

OFFICE LOCATION -
DOWNSVIEW AVE.,
KEELE ST. - HIGHWAY 401
TORONTO, ONTARIO.



ONTARIO
DEPARTMENT OF HIGHWAYS

23-65-134
POSTAL ADDRESS -
DEPARTMENT OF HIGHWAYS
PARLIAMENT BUILDINGS,
TORONTO 5, ONTARIO.

Bridge Division,
December 1, 1961.

MEMORANDUM TO:

Mr. A. G. Stermac,
Principal Foundation Eng.,
Department of Highways,
Room 107, Lab. Bldg.,
DOWNSVIEW, Ontario.

RE: W.P. 275-60
Carp River Bridge
Hwy. 44 - Dist. #9

Enclosed find one print of the preliminary
plan for the subject structure.

The designer has chosen to use bearing piles
because of construction problems as the old struc-
ture is to be used to maintain traffic while a
section of the new structure is built.

Would you kindly let us have any comments you
might like to make.

A handwritten signature in dark ink, appearing to be 'J. B. Curtis'.

JBC/ea
cc. D. Smith

J. B. Curtis,
Bridge Location Engineer.

Mr. A. M. Teye,

August 31, 1961.

Bridge Engineer.

Materials & Research Section,

(Foundations Office).

Attention: Mr. Bruce Davis.

Re: Proposed Crossing - Hwy. 44 and Carp River,
Township of Huntley, District No. 9,
W.P. 275-60.

This is to confirm the telephone conversation of August 29th about the foundations of the above-mentioned bridge.

If steel 'H' end-bearing piles, driven down to bedrock are decided upon, 50 - 60 ton loads per pile can be used. On the other hand, if short friction piles, driven only 20 ft. into the ground are decided upon, a safe load of only 10 tons per pile can be used. This value has been arrived at by calculations using the available soil properties and assuming a factor of safety of 3. The calculated value is applicable to 12 BP 53 'H' piles or displacement piles of 12" diameter.

There is a strong possibility that higher safe load values could be obtained for the short friction piles, but this would have to be established by a pile loading test. It is recommended that such a test be decided upon. Three piles would have to be driven, two of which would be utilized as supports for the testing of the third middle one.

AGS/MdeF

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

cc: Foundations Office
Gen. Files.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

Short friction piles in clay:

a) 12" dia. tubular piles

$$Q_1 = \pi \times D \times L \times C_a + C_L \times C_a$$

$$= 3 \times 2000 \times \pi \times \frac{D^2}{4} + \pi \times D \times 20 \times 700$$

$$= \frac{9 \times 2000 \times 3.14 \times 1}{2000} + \frac{3.14 \times 1 \times 38 \times 700}{2000}$$

$$Q_1 = 7.2 + 9.2 = 28$$

$$P_a = \frac{Q_1}{F.S.} = \frac{28}{3} = \text{say } 10 \text{ tons/pile}$$

b) Using 12" dia. piles (12 BP 33) :

$$Q_2 = 3 \times 2000 \times \frac{16}{24} + \frac{30 \times 700 \times 4}{2000}$$

$$= 14.28 + 9.2 \text{ Tons}$$

$$P_a = \frac{Q_2}{F.S.} = \frac{23}{3} = \text{say } 10 \text{ Tons/4 pile}$$

SUMMARY OF RECOMMENDATIONS

1. Short friction piles in clay of approx. 16 ft dia. piles will provide a safe load of 10 tons/pile. 12" dia. tubular piles or 12 BP 33 (12 BP 33) can be used.
2. Higher values of pile bearing load can only be used if hardening of the pile loading is made in concrete or steel.
3. The design of bridge.

2. In view of the fact of the low bearing values for the
bottom pile, it is recommended that low bearing
pile should be used.

If the designer wishes to use spread footings the safe
bearing pressure will be ^{the same} as already indicated
in the previous letter.

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ONTARIO
DEPARTMENT OF HIGHWAYS

POSTAL ADDRESS -
DEPARTMENT OF HIGHWAYS
PARLIAMENT BUILDINGS,
TORONTO 5, ONTARIO.

Bridge Division,
August 23, 1961.

MEMORANDUM TO:

Mr. L. G. Soderman,
Principal Soils & Foundation Eng.,
Department of Highways,
Room 107,
Downsview, Ontario.

W. K. Selby.

RE: W.P. 275-60
Carp River Bridge
Hwy. 44 0.25 miles S. of
Hwy. 17 District 9

This will confirm our telephone conversation of the 23rd instant in which I advised you that we intend to construct a completely new structure at the above site and not a widening as was lately considered. The new line is approximately twelve feet west of the existing centre line and the grade is to be raised slightly more than two feet over the existing grade.

It is my understanding that the original soil report dated Jan. 11, 1961 and a subsequent memo dated April 11, 1961 are still applicable. Would you kindly review these and let us have a note for our records.

A handwritten signature in dark ink, appearing to read 'J. B. Curtis'.

JBC/et

J. B. Curtis,
Bridge Location Engineer.

Mr. S. McCombie,

April 11, 1961.

Bridge Planning Engr.

Materials & Research Section -

(Foundations Office)

Attention: Mr. John Curtis.

Re: Proposed Crossing Hwy. 44 and Carp River,
Twp. of Huntley, District No. 9,
W.P. 275-60.

We were advised that the above mentioned proposed structure will not be built, and that only a widening of the existing structure is contemplated. In connection with this widening, we have carried out settlement analyses in order to find out how the building of the new structure is going to affect the existing one. In our calculations, we have assumed that the road width is going to be 48 feet.

If the recommended safe bearing pressure of 1,600 lb./sq.ft. is being used, a settlement in the order of $3/4$ of an inch can be expected. Some additional settlement will also occur due to the load of the widened embankment. We are of the opinion that the resultant settlement will not exceed about one inch, and the new structure will therefore not affect the old one - i.e., no damage should be expected. However, some maintenance will be necessary during a certain period of time. We would suggest that the new structure - i.e., the addition - be built as a separate unit and not be doweled with the old one.

L. G. Soderman,
PRINCIPAL FOUNDATION ENGR.
Per:

A. G. Stermac

AGS/MdeF

(A. G. Stermac,
SUPERVISING FOUNDATION ENGR.)

cc: Foundations Office
Gen. Files.

Mr. A. M. Toye,
Bridge Engineer.

January 11, 1961.

Materials & Research Section.

FOUNDATION INVESTIGATION REPORT
by: H. C. Acres & Company, Limited.

Attention: Mr. S. McCombie.

Re: Proposed Crossing Hwy. 44 and Carp River,
Twp. of Huntley -- District No. 9
W.P. 275-60.

Attached to this memo, we are forwarding to you the above mentioned report submitted by the Consultant, H.C. Acres & Company, Ltd. We have reviewed the report and have found the factual data well presented. It seems to us that the necessity of scheduling the construction is not stressed enough in the report. If the approach fills are not built in advance and enough time allowed for the settlements to take place, it would not be possible to construct - i.e., place the abutment footings on the fill. One year is considered to be the desirable period of time that should be allowed between the construction of the embankments and the bridge structure.

We believe that the recommendations contained in the report and supplemented in this memo, will be adequate for your future work. However, should there be any other question you would like to discuss, please feel free to call on our Office.

AGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
J. Ford
L. E. Walker
J. E. Grusnier
A. Watt

Foundations Office
Gen. Files.

L. C. Soderman,
PRINCIPAL FOUNDATION ENGR.
Per:

Asterma
(A. G. Sternac,
FOUNDATION OFFICE ENGR.)

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT

on

FOUNDATION INVESTIGATION

PROPOSED CROSSING
HIGHWAY 44 AND CARP RIVER
TOWNSHIP OF HUNTLEY, DISTRICT NO. 9
WP 275-60

H.G. ACRES & COMPANY LIMITED
Consulting Engineers
Niagara Falls, Canada

November, 1960

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT

on

FOUNDATION INVESTIGATION

PROPOSED CROSSING
HIGHWAY 44 AND CARP RIVER
TOWNSHIP OF HUNTLEY, DISTRICT NO. 9
WP 275-60

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ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT

on

FOUNDATION INVESTIGATION

PROPOSED CROSSING
HIGHWAY 44 AND CARP RIVER
TOWNSHIP OF HUNTLEY, DISTRICT NO. 9
WP 275-60

Introduction

At the request of the Ontario Department of Highways, soil explorations were carried out by H.G. Acres & Company Limited at the above-mentioned site to determine the foundation conditions for the bridge and its approach embankments. A plan of the site is shown on Plate I.

The F.E. Johnston Drilling Company Limited performed the drilling and soil sampling operations, under the supervision of Mr. J.A. MacLeod of H.G. Acres & Company Limited. The field work commenced on November 8, 1960, and was completed on November 18, 1960. Laboratory testing of the soil samples was completed in November 1960.

The results of the field and laboratory work are presented in this report, together with our

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interpretations of these data and our recommendations for the foundation design.

Geology of the Site

The site of the proposed crossing is the flat bottom of a valley which extends away from the Ottawa River. The site is underlain by limestone which outcrops as the land rises gently to the southwest. The limestone is of Ordovician age and believed to be of the Ottawa formation (Trenton or Blackriver). About 900 feet north of this site an outcrop of igneous and metamorphic rock rises abruptly above the surrounding soils.

During the most recent glacial period, the bedrock surface was generally denuded and subsequently the low lying areas were covered with a variable thickness of till, which is composed mainly of sand and gravel. After the glaciers retreated, the area was inundated by the Champlain Sea and in this marine environment, the till was buried beneath deposits of fine sand, silt and clay. At this particular site these deposits are stratified to a high degree and are lacking in horizontal uniformity. Subsequent uplift has raised the soils considerably above sea level with the result that weathering and desiccation

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have developed a stiff crust, about ten feet thick, on the stratified silty clay marine deposit.

The Carp River, a meandering stream only about four feet deep, flows along the axis of the valley. Meander scars indicate that the stream has wandered in a wide belt across the valley. It is possible that the stream was previously much larger and may have effected the erosion of a significant depth of the marine deposits.

Exploratory Work

Two diamond drills were used in the exploration work. In the marine deposit, the wash boring method was employed and NX or BX casing was used to advance the hole. Two-inch diameter Shelby tube samples were taken at 6- or 12-foot intervals in all but the initial hole, No. 908-1. In this hole, 3-inch fixed-piston samples were taken at 5-foot intervals. These piston samples were obtained in an attempt to reduce sample disturbance, which experience has shown to be large in sensitive clays with the standard 2-inch Shelby tube samples. Where possible, in situ vane tests were performed 18 inches below the lower elevations of all tube samples, immediately after the samples were removed, or 18 inches below the bottom of the casing if a sample was not taken.

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When sand or gravel was encountered, standard penetration tests were performed and the split-spoon samples were retained. When rock was encountered, the NX or BX casing could not be advanced; under these circumstances, hole No. 908-1 was advanced by diamond drilling with AX rod 5.0 feet through sound limestone. In all holes except hole No. 908-5, the casing was advanced until rock was encountered. Hole No. 908-5 was stopped at a depth of approximately 73 feet.

A total of five holes were drilled and sampled, and vane tests and ground water observations were made in each of these.

The program of work is outlined in Appendix A.

Site Conditions and Soil Properties

The site investigated is in the flat bottom of a valley whose average ground surface elevation is approximately 300 feet. The land is poorly drained and used mostly for pasture.

The soils which were encountered in the exploratory holes are described in the attached drilling reports, Plates II to VI inclusive.

(a) - Existing Embankment - The existing Highway No. 44 consists of an embankment, about 6 feet high,

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which contains a heterogeneous mixture of sand and gravel. This embankment rests immediately upon the crust of the underlying silty clay deposit.

(b) - Silty Clay Crust - This is the stiff, weathered and desiccated surface layer of the marine deposit. The crust has a variable thickness as shown on Plate I, and the transition to the underlying deposit is a gradual one rather than an abrupt boundary which the single dotted line on Plate I might suggest.

Within this layer, the soil has the following average properties:

Liquid limit = 59.5 per cent
Plastic limit = 27.5 per cent
Water content = 36.8 per cent

Vane tests generally were not performed because this soil was too stiff to fail by this means. The natural undrained shear strength, determined from a laboratory compression test, was 2,100 psf at an elevation of 291.6 feet.

