 1" = 50'

**NOTE:**  
 \* DENOTES CATCHBASIN  
 \* DENOTES CATCHBASIN-MANHOLE  
 \* DENOTES MANHOLE

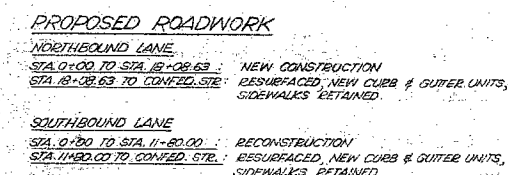


REV	BY	DATE	DESCRIPTION



NOTE:

- B DENOTES CATCHBASIN
- (B) DENOTES CATCHBASIN-MANHOLE
- (M) DENOTES MANHOLE



D. H. C.  
TORONTO  
RECEIVED  
FEB 12 1962  
BRIDGE  
OFFICE

## LAYOUT OF APPROACHES

					DESIGN <i>D</i>	CHECKED <i>NEB</i>	APPROVED BY	DRAWING NO.  691-P <sub>2</sub>	LATEST REVISION
					DRAWING <i>e2</i>	CHECKED <i>D</i>			
					TRACING	CHECKED			
REV	BY	DATE	DESCRIPTION	REFERENCE	DATE NOVEMBER 24, 1961				

Mr. A. M. Teye,  
Bridge Engineer.  
Materials & Research Division,  
(Foundation Section)

March 7, 1962.

REVIEW OF REPORT BY RACEY,  
MAC CALLUM & ASSOC., and  
PRELIMINARY PLAN BY DAMAS AND

SMITH, LTD.

(Bridge Office Ref. BA 1353)

Attention: Mr. K. L. Kleinsteinber,  
Municipal Bridge Liaison Engr.

Re: Soil Investigation, C.N.R.  
Overpass, Vidal Street South,  
Barnia, Ont. (D.H.C. Dist. #1)

We have reviewed the above-mentioned report submitted by the Consultant, Racey, MacCallum and Assoc., and also the preliminary bridge design prepared by the bridge consultant, Damas and Smith, Ltd. Below, we are submitting our comments for your consideration:-

The final recommendation of the soil consultant, and accepted by the bridge designer, is to use piled foundations for the new structure. This recommendation is based not on the inadequate safe bearing capacity, but on the fact that the settlements will be too great. A calculation presented in the report gives 5.29" as the maximum settlement.

An additional settlement calculation was carried out in this office taking a load of 2,300 p.s.f. (the Consultant used 2,500 p.s.f. in his calculation), and using the same two compression curves. The value obtained was 4.37" which was then corrected for overconsolidation and a final value of 2.6" was arrived at.

The above final value represents, in our opinion, the maximum possible consolidation settlement based on the two compression curves given in the report. We are not in the position to evaluate to what extent these two curves are representative for the particular subsoil, and are therefore unable to dispute the above given settlement figures. However, we are of the opinion

Attn: Mr. K. L. Kleinstreiber,  
Municipal Bridge Liaison Engr.

March 7, 1962.

that they represent a maximum and that very likely, much smaller settlements will take place. This opinion of ours is based on the following information and its interpretation:-

It is a known fact that the railway cut was excavated some 60 - 70 years ago. Prior to that, the ground surface was relatively flat and had a general elevation of about 605. The cut bottom elevation under the present bridge is 565 which means that approximately 40 ft. of overburden was removed. If we assume that 20 ft. of that material was submerged, we get an approximate load that was acting on the soil at elevation 565 as:

$$(20 \times 120) + (20 \times 60) = 2,400 + 1,200 = 3,600 \text{ lb./sq.ft.}$$

This load was removed and the soil has rebounded. Any new load that would be put on the soil and would be less than the one removed, and under which the soil was consolidated, should cause only very small settlements. Greater settlements could only be expected if the soil is of a swelling nature.

In this particular case, a load of 2,300 p.s.f. is added, which is quite smaller than the preconsolidation load and therefore, the settlements should be very small. For the piers placed at mid-height of the present cut, where the overconsolidation pressure is only 2,400 p.s.f., the new load would be only slightly smaller and, again, the settlements should be small. As to the abutments, settlements should be also very small because the footings would be placed within the upper desiccated crust which is much less compressible than the deeper layers.

The above reasoning certainly points to the possibility that the settlements will be smaller than the computed ones, and we would therefore recommend that spread footings be used. We would also like to point out that by the use of friction piles, settlements are not eliminated: sometimes, not even reduced. In this particular case, since the properties of the subsoil do not improve appreciably with depth, the reduction may be insignificant. It is also our opinion that spread footings could be used even if the settlements would be in the order of 2 - 3 inches, because the differential settlements, the ones that are to be considered, would not be more than 1-2 inches. Such differential settlements could be taken without damage by the proposed structure.

If you feel that there are some other questions or problems that you would like to discuss, please feel free to call on our office.

AGS/ndef

cc: Foundations Office  
Gen. Files.

*A. G. Stermac*  
A. G. Stermac,  
PRINCIPAL FOUNDATION ENGINEER



ONTARIO

DEPARTMENT OF HIGHWAYS

Bridge Division.

Memo to Mr. A. G. Stermac,  
Principal Foundation Engineer,  
Room 107, Lab. Building,

Date February 11, 1962.

Subject City of Sarnia  
Proposed Grade Separation  
Vidal Street and C.N.R.

From Mr. A. L. Kleinstemper

We are enclosing, herewith, a copy  
Racey MacCallum and Associates report for  
the above site (BA153). In addition, we  
are enclosing one set of Preliminary Plans.

We would appreciate your comments and  
recommendations regarding the report and  
the proposed structure.

ALK/ea

A. L. Kleinstemper,  
Municipal Bridge Liaison Engineer.

# RACEY, MacCALLUM AND ASSOCIATES

LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

**Consulting Engineers**  
AND ASSOCIATED STAFF

MONTREAL



OTTAWA

TORONTO

DONALD C. MACCALLUM, B.ENG., M.E.I.C., P.ENG.

H. JOHN RACEY, B.SC., M.E.I.C., P.ENG.

GEORGE L. HOUGHTON, A.M.I.MECH.E., M.E.I.C., P.ENG.

TORONTO DIVISION  
27 CARLTON STREET

Reference: S-501/T-3189  
- Report -

8th September, 1961.

Lazarides, Damas & Smith Limited,  
209, Davenport Road,  
TORONTO - Ontario.

61-F-247M

Attention: Mr. N.E. Damas, P.Eng.

RE: SOIL INVESTIGATION, RAILWAY OVERPASS,  
VIDAL STREET SOUTH,  
SARNIA, ONTARIO.

Dear Sirs,

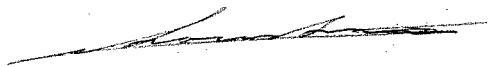
The enclosed report presents the results of our soil investigation at the above location.

We hope this report is satisfactory to you; if you have any questions about it please do not hesitate to get in touch with us.

Thank you for this opportunity of being of service to you.

Yours very truly,

RACEY, MacCALLUM AND ASSOCIATES LIMITED,

  
J. J. Schoustra, P. Eng.,  
Divisional Soil Engineer.

JJS/HK



Lazarides, Damas & Smith Limited,  
209, Davenport Road,  
Toronto - Ontario.

SOIL INVESTIGATION, RAILWAY OVERPASS,  
VIDAL STREET SOUTH,  
SARNIA, ONTARIO.

Reference: S-501/T-3189  
- Report -

Racey, MacCallum & Associates  
Limited.

8th September, 1961.

# RACEY, MACCALLUM AND ASSOCIATES LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

**Consulting Engineers**  
AND ASSOCIATED STAFF

MONTREAL



OTTAWA

TORONTO

DONALD C. MACCALLUM, B.ENG., M.E.I.C., P.ENG.

H. JOHN RACEY, B.SC., M.E.I.C., P.ENG.

GEORGE L. HOUGHTON, A.M.I.MECH.E., M.E.I.C., P.ENG.

TORONTO DIVISION  
27 CARLTON STREET

Reference: S-501/T-3189

- Report -

8th September, 1961.

RE: SOIL INVESTIGATION, RAILWAY OVERPASS,  
VIDAL STREET SOUTH,  
SARNIA, ONTARIO.

## INTRODUCTION :

The purpose of the investigation was to determine the engineering properties of the soil relevant to the replacement of an existing bridge over the Canadian National Railways' main line in Sarnia.

Four boreholes were put down, samples of soil taken and field and laboratory tests carried out.

This report gives the results of the investigation and presents recommendations for design and construction.

The investigation was authorised by City of Sarnia purchase order No. 30296 of July 21, 1961.

## FIELD WORK :

The locations of the boreholes are as shown on the accompanying site plan ( Enclosure No. 1). Borehole No. 1 was put down five feet north of the position indicated on the Client's plan SK 691-1 and borehole No. 3 was located ten feet south of the specified position. The reason for these changes is that the slopes of the cut were of a terraced nature and the revised positions of the boreholes placed then on the comparatively level terraces instead of on the steepest part of the slope immediately above the small retaining wall and where there was a real danger of the drill falling on to the railway track while it was being manoeuvred into position. It is not considered likely that there is any significant difference in soil conditions between the specified and actual positions.

- 2 -

Reference: S-501/T-3189

- Report -

8th September, 1961.

The boreholes were carried out by a skid-mounted diamond drill rig and wash boring methods were used to advance the holes. Split-spoon samples, in-situ vane tests and Shelby (thin-wall) tube samples were taken at intervals of five feet.

The two-inch split-spoon sampler was driven by a 140 lb. weight falling a distance of 30 inches. The number of blows required to drive the sampler one foot constitutes the standard penetration test for soils.

The blows were recorded and are plotted on the borehole data sheets (Enclosures 2 to 5).

Elevations given on the borehole data sheets are true geodetic and are derived from the elevation of the top of the south rail of the C.N.R. track under the east hand-rail of the bridge. The elevation of the top of the track rail was 565.90.

#### SITE AND GEOLOGY :

The site is located approximately one thousand feet east of the east portal of the St. Clair railway tunnel, through which the Grand Trunk main line of the C.N.R. passes under the St. Clair River. The gradient of the track falls to the west from a general ground elevation of about 605 down to an elevation of 565 under the existing bridge, so that the total depth of the cut at the site is about 40 feet.

The slopes of the cut, more markedly at the western or deep end, have terraced surfaces. It is though probable that these terraces or berms were part of the original construction since, on the south slope of the cut between the bridge and the tunnel, there are two terraces and three slopes. The terraces run horizontally and approximately one-third and two-thirds of the way down the slope. The even alignment of the terraces suggest that they were man-made and not a natural phenomenon. The backs of the terraces are generally at a slightly lower elevation than the crests and in some places pipes have been laid in the channel so formed, thereby assisting longitudinal drainage. The mean slope of the cut near the tunnel was about  $2\frac{1}{2}$  horizontal to one vertical.

The existing bridge is a five-span structure consisting of steel plate-girder spans carried on four two-column bents and two concrete abutments. The bents rest on concrete pedestal footings which are apparently pyramid-shaped. There was no visible evidence of settlement of the bridge, which was of comparatively light construction.

Reference: S-501/T-3189  
- Report -

8th September, 1961.

In common with most of the area within Lambton and Essex Counties, the City of Sarnia is situated on a deep deposit of grey silty clay which extends down to a shale bedrock at depths in excess of 120 feet and which has been subjected only to light consolidation loads. This clay stratum is known to have a desiccated zone near the surface which is harder than the material lower down. However, since the boreholes were commenced at elevations twenty feet below the general ground elevation at the top of the cut, it was considered probable that this "crust" would be by-passed and not be encountered in the boreholes.

#### SOIL CONDITIONS :

Beneath six to eight inches of clay loam topsoil, the first main soil stratum was a grey-brown damp to moist clayey silt. The resistance of this material to the split-spoon sampler was high enough to imply that reasonably good bearing values could be obtained if this magnitude of penetration resistance persisted at lower depths. However, the resistance decreased permanently after a few feet of drilling and it is assumed that the initial harder section was the lower part of the "crust" zone referred to earlier or a new crust formed by weathering since the construction of the cut seventy years ago.

The main stratum encountered throughout most of the footage of the boreholes was a medium stiff grey silty clay stratum commencing just below the clayey silt and extending down almost to the limit of the borehole. The penetration resistance of this material was uniformly low except near the lower limits of the boreholes. Shear strengths as measured by vane tests, fluctuated with depth but had an average of 1100 - 1300 psf. On remoulding, the vane shear strength was found to be consistently about half or two-thirds of the undisturbed value.

