Mr. A. R. Toye,
Bridge Engineer.
Materials & Research Section.

December 22, 1960
W.P. 175-60

Attention: Mr. J. E. Curtis

Re: W.P. 175-60
Lancaster Twp. Br. #15
Hwy. 401 at R.A. between
Lots 1 & 2, Conc. I,
Lancaster Twp., Dist. 9

With reference to your letter of December 8, 1960, we have received the preliminary plan of the above mentioned structure and have no comments.

ASiam

c.c. to Foundations (2)
General File

L. G. Soderman
Principal Foundation Engineer

Per:

A. Stermac
Foundations Office Engineer

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

ONTARIO
DEPARTMENT OF HIGHWAYS

POSTAL ADDRESS
DEPARTMENT OF HIGHWAYS
PARLIAMENT BUILDINGS,
TORONTO 2, ONTARIO.

Bridge Division,
December 8th, 1960.

MEMORANDUM TO:

Mr. L.G. Soderman,
Principal Soils & Foundations Engineer,
Room 107,
Downsview, Ontario.

H.G. ACRES
JUL CC

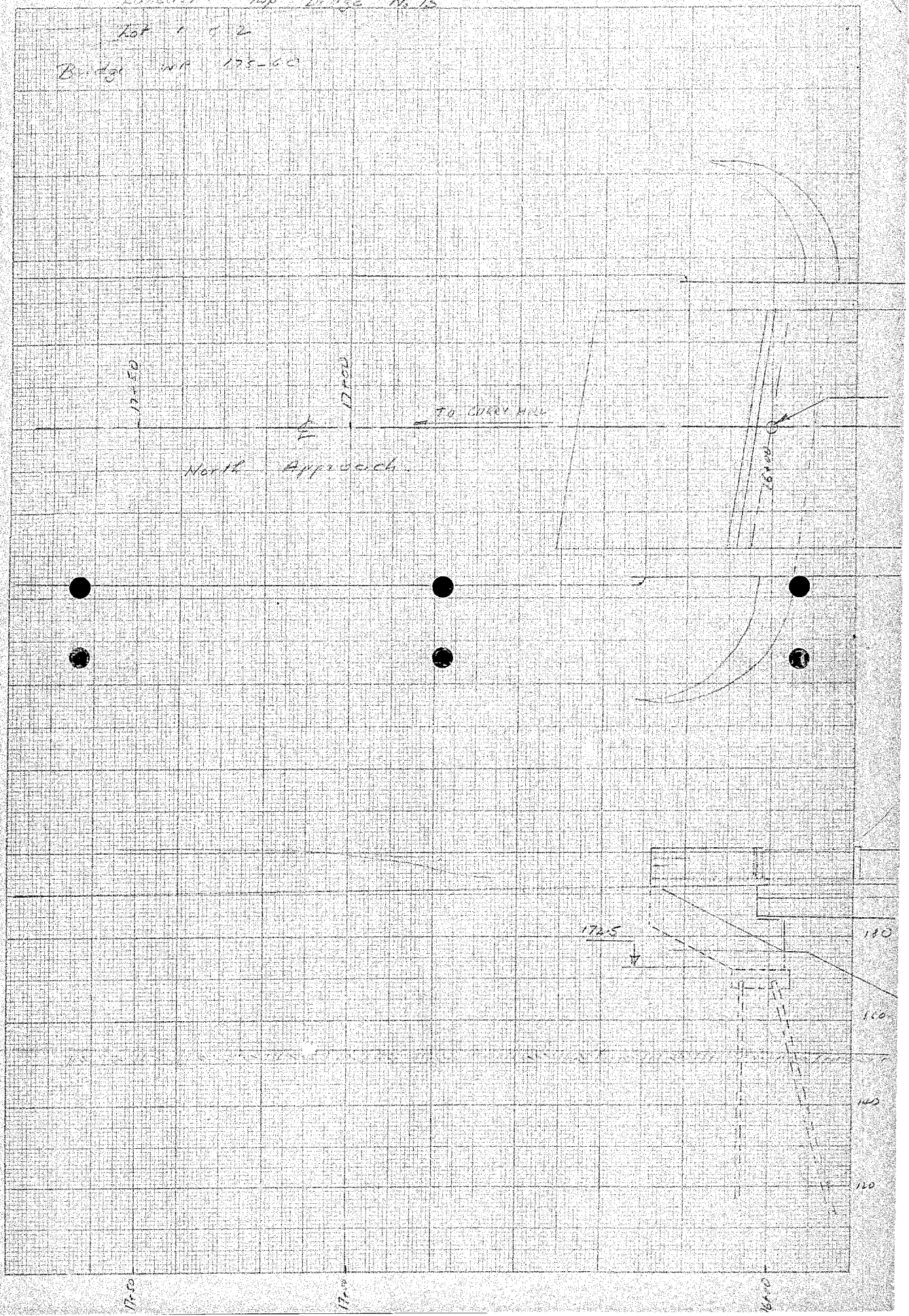
Re: W.P. 175-60
Lancaster Twp. Br. #15
Hwy. 401 at P.A. between
Lots 1 & 2, Conc. I,
Lancaster Twp., Dist. 9

Enclosed find one copy of the preliminary plan for the above structure.