(c) - Silty Clay - This soil is not a homogeneous deposit, and it has been called silty clay only because this is the predominant grain size. At this particular site the deposit is extremely stratified and contains layers of fine sand, silt and clay. The approximate limits of this deposit are shown on Plate I. The

stratification, as observed in the tube samples, varied from horizontal to a slope as great as 20 degrees. Some 3-inch piston samples showed such a distortion of stratification that the only explanation, other than sample disturbance, must be that of slope failures during deposition.

On the basis of the samples tested, this deposit should not be assigned average properties. However, as shown on Plates VII and XI, there is a trend toward decreasing plasticity and water content with increasing depth. The natural undrained shear strength was measured by means of field vane tests and laboratory compression tests on samples trimmed from 3-inch piston samples. It was found that for samples that failed at strains less than 2 per cent, the results of the compression tests agree very closely with the results of the field vane tests. This is shown on Plate VII where all the test results for hole No. 908-1 are summarized. Because of this close correlation the reliability of the vane test results have been established for this particular deposit. On the basis of the vane tests, the natural undrained shear strength of this deposit can be taken as 900 psf from elevation 286 feet to elevation 260 feet, and at least 1,200 psf from elevation 260 feet to elevation 237 feet.

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The results of the laboratory tests are summarized in Appendix B and are shown graphically on Plates VII and XI. The results of the field vane tests are summarized in Appendix C and are shown graphically on Plates VII and XI.

The maximum sensitivity of this clay indicated by the field vane tests is 8.3, but experience has shown that sensitivities measured by this method are generally lower than those measured in the laboratory. Unfortunately, when remoulded in the laboratory, the soil, except in the crust, was too soft to test with the conventional equipment available, and, for this reason, the sensitivity could not be measured. Therefore, no remoulded tests were attempted, but rather the number of shocks in the liquid limit device at natural water content was recorded. These are all less than the liquid limit with a minimum value of one blow being obtained. It has been suggested that the sensitivity of this material exceeds 100⁽¹⁾.

(1) Eden, W.J. and Crawford, C.B. 1957. "Geotechnical Properties of Leda Clay in the Ottawa Area" Proc. 4th International Conference on Soil Mechanics, Vol. 1. p.p. 22-27.

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Consolidation tests were made on samples taken from the more uniform silty clay strata and the "p-e" curves are presented on Plates VIII to X. From these data, it may be deduced that this deposit has experienced overconsolidation even below the crust. The curves for the silty clay below the crust have the shape characteristic of a sensitive soil, and if the preconsolidation pressure is exceeded, this soil is very compressible. The apparent preconsolidation pressures have been estimated and the results are summarized on Plate XI. Sample disturbance reduces the apparent preconsolidation pressure, and if failure strain in the compression test can be taken as a measure of sample disturbance, the estimates of preconsolidation pressure are probably low.

(d) - Stratified Sand - This is an extremely stratified deposit and although the layers are predominantly sand, it also contains many layers of silt and some clay layers. The density of the sand as indicated by the number of blows in the standard penetration test, ranges from loose to medium. The approximate limits of this deposit are shown on Plate I.

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(e) - Sand and Gravel - This is a heterogeneous deposit of gravel with some sand and ranges from medium to dense. It constitutes only a thin layer overlying the bedrock surface as shown on Plate I. This is probably a glacial till.

(f) - Bedrock - The bedrock is a sound, horizontally bedded limestone probably belonging to the Ottawa formation.

(g) - Ground Water Conditions - The elevation of the free ground water surface in the silty clay and silty clay crust was observed in the bore holes after drilling was completed. The steady state elevations are recorded on the drilling reports. The average elevation for the holes near the Carp River is 299.8 feet. From these observations it can be seen that an excess hydrostatic pressure exists in the stratified sand overlying the bedrock surface, as all the elevations are above the surface of the Carp River. The elevation shown on Plate XI is the average elevation of the surface of the Carp River.

Design Considerations

(a) - Bearing Capacity -

Road Embankment - The critical condition for stability of an embankment founded on sensitive clays

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generally occurs during construction or shortly after completion of construction. Even though it is probable that some consolidation of the silty clay would take place during construction due to the presence of the sand layers, the $\phi = 0$ method of stability analysis is considered to be the most applicable in this case. In this method of analysis, stability is governed by the applied loads and by the stress strain and shear strength properties of the foundation and embankment soils.

The embankment for the approaches to the overpass structure will be approximately 31 feet high with respect to the original ground surface at station 476+00 and 22 feet high at the bridge. The crest width will be 52 feet, and the unit weight of the embankment material will be approximately 130 pcf. Using these loading requirements, the stability analyses indicate that the factor of safety will be greater than 1.36 for an embankment 31 feet high with 1.5 to 1 side slopes, and greater than 1.39 with 2 to 1 side slopes.

These factors of safety were obtained assuming the shear strength of the foundation material to be 900 psf throughout its depth and the shear strength of the embankment soil to be zero.

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The actual factors of safety are greater than these because of the greater shear strength of the crust and the silty clay below elevation 260 feet, but since a factor of safety of 1.36 is considered adequate for the stability of the embankment, a more refined analysis is not warranted.

Bridge Structure - With an embankment at the bridge only 22 feet high, a rigid frame might be considered the most suitable type of bridge structure. However, using a $\phi = 0$ analysis, the factor of safety against a stability failure of the end of the embankment is only 1.52, and whereas this would be sufficient in the case of embankment stability alone, it is not considered sufficient where a bridge structure is involved. The soil properties and details of the analyses are considered in some detail below and shown on Plate XII.

(i) - Shear Strength of the Silty Clay - The 3-inch fixed-piston samples were the best possible undisturbed samples that could be obtained for laboratory compression tests. Since the natural undrained shear strength measured in these samples agreed closely with the vane test results in the same hole, No. 908-1, the use of the vane test results without modification appears justified for this analysis.

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From the vane test results, a horizontal as well as vertical variation exists in the shear strength of the silty clay deposit. For the purpose of the stability analysis, average values must be used; therefore, the following are considered the best estimates of the shear strength profile for this stability analysis:

<u>Elevation</u>	<u>Shear Strength</u>
286 to 260	900
260 and below	1,200

In a marine deposit of such very high sensitivity, a shear failure would be expected at strains of less than one per cent with large strength reductions at larger strains. The minimum value measured in the compression tests was 1.25 per cent, but since even the best samples are subject to some disturbance, the in situ failure strain has been assumed to be 0.7 per cent.

(ii) - Shear Strength of the Crust - The natural undrained shear strength of the crust on the basis of the laboratory compression test is approximately 2,100 psf. However, experience suggests that this strength can be developed only at strains of about two per cent. Therefore, by the time this shear strength could be mobilized along the failure surface in the crust, the

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shear strength of the underlying sensitive clay would be reduced to much less than its maximum value. For a strain of 0.7 per cent along the failure surface, the strength that could be developed in the crust would be only about 1,300 psf.

(iii) - Shear Strength of the Embankment -

At 0.7 per cent strain, the shear strength developed in the compacted fill of the embankment will probably be very small and, therefore, it is assumed to be zero in the calculations.

(iv) - Method of Analysis and Factor of

Safety - The $\phi = 0$ method of analysis is not strictly applicable if the main embankment is placed a sufficient length of time prior to the construction of the bridge structure. However, it is not considered that the increased stability would be large enough to raise the factor of safety of 1.52 to an acceptable value. For this reason we believe that a rigid frame structure would not be suitable under these site conditions.

A suitable type of bridge structure would be a central span supported by piers and approach spans supported by abutments resting on the embankments.

The abutments do not decrease the overall stability of the embankment. The increase in foundation stresses due to the abutment load is offset by

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the greater bearing capacity at the end of the fill. However, the abutment loads might cause a critical condition with respect to the stability of the end slope within the embankment itself. This can only be determined, as a design consideration, when the in-place properties of the embankment materials are known.

Bridge Piers - One method of supporting the bridge would be with spread footings. The net bearing pressure for a shallow footing on clay is given by the following expression:

$$q_{net} = 5 \left(1 + 0.2 \frac{D}{B} \right) \left(1 + 0.2 \frac{B}{L} \right) S_u$$

where: q_{net} denotes the net bearing capacity of the foundation soil.

D denotes the depth of the base of the footing below the surface of the overburden.

B denotes the footing width.

L denotes the footing length.

S_u denotes the natural undrained shear strength of the foundation soil.

The base of the footings would be located at a minimum depth of approximately six feet below the

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original ground surface, and near the river where the crust is thin or even non-existent at this depth and where an average value of shear strength of 900 psf would be applicable. Since the depth of influence of a footing is approximately limited to its width, this shear strength will apply throughout the full depth of influence. In addition, to make allowance for the possibility of scour by the stream, the value of D has been assumed to be zero. Applying a factor of safety of 3, the allowable bearing pressures have been calculated, and are plotted in Plate XIII for various values of the ratio B/L .

This type of bridge would be approximately 32 feet wide, and would consist of a centre span 52 feet long with two approach spans approximately 40 feet long, if 2 to 1 end slopes are used. The total load on the centre piers will be approximately 650 kips. For a centrally loaded footing with a 30-degree skew, the dimension of the footing would be 36 feet by 11 feet. However, to take into account the effect of eccentric loading, the footing width must be increased so that the maximum stress beneath the footing, due to the eccentric load, does not exceed the value given on Plate XIII for the final B/L ratio.

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The alternative to spread footings for the support of the bridge piers is bearing piles driven to rock. Due to the high sensitivity of the silty clay, a non-displacement type pile, such as steel H-piles, would have to be used. However, the depth of overburden would probably make this type of foundation less attractive economically.

(b) - Settlement - The settlement of the embankment and of the end abutments of the bridge will be governed primarily by the embankment loading. The loading conditions which have been assumed are shown on Plate XIV. The consolidation characteristics of the clay are given on Plates VIII to X, and the apparent preconsolidation pressures are summarized on Plate XI. The value of the apparent modulus of elasticity which has been used to calculate the immediate settlements is 80 tons per square foot. The calculated settlements are shown on Plate XIV, and it must be noted that they are very large. The most important factors contributing to these large settlements are the apparent preconsolidation pressures and the steep initial portions of the "p-e" curves immediately beyond the points of preconsolidation. However, the samples were probably disturbed, and

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these disturbances, especially in sensitive clays, will cause reductions in the apparent preconsolidation pressures. Therefore, if the preconsolidation pressures were higher, the calculated settlements would be much lower. For this reason, the calculated settlements are considered to be too large, and the error in estimation may be appreciable.

As the deposit is extremely stratified and contains many sand layers throughout its depth, much of the settlement will probably occur during construction or will be complete within a few weeks thereafter. The stratification makes it impossible to estimate this time accurately. However, by observation of the settlement of the embankment after its completion, the final magnitude and time for complete consolidation can best be determined.

The footings which support the centre span of the bridge will only be subjected to small elastic and consolidation settlements, because the soil pressures beneath these footings will not exceed the preconsolidation pressures. These settlements will occur primarily during construction. Thus the worst condition which might result from settlement would be the differential settlement between the abutments

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and the piers. The most satisfactory method of reducing this differential settlement to an acceptable amount would be to schedule the construction program to allow time for the settlement of the embankment to take place before the bridge is erected. This time can best be determined by field observations.

Conclusions

(a) - From the drilling done at the site, the overburden was found to be approximately 80 feet thick. The soils are of marine origin and consist of stratified fine sand, silt and clay with the fine sand becoming more predominant with depth. There is a horizontal surface crust approximately 14 feet thick. Beneath this crust, the silty clay and other strata are extremely sensitive. Below a depth varying from 36 to 62 feet, the deposit consists of stratified fine sand, with occasional layers of silt and clay. The bedrock is sound, horizontally bedded, limestone.

(b) - The properties of the foundation soils are summarized on Plates VII to XI.

(c) - The 31-foot high embankment with a crest width of 52 feet, and with side and end slopes as steep as 1.5 to 1, can be safely supported on the undisturbed foundation soils. However, the design of

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the end slopes must also consider the stability within the embankment due to the abutment loads. The calculated settlements of the embankment are large, but the actual settlements will probably be much smaller.

(d) - The bridge can be supported on spread footings, and the footings should be placed at a depth of six feet below the original ground surface. The maximum net contact pressure at any point beneath the footings should not exceed the values obtained from Plate XIII.

Recommendations

The embankment should be constructed with side and end slopes not steeper than 1.5 to 1.

The bridge structure should be supported on spread footings with the base of the footings six feet below the original ground surface. The maximum soil pressure should not exceed the values obtained from Plate XIII.

APPENDIX AProgram of Work

November 8, 1960 - Diamond Drills Nos. 1 and 2 arrived at the site. Holes Nos. 908-1 and 908-4 were commenced.