At an elevation of about 535, the penetration resistance of the clay increased as a result of a change from the silty clay to a grey clayey fine sand. This was in accord with the results of a previous site investigation in Sarnia where the boreholes went down to a depth of 120 feet. A fine sand stratum was encountered at approximately elevation 530 but the soil later reverted to grey silty clay very similar to that above the sand.

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Reference: S-501/T-3189

- Report -

8th September, 1961.

WATER CONDITIONS :

Observations were made of water levels in the boreholes but in view of the general soil conditions - particularly the very low permeability - it is considered that the readings are not likely to be of any assistance in indicating the level of the ground water table.

The level of the water in the St. Clair River is known to be 581 and, as this is approximately the same as the ground surface at the boreholes, it is probable that the ground water table is very close to the surface. Because of the very low permeability of soil there is not likely to be any difficulty with ground water during construction.

LABORATORY TESTING :

At the beginning of the site investigation it became quite clear that the silty clay stratum would be the one in which the foundations of the new bridge would be constructed. It was, therefore, from this material that nearly all the Shelby tube samples were taken and on which all the laboratory tests were performed.

Atterberg Limits were obtained for two samples of the silty clay from borehole No. 4. A sample from a depth of 14 feet had a liquid limit of 51.3% and a plastic limit of 15.0%. At a depth of 40 feet, the liquid limit was 22.9% and the plastic limit 13.2%. A sieve and hydrometer analysis (Enclosure No. 6) of the former sample indicated that the material was composed of 35% clay and 45% silt. Thus, its identification as a silty clay was confirmed since these percentages are within the range of definition of a silty clay according to the standard soil classification.

Since the plasticity index of the material was approximately the same as the percentage of clay, the clay may be classified as one of normal activity as defined by Skempton.

Unconfined compression tests were carried out in the laboratory on twelve of the Shelby tube samples. The results are shown in Enclosure No. 7. Failure of the samples generally occurred at quite high percentage strains and it will be seen that the shear strengths corresponding to the compressive strengths at failure are somewhat lower than the shear strength values obtained in the vane tests in the field. This phenomenon is quite normal since the "undisturbed" sampling is not completely undisturbed and even the most

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Reference: S-501/T-3189

- Report -

8th September, 1961.

Careful transportation causes some disturbance within the soil. This has been demonstrated by the fact that field unconfined compression tests yield compressive strengths higher than tests on samples taken back to the laboratory. Thus the laboratory compression tests usually give a slightly pessimistic evaluation of the shear strength of the soil and this should be borne in mind when they are compared with the vane test results. As a result of these considerations, it was decided, for design purposes, to assume the soil is a frictionless material with a cohesive strength of 1,000 pounds per square foot.

As it was apparent that the permissible settlement of the structure would govern the choice of type of foundation, two consolidation tests were carried out and the results of these tests are shown in Enclosure No. 8 and 9. The two samples were representative of different depths in view of the difference between the liquid limits of the samples on which the Atterberg tests were performed.

#### DESIGN CONSIDERATIONS :

With regard to the cost of the structure above ground, the most economical design under consideration was a bridge composed of three equal spans with two piers close to the existing retaining walls near the track. This arrangement would necessitate grading the sides of the cut to a two-to-one slope directly back from the tops of the retaining walls.

The minimum factor of safety recommended for earth slopes is 1.5 when soil characteristics have been determined by field and laboratory tests. Using this factor of safety together with the cohesive strength of 1,000 lb./sq.ft. referred to earlier, classical soil mechanics design principles indicate that the maximum height of a cut with two-to-one slopes is thirty feet. With the material involved, it is most unlikely that this approach errs on the side of conservatism.

As the total height of the slope would be forty feet, it is therefore recommended that the three-span design should not be used.

The main alternative to the three-span structure was one consisting of five spans which would have the slopes of the cut at about their present contours and would have piers and abutments at approximately the same position as in the existing bridge.

Reference: S-501/T-3189

- Report -

8th September, 1961.

In this proposed arrangement, the most critical shear failure conditions occur near the bases of the two centre piers which are close to the existing retaining walls. Potential shear failure would occur along and are passing beneath the retaining walls and up through the railway roadbed. The spread footings proposed for this design were nine feet wide and their bases were intended to be four feet below the bottom of the drainage channels on each side of the roadbed.

In contrast to the factor of safety for a slope, the recommended factor of safety for a footing is 3.0. Again assuming a frictionless soil with a cohesive strength of 1,000 lb./sq.ft., the permissible bearing stress is 2,800 lb./sq.ft. It is understood that the probable bearing stress on this type of footing would be of the order of 2,300 lb./sq.ft. The proposed spread footing would therefore be safe against shear failure.

The other design consideration is the avoidance of excessive settlement. In order to investigate this, a settlement computation was carried out for a strip footing nine feet wide, with a bearing stress of  $1\frac{1}{4}$  tons/sq.ft. For the purpose of the calculation the soil beneath the footing was divided into eight layers: five of four feet immediately below the footing and three of ten feet. Thus a total of fifty feet of soil below the proposed footing level was taken into consideration for the settlement calculation.

The calculation is shown in tabular form in Enclosure No. 10. The consolidation characteristics for the higher sample on which the consolidation tests were performed were used for determining the settlement contributed by the uppermost three four-foot layers. The properties of the lower consolidation sample were used for the remainder of the layers.

As will be seen, the overall settlement is in excess of 5 inches which almost certainly could not be permitted in any orthodox design. Since there is no significant depth of soil of lower compressibility with a reasonable range, the settlement can only be reduced by decreasing the increment of the stress on the soil in relation to the original stress before loading.

Friction piles have the effect of transferring the stresses to a much lower depth than the base of, say, spread footings without removal of the soil above this depth. This means that the initial pressure due to overburden is not zero as in the case of the spread footing settlement calculations, with the result that the increase in

- 7 -

Reference: S-501/T-3139

- Report -

8th September, 1961.

stress is represented on the appropriate consolidation versus void ratio curve by a section of the graph further to the right than with the spread footing case. In this manner, a smaller reduction in void ratio is obtained for the same increase in stress.

If the settlement is still excessive, the use of raker piles instead of vertical ones increases the area between them at any particular level and thus the stress producing settlement is decreased.

A suggested piling scheme for the proposed bridge consisted of two rows of piles for each pier, each row consisting of 27 piles, four feet apart with the rows three feet apart. The probable dead load per pile was calculated to be 31,600 lb. or just under 35,000 for dead load plus live load. With a factor of safety of three, it will therefore be necessary for each pile to support, say, 100,000 lb. by friction alone since the bearing capacity derived from end-bearing would be negligible. Assuming a surface adhesion of 750 lb./sq.ft. as appropriate to this type of clay and a mean pile diameter of 11", the required length of the piles is 46 feet.

An inclination to the vertical of one in six is recommended for the piles. If this is done, the total settlement is expected to be from 1-3/4" to 2". Differential settlements should be insignificant and well within the magnitude which can be accommodated by a statically determinate structure of this type. Approximately half this consolidation settlement may be expected to take place during construction.

As has been remarked upon earlier, failure of the clay takes place at high strains and some settlement as a result of vertical shear forces may be expected. This will occur immediately on loading and is likely to be much less than the settlement due to consolidation since the unconfined compression tests show that the strain at one-third of failure stress is only 2%. Furthermore, a large part of this is expected to be nullified by stresses induced during the driving of the piles.



- 8 -

Reference: S-501/T-3189

- Report -

8th September, 1961.

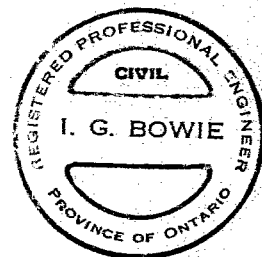
RECOMMENDATIONS AND CONCLUSIONS :

The main results of the investigation may be summarised as follows:

1. The site of the proposed bridge is located on a deep deposit of medium-stiff silty clay.
2. A slope of two horizontal to one vertical in this material with a height of forty feet would not possess the minimum factor of safety required by correct engineering design.
3. Spread footings as indicated in the provisional design of the bridge would be safe against shear failure but the settlement would be in excess of five inches for a bearing stress of 2,500 lb./sq.ft.
4. This settlement may be reduced by using friction piles, in particular, raker piles. By this means, the total settlement may be cut down to less than two inches, approximately half of which is expected to take place during the construction period.
5. There is not likely to be any difficulty with ground water during excavation for construction.

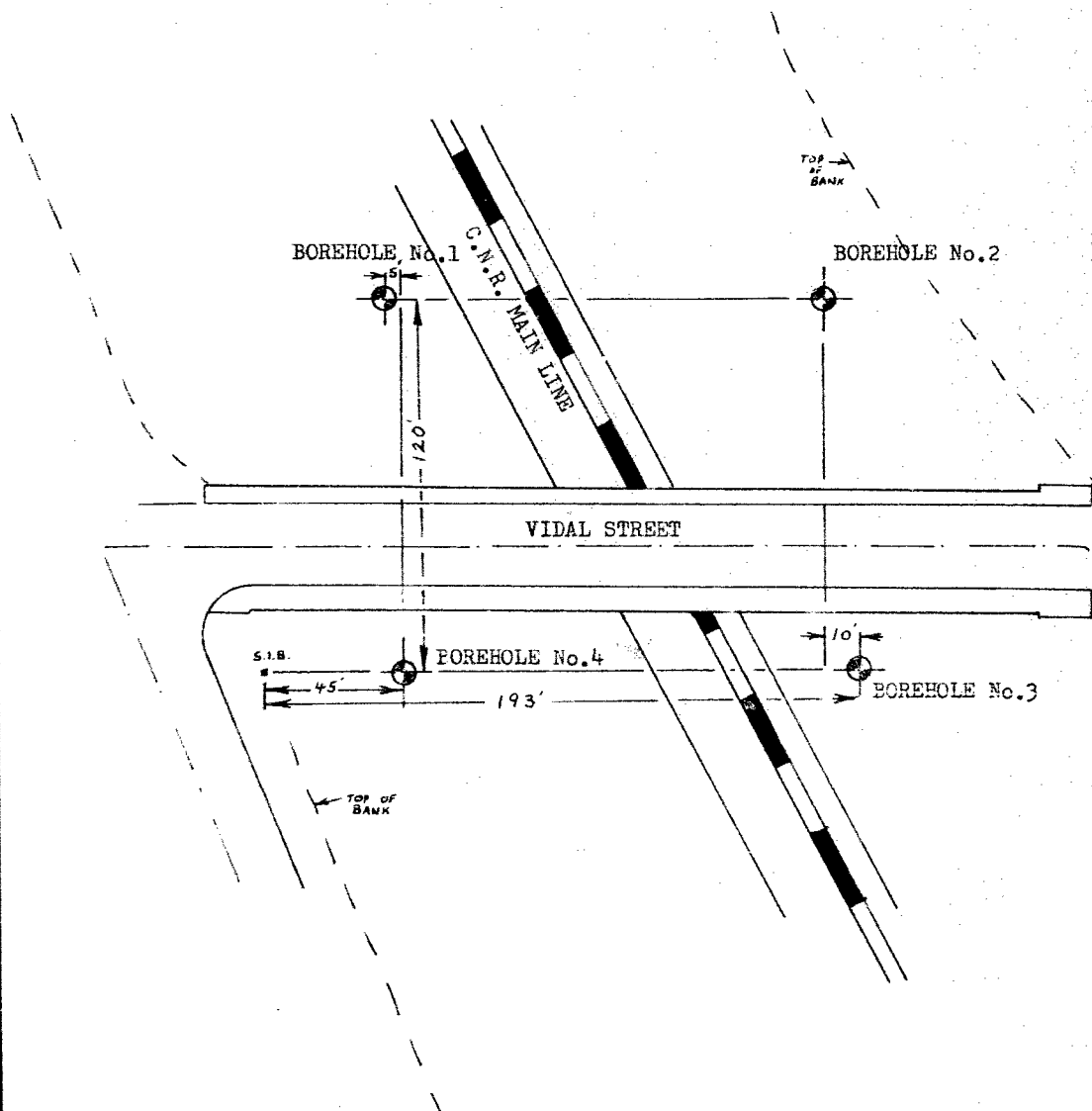


I. G. Bowie, P. E.,  
Struct. Engineer.



IGB/HK

Prep. By I.G.B.



VIDAL STREET OVERPASS -- SARNIA

LOCATIONS OF TEST BOREHOLES

## RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 1

Project: SOIL INVESTIGATION, RAILWAY OVERPASS,  
 Location: VIDAL STREET, SARNIA,  
 Hole Location: See Enclosure No. 1,  
 Hole Elevation and Datum: 579.2  
 Field Supervisor: I.G.B. Prep.: I.G.B.  
 Driller: H.D. Checked: J.J.S. Date: 4.8. '61.