The designer appears to have complied with the requirements of the foundation report but we would appreciate any comments you wish to make.

J.B. Curtis,
Bridge Location Engineer.

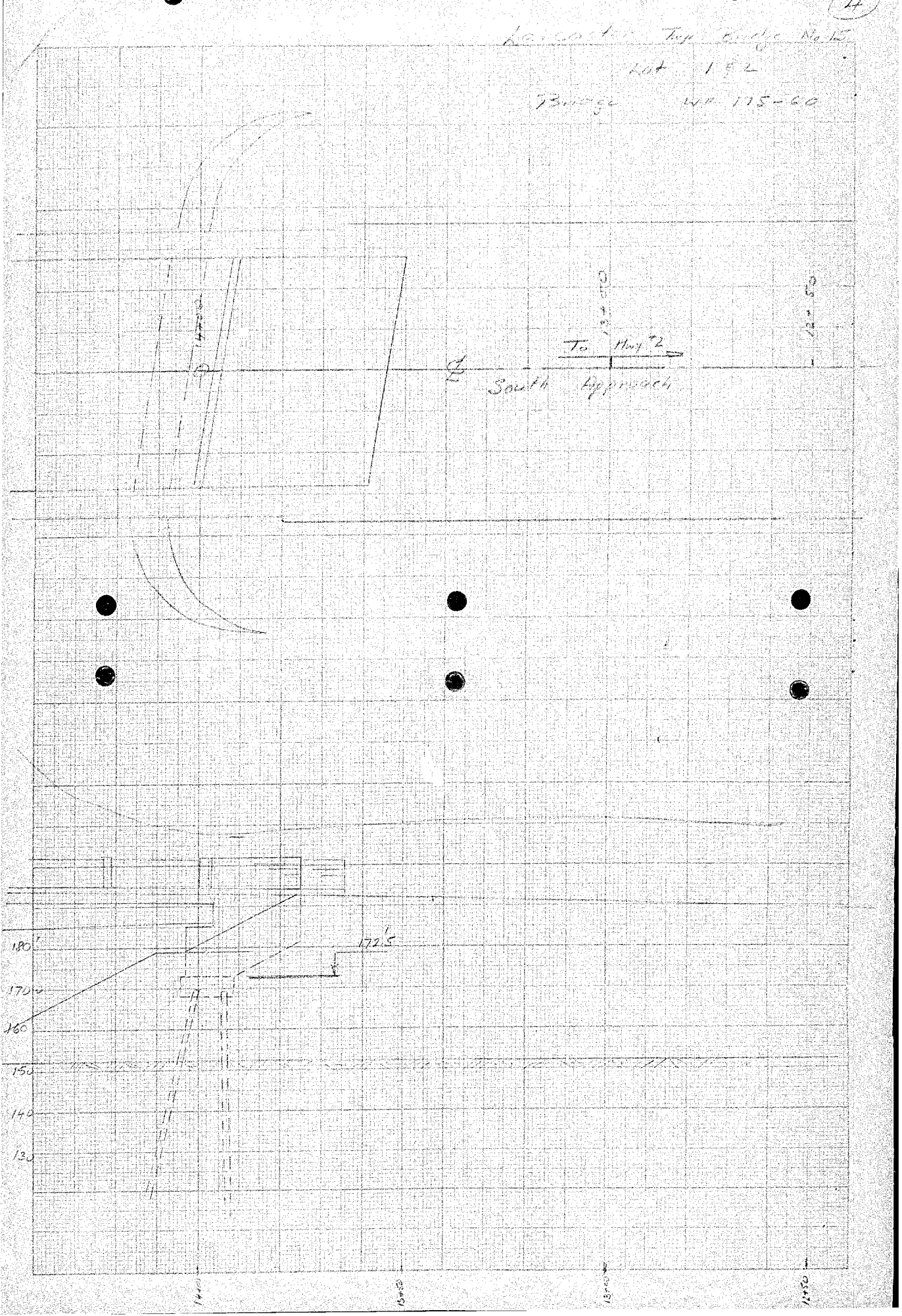
JBC/ek


Lincoln Twp Bridge No. 15
Sht 1 of 2
Bridge WR 175-60
17+50
17+00
To Quary Hill
Mark Approach
17+25
110
100
90
120

Lancaster Twp Bridge No. 15

Lot 162

Bridge Map 175-60



Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Section.

October 28, 1960.

FOUNDATION INVESTIGATION REPORT

by: M.C. Acres & Company, Limited.

Attention: Mr. S. McCombie.

Re: Proposed Crossing, Highway 401 and Road Allowance Between Lots 1 and 2, Concession 1, Township of Lancaster, District No. 9 -- W.P. 175-60.

Enclosed herewith, is the detailed foundation report prepared by M. C. Acres & Co., Ltd.