November 10, 1960 - Hole No. 908-4 was completed. Hole No. 908-5 was commenced.

November 11, 1960 - Hole No. 908-1 was completed.

November 12, 1960 - Hole No. 908-5 was completed.

November 16, 1960 - Holes Nos. 908-2 and 908-3 were commenced.

November 17, 1960 - Hole No. 908-3 was completed.

November 18, 1960 - Hole No. 908-2 was completed.

Summary of Time

<u>Work Type</u>	<u>No. of Holes</u>	<u>Total Length Feet</u>	<u>Total Time Hours</u>
Modified Wash Boring	5	373.0	105.5
Diamond Drilling	1	5.0	4.0

APPENDIX BSummary of Laboratory Test Results

Hole No.	Sample No.	Elevation Feet	Water Content %	Liquid Limit %	Plastic Limit %	S _{un} Psf	e _f %	Remoulded Test *
908-1	1	291.6	36.8	59.5	27.5	2,100	8.00	-
	2	286.6	34.0	-	-	-	-	-
	3	281.6	57.2	38.3	20.3	713	4.50	B6
	4	276.6	41.0	23.0	13.8	534	8.50	B1
	5	271.6	47.5	36.3	20.2	1,435	1.25	B4
	6	266.6	52.3	36.4	19.6	804	2.50	B6
	7	261.6	35.7	-	-	-	-	-
	8	256.6	40.1	31.4	18.1	1,235	2.00	B8
	9	251.6	29.3	23.4	14.8	1,178	2.00	B7
	10	246.6	31.9	-	-	-	-	-
	11	241.6	33.6	-	-	-	-	-

e_f - Failure strain.

S_{un} - Natural undrained shear strength.

B6 - Number of shocks in liquid limit device.

* - Because most remoulded samples were too soft for a compression test, the number of shocks in the liquid limit device at natural water content is given.

Note: All laboratory compression tests were "Undrained Triaxial Compression" tests.

APPENDIX CSummary of Field Vane Test Results

Hole No.	Elevation Feet	Undrained Shear Strength Psf		Sensitivity
		Natural	Remoulded	
908-1	279.6	886	217	4.1
	274.6	790	170	4.7
	269.6	1,270	248	5.1
	264.6	775	140	5.5
	254.6	1,600	243	6.6
	249.6	2,320	496	4.7
	244.6	2,230	778	2.9
	239.6	2,080	418	5.0
908-2	282.2	868	186	4.7
	274.7	930	233	4.0
	270.2	1,023	124	8.3
	262.7	837	155	5.4
	258.2	1,581	372	4.3
	250.7	1,488	264	5.6
908-3	280.2	1,116	279	4.0
	272.7	1,054	248	4.3
	268.2	1,550	372	4.2
	252.7	1,581	279	5.7
	242.7	1,860	527	3.5

Appendix C - 2

Hole No.	Elevation Feet	Undrained Shear Strength Psf		
		Natural	Remoulded	Sensitivity
908-4	280.3	805	140	5.8
	272.8	1,610	264	6.1
	262.3	1,120	280	4.0
	254.8	1,550	294	5.3
	250.3	1,300	279	4.7
908-5	286.8	992	205	4.8
	279.3	1,680	240	7.0
	273.3	837	171	4.9
	268.8	1,580	310	5.1
	255.3	2,139	496	4.3

APPENDIX D

List of Plates

- Plate I - Exploratory Holes, Plan and Section.
- Plate II - Drilling Report, Hole No. 908-1.
- Plate III - Drilling Report, Hole No. 908-2.
- Plate IV - Drilling Report, Hole No. 908-3.
- Plate V - Drilling Report, Hole No. 908-4.
- Plate VI - Drilling Report, Hole No. 908-5.
- Plate VII - Summary of Drilling and Testing Results, Hole No. 908-1.
- Plate VIII - Consolidation Test Hole No. 908-1, Sample Elevation 281.6 Feet.
- Plate IX - Consolidation Test Hole No. 908-1, Sample Elevation 266.6 Feet.
- Plate X - Consolidation Test Hole No. 908-1, Sample Elevation 241.6 Feet.
- Plate XI - Summary of Drilling and Testing Results, Comparison of all Tests.
- Plate XII - Summary of the Analysis, Overall Stability of a Rigid Frame Structure.
- Plate XIII - Footing Design Chart.
- Plate XIV - Foundation Settlements Due to Embankment Load.

DRILLING REPORT

CLIENT Ontario Department of Highways JOB No. 908
 PROJECT WP 275-60 HOLE No. 908-1
 SITE Highway 44 and Carp River SHEET No. 1 OF 2
 CONTRACTOR: F.E. Johnston Drilling Company Limited STARTED 1:00 P.M. November 8, 19 60
 FINISHED 3:00 P.M. November 11, 19 60
 METHOD OF DRILLING: SOIL Modified Wash Boring CASING DIAM. NX
 ROCK Diamond Drill CORE DIAM. AI
 LOCATION: ~~Station~~ Chainage 473+92 ELEVATIONS: DATUM G.S.C.
 DEPARTURE 28 Feet right DRILL PLATFORM
 BEARING GROUND SURFACE 299.6
 INITIAL DIP 90 degrees ROCK SURFACE 221.1
 OTHER DIPS BOTTOM OF HOLE 216.1
 WATER TABLE 300.1

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST
			NO.	TYPE *	SIZE	DEPTH	RET'D	
<u>Feet</u>					<u>In.</u>	<u>Ft</u>	<u>In.</u>	<u>Blows *</u>
<u>0.0</u>	<u>Silty clay</u>	<u>Mottled, grey brown, weathered, very stiff to stiff</u>	<u>1</u>	<u>BO</u>	<u>2</u>	<u>7.0</u>		<u>Bar</u>
						<u>8.3</u>	<u>15</u>	<u>Pushed</u>
			<u>2</u>	<u>BO</u>	<u>2</u>	<u>12.0</u>		<u>Bar</u>
						<u>13.6</u>	<u>18</u>	<u>Pushed</u>
<u>14.0</u>	<u>Silty clay</u>	<u>Grey medium to stiff containing many ss and silt layers</u>	<u>3</u>	<u>CO</u>	<u>3</u>	<u>17.0</u>		
						<u>18.4</u>	<u>17</u>	<u>Pushed</u>
					<u>Vane test</u>	<u>19.9</u>		
			<u>4</u>	<u>CO</u>	<u>3</u>	<u>22.0</u>		
						<u>23.4</u>	<u>17</u>	<u>Pushed</u>
					<u>Vane test</u>	<u>24.9</u>		
			<u>5</u>	<u>CO</u>	<u>3</u>	<u>27.0</u>		
						<u>28.4</u>	<u>17</u>	<u>Pushed</u>
					<u>Vane test</u>	<u>29.9</u>		
			<u>6</u>	<u>CO</u>	<u>3</u>	<u>32.0</u>		
						<u>33.4</u>	<u>17</u>	<u>Pushed</u>
					<u>Vane test</u>	<u>34.9</u>		

SAMPLING METHOD

* A - SPLIT TUBE
 B - THIN WALL TUBE
 C - PISTON SAMPLER
 D - CORE BARREL

E - AUGER
 F - WASH

SHIPPING CONTAINER

N - INSERT
 O - TUBE
 P - WATER CONTENT TIN
 Q - GLASS JAR

R - CLOTH BAG
 S - PLIOFILM BAG
 Z - DISCARDED

INSPECTOR J. Bateson

LOGGED BY J. MacLeod

APPROVED

DATE

November, 1960

D. H. MacDonald

DRILLING REPORT

CLIENT Ontario Department of Highways
 PROJECT WP 275-60
 SITE Highway 44 and Carp River

JOB No. 908
 HOLE No. 908-1
 SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTION, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST
			NO.	TYPE	SIZE	DEPTH	RET'D	Blows *
Feet					In.	Ft	In.	
			7	CO	3	37.0		
						38.4	17	Pushed
			8	CO	3	42.0		
						43.4	17	Pushed
				Vane test		44.9		
		* Penetration Test	9	CO	3	47.0		
		The value given is the number of blows of a 140-pound weight falling freely 30 inches required to advance the standard split-spoon sampler 6 inches to the depth indicated.				48.4	17	Pushed
				Vane test		49.9		
			10	CO	3	52.0		
						53.4	17	Pushed
				Vane test		54.9		
			11	CO	3	57.0		
						58.4	17	Pushed
				Vane test		59.9		
62.0	Sand	Fine, loose to medium density containing silt and clay layers		CZ		62.0		
						63.0	N11	Pushed
				AZ		63.0		
						63.5		11
						64.0		3
						64.5	N11	5
			12	AQ		67.0		
						67.5		1
						68.0		2
				With trap		68.5	6	4
77.0	Gravel	Grey dense						
78.5	Rock	Bedrock or boulder	13	DZ		78.5		
						83.5	100%	
83.5		End of hole						

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 908

PROJECT WP 275-60

HOLE No. 908-2

SITE Highway 44 and Carp River

SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling
 Company Limited

STARTED 10:30 A.M. November 16, 1960
 FINISHED M. November 18, 1960

METHOD SOIL Modified Wash Boring

CASING DIAM. BX

OF
 DRILLING: ROCK Diamond Drill

CORE DIAM.

LOCATION: ~~EAST~~ Chainage 473+24

ELEVATIONS: DATUM G.S.C.

DEPARTURE 26 feet right

DRILL PLATFORM

BEARING

GROUND SURFACE 301.7

INITIAL DIP 90 degrees

ROCK SURFACE 229.7

OTHER DIPS

BOTTOM OF HOLE 229.7

WATER TABLE 300.0

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC	SAMPLE					PENETRATION TEST
			NO	TYPE *	SIZE	DEPTH	RET'D	
Feet					In.	Ft	In.	Blows *
0.0	Silty clay	Mottled, grey brown, weathered, very stiff to stiff, stratified	1	BO	2	6.0		Bar
						7.5	6	Pushed
			2	BO	2	12.0		Bar
						13.5		Pushed
14.0	Silty clay	Grey, medium to stiff, stratified, containing many silt and sand layers			Vane test	19.5		
			3	BO	2	24.0		
						25.5	8	Pushed
					Vane test	27.0		
					Vane test	31.5		
			4	BO	2	36.0		
						37.5	18	Pushed
					Vane test	39.0		
					Vane test	43.5		
			5	BO	2	48.5		
						49.5	18	Pushed
					Vane test	51.0		

SAMPLING METHOD

* A - SPLIT TUBE
 B - THIN WALL TUBE
 C - PISTON SAMPLER
 D - CORE BARREL

E - AUGER
 F - WASH

SHIPPING CONTAINER

N - INSERT
 O - TUBE
 P - WATER CONTENT TIN
 Q - GLASS JAR

R - CLOTH BAG
 S - PLIOFILM BAG
 Z - DISCARDED

INSPECTOR J. MacLeod

LOGGED BY J. MacLeod

APPROVED

D. H. MacDonald

DATE

November, 1960

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 908

PROJECT WP 275-60

HOLE No. 908-2

SITE Highway 44 and Carp River

SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST
			NO	TYPE	SIZE	DEPTH	RET'D	
Feet					In.	Ft	In.	Blows *
52.0	Silty sand	Fine loose, stratified containing many silt and clay layers	6	BO	2	54.0		
						55.5	18	Pushed
			1	AQ	2	66.0		
						66.5		1
						67.0		2
						67.5	18	1
71.0	Gravel	Grey, dense						
72.0	Rock	Bedrock or boulder						
		End of hole						
		* Penetration Test						
		The value given is the number of blows of a 140-pound weight falling freely 30 inches required to advance the standard split-spoon sampler 6 inches to the depth indicated.						

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 908

PROJECT WP 275-60

HOLE No. 908-3

SITE Highway 44 and Carp River

SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling
Company Limited

STARTED 10:30 A.M. November 16, 1960

FINISHED 5:00 P.M. November 17, 1960

METHOD OF SOIL Modified Wash Boring

CASING DIAM. BX

DRILLING: ROCK Diamond Drill

CORE DIAM.

LOCATION: ~~Station~~ Chainage 473+04

ELEVATIONS: DATUM G.S.C.