## LEGEND:

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

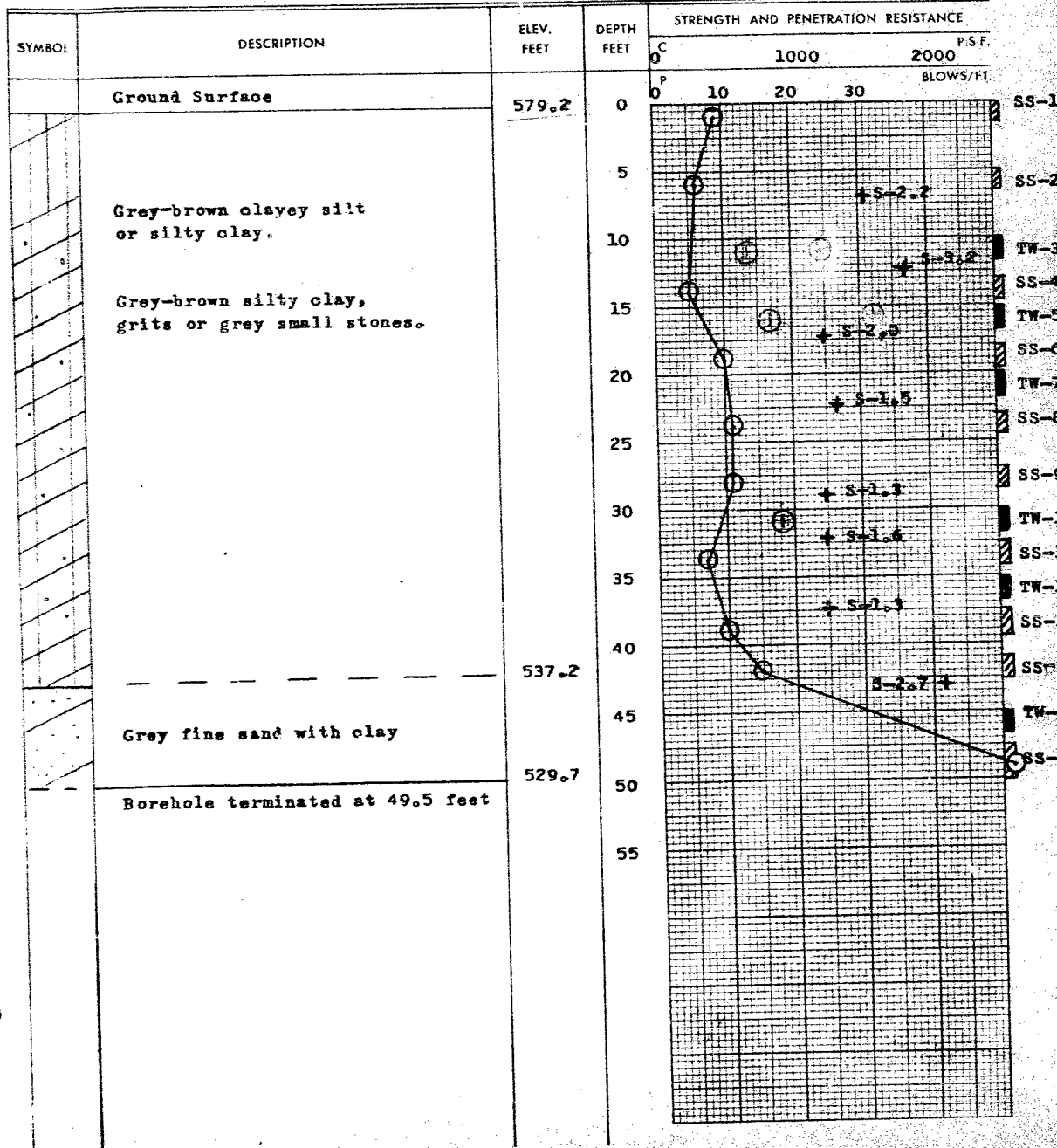
2" Split tube

2" Dia. Cone

Casing

⊕  
4°

⊕ ⊕



Foundation Engineering Division

## Engineering Data Sheet for Borehole: 2

### LEGEND

Shear Strength (C)

### Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance ( $P_i$ )

2' Split tube

2<sup>nd</sup> Dia. Cone

## Casing

[illegible]

**RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole: 3

Project: SOIL INVESTIGATION, RAILWAY OVERPASS,  
 Location: VIDAL STREET, SARNIA,  
 Hole Location: See Enclosure No. 1,  
 Hole Elevation and Datum: 577.3  
 Field Supervisor: I.G.B. Prep.: I.G.B.  
 Driller: H.D. Checked: J.J.S. Date: 9.8.'61

**LEGEND**

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

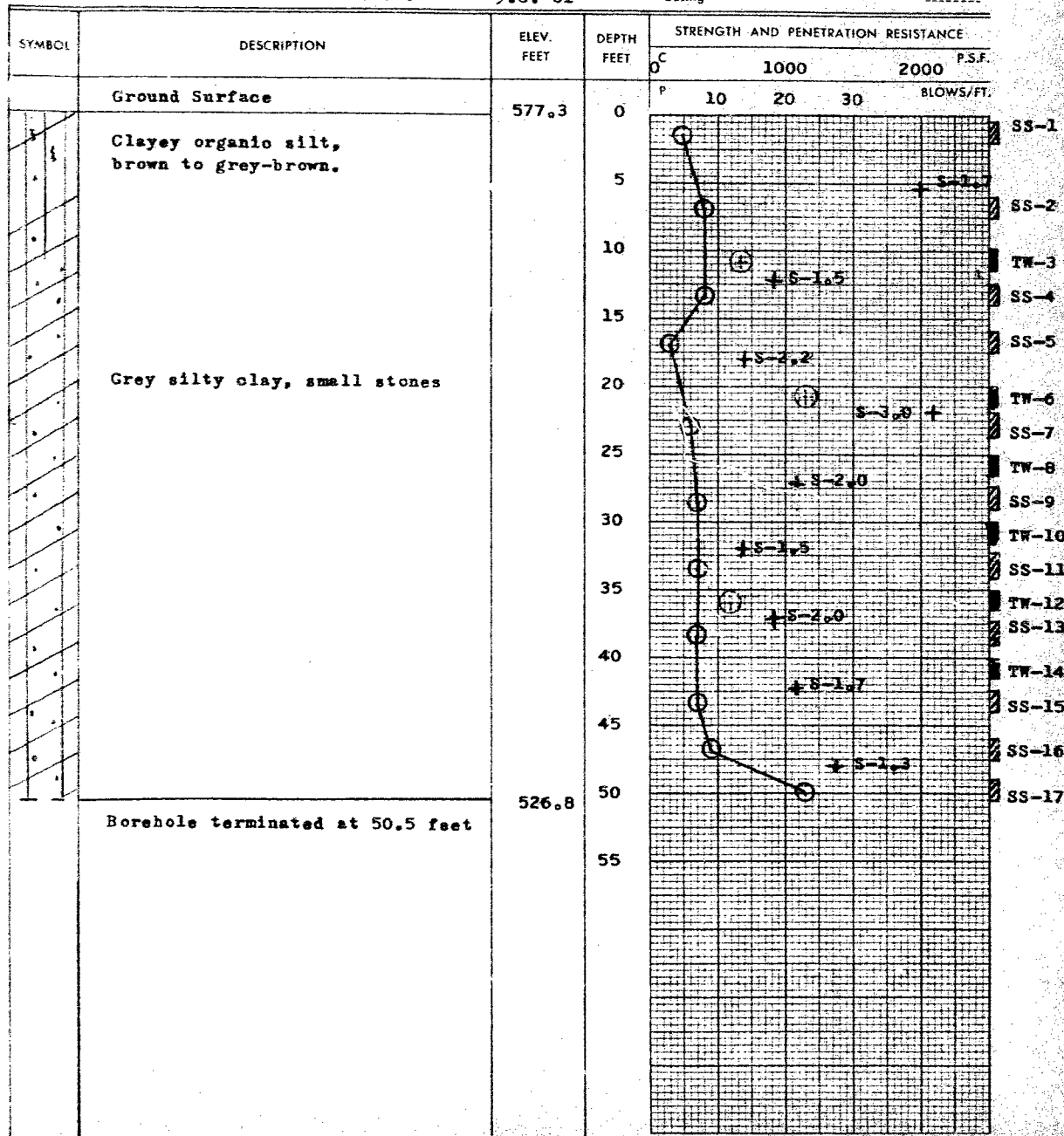
2" Split tube

2" Dia. Cone

Casing

⊕  
+S

⊕ ⊕



## RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 4

Project: SOIL INVESTIGATION, RAILWAY OVERPASS,  
 Location: VIDAL STREET, SARNIA,  
 Hole Location: See Enclosure No. 1,  
 Hole Elevation and Datum: 584.3  
 Field Supervisor: I.G.B. Prep.: I.G.B.  
 Driller: H.D. Checked: J.J.S. Date: 2.8.'61

## LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

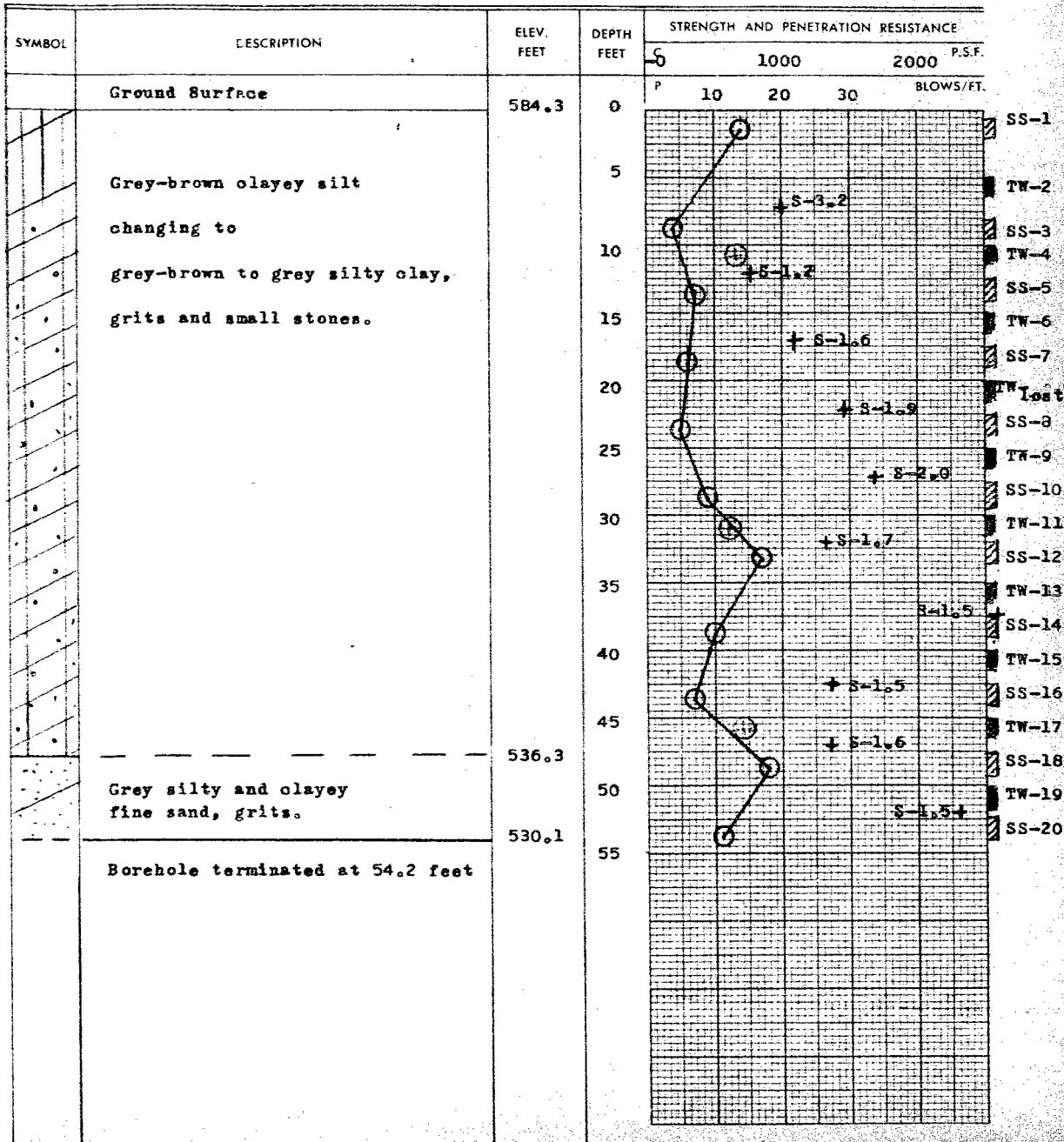
2" Split tube

2" Dia. Cone

Casing

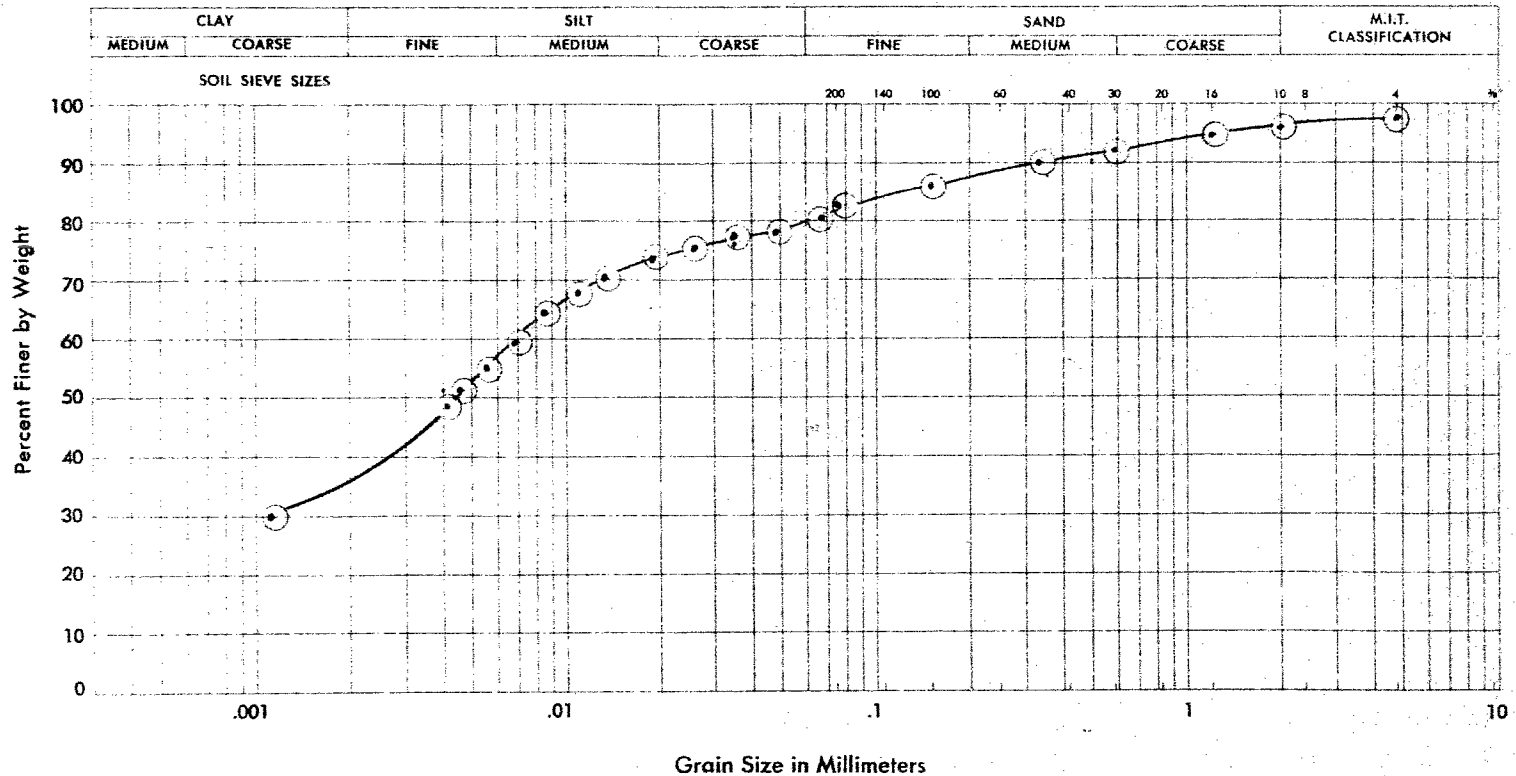
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# RACEY MacCALLUM AND ASSOCIATES LTD.

## GRAIN SIZE DISTRIBUTION



Project PROPOSED RAILWAY OVERPASS, VIDAL STREET, SARNIA.

Order No. S-501/T-3189

Legend BOREHOLE 4, SAMPLE 5 - GREY SILTY CLAY

Order No. S-510/T-3189

Enclosure No. 7

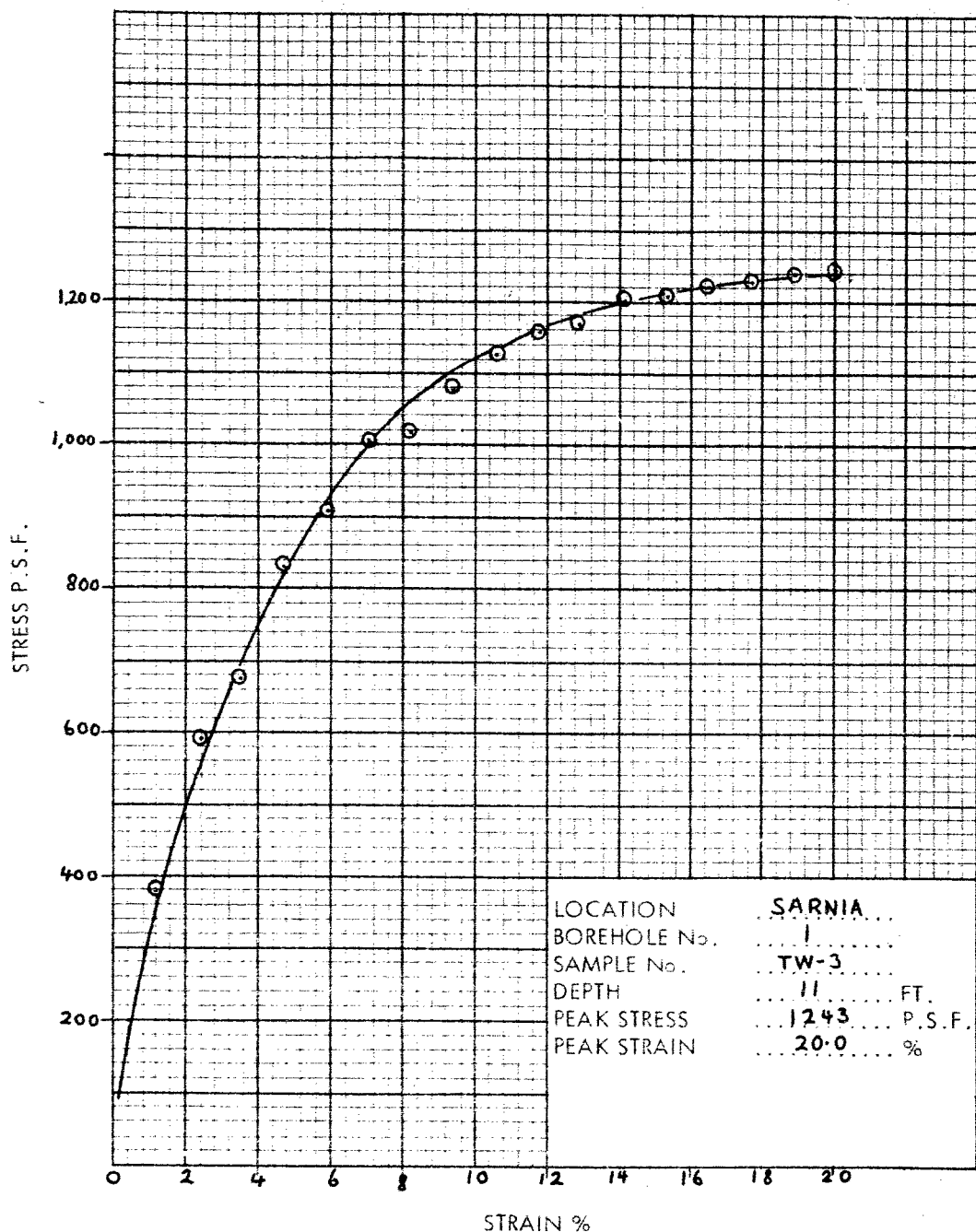
UNCONFINED COMPRESSION TEST RESULTS

<u>Borehole</u>	<u>Sample No.</u>	<u>Depth (feet)</u>	<u>Moisture Content</u>	<u>Compressive Strength (lb./sq.ft.)</u>	<u>Strain at Failure</u>	<u>Density (lb./ccu.ft.)</u>
1	3	11	31.6%	1243	20.0%	124.0
1	5		24.5%	1580	19.8%	124.0
1	10		28.6%	1720	19.0%	121.5
2	6		27.4%	1990	20.0%	119.0
2	10	31	25.9%	1620	15.5%	121.0
2	15	46	29.2%	2340	18.1%	120.5
3	3	11	24.4%	1215	18.8%	127
3	6	20	26.8%	2280	18.6%	124.0
3	12	36	31.4%	1110	19.4%	114.0
4	4	11	23.9%	1250	19.0%	117.6
4	11	31	22.7%	1015	16.3%	118.0
4	17	46	27.8%	1320	19.0%	118.0



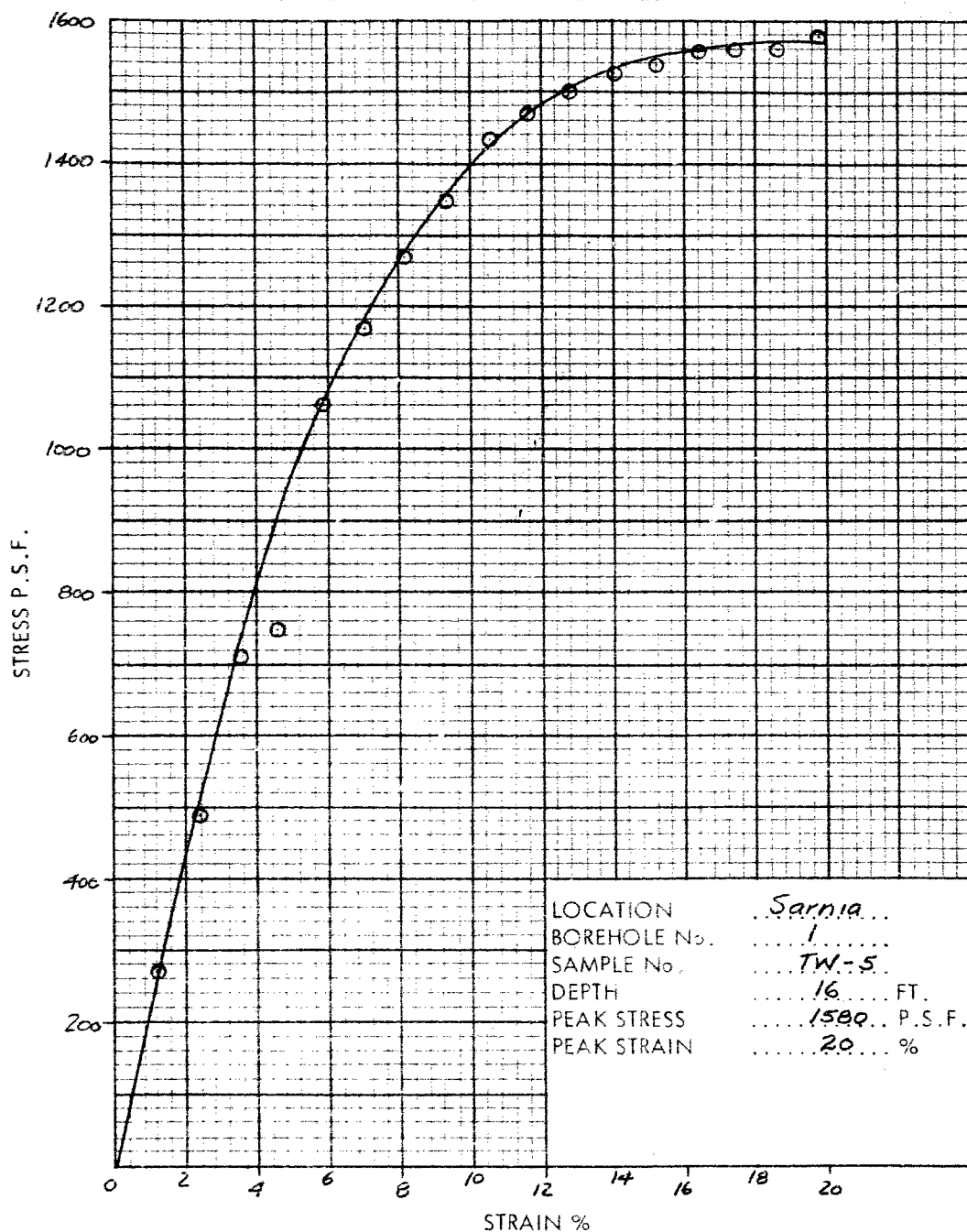
Prep. By I. G. B.