Considerable discussion has taken place concerning settlements in the embankments, between staff in our Section and representatives of M.C. Acres.

Your attention is drawn to the following:-

1. The proposed fill section can be designed using standard 2:1 side slopes.

2. Because of the deposit of soft, sensitive, highly compressible marine clay underlying the approach embankments, settlements resulting from the application of embankment loadings, will be high. The theoretical values are in the order of 4 to 5 feet; however, these values are felt to be conservative and a probable value of 2 feet can be expected. Because of the high settlements, it is our opinion that the embankment should be built, and in place, for a period approaching one year prior to construction of the bridge.

3. The Consultants have suggested that the piling for the structure be placed prior to embankment construction. We do not agree with this procedure and recommend that the embankment be built prior to substructure construction. Small displacement type 'H' piles, as suggested by the Consultants, should most certainly be used to support pier and abutment footings.

cont'd. /2 ...

Recommendations: (cont'd.) ...

4. In view of the high settlements which are anticipated at this site, and in view of the uncertainties with respect to the time rate of settlement, it is our recommendation that the subsoil be instrumented prior to embankment construction. This can be done by the Materials and Research Section. The results of this instrumentation will allow a reliable prediction of final settlements to be made and allow adjustments in structure length to be made, should they be necessary.

If there are any queries with respect to the contents of this report, or our foregoing comments, please do not hesitate to contact our Office.

L. G. Sodeman

LGS/MacF
Attach.

L. G. Sodeman,
PRINCIPAL FOUNDATIONS ENGINEER

cc: Messrs. A. M. Toye (2)
H. A. Tregeskes
D. G. Ramsay
J. Ford
L. E. Walker
J. E. Gruspier
A. Watt

Foundations Office ✓
Gen. Files.

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

IN YOUR REPLY REFER TO
FILE 892

October 6, 1960

Ontario Department of Highways,
Materials and Research Section,
Parliament Buildings,
Toronto 2, Ontario

Attention: Mr. A. Rutka,
a/Materials and Research Engineer.

Gentlemen: Ontario Department of Highways
Proposed Crossing, Highway 401
and Road Allowance Between Lots
1 and 2, Concession 1, Township
of Lancaster, District No. 9
WP 175-60

Enclosed with this letter are ten final copies of the above-mentioned report. Three preliminary copies of the report on this project were sent to you on September 12, 1960. It was necessary for this preliminary report to be issued without my approval because I was unavoidably absent from this office.

Mr. L.G. Soderman of your staff, in a telephone conversation with Mr. T.C. Kenney of our Company, enquired about the possibility of changing the construction procedure which had been recommended in the preliminary report on this project. In answer to this enquiry, the section on "Construction Problems" in the final report has been extended to cover this possibility.

However, the only change in the conclusions and recommendations given in the final report has been to increase the allowable capacity of the steel H-piles from 45 to approximately 50 tons per pile.

Yours very truly,

H.G. ACRES & COMPANY LIMITED

RJC

D.H. MacDonald
Geotechnical Engineer

RJC:sm
Encls:

Department of Highways

COPY

For the Information of:

Mr. L. Soderman,
Principal Soils &
Foundations Engr.,
D.H.O. - Room 107,
Downsview, Ontario.

Bridge Division,
February 2, 1961.

MEMORANDUM TO:

Mr. N. D. Smith,
Soils & Materials
Planning Supervisor,
Materials & Research Br.,
Downsview, Ontario.

RE: Structure sites involving marine clays. District #9

We have received a copy of your memorandum to Mr. L. G. Soderman of January 30.

It was my understanding that the Material & Research Section wanted to instrument the fills at W.P. 108-59 - Hwy. 401 and 34, and W.P. 176-60 - Bainsville Rd. and Hwy. 401 as well as the other two locations mentioned in your memorandum.

The structure at Hwy. 401 and Road to St. Andrews, W.P. 78-59 is situated on fill material and no stability or settlement problems are anticipated. Hwy. 401 to the east of the structure is, I believe, in marine clay but this will have no bearing on the structure or its approaches.

At Wesley Creek and Hwy. 401 (W.P.115-59) there is no significant approach fills to the structure since Hwy. 401 is now proposed to pass under Westley Point Rd. thus stage construction is not necessary.

On Hwy. 44 at Carp River (W.P. 275-60) and Huntley Tow. C.N.R. O'head (W.P. 274-60) I believe there is a verbal agreement between myself and Mr. A. Stermac that some one from the Material & Research Section will inspect the consolidation of the approach fills if and when I tell him of the construction date. As you know it is proposed to place abutments directly on the approach fills without the use of piles.

.....2

Re: Structure sites involving
Marine clays. District #9

I believe that the Materials & Research Section would also like to instrument any fills of significant size in the Sutherland Creek area.

Would you kindly clarify these points.

JBC/bm

J. B. Curtis.
Bridge Location Engineer

ONTARIO
DEPARTMENT OF HIGHWAYS

Memo to Mr. L. G. Soderman, Principal Foundations Engr. *Date* February 10, 1961.