DEPARTURE 28 feet left

DRILL PLATFORM

BEARING

GROUND SURFACE 299.7

INITIAL DIP 90 degrees

ROCK SURFACE 228.7

OTHER DIPS

BOTTOM OF HOLE 228.7

WATER TABLE 299.0

DEPTH	SOIL TYPE	DESCRIPTION, COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST
			NO.	TYPE *	SIZE	DEPTH	RET'D	
Feet					In.	Ft	In.	Blows *
0.0	Silty clay	Mottled, grey brown, weathered, very stiff to stiff	1	BO	2	6.0		Bar
						7.5	18	Pushed
			2	BO	2	12.0		Bar
						13.5	18	Pushed
16.0	Silty clay	Grey, medium to stiff, stratified, containing many sand and silt layers			Vane test	19.5		
			3	BO	2	24.0		
						25.5	18	Pushed
					Vane test	27.0		
					Vane test	31.5		
36.0	Sand	Fine, loose, stratified containing silt and clay layers		BZ	2	36.0		
						37.5	2	Pushed
			4	BO	2	38.0		
						39.5	18	Pushed
			5	BO	2	44.0		
						45.5	18	Pushed
					Vane test	47.0		

SAMPLING METHOD

* A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARRELE — AUGER
F — WASH

SHIPPING CONTAINER

N — INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JARR — CLOTH BAG
S — PLIOFILM BAG
Z — DISCARDED

INSPECTOR J. MacLeod

APPROVED

A. H. MacDonald.

LOGGED BY J. MacLeod

DATE

November, 1960

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 908

PROJECT WP 275-60

HOLE No. 908-3

SITE Highway 44 and Carp River

SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTION, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST
			NO.	TYPE	SIZE	DEPTH	RET'D	
Feet					In.	Ft	In.	Blows *
			6	BO	2	50.0		
						51.5	18	Pushed
				Vane test		57.0		
			7	AQ	2	62.0		
						62.5		1
						63.0		2
						63.5	18	1
70.0	Gravel	Gray, dense						
71.0	Rock	Bedrock or boulder						
		End of hole						
		* Penetration Test						
		The value given is the						
		number of blows of a 140-						
		pound weight falling						
		freely 30 inches required						
		to advance the standard						
		split-spoon sampler 6						
		inches to the depth in-						
		dicated.						

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 908

PROJECT WP 275-60

HOLE No. 908-4

SITE Highway 44 and Carp River

SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling
 Company Limited

STARTED 1:00 P.M. November 8, 1960
 FINISHED 2:00 P.M. November 10, 1960

METHOD OF SOIL Modified Wash Boring

CASING DIAM. BX

DRILLING: ROCK Diamond Drill

CORE DIAM.

LOCATION: ~~latitude~~ Chainage 473+69

ELEVATIONS: DATUM G.S.C.

DEPARTURE 26 feet left

DRILL PLATFORM

BEARING

GROUND SURFACE 299.8

INITIAL DIP 90 degrees

ROCK SURFACE 221.8

OTHER DIPS

BOTTOM OF HOLE 221.8

WATER TABLE 300.0

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC	SAMPLE					PENETRATION TEST
			NO	TYPE *	SIZE	DEPTH	RET'D	
Feet					In.	Ft	In.	Blows *
0.0	Silty clay	Mottled, grey brown, weathered, very stiff to stiff, stratified	1	BO	2	6.0		Bar
						7.5	18	Pushed
			2	BO	2	12.0		
13.0	Silty clay	Grey, medium to stiff stratified containing many sand and silt layers				13.5	18	Pushed
					Vane test	19.5		
			3	BO	2	24.0		
						25.5	18	Pushed
					Vane test	27.0		
			4	BO	2	30.0		
						31.5	18	Pushed
					Vane test	37.5		
			5	BO	2	42.0		
						43.5	18	Pushed
					Vane test	45.0		
					Vane test	49.5		
50.0	Sand	Grey fine loose, stratified						

SAMPLING METHOD

* A — SPLIT TUBE
 B — THIN WALL TUBE
 C — PISTON SAMPLER
 D — CORE BARREL

E — AUGER
 F — WASH

SHIPPING CONTAINER

N — INSERT
 O — TUBE
 P — WATER CONTENT TIN
 Q — GLASS JAR

R — CLOTH BAG
 S — PLIOFILM BAG
 Z — DISCARDED

INSPECTOR J. MacLeod

APPROVED

D. H. MacDonald

LOGGED BY J. MacLeod

DATE

November, 1960

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 908

PROJECT WP 275-60

HOLE No. 908-4

SITE Highway 44 and Carp River

SHEET No. 2 OF 2

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST
			NO.	TYPE	SIZE In.	DEPTH Ft	RET'D In.	Blows *
			6	AQ	2	54.0		
						54.5		3
						55.0		2
						55.5	18	1
			7	AQ	2	60.0		
						61.5	18	Pushed
			8	AQ	2	66.0		
						66.5		2
						67.0		3
						67.5	18	1
76.0	Sandy gravel	Grey, dense	9	AQ	2	76.0		
						76.5		23
						77.0		30
						77.5	18	40
78.0	Rock	Bedrock or boulder End of hole						
		* Penetration Test						
		The value given is the number of blows of a 140-pound weight falling freely 30 inches required to advance the standard split-spoon sampler 6 inches to the depth indicated.						

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 908

PROJECT WP 275-60

HOLE No. 908-5

SITE Highway 44 and Carp River

SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling
 Company Limited

STARTED 4:00 P.M. November 10, 1960

FINISHED 3:00 P.M. November 12, 1960

METHOD SOIL Modified Wash Boring

CASING DIAM.

OF
 DRILLING: ROCK Diamond Drill

CORE DIAM.

LOCATION: ~~LATITUDE~~ Chainage 475+42

ELEVATIONS: DATUM G.S.C.

DEPARTURE 20 feet left

DRILL PLATFORM

BEARING

GROUND SURFACE 306.3

INITIAL DIP 90 degrees

ROCK SURFACE

OTHER DIPS

BOTTOM OF HOLE 232.8

WATER TABLE 303.2

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST
			NO	TYPE *	SIZE	DEPTH	RET'D	
Feet					In.	Ft	In.	Blows *
0.0	Gravel	Embankment of existing highway	1	AQ	2	6.0		
						6.5		2
						7.0		2
						7.5	18	1
8.0	Silty clay	Mottled, grey brown, weathered, stiff stratified	2	BO	2	12.0		Bar
						13.5	12	Pushed
15.0	Silty clay	Grey medium to stiff, stratified, containing many thin sand and silt layers		Vane test		19.5		
			3	BO	2	24.0		
						25.5	11	Pushed
				Vane test		27.0		
			4	BO	2	30.0		
						31.5	18	Pushed
				Vane test		33.0		
				Vane test		37.5		
42.0	Sand	Grey, fine, loose to medium density stratified containing silt and clay layers	5	BO	2	42.0		
						43.5	18	Pushed

SAMPLING METHOD

* A — SPLIT TUBE
 B — THIN WALL TUBE
 C — PISTON SAMPLER
 D — CORE BARREL

E — AUGER
 F — WASH

SHIPPING CONTAINER

N — INSERT
 O — TUBE
 P — WATER CONTENT TIN
 Q — GLASS JAR

R — CLOTH BAG
 S — PLIOFILM BAG
 Z — DISCARDED

INSPECTOR J. MacLeod

APPROVED

D. H. Macdonald.

LOGGED BY

J. MacLeod

DATE

November, 1960

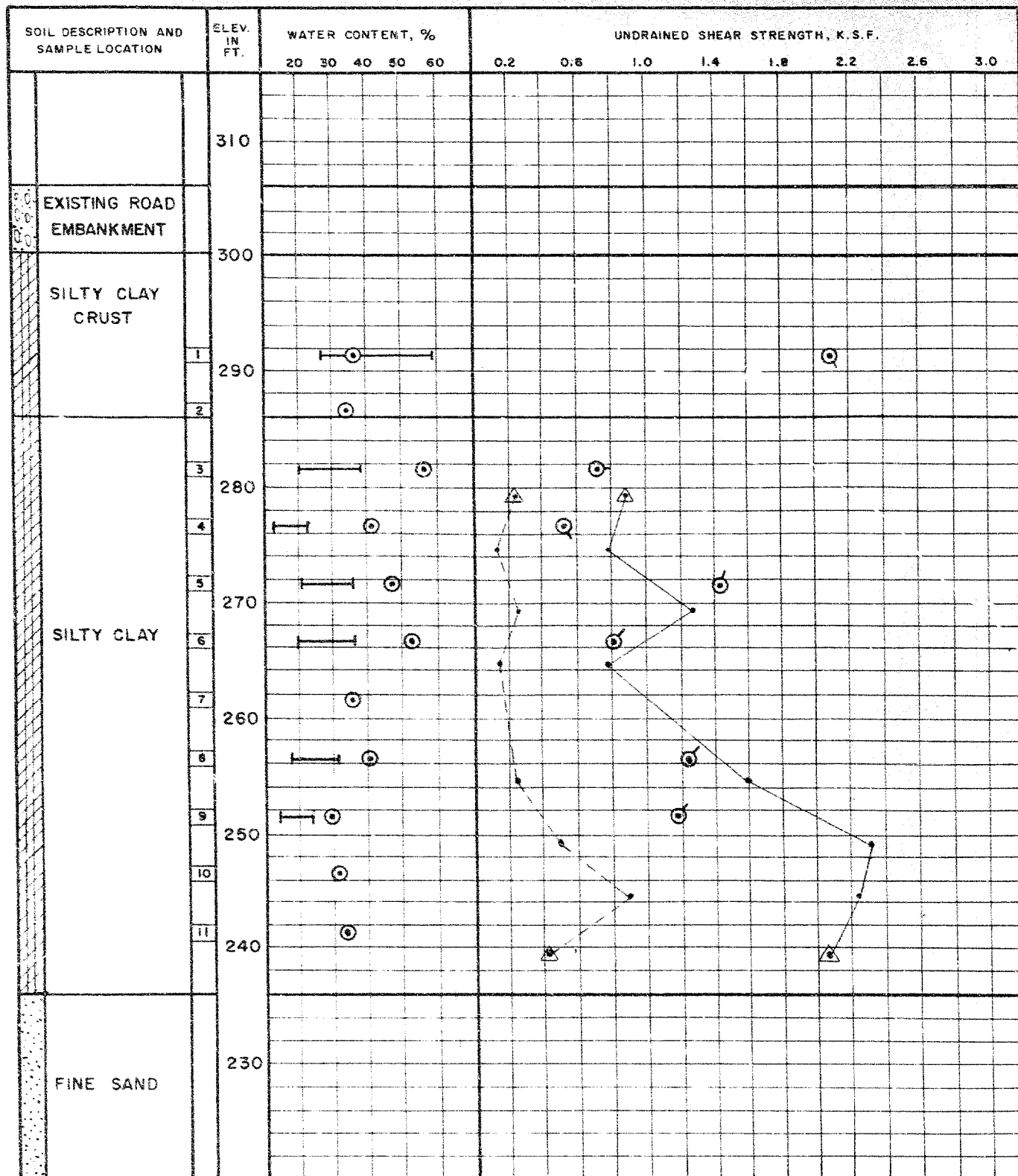
H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
 NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Ontario Department of Highways
 PROJECT WP 275-60
 SITE Highway 44 and Carp River

JOB No. 908
 HOLE No. 908-5
 SHEET No. 2 OF 2

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTION, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST
			NO	TYPE	SIZE In.	DEPTH Ft	RET'D In.	Blows *
			6	BO	2	48.0 49.5	18	Pushed
				Vane test		51.0		
			7	BO	2	54.0 55.5	18	Pushed
			8	BO	2	60.0 61.5	18	Pushed
			9	BO	2	66.0 67.5	15	Pushed
			10	AQ	2	72.0 72.5 73.0		12 27
73.5		End of hole				73.5	18	31
* Penetration Test The value given is the number of blows of a 140-pound weight falling freely 30 inches required to advance the standard split-spoon sampler 6 inches to the depth indicated.								