## UNCONFINED COMPRESSION TEST



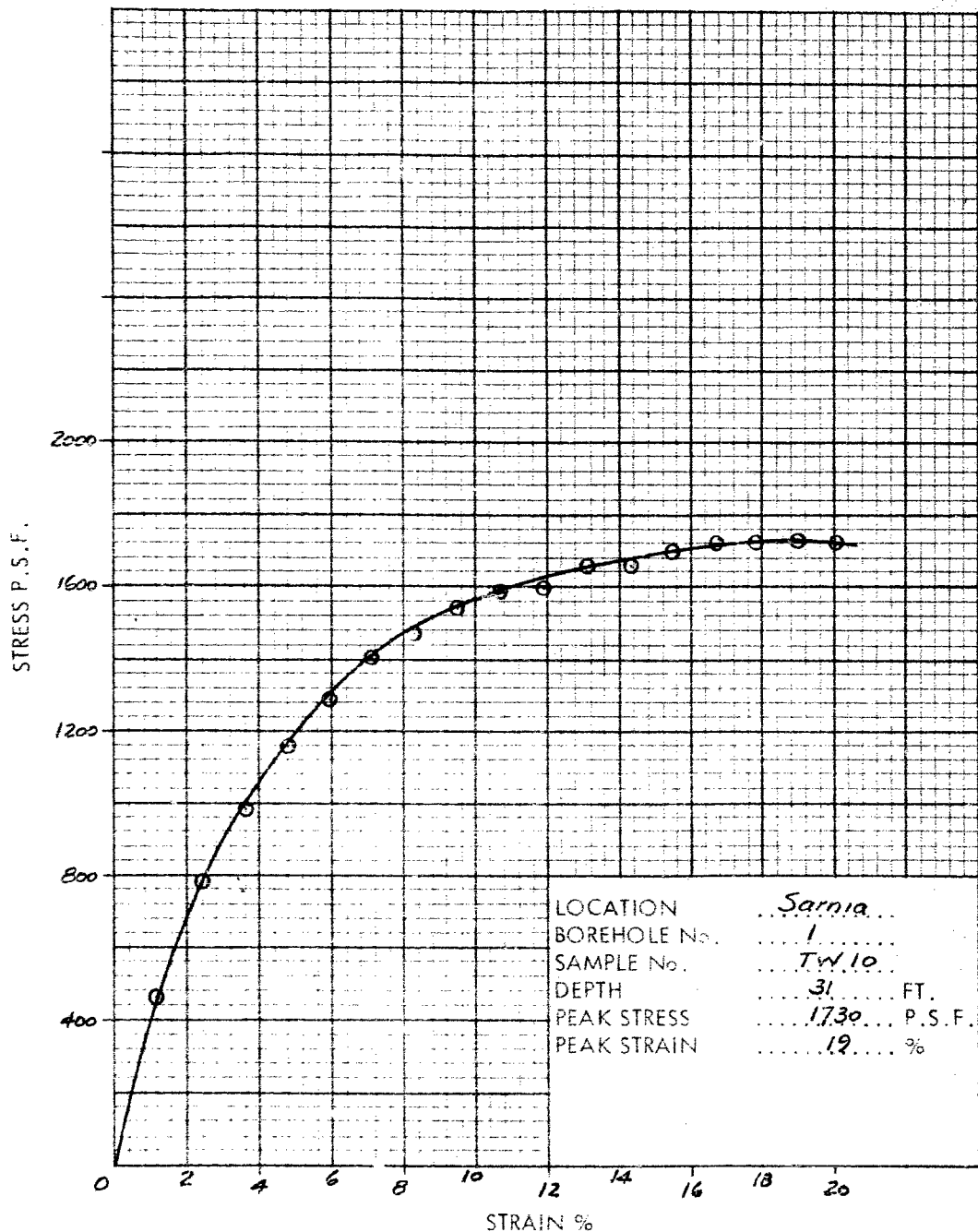
Prep. By PJA

## UNCONFINED COMPRESSION TEST



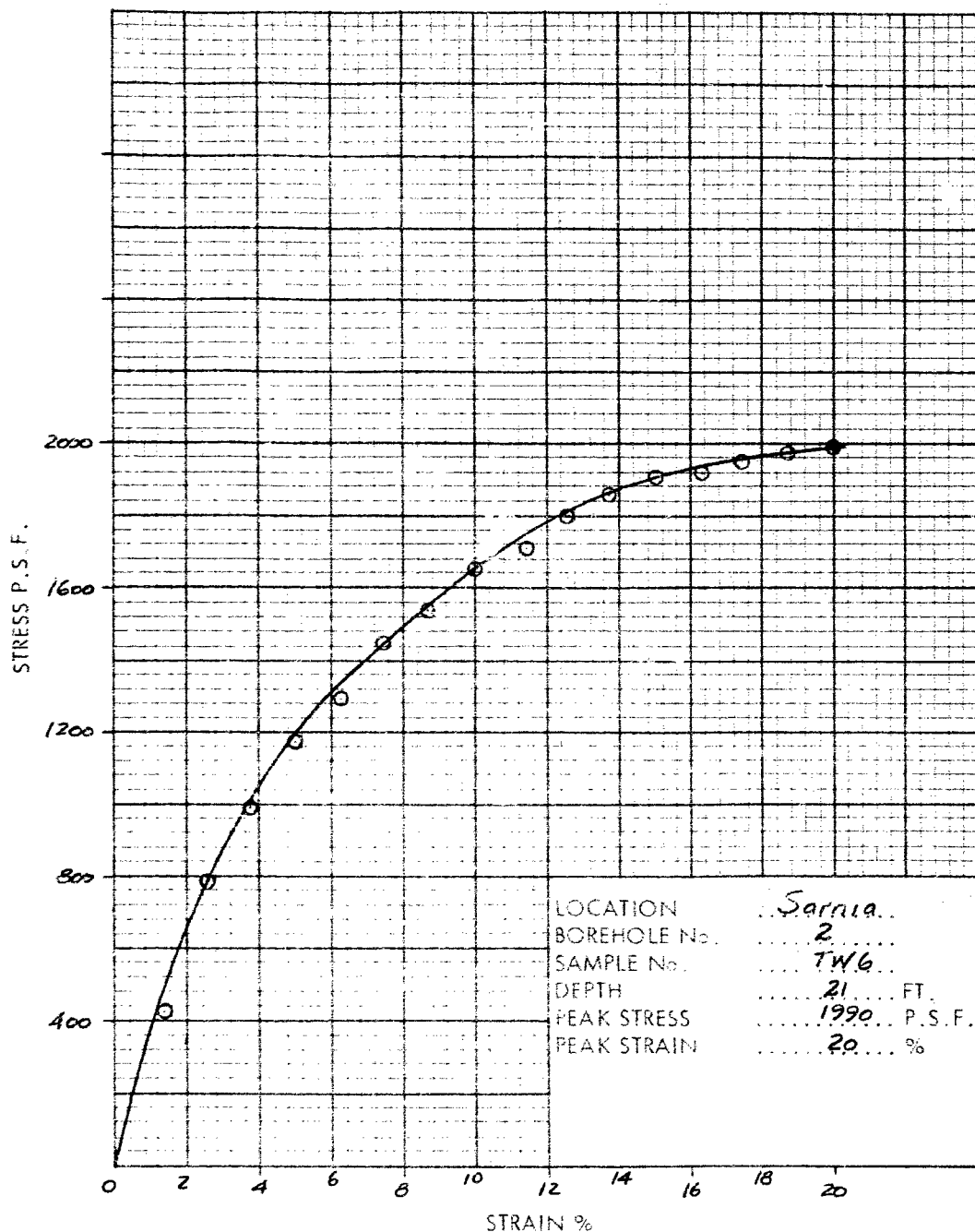
Prep. By PJA

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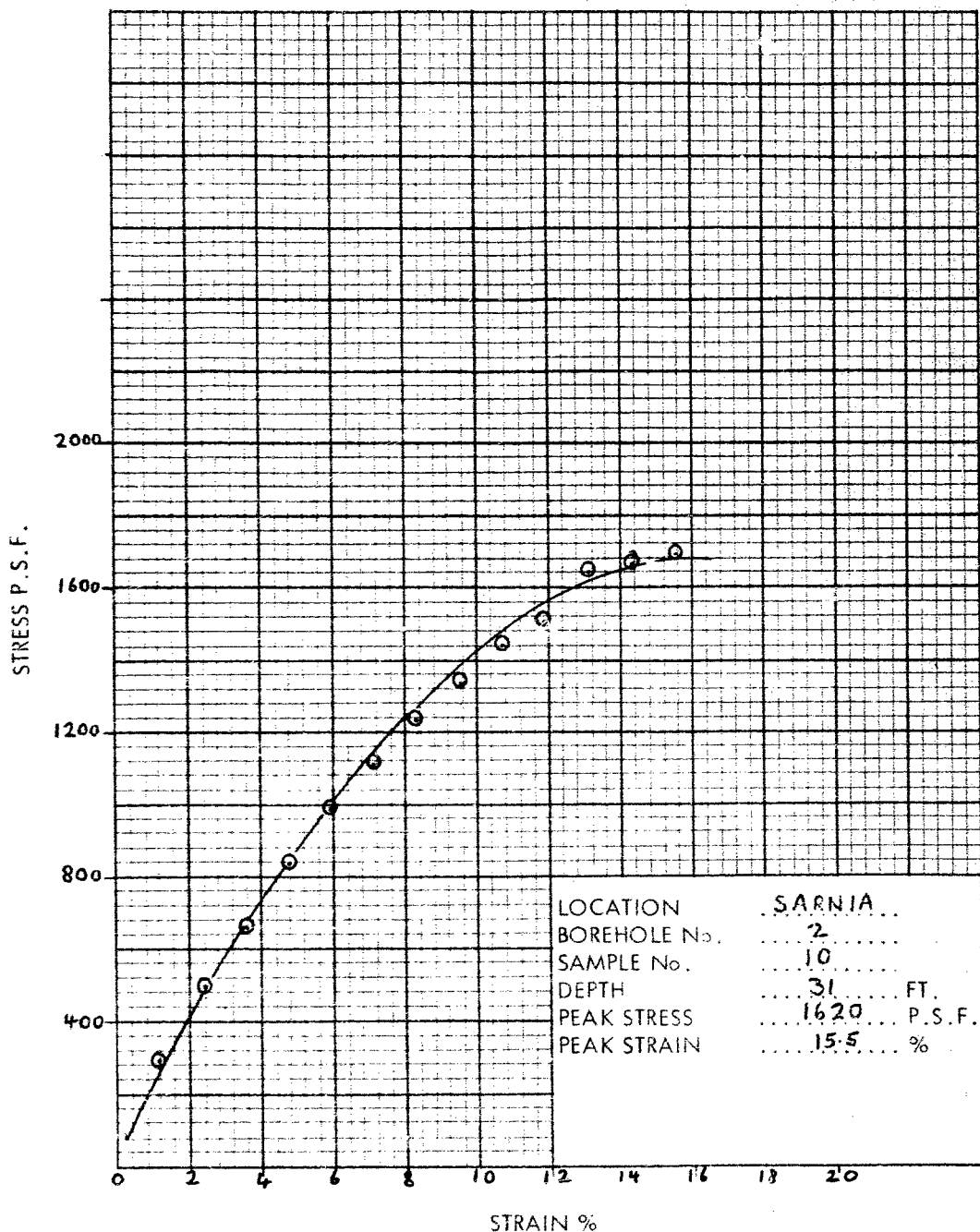
Prep. By PJA