From N. D. Smith *Subject* Re: Structure sites involving marine clays.

Further to my memorandum of January 30, 1961, I would point out that further information from the Bridge Office shows no problems would be anticipated for WP115-59, Hwy. 401, Wesley Creek, since the crossing at the adjacent Westley Point Rd. has been changed to an underpass. John Curtis also pointed out that the embankment stability and settlement problems at WP78-59, Hwy. 401, Rd. to St. Andrews, is not immediately adjacent to the structure, but is slightly to the East.

John Curtis also referred to an agreement between himself and Tony concerning the structures on the Carp By-Pass, WP274-60 and 275-60. However I understand, from the Program Section, that these have been entirely deleted from the programme for the present time at least, and hence do not have to be considered at this time.

John mentioned possible verbal committments to instrument the fills for the following Projects:

WP108-59, Hwy. 401, Hwy. 34 Interchange,
Reported by H. G. Acres: recommended building fills 1 year in advance.

The contract for the approach fills will be awarded on October 21, 1961, approx., and the contract for the structure will be awarded during 1962-63.

WP176-60, Hwy. 401, Twp. of Lancaster,
Reported by Associated Geotechnical: recommended berms, and building fills first.

This contract is not scheduled for award until 1962-63. Other projects which should be considered for instrumentation would be the following:

cont'd. /2 ...

WP103-59, Hwy. 401, Cornwall Twp.,
Reported by Racey, MacCallum; recommended berms.

According to the Program Section, the contract for the approach grading will be awarded May 10, 1961 approx., and the structure sometime in 1962 or 1963.

WP106-59, Hwy. 401, Charlottenberg Twp.,
Reported by H. G. Acres; recommended building approaches 1 year in advance. This project is not scheduled until at least 1964.

Finally, we would draw your attention to the fact that, of the projects for which a definite recommendation for instrumentation has been made, WP178-60 is scheduled for award in 1962 or 1963, and WP175-60 is not scheduled for award until 1964 or later.

N. D. Smith
Soils & Materials Planning Supervisor

NDS/tt
c.c. J. Curtis
G. Wrong
J. E. Gruspier
N. D. Smith
Files

ONTARIO
DEPARTMENT OF HIGHWAYS

Memo to Mr. L. G. Soderman, Principal Foundations Engr. *Date* January 30, 1961.

From Materials & Research. *Subject* Re: Structure sites involving marine clays.

A review of the proposed structure projects in Ottawa District has shown only two projects for which a definite recommendation for instrumentation has been made.

These are:

W.P.175-60, Hwy. 401, Rd. Allce., Twp. of Lancaster.
Investigated by Acres and reportec in July 1960.

W.P.178-60, Hwy. 401, Rd. Allce., Twp. of Lancaster.
Investigated by Racey, MacCallum and reported June 28, 1960.
In addition, some instrumentation has apparently been carried out at the following site:

W.P.114-59) Hwy. 401, C.P.R., E. of Cornwall
Cont. 60-103) Investigated by Geocon and Dr. Golder.

Finally, the reports for the following projects indicate high settlements, in most cases a recommendation is made for stage construction, and in one case a surcharge was recommended:

W.P.115-59, Hwy. 401, Wesley Creek
Reported by Racey, MacCallum on June 28, 1960.

W.P.78-59, Hwy. 401, Rd. to St. Andrews
Reported by Associated Geotechnical in June 1960.
A surcharge was recommended at this site.

W.P.103-59, Hwy. 401, Interchange Cornwall Twp.
Reported by Racey, MacCallum on March 16, 1960.
Requires berms.

cont'd /2 ...

W.P.108-59, Hwy. 401, Hwy. 34 Interchange.
Reported by H. G. Acres in July, 1960.
Build fill 1 year in advance.

W.P.106-59, Hwy. 401, Interchange, Charlottenberg Twp.
Reported by H. G. Acres in August 1960.
Build fill in advance.

W.P.176-60, Hwy. 401, Rd. Allce., Twp. of Lancester.
Reported by Associated Geotechnical on Aug. 1960
& Nov. 25, 1960.
Requires berms, and build fill first, then structure.

W.P.274-60, Hwy. 17, C.N.R., Carp By-Pass
Reported by H. G. Acres in Nov. 1960.
Build fill 1 year in advance.

W.P.275-60, Hwy. 17, Carp River
Reported by H. G. Acres in Nov. 1960.
Build fill 1 year in advance.

W.P.111-59,) Hwy. 401, S. Raison River
Cont.58-118)
Reported by E. M. Peto on Feb. 10, 1958.
Build fill first.

The foregoing are, to the best of our knowledge, the only additional projects in this area, where instrumentation could be required. Should you be aware of any others, we would appreciate these being brought to our attention, in order that we might add these to our record.