3 SOIL SAMPLE
 ○ NATURAL WATER CONTENT
 — LIQUID LIMIT
 — PLASTIC LIMIT

○ UNDRAINED COMPRESSION TEST
 △ FIELD VANE TEST
 — NATURAL STRENGTH
 --- REMOULDED STRENGTH

15 — 0 — 5
 10
 FAILURE STRAIN

H. G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

SUMMARY OF DRILLING AND TEST RESULTS

HOLE 908 - 1

ONTARIO DEPARTMENT OF HIGHWAYS

APPROVED

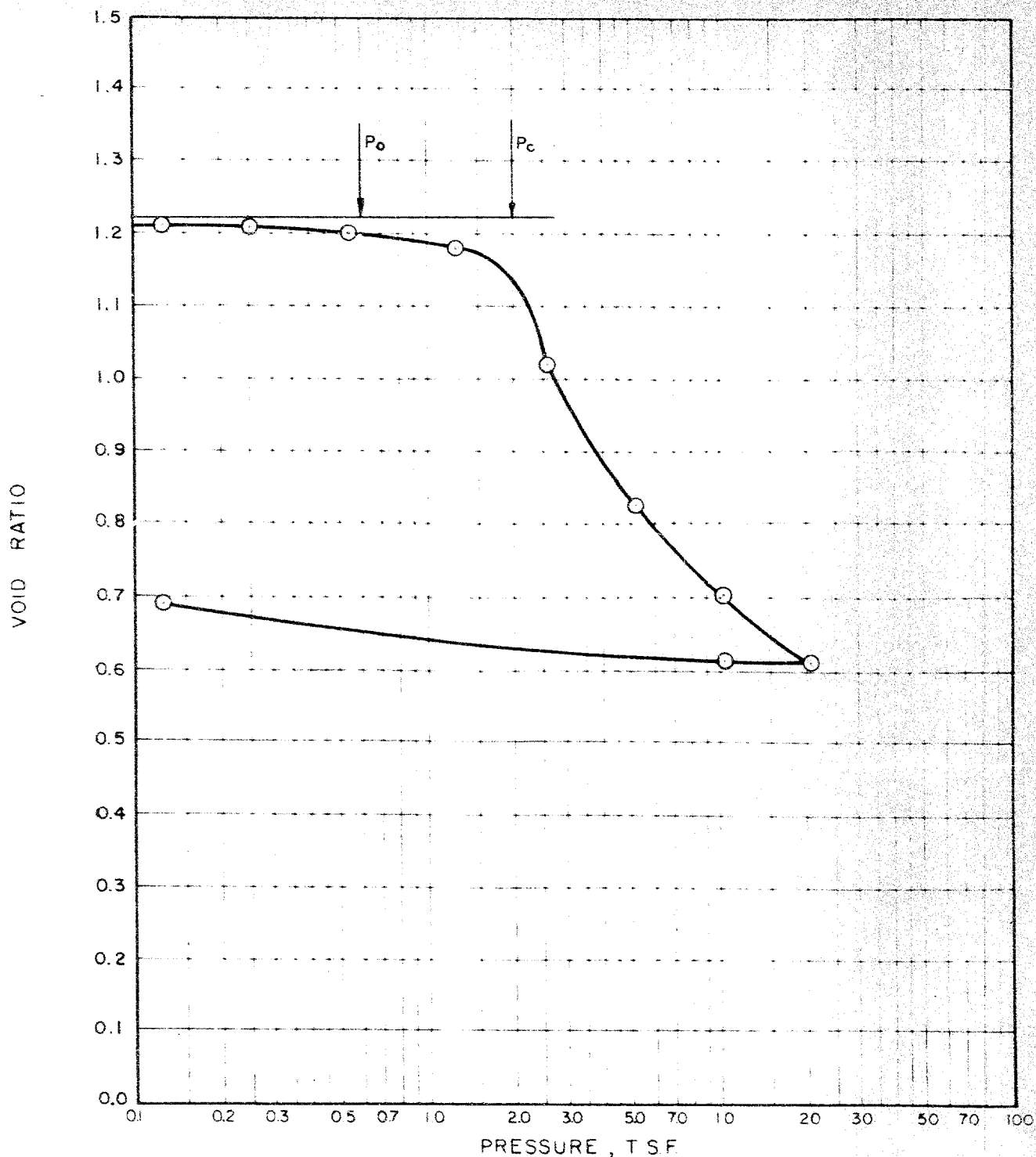
DATE NOVEMBER 1960

WP - 275 - 60

H. G. ACRES & COMPANY LTD.

JOB No. 908

PLATE VII



OVERBURDEN PRESSURE — $P_0 = 0.59$ TSF
 CONSOLIDATION PRESSURE — $P_c = 2.00$ TSF

NATURAL WATER CONTENT 44.2%
 LOADING INTERVAL 100% PRIMARY CONSOLIDATION

SAMPLE No 908-CO-3
 TEST No 908-9-1

TEST DATE NOVEMBER 15, 1960
 TESTED BY B.H.

H. G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

CONSOLIDATION TEST

HOLE No 908-1 SAMPLE ELEV 281.6'

ONTARIO DEPARTMENT OF HIGHWAYS

APPROVED

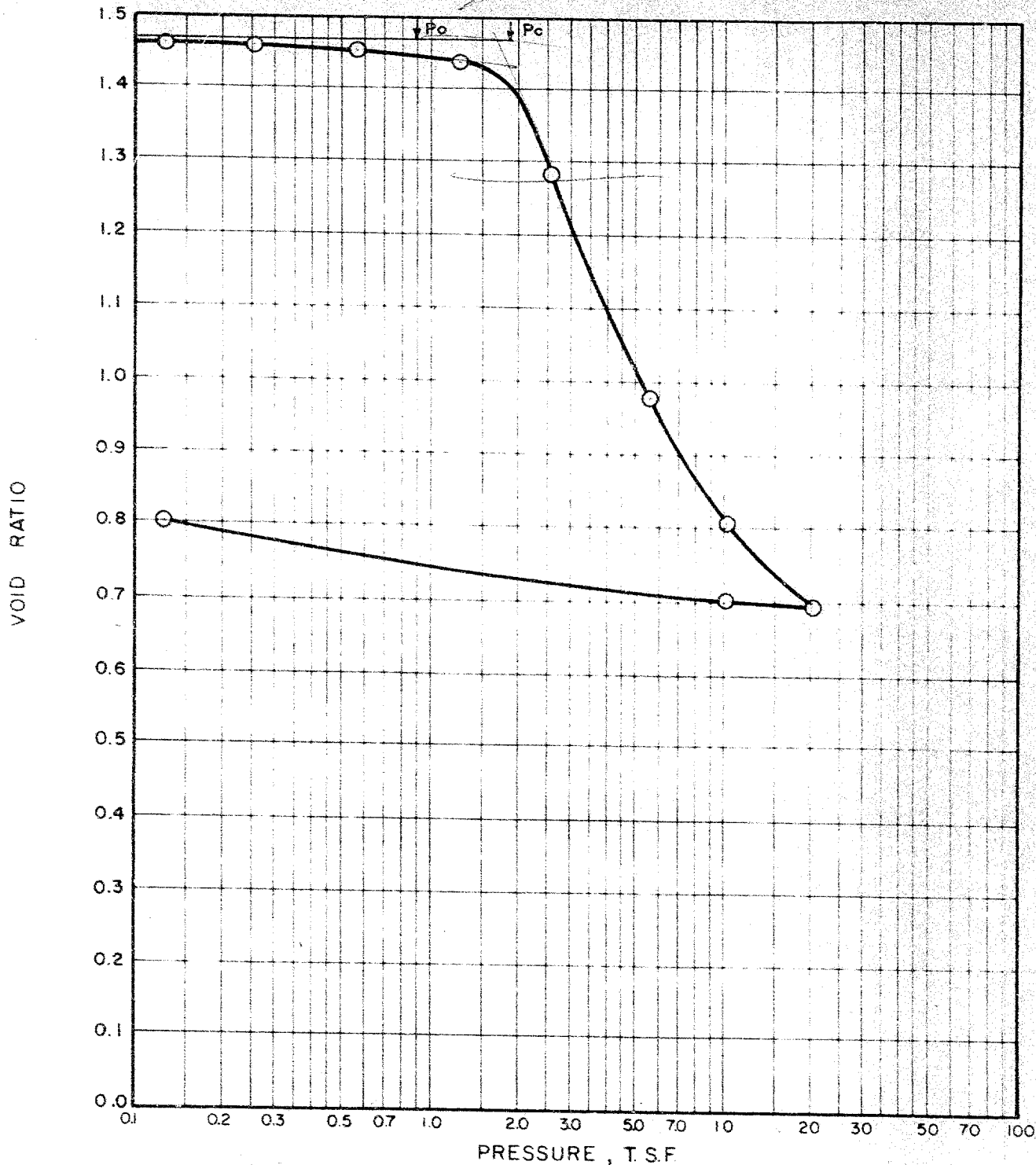
DATE NOVEMBER 1960

WP-275-60

B. H. Buckland
 H. G. ACRES & COMPANY LTD

JOB No 908

PLATE VIII



OVERBURDEN PRESSURE - $P_0 = 0.94$ T.S.F.
 CONSOLIDATION PRESSURE - $P_c = 1.85$ T.S.F.

NATURAL WATER CONTENT 53.0 %
 LOADING INTERVAL 100% PRIMARY CONSOLIDATION

SAMPLE No. 908-CO-6
 TEST No 908-9-2

TEST DATE NOVEMBER 16, 1960
 TESTED BY R.L.

H. G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

CONSOLIDATION TEST

HOLE No. 908-1 SAMPLE ELEV. 266.6'

ONTARIO DEPARTMENT OF HIGHWAYS

APPROVED

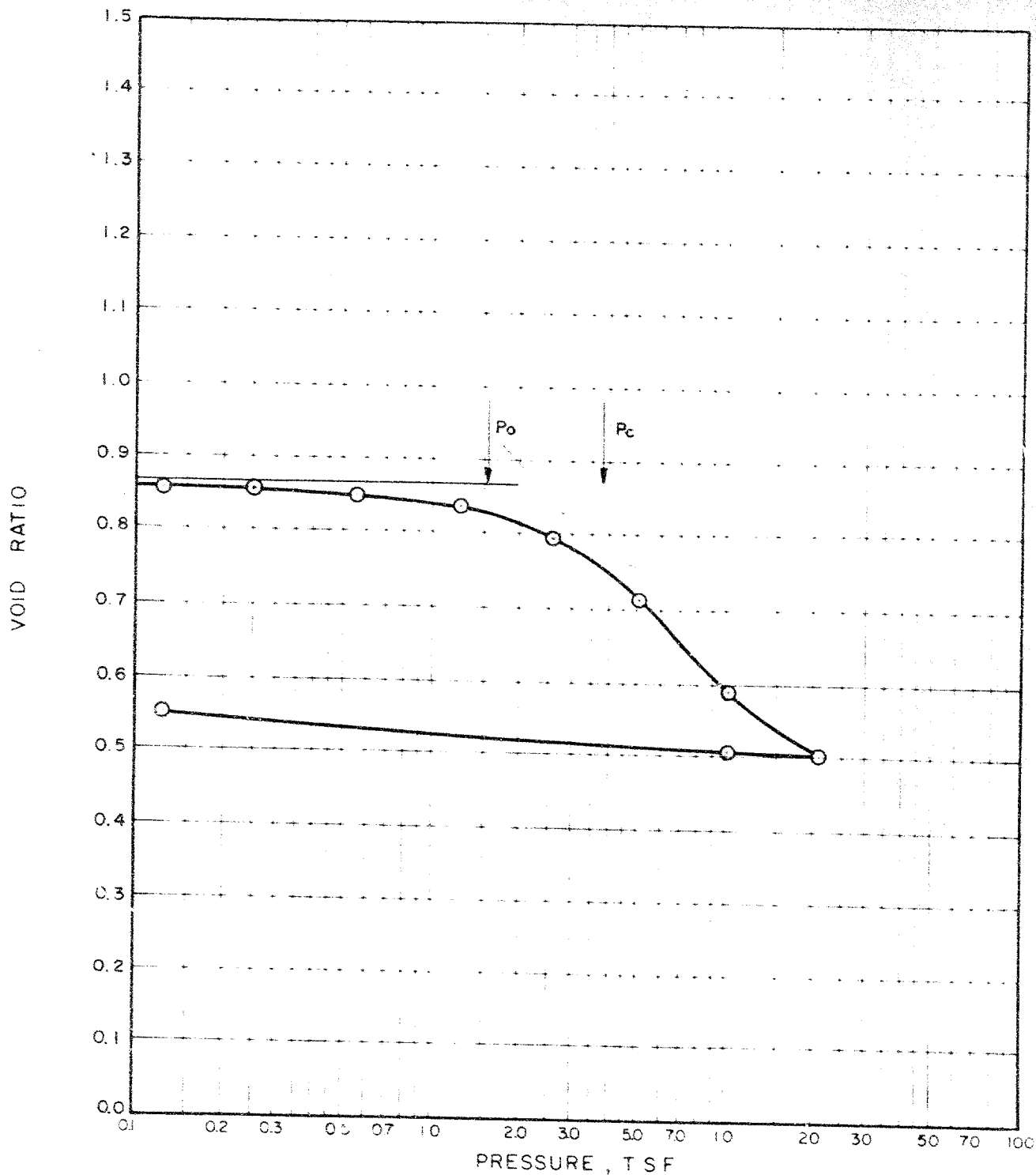
DATENOVEMBER 1960

WP - 275 - 60

HG ACRES & COMPANY LTD.