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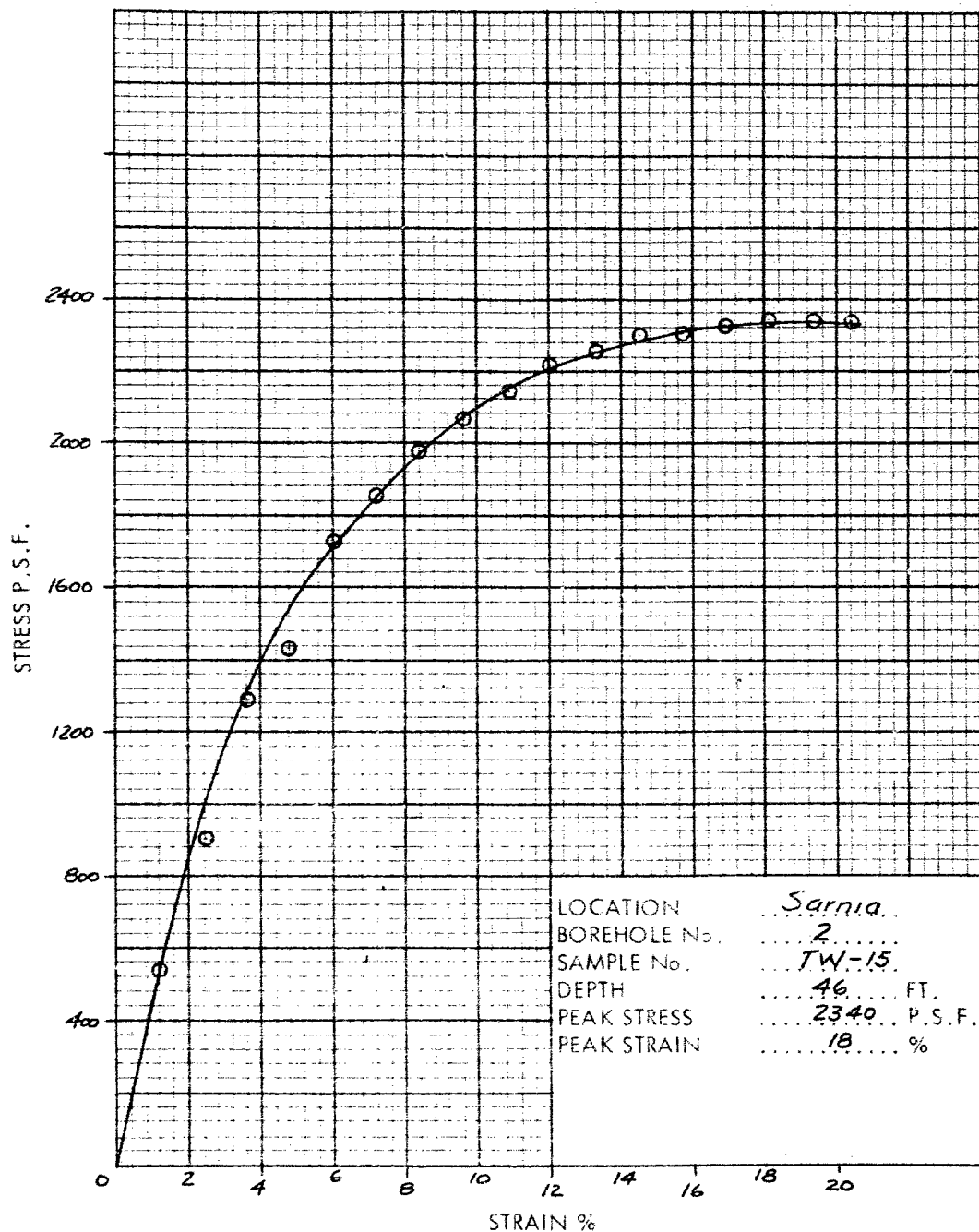
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## UNCONFINED COMPRESSION TEST



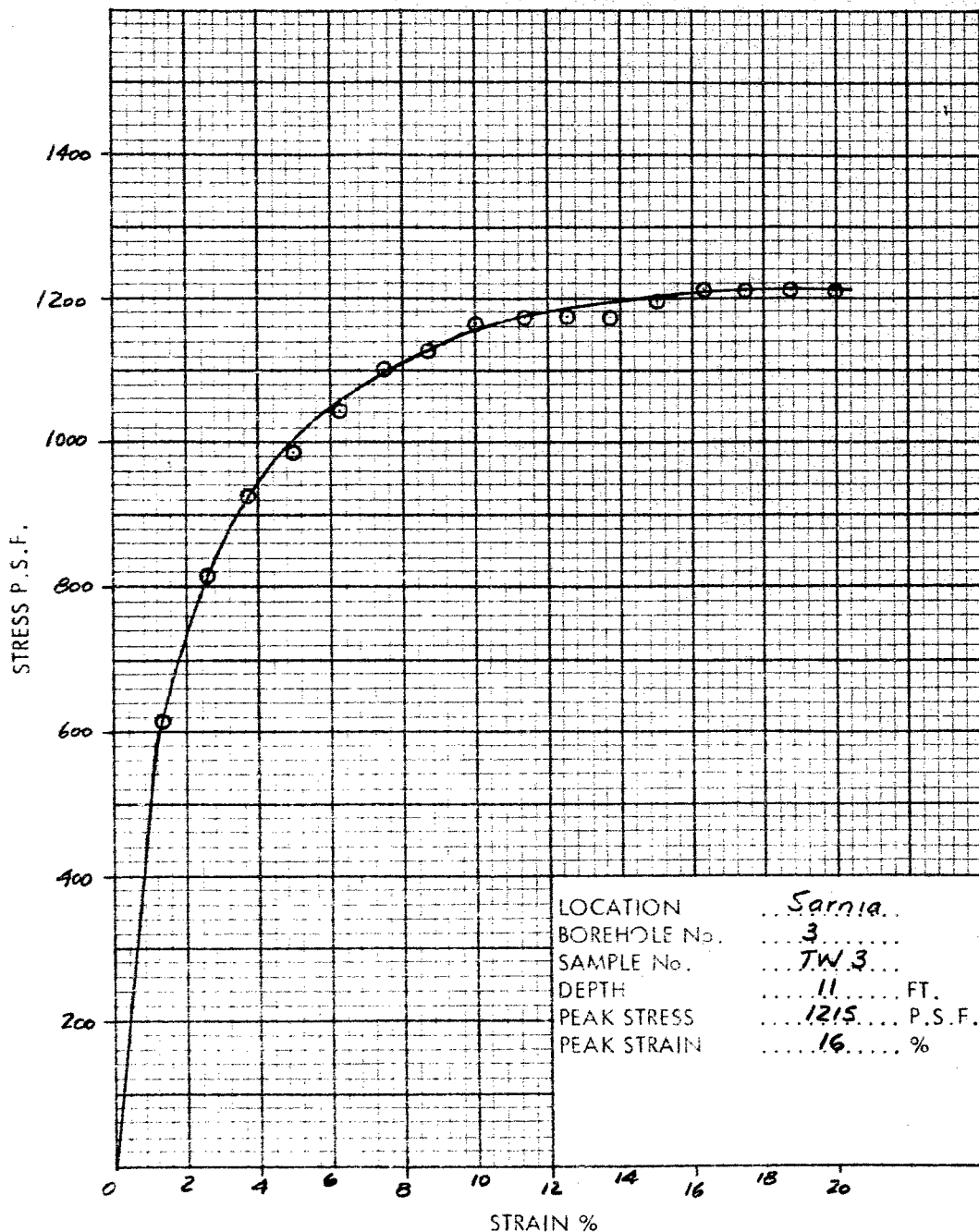
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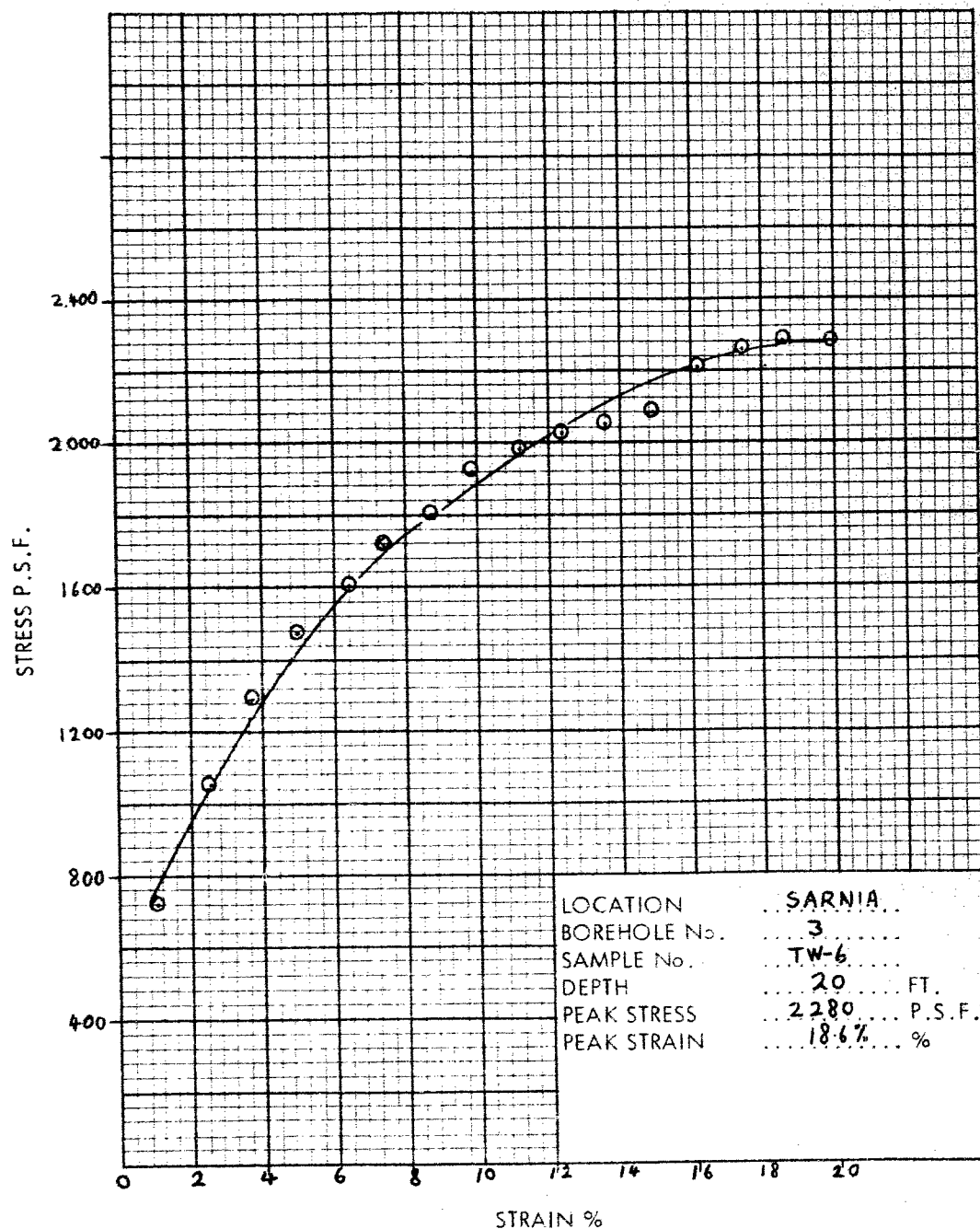
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## UNCONFINED COMPRESSION TEST