N. D. Smith
Soils & Materials Planning Supv

NDS/tt
c.c. J. Curtis
G. Wrong
J. E. Gruspier
NDS
Files

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT

on

FOUNDATION CONDITIONS

PROPOSED CROSSING
HIGHWAY 401 AND ROAD ALLOWANCE
BETWEEN LOTS 1 AND 2, CONCESSION 1,
TOWNSHIP OF LANCASTER, DISTRICT NO. 9
WP 175-60

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

July, 1960

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT

on

FOUNDATION CONDITIONS

PROPOSED CROSSING
HIGHWAY 401 AND ROAD ALLOWANCE
BETWEEN LOTS 1 AND 2, CONCESSION 1,
TOWNSHIP OF LANCASTER, DISTRICT NO. 9
WP 175-60

Table of Contents

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT

on

FOUNDATION CONDITIONS

PROPOSED CROSSING
HIGHWAY 401 AND ROAD ALLOWANCE
BETWEEN LOTS 1 AND 2, CONCESSION 1,
TOWNSHIP OF LANCASTER, DISTRICT NO. 9
WP 175-60

Introduction

This report contains the results of an investigation into the foundation conditions at the proposed crossing of Highway 401 and the Road Allowance between Lots 1 and 2, Concession 1, in Lancaster Township. At the Department's request, soil explorations were carried out by H.G. Acres & Company Limited to determine the foundation conditions for a bridge structure and its approach embankments to carry the road allowance over Highway 401. 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 June 27, 1960, and was completed on July 6, 1960. Laboratory testing of the soil samples was completed in August 1960.

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

Geology of the Site

The site of the proposed crossing is in a flat lowland plain which extends about eight miles back from the north shore of the St. Lawrence River. The area is underlain by limestone bedrock at an undetermined depth. During glacial times the bedrock surface was covered with a variable thickness of till which is composed mainly of sand and gravel with many large boulders. 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. Subsequent uplift has resulted in the present non-marine environment. Weathering and desiccation have developed a stiff crust about ten feet thick on the marine deposit.

Exploratory Work

Two diamond drills were used for the exploration work. In the marine deposits, the wash boring method was employed and BX casing was used to advance the holes. Two-inch diameter Shelby tube samples were taken, generally at six-foot intervals. Where possible, in situ vane tests were performed 18 inches below the elevation of the bottom of the Shelby tube samples, immediately after the samples were removed.

When the till was encountered, standard penetration tests were performed and the split-spoon samples retained. When boulders were encountered, the BX casing could not be advanced. Holes No. 892-1 and No. 892-4 were advanced by diamond drilling tnrough a succession of boulders but bedrock was not encountered. All other holes were stopped when they reached boulders or when the casing could not be advanced in the dense till without causing damage to it.

A total of five holes were drilled and sampled, and in each of these holes, vane tests were made. The program of work is given in Appendix A.

Site Conditions and Soil Properties

The site investigated lies in a flat lowland plain, the general ground surface elevation of which is approximately 162 feet. The land is used mostly for pasture or is brush covered. It is poorly drained with only a few open ditches to carry away the surface water.

The materials encountered in the exploratory holes are described in the attached drilling reports, Plates II to VI, inclusive. The soil conditions are quite uniform in the horizontal direction except for the variable surface of the till layer.

(a) - Clay Crust - This is the stiff, weathered and desiccated crust of marine clay. The crust constitutes a horizontal layer about 10 feet thick with a gradual transition to the underlying clay deposit.

Within this 10-foot layer, the soil has the following average properties:

Liquid limit = 60 per cent
Plastic limit = 29 per cent
Water content = 53 per cent

Vane tests were not performed because the soil was too stiff to fail by this means. The natural undrained shear strengths determined from laboratory compression

tests and field vane tests, decrease from 2,000 psf at an elevation of 156 feet to about 1,200 psf at an elevation of 152 feet. The sensitivity of this material is about 10 at an elevation of 156 feet and it increases with depth.

(b) - Clay - This is the soil commonly called Leda Clay, and it generally appears homogeneous, although some samples do show stratification when dried. The thickness of this deposit varies because of the irregular surface of the underlying glacial till, as shown on Plate I.

The results of the field and laboratory tests indicate two marine deposits at this site, resulting from separate marine depositions. All the test results, including the Atterberg limits, the shear strengths and the consolidation test results, support this conclusion. The interface between the two deposits occurs at the approximate elevation of 132 feet.

From the samples obtained, the clay above elevation 132 feet was found to have the following average properties:

Liquid limit = 65 per cent
Plastic limit = 29 per cent
Water content = 80 per cent

Below elevation 129 feet the clay was found to have the following average properties:

Liquid limit = 40 per cent
Plastic limit = 25 per cent
Water content = 50 per cent

The natural undrained shear strengths were measured by field vane tests and by laboratory compression tests. It was found that the compression test results generally combined low strengths and high values of failure strain as shown on Plate XIII, whereas it is known that this type of clay, if undisturbed, should fail at very low values of strain. Therefore, it is considered that those results which display high failure strains are indicative of sample disturbance. The tests in which failure occurred at strains of 2 to 3 per cent indicate relatively undisturbed samples, and in these cases the shear strengths obtained agree closely with the vane test results. The average natural undrained shear strength obtained from the vane tests is approximately 680 psf.