JOB No. 908

PLATE IX



OVERBURDEN PRESSURE — $P_0 = 1.55$ TSF
 CONSOLIDATION PRESSURE — $P_c = 3.75$ TSF

NATURAL WATER CONTENT 31.4%
 LOADING INTERVAL 100% PRIMARY CONSOLIDATION

SAMPLE No 908-CO-II

TEST DATE NOVEMBER 23, 1960

TEST No 908-9-3

TESTED BY R.G.

H. G. ACRES & COMPANY LIMITED
 CONSULTING ENGINEERS
 NIAGARA FALLS CANADA

CONSOLIDATION TEST

ONTARIO DEPARTMENT OF HIGHWAYS

HOLE No 908-1 SAMPLE ELEV 241.6'

APPROVED

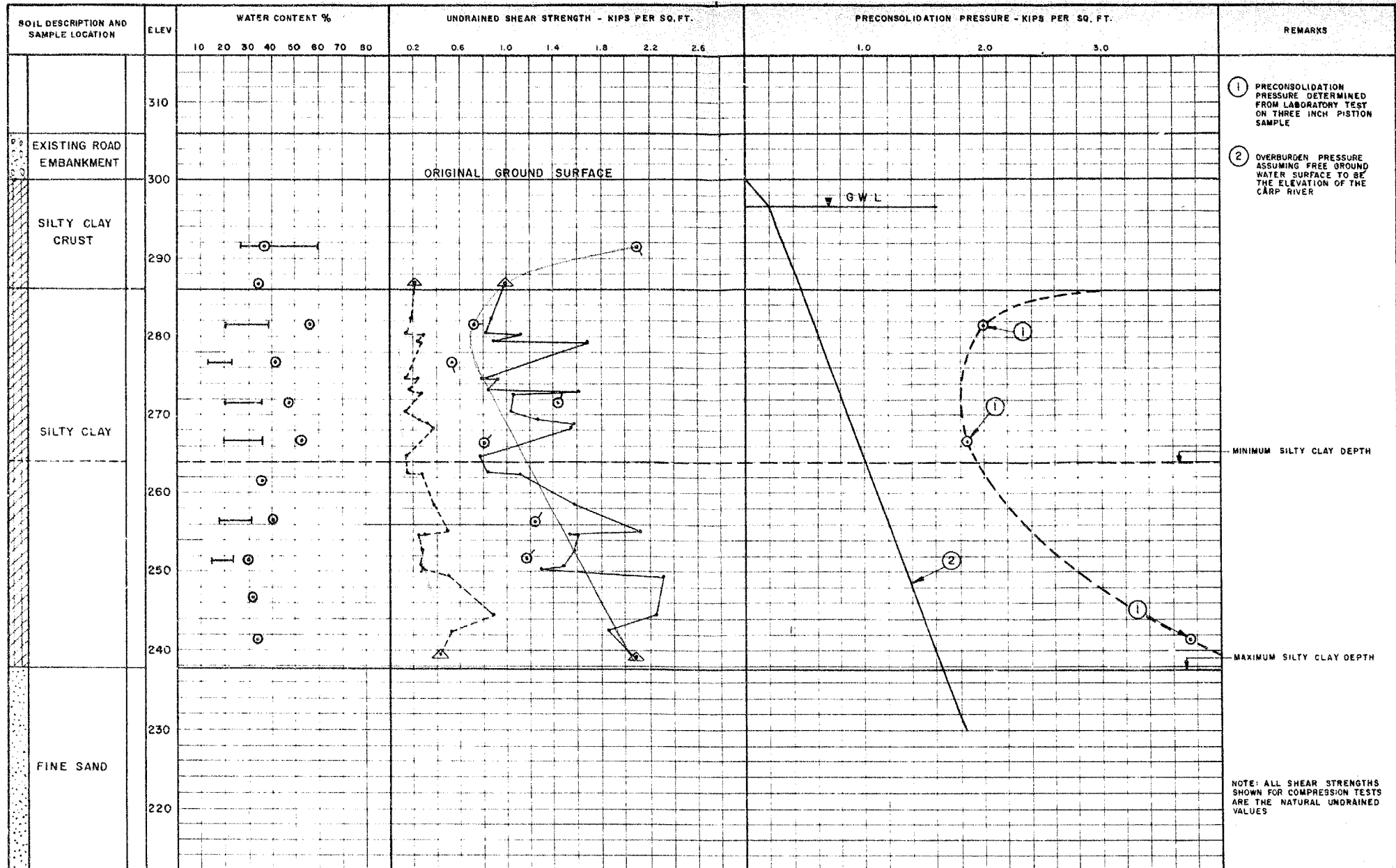
DATE NOVEMBER 1960

WP-275-60

H. G. ACRES & COMPANY LTD

JOB No 908

PLATE X



③ SOIL SAMPLE

⊙ NATURAL WATER CONTENT

— LIQUID LIMIT

— PLASTIC LIMIT

⊙ UNDRAINED COMPRESSION TEST

△ FIELD VANE TEST

— NATURAL STRENGTH

--- REMOULDED STRENGTH

0
15 — 5
10
FAILURE STRAIN

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ONTARIO DEPARTMENT OF HIGHWAYS

W.P. 275 - 60

SUMMARY OF DRILLING AND TEST
RESULTS

COMPARISON OF ALL TESTS

APPROVED

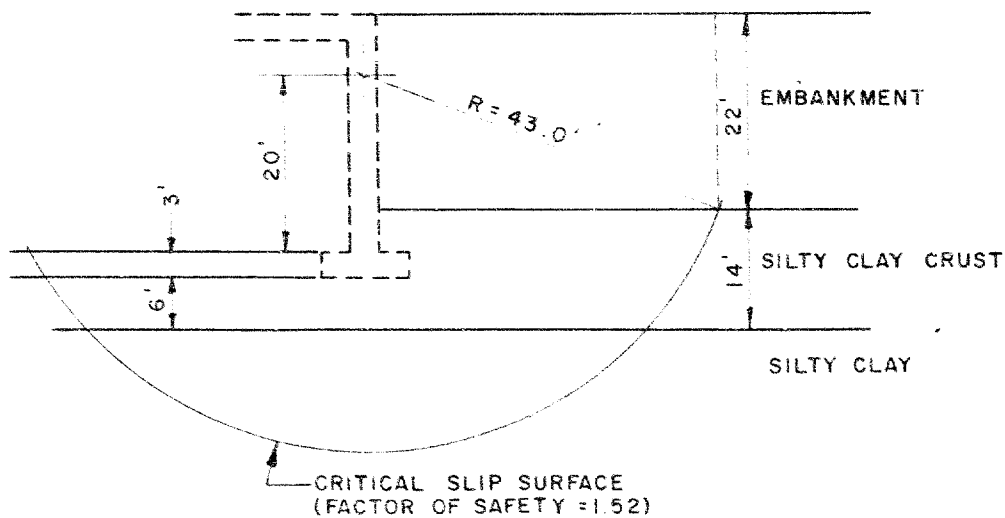
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JOB No. 908

PLATE - XI

NOTE: ALL SHEAR STRENGTHS
SHOWN FOR COMPRESSION TESTS
ARE THE NATURAL UNDRAINED
VALUES



SHEAR STRENGTHS

EMBANKMENT $\phi = 0^\circ$
 $C = 0$ P.S.F.

SILTY CLAY CRUST $\phi = 0^\circ$
 $C = 1300$ P.S.F.

SILTY CLAY $\phi = 0^\circ$
 $C = 900$ P.S.F.