Prep. By I. G. B

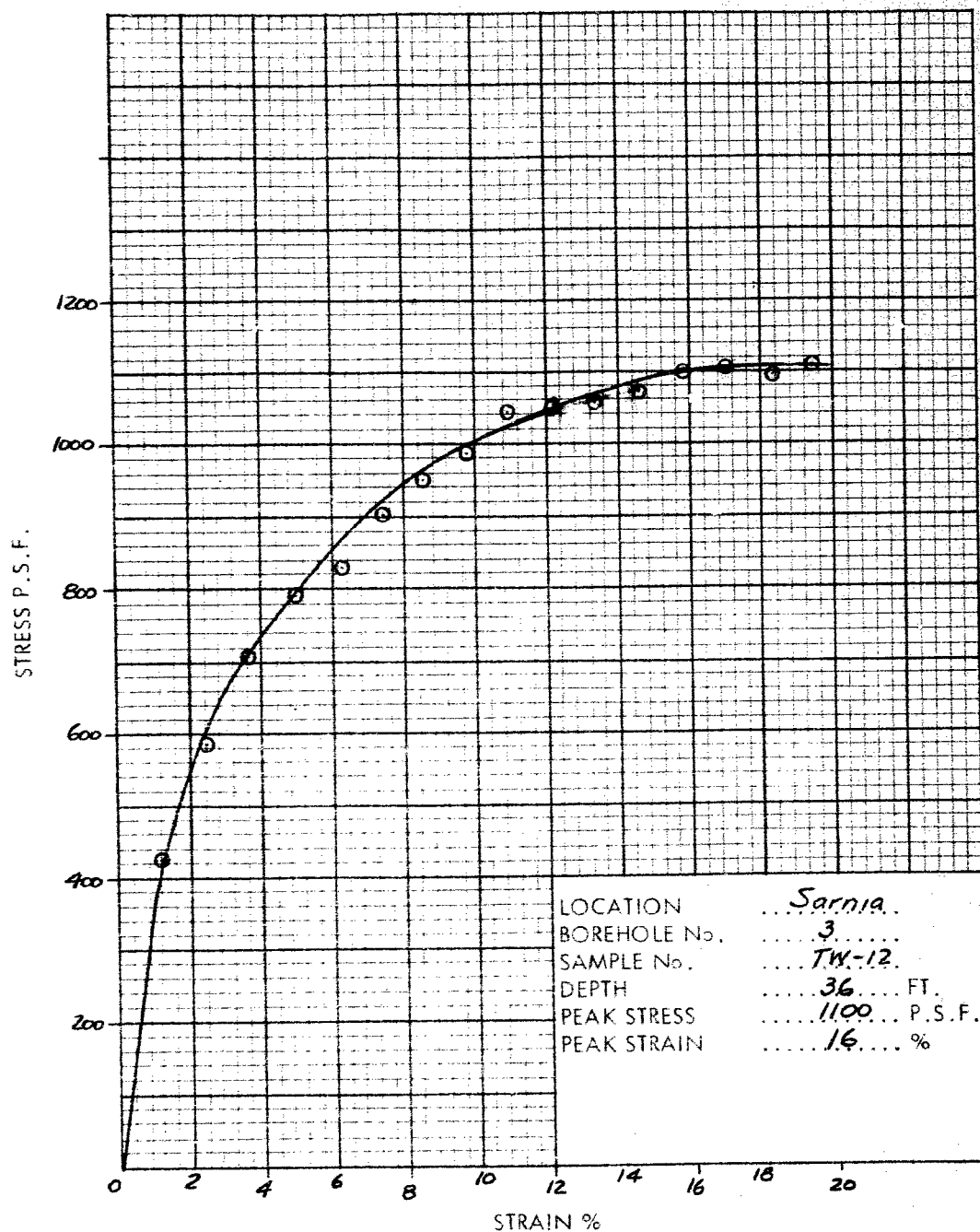
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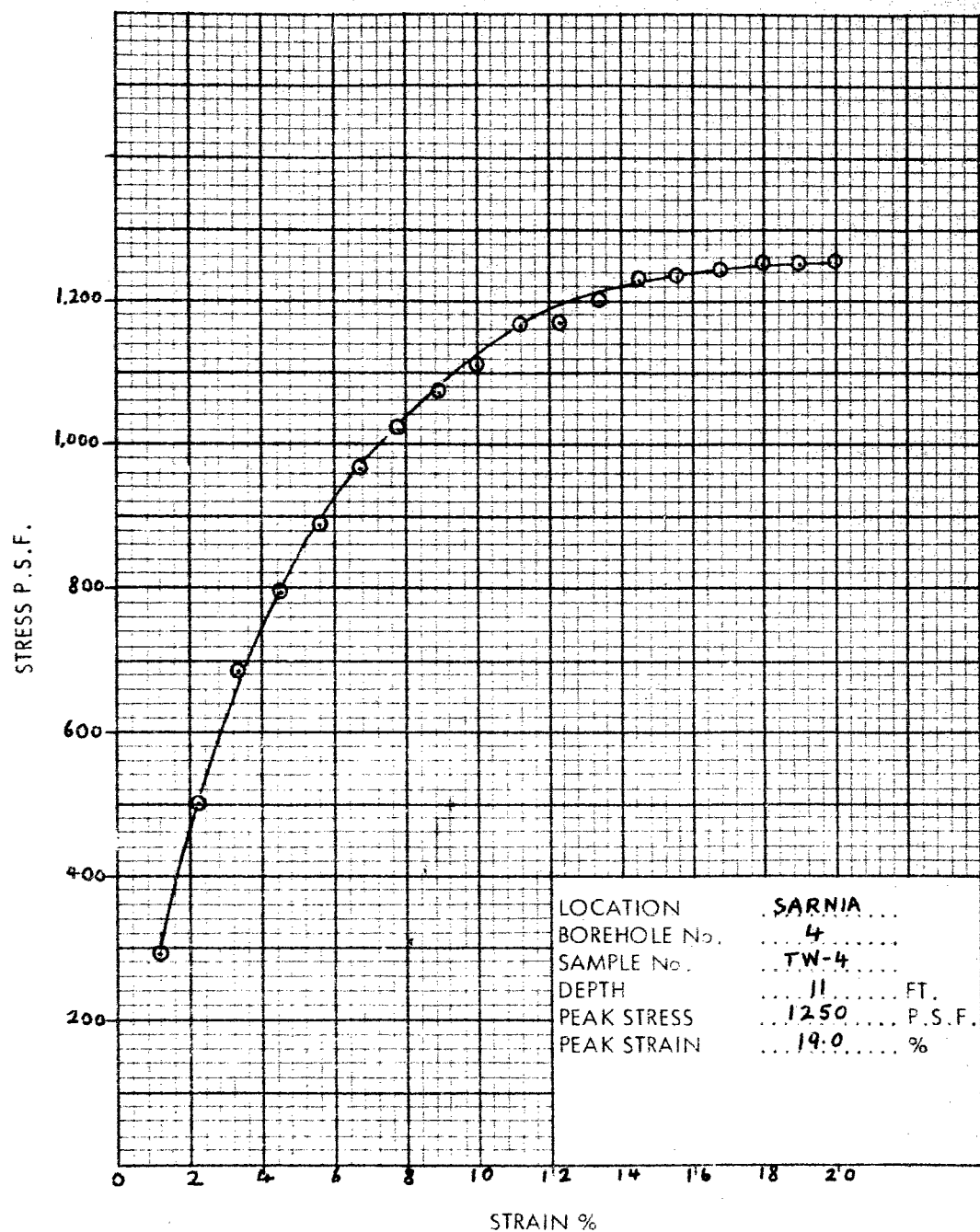
Prep. By *PJA*

## UNCONFINED COMPRESSION TEST



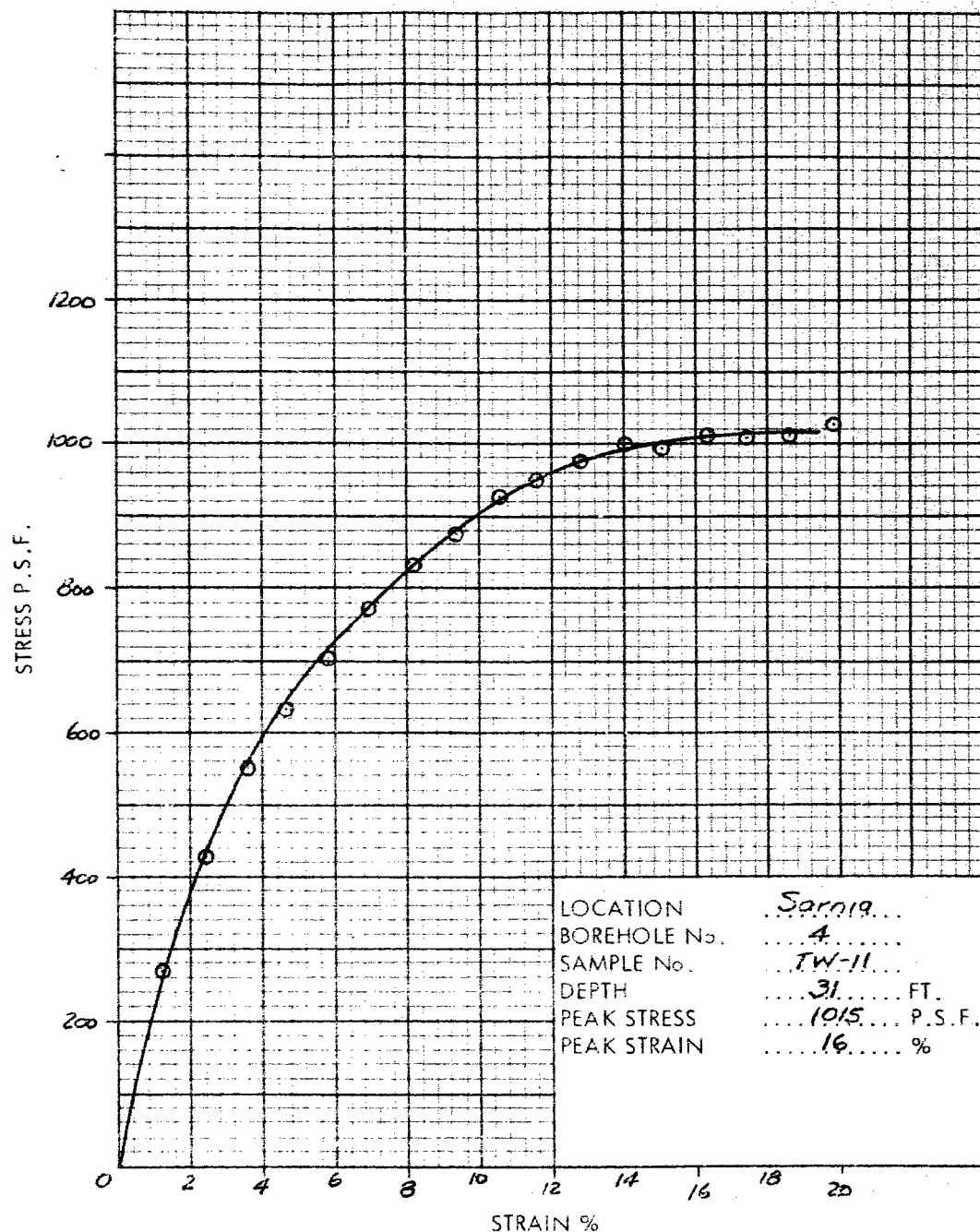
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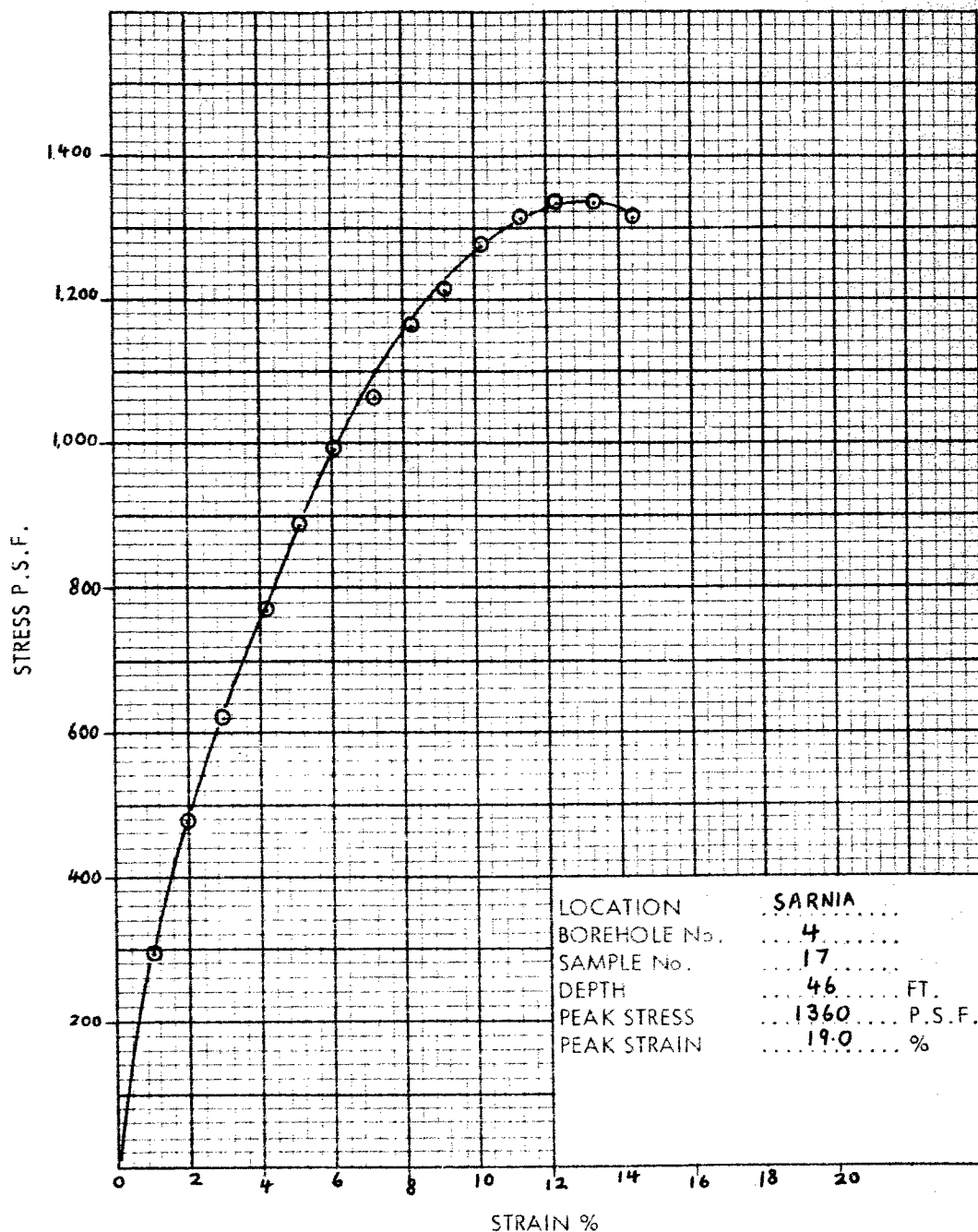
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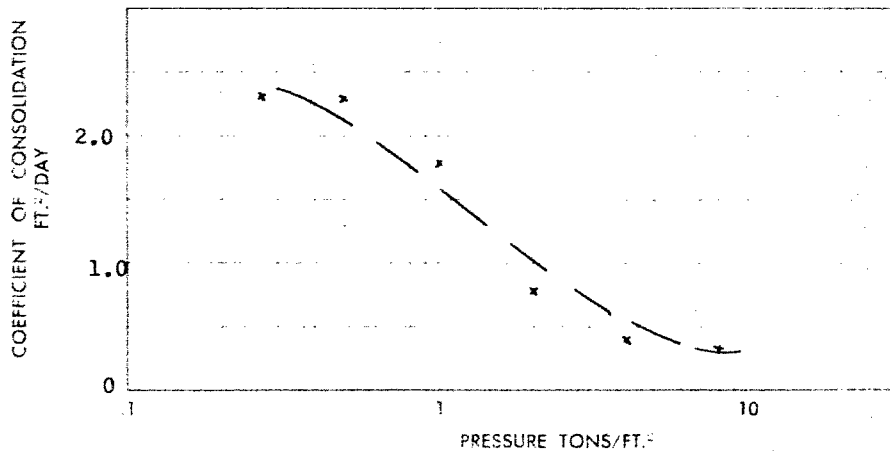
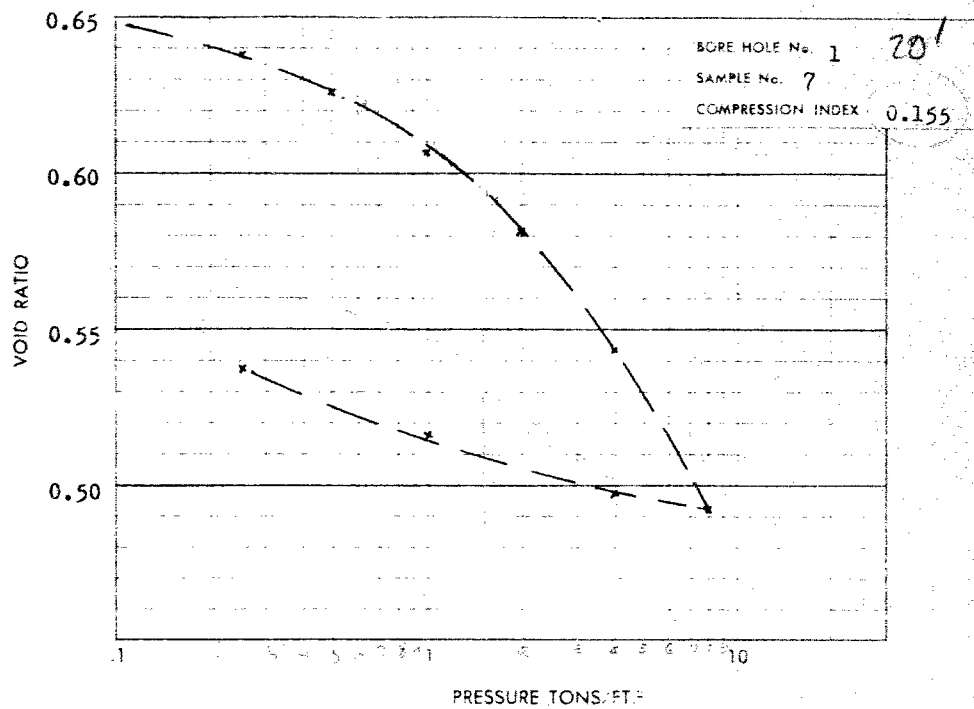
Prep. By I.G.B.