The large increase in shear strength between elevation 132 feet and 129 feet along with lower water contents indicate the stiffer crust of the lower marine deposit.

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

The maximum sensitivity of this clay indicated by the field vane tests is 7.8, 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 was too soft to test with the available equipment and, for this reason, the sensitivity could not be determined. It has been suggested, however, that the sensitivity of this material exceeds 100(1).

Consolidation tests were run on this clay and its overlying crust. The "p-e" curves are presented on Plates VII to XII. From these data it can be deduced that the clay has experienced some overconsolidation below the crust. The curves for the clay have the shape

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

characteristic of sensitive soil, and beyond the preconsolidation pressure this clay is very compressible. The apparent preconsolidation pressures have been estimated, and the results summarized on Plate XIV. Sample disturbance reduces the value of apparent preconsolidation pressure, and because the laboratory compression tests have indicated that many of the samples have been disturbed, the estimates of preconsolidation pressure are probably low.

The large increase in the preconsolidation pressure at a depth of 129 feet is further indication of the presence of the crust of the lower marine deposit.

(c) - Silt - A layer of silt 5-1/2 feet thick was encountered beneath the clay in hole No. 892-4. Since it was not observed in the other borings, it probably is not continuous. Vane tests in this material indicate a lower value of natural undrained shear strength than in the clay. It is believed, however, that this is indicative of high water pressures transmitted from the underlying till rather than the natural consistency of the silt layer.

(d) - Sand and Gravel Till - This is a heterogeneous deposit of sand and gravel with some silt and clay. In general, it is of medium density as indicated by the N-values from the standard penetration tests. It contains many large boulders. An outcrop of the till about one mile from the site was examined; here the boulders were as much as 6 feet in diameter. Hole No. 892-1 was advanced through the sand and gravel till 11.5 feet until a boulder was encountered. The hole was continued 12 feet by drilling through a succession of boulders. Hole No. 892-4 was advanced 4 feet through boulders. Bedrock was not encountered in either of these holes. Because the drilling proved the existence of boulders, it would not be practical to rest the foundation of a structure of the nature proposed on bedrock. For this reason, all other exploratory holes were discontinued when boulders were encountered. The cores from the drilling of the boulders in hole No. 892-1 indicated the boulders to be limestone.

(e) - Ground Water Conditions - No piezometric observations of the ground water level were made at this site. However, from the surface of a pond in a nearby field the ground water level is estimated to be at about elevation 157 feet.

Design Considerations

(a) - Bearing Capacity

Road Embankment - The critical condition for stability of an embankment founded on soft sensitive clays generally occurs during construction or shortly after completion of construction. Since it is probable that negligible consolidation of the clay would take place during construction, the $\phi = 0$ method of stability analysis is considered to be 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 approach to the over-pass structure will be approximately 25 feet high and 50 feet wide at the top, and the unit weight of the embankment material will be approximately 130 pcf. Using these loading requirements, the analyses indicate that the factor of safety can range from 1.0 to 1.6, depending on the choice of soil properties and the geometry of the embankment. For this reason, the soil properties and the results of the analyses are considered in some detail below:

(i) - Shear Strength of the Clay - On the basis of the laboratory compression tests alone, an average value for the natural undrained shear strength of the clay would be about 550 psf. The field vane tests alone would justify a value of 680 psf. This discrepancy is appreciable, but in view of the sample disturbance, which has already been discussed, it is considered that the best estimate of the in situ undrained shear strength of the clay is given by the field vane tests. In estimating the shear strength from test results, the high values at about elevation 130 feet have not been considered. These are due to the crust of the first marine deposit which is only about four feet thick and would contribute very little to the stability of the embankment.

A marine clay of such very high sensitivity would be expected to fail at strains less than one per cent with large strength reductions at larger strains. Since no samples failed in this range, the in situ failure strain has been estimated at 0.7 per cent.

In summary, the limiting values of the natural undrained shear strength are:

Maximum	S_u = 680 psf
Minimum	S_u = 550 psf

and the best estimate of S_u for the clay would be about 680 psf.

(ii) - Shear Strength of the Crust - The average natural undrained shear strength of the crust on the basis of laboratory compression tests is approximately 2,000 psf. However, 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 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 1,300 psf. For a lower limit, a minimum value of shear strength for the crust would be the same as that for the underlying clay.

In summary, the limiting values of the natural undrained shear strength of the crust are:

Maximum	S_u = 2,000 psf
Minimum	S_u = 550 to 680 psf

and the best estimate for S_u for use in the stability analyses would be about 1,300 psf.

(iii) - Shear Strength of 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) - Factor of Safety - The minimum depth of the clay layer is 32 feet and the maximum depth is 44 feet. Using the maximum depth in the analyses, the minimum factors of safety are obtained.

The factors of safety have been calculated for embankments with 2:1 and 3:1 side slopes and varying widths of berms to demonstrate the effect of the geometry of the embankment.