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SUMMARY OF ANALYSIS
 OVERALL STABILITY
 OF A RIGID FRAME STRUCTURE

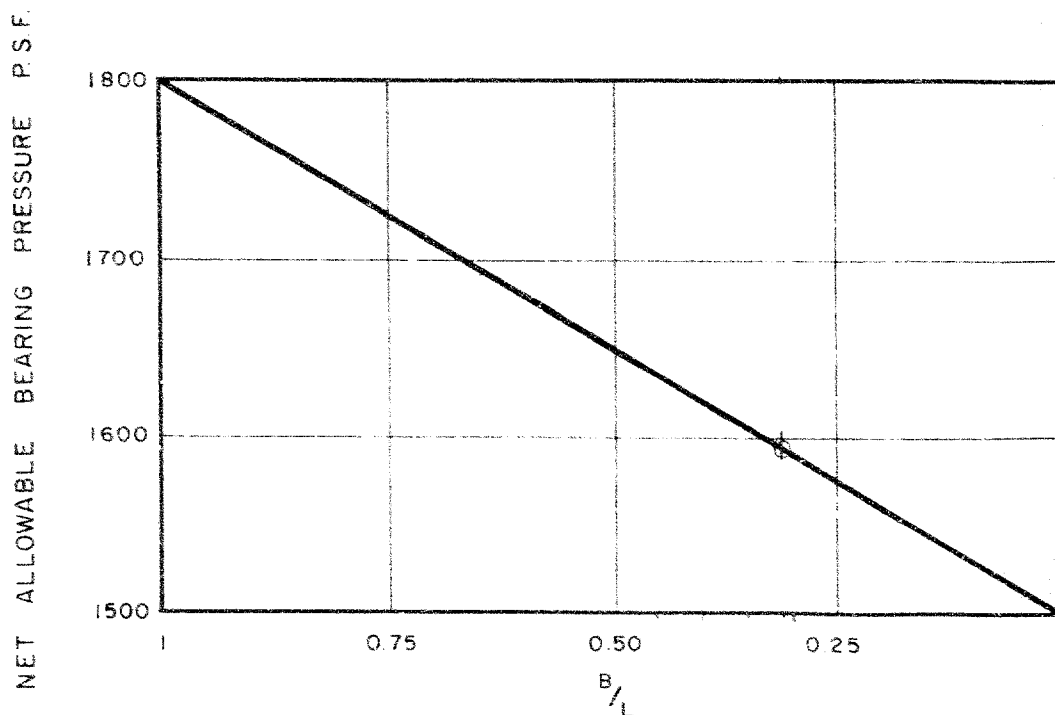
APPROVED

DATE NOVEMBER, 1960

D. H. MacDonald
 H. G. ACRES & COMPANY LIMITED

SCALE JOB No.
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PLATE XII



NOTE

B DENOTES FOOTING WIDTH

L DENOTES FOOTING LENGTH

FACTOR OF SAFETY FOR THIS CHART IS 3

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FOOTING DESIGN CHART

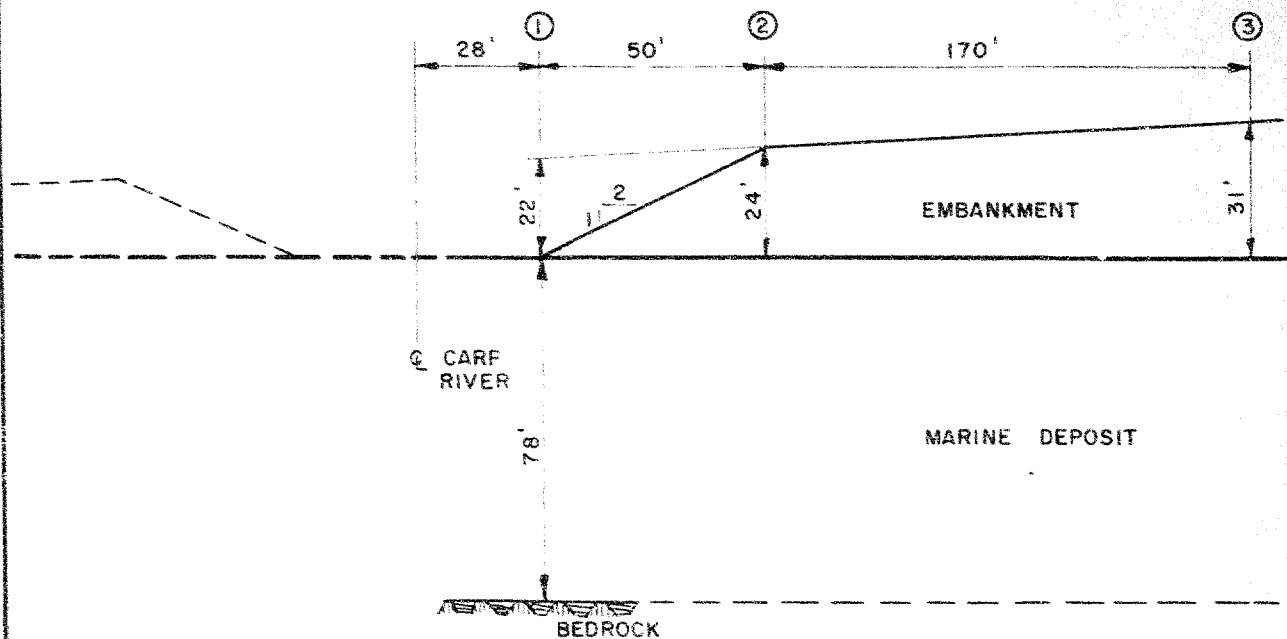
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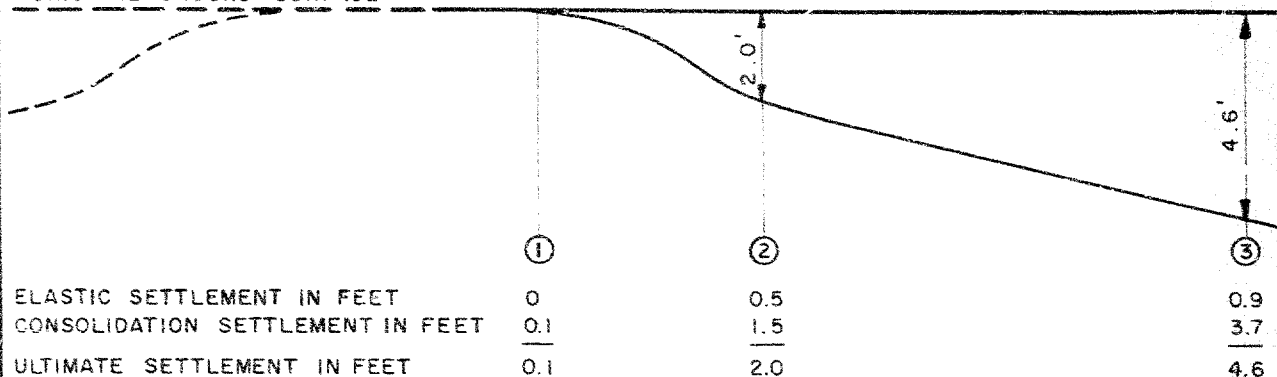
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PLATE - XIII



SECTION ALONG C HIGHWAY 44

ORIGINAL GROUND SURFACE



RESULTS OF SETTLEMENT CALCULATIONS

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FOUNDATION SETTLEMENTS
DUE TO EMBANKMENT LOAD

APPROVED

DATE NOV. 1960

SCALE

JOB No.
908

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PLATE XIV

#60-F-211

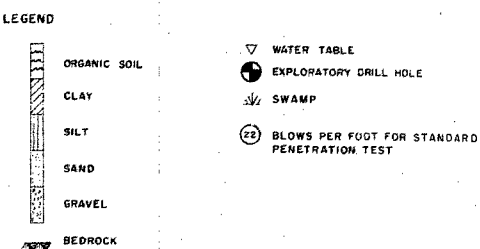
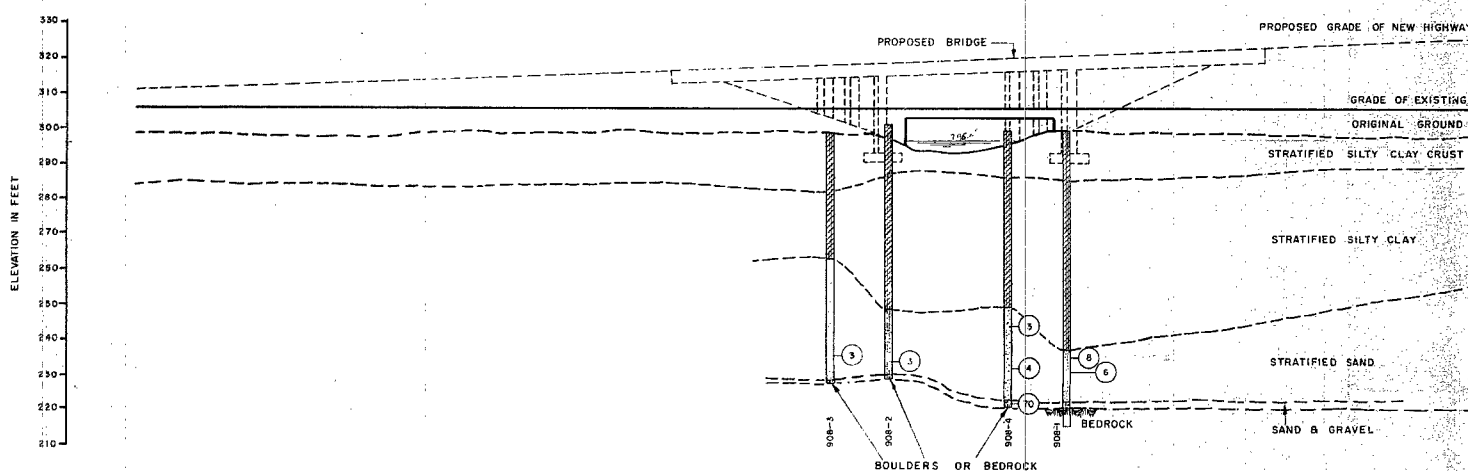
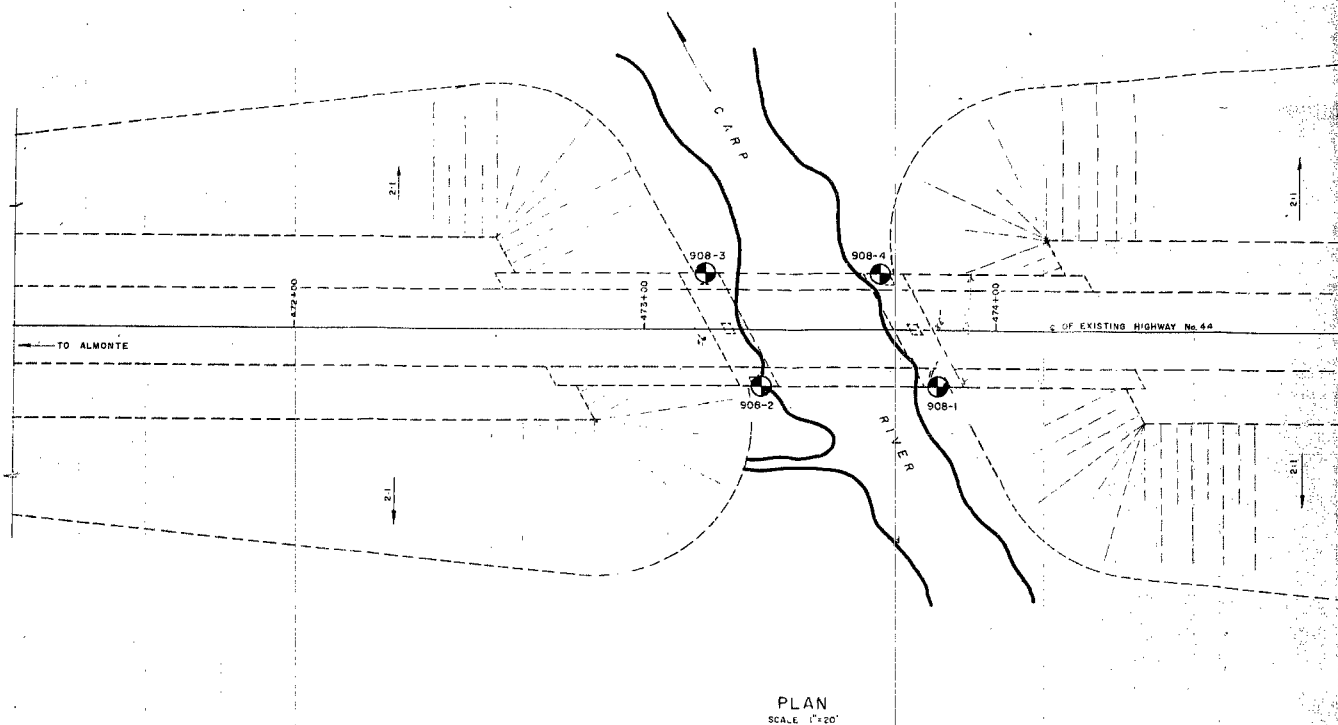
W.P. #275-60

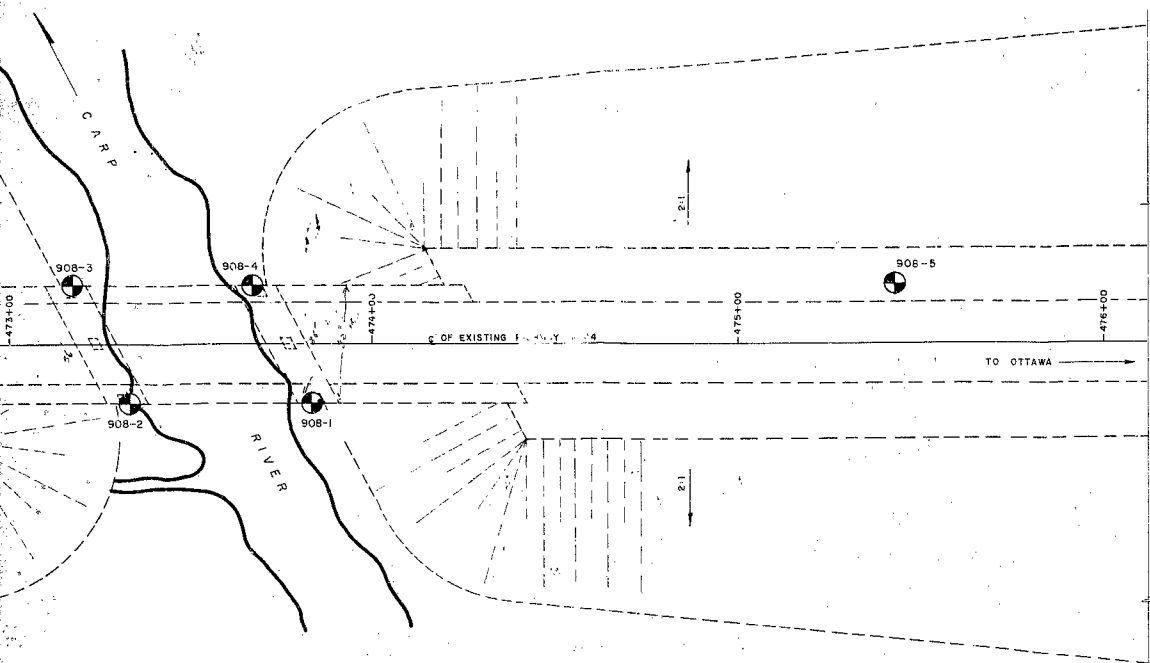
Hwy. #44

PROP. CROSSING

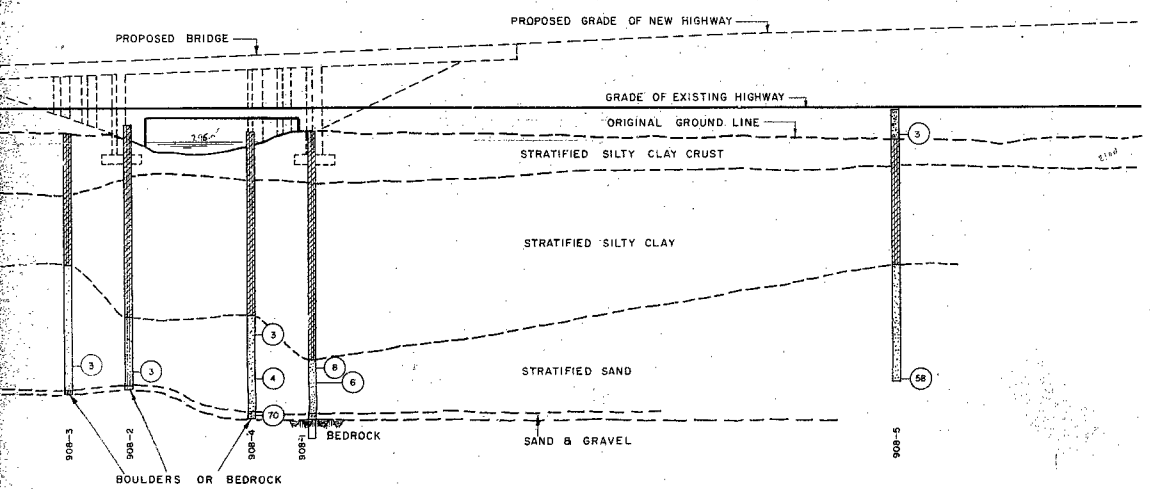
CARP RIVER

HUNTLEY TWP.