## UNCONFINED COMPRESSION TEST



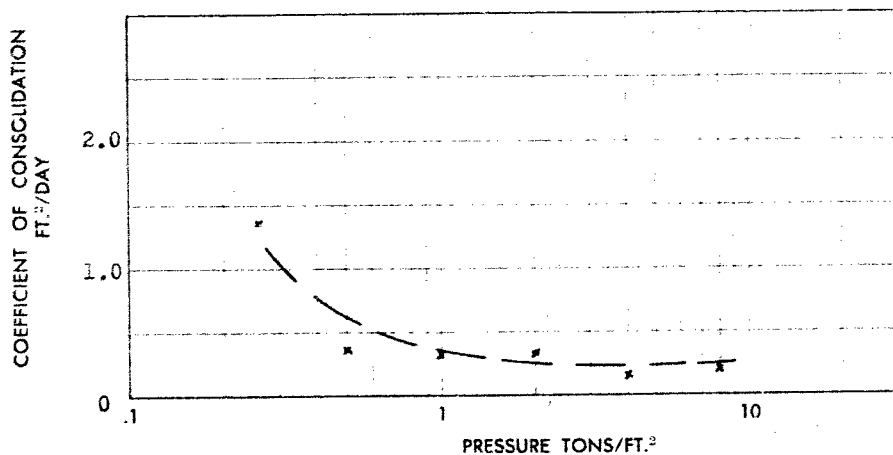
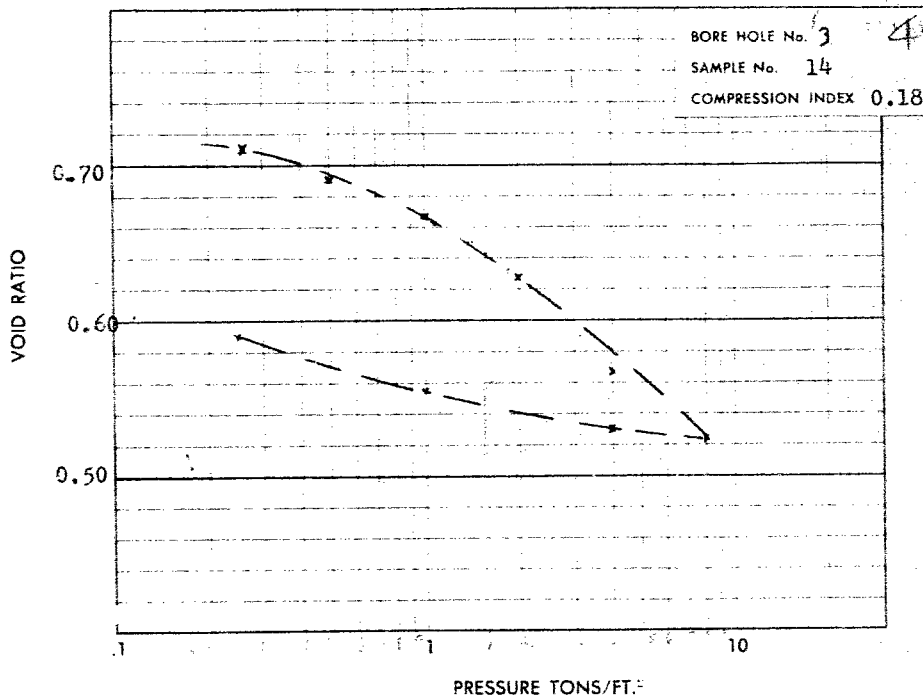
## RACEY MacCALLUM AND ASSOCIATES LTD.

## CONSOLIDATION TEST



## RACEY MacCALLUM AND ASSOCIATES LTD.

## CONSOLIDATION TEST



Order No. S-510/T-3189

Enclosure No. 10

SETTLEMENT CALCULATION

Assumed: Bearing Stress - 2,500 lb./sq.ft.;  
Strip Footing 9 feet wide.

<u>Depth below Footing</u>	<u>Depth of Centre</u>	<u>Stress Coefficient</u>	<u>Stress (Ton/sq.ft.)</u>	<u>Void Ratio Difference</u>	<u>Settlement</u>
0					
4'	2'	0.972	1.22	0.045	1.31"
8'	6'	0.760	0.95	0.036	1.05"
12'	10'	0.504	0.630	0.029	0.84"
16'	14'	0.380	0.475	0.02	0.58"
20'	18'	0.308	0.375	0.014	0.39"
30'	25'	0.224	0.280	0.009	0.63"
40'	35'	0.160	0.200	0.004	0.28"
50'	45'	0.128	0.160	0.01	0.07"

Total Settlement:

5.29"

Order No. S-501/T-3189

Enclosure No. 11

REF CES

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Canada Department of Agriculture, Report No. 22,  
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- A.W. SKEMPTON: "The Colloidal 'Activity' of Clays"  
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4th International Conference on Soil Mechanics, 1957.
- L.G. SODERMAN, T.C. KENNEY & A.K. LOH: "Geotechnical Properties of  
Glacial Clays in Lake St. Clair Region of Ontario"  
Proceedings 14th Canadian Soil Mechanics Conference,  
October 1960.



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London  
August 15th, 1968.

ASSOCIATED COMPANY  
SOIL TESTING AND ENGINEERING LTD.  
35 BRENTFORD ROAD  
KINGSTON 3, JAMAICA  
WEST INDIES

Ref: 8-8-L8.

Nisbet Leckham Limited,  
Consulting Engineers,  
206 Water Street,  
Sarnia, Ontario.

Gentlemen:

Vidal Street Bridge,  
Pile Load Test.

The following paragraphs summarise the results of the pile load test carried out on pile No.5 of footing P1 on August 12 & 13, 1968.

METHOD.

The pile was loaded in 10 ton increments and each load was maintained on the pile until no settlement had taken place over a 2-hour interval. When the load reached 40 tons and no further settlement was observed, it was then removed and the rebound or elastic recovery of the pile was measured. The 40-ton load was then re-applied and further increments of 10 tons added until plastic failure was observed.

The load-settlement characteristics of the test are plotted graphically on Enclosure 1. of this report and form the basis for the conclusions which have been drawn.

RESULTS.

The load-settlement curve for the pile test shows that a distinct failure occurred between the 50 and 60 ton loads therefore for the purpose of analysis we have assumed that the ultimate capacity of the pile is 55 tons.

Based on an embedded pile length of 73 feet the adhesion developed by the pile is therefore 500 p.s.f. This value is equivalent to 25% of the average undrained shear strength of the soil in which the pile is embedded, and falls far short of the normal

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40 to 60% which can usually be mobilized.

#### CONCLUSIONS.

The low result obtained from the test can be attributed to two conditions:-

- 1) Insufficient time has elapsed between the driving and testing dates to allow the pile to develop the full adhesion.
- 2) The subsoil at this site is not capable of mobilizing the adhesion assumed in the original design.

Both of the conditions mentioned above probably apply to the test result therefore the effect of each will be considered separately.

First of all we anticipate that the adhesive strength could increase with time by 50%, which would result in an ultimate resistance of 750 p.s.f. Based on this value, a 50 ton capacity pile would require an embedment of 75 feet assuming a factor of safety of 2. We can therefore assume that the present 50 foot long piles will require to be extended to 75 feet to achieve the 50 ton working load, however further pile testing would be required to confirm this value.

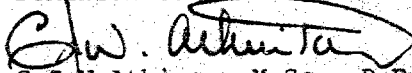
Due to this increase in pile length of 50 per cent to achieve the 50 ton working load, it now becomes more economical to consider end-bearing piles driven to bedrock which would require an increase in length of about 35 feet per pile. This would allow an increase in the working load to 75 tons per pile and consequently only two-thirds of the number of piles would be required. The net increase in total pile length is therefore less for the end-bearing than for the friction type of pile foundation.

Other factors favouring the end-bearing type of pile are that settlement will be restricted to elastic compression of the pile and bedrock, and also with the use of end-bearing piles driven to bedrock pile load testing is not considered necessary.

We trust that the above remarks explain the results of the pile load test and also enable the pile design to be finalised. If further explanation is necessary please do not hesitate to contact us.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED.

  
C.J.W. Atkinson M.Sc., P.Eng.  
Branch Manager.

CJWA.jb.

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