Plates XV and XVI give a summary of the factors of safety for all combinations of the variables. On the basis of our "best estimate" values, an embankment with 2:1 side slopes will have a factor of safety of 1.42. It is considered that this factor of safety, with the choice of variables involved, is satisfactory.

Bridge Piers - One method of supporting the bridge would be with spread footings, the bases of which would be at a depth of about six feet. The average natural undrained shear strength of the foundation soil is approximately 680 psf, and using a factor

of safety of three against ultimate failure and a one-third reduction for eccentric loading, the maximum allowable uniform bearing pressure would be about 960 psf. Assuming a bridge pier load of 30 kips per foot length of pier, the footing would have the impractical width of 31 feet.

The alternative to footings for the support of the bridge piers is bearing piles driven to refusal in the coarse till underlying the marine clay. However, due to the high sensitivity of the clay, a non-displacement type pile should be used In this category, two types of cast-in-place concrete piles, as well as steel H-piles, are available.

Both types of cast-in-place concrete piles require a steel casing. In one type, a casing is is advanced and the soil cleaned out of the casing simultaneously. In the other type, the casing is driven open ended and cleaned out subsequent to completion of driving If the latter type was used at this site, it would be necessary to clean out the casing when it had been advanced through the crust in order to prevent the crust from plugging the end of the casing and thereby cause full displacement of the sensitive clay.

Such a pile might best be placed by drilling an uncased hole through the crust prior to driving the casing. The possibility of hydrostatic pressure in a continuous aquifer in the sand and gravel till might result in a blowout in the bottom of the hole if either of these preceding types was used. In order to prevent such a blowout, the holes would have to be kept full of water and the concrete placed under water. Such a method always carries with it some doubt concerning the conditions at the bottom of the pile during the concreting operation.

Steel H-piles are non-displacement piles, and their installation and performance would not be affected adversely by the existing ground water conditions. It would be difficult, however, to develop a pile load in the sand and gravel till which would allow an efficient use of steel bearing piles. Some piles might come to rest on large boulders, whereas others would be driven to refusal in the sand and gravel till. Therefore, the capacities of adjacent piles would probably vary markedly. It is impossible to predict accurately the capacities of these piles and it would be uneconomical to load test even a few of them. However, H-piles

driven to a refusal of one-quarter inch per blow with a 15,000-foot pound hammer could develop a working capacity of approximately 50 tons.

(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 XVII. The consolidation characteristics of the clay are given on Plates VII to XII, and the apparent preconsolidation pressures are summarized on Plate XIV. The value of the apparent modulus of elasticity which has been used to predict the immediate settlements is 80 tons per square foot. The calculated settlements are shown on Plate XVII, and it can be seen that they are very large. The most important factors contributing to these large settlements are the apparent preconsolidation pressures and the initial steep portions of the "p-e" curves beyond the points of preconsolidation. However, the samples were definitely disturbed, and 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 may not even be of the right order of magnitude.

Nevertheless, the worst condition which will result due to settlement will be the differential settlement between the abutments and the pile supported bridge. The abutments could be supported on piles, but this is not necessary to ensure stability of the embankment. The increase in foundation stress due to the abutment load is offset by the greater bearing capacity at the end of the fill.

(c) - Construction Problems

Because of the low factor of safety of the embankment against a bearing capacity failure, and because of the high sensitivity of the foundation soil, the construction procedure is important. The piles should be driven before placement of the embankment, and observations should be made to detect possible distress of the embankment while it is being placed. The failure of the base of an embankment on clay is commonly preceded by the gradual heave of broad belts located on either side of the fill. Observations should be made to detect such heave in its initial state.

Lateral pressure may be developed on the bearing piles during and after placement of the embankment due to shear and consolidation strains in the clay. Such conditions could be detected by observations of any movements of the piles.

If it should be considered that the construction of the embankment would create a condition of imminent failure, then the embankment should be constructed first. In this case an embankment failure would not result in the loss of the bridge footings and piles. However, to be consistent with this reasoning, time must be allowed for consolidation of the foundation soils to take place with the resultant increase in factor of safety before the piles are driven. If this is not done, the driving of the piles might initiate failure.

Conclusions

(a) - From the drilling work done at the site, the general soil profile was found to consist of a surface deposit of marine clay varying in depth from 32 to 44 feet. This clay has a horizontal surface crust approximately 10 feet thick. Beneath the marine clay is a coarse, granular till containing many large boulders,

the irregular surface of which causes the variations in the thickness of the clay layer.

(b) - The properties of the foundation soils are summarized on Plates XIII and XIV.

(c) - The 25-foot high embankment, with a crest width of 50 feet and with two to one side slopes, can safely be supported on the undisturbed foundation soils provided two to one slopes are maintained on the end of the embankment as well as the sides. The calculated settlement of the embankment is very large, but actual settlements will probably be much smaller

(d) - The bridge should be supported by bearing piles of a non-displacement type. Steel H-piles fall within this category and their use would eliminate any ground water problem that might be encountered with other possible pile types.

(e) - The construction procedure is important. The piles should be driven before placement of the embankment and observations should be made to detect any movements while placing the embankment.