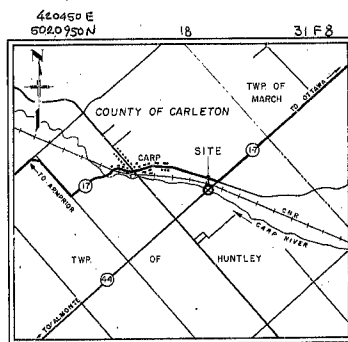




PLAN
SCALE 1"=20'



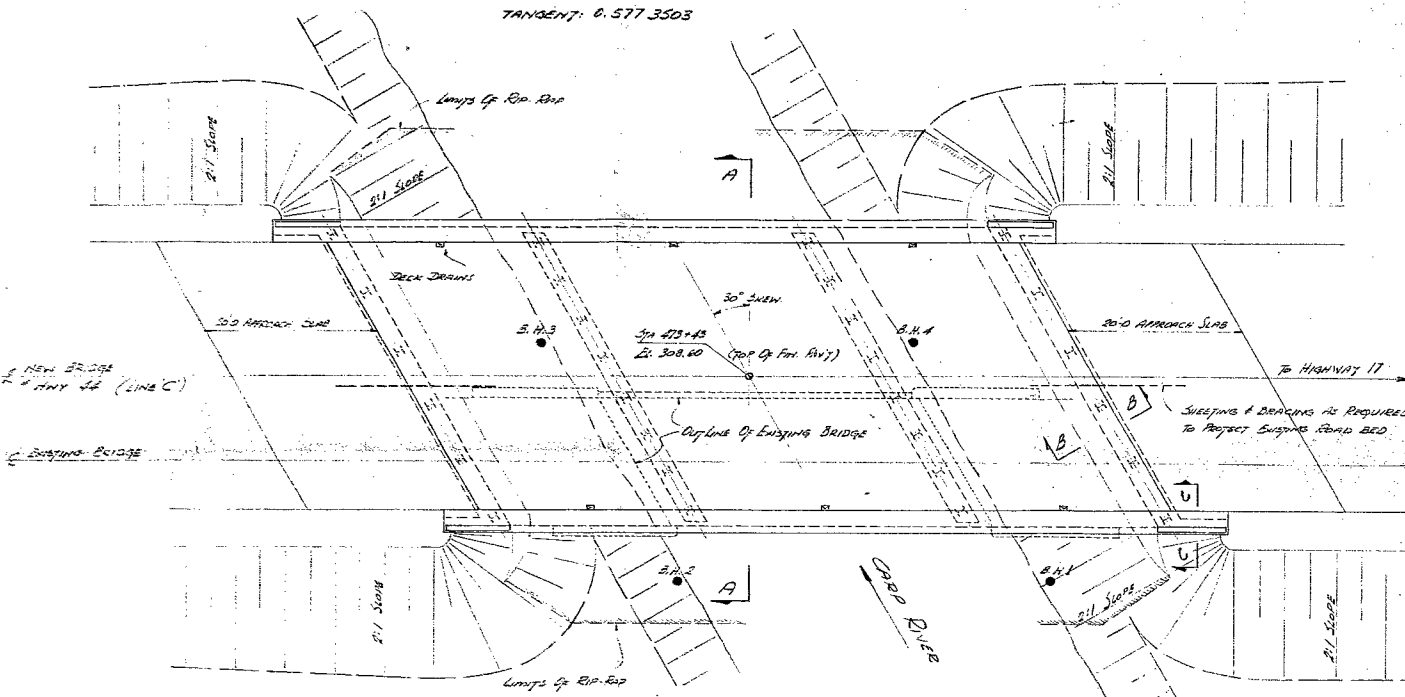
SECTION ALONG C OF EXISTING HIGHWAY
SCALE 1"=20'



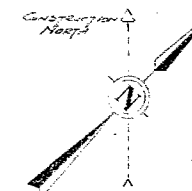
KEY PLAN
SCALE 1 IN. = 1 MI.

H. G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS NIAGARA FALLS CANADA	
ONTARIO DEPARTMENT OF HIGHWAYS	
WP-275-60	
EXPLORATORY HOLES PLAN AND SECTION	
APPROVED	DATE NOVEMBER 1960
<i>H. G. Acres</i>	SCALE AS NOTED JOB No. 908
H. G. ACRES & COMPANY LIMITED	PLATE - I

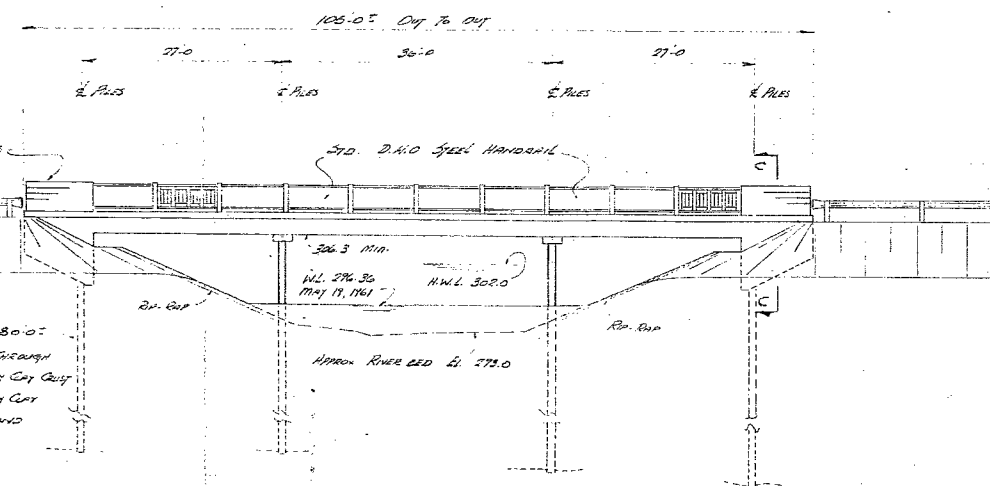
SKEW ANGLE: $30^{\circ}00'00''$
 SINE: 0.500 0000
 COSINE: 0.866 0254
 TANGENT: 0.577 3503



PLAN
 Scale: 1 in. = 10 ft.



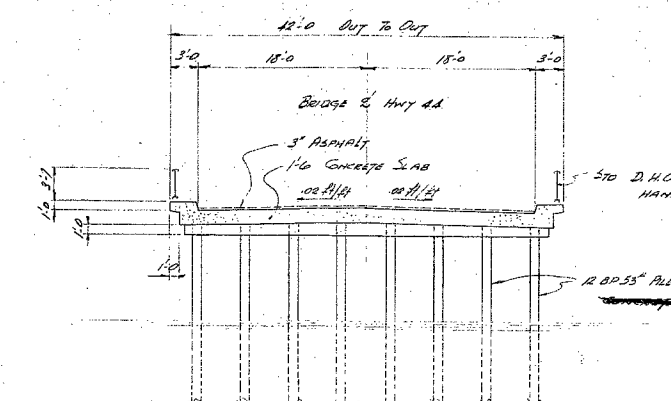
EXISTING BRIDGE TO BE DEMOLISHED TO 1.0' BELOW GRADE LINES. OLD CONCRETE TO BE USED FOR R.P. RAP.



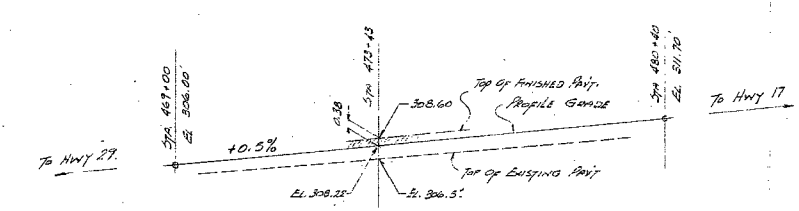
ELEVATION
 Scale: 1 in. = 10 ft.

GEODETIC BENCH MARK, N° SCCLX EL. 307.33
 C.N.R. BRIDGE OVER CARP RIVER, 1/2 MILE WEST OF STATION AND 2000 FEET WEST OF CROSSING OF OPPAWICA RIVER HIGHWAY. NORTH FACE OF NORTH STONE REMAINING WALL AT EAST END OF BRIDGE 6 FEET FROM EAST END OF WALL. AND IN SECOND COURSE ABOVE BRIDGE SEAT. BUT SET HORIZONTALLY.

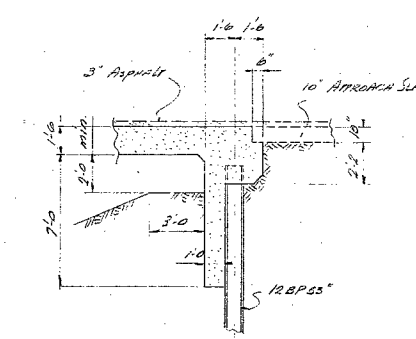
Comment:
 The pier at the Abutment location should be entered in order to make the horizontal thrust.



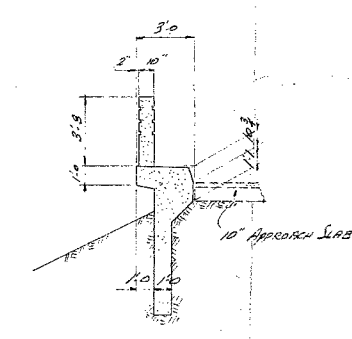
SECTION A-A
 Scale: 1 in. = 8 ft.



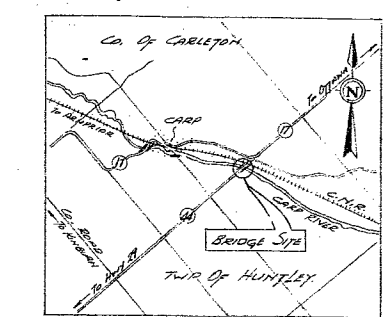
PROFILE AT & HIGHWAY 44
 PROFILE GRADE 0.38' BELOW FINISHED PAVEMENT. (N.T.S.)



SECTION B-B
 Scale: 1 in. = 4 ft.



SECTION C-C
 Scale: 1 in. = 4 ft.



KEY PLAN
 Scale: 1 in. = 1 mi.

NOTES:
 DESIGN SPECIFICATIONS - C.S.A. S6-1952 AND A.R.S.H.O. SPECIFICATIONS FOR HIGHWAY BRIDGES.
 LIVE LOAD - H20-S16
 CONCRETE STRENGTH - 3000 P.S.I. THROUGHOUT.
 NO PROVISION FOR LIGHTING HAS BEEN MADE ON THE STRUCTURE.
 REFER TO BA 1111 FOR COMPLETE SOILS REPORT.
 A CONSTRUCTION JOINT IS TO BE LOCATED AT THE E OF THE NEW STRUCTURE. THIS WILL ALLOW USE OF THE EXISTING BRIDGE DURING CONSTRUCTION OF THE NORTH HALF OF THE NEW BRIDGE.
 FALSEWORK MAY BE SUPPORTED ON MUDSILLS FOUND ON SILTY CLAY CRUST - ALLOWABLE BEARING PRESSURE 1500 P.S.F.

W.P. 275-60	
DE LEVIN, CATHY & COMPANY OF CANADA LTD. CONSULTING ENGINEERS OTTAWA	
DEPARTMENT OF HIGHWAYS-ONTARIO BRIDGE OFFICE-TORONTO	
CARP RIVER BRIDGE 0.25 MILES SOUTH OF HIGHWAY 17	
THE KING'S HIGHWAY NO. 44	DIST. NO. 9
CO. CARLETON	CON. 11
TWP. HUNTLEY	LOT 15 & 16
GENERAL PLAN OF PROPOSED STRUCTURE	
APPROVED	
OCT 23 1961	
BRIDGE ENGINEER	DESIGN ENGINEER
DESIGN 9.5.5	CHECK 2.1.1
DRAWING 17	CHECK 6.5.5
TRACING 120	CHECK 5.10
DATE SEPT. 15 1961	DRAWING NUMBER D4727-P1