Recommendations

The embankment can be constructed with two to one side and end slopes.

It is recommended that the bridge structure be supported on non-displacement bearing piles. Steel H-piles are the most suitable type at this particular site, and these could be driven into the till to develop a capacity of approximately 50 tons per pile.

The piles should be driven before placement of the embankment.

APPENDIX A

Program of Work

June 27, 1960 - Diamond drill No. 1 arrived at the site. Hole No. 892-1 was commenced.

June 28, 1960 - Diamond drill No. 2 arrived at the site. Hole No. 892-2 was commenced.

June 29, 1960 - Holes No. 892-1 and No. 892-2 were completed.

June 30, 1960 - Holes No. 892-4 and No. 892-5 were commenced.

July 4, 1960 - Hole No. 892-5 was completed.

July 5, 1960 - Hole No. 892-4 was completed and hole No. 892-3 was commenced.

July 6, 1960 - Hole No. 892-3 was completed.

Summary of Time

Work Type	No. of Holes	Total Length Feet	Total Time Hours
Modified wash boring ..	5	247	71
Diamond drilling	2	16	11

APPENDIX B

Summary of Laboratory Test Results

Hole No.	Sample No.	Elevation Feet	Water Content %	Liquid Limit %	Plastic Limit %	S_{u_n} Psf	e_f %	S_{u_r} Psf	St
892-1	2	152	67.5	56.7	26.3	457	3.5	B17	-
	3	147	80.2	58.5	25.7	467	3.5	B9*	
	4	142	87.0	68.4	27.4	373	6.0	B11	
	5	137	75.4	-	-	834	4.0	88	9.5
	6	132	53.6	54.2	27.3	854	7.5	115	7.4
892-2	3	147	77.6	-	-	580	2.5	B13	-
	4	138	40.9	-	-	855	11.5	164	5.2
892-3	1	156	57.3	60.8	28.3	1,873	2.5	142	13.2
	2	150	80.2	63.1	29.3	192	11.5	B9	-
	3	144	86.5	62.2	27.5	468	2.0	B8	-
	4	138	81.0	-	-	510	4.0	B10	-
	5	132	73.5	69.9	32.7	849	3.0	B31	-
892-4	1	156	48.3	58.9	30.6	2,295	4.0	313	7.3
	3	148	82.4	-	-	432	6.0	B10	-
	4	142	90.1	-	-	399	6.5	B8	-
	5	137	75.4	-	-	505	6.0	B3	-
	7	129	64.8	48.1	30.4	554	4.0	B12	-
	9	121	36.5	31.3	19.9	565	8.0	B9	-
892-5	3	147	82.3	-	-	421	5.0	B10	-

e_f - Failure strain

S_{u_n} - Natural undrained shear strength

S_{u_r} - Remoulded undrained shear strength

St - Sensitivity

B9 - Number of shocks in liquid limit device.

* When a remoulded sample was too soft for a compression test, the number of shocks in the liquid limit device at natural water content is given.

APPENDIX C

Summary of Field Vane Test Results

Hole No.	Elevation Feet	Undrained Shear Strength Psf		Sensitivity
		Natural	Remoulded	
892-1	149	670	130	5.2
	144	632	121	5.2
	140	911	177	5.2
892-2	149	465	102	4.6
	140	527	223	2.4
	132	1,395	362	3.9
892-3	154	1,720	352	4.9
	148	721	166	4.3
	142	777	203	3.8
	136	1,110	222	5.0
	130	1,239	222	5.6
892-4	150	777	102	7.6
	145	685	139	4.9
	140	546	111	4.9
	135	777	102	7.6
	131	1,690	306	5.5
	127	648	83	7.8
	122	703	157	4.5
	118	342	157	2.2
892-5	149	666	130	5.1
	143	592	120	4.9
	137	906	186	4.9
	131	1,369	212	6.5
	127	998	296	3.4

APPENDIX D

List of Plates

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

DRILLING REPORT

CLIENT: Ontario Department of Highways — JOB No. 892

PROJECT: WP 175-60 — HOLE No. 892-1

SITE: Highway 401 and Forced Road, Lancaster Twp. — SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Company Limited — STARTED 12.00 A.M. June 27, 19 60 — FINISHED 5.00 P.M. June 29, 19 60

METHOD OF DRILLING: SOIL Modified Wash Boring — CASING DIAM BX and AX

ROCK Diamond Drill — CORE DIAM AXT

LOCATION: ~~EASTING~~ Chainage 463+19 — DEPARTURE On Centreline — BEARING — INITIAL DIP 90 Degrees — OTHER DIPS

ELEVATIONS: DATUM GSC — DRILL PLATFORM — GROUND SURFACE 162.5 — ROCK SURFACE — BOTTOM OF HOLE 106.5 — WATER TABLE

DEPTH	SOIL TYPE	DESCRIPTION, COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC	SAMPLE NO	SAMPLE TYPE*	SAMPLE SIZE	SAMPLE DEPTH	SAMPLE RET'D	PENETRATION TEST Blows *
Feet					In.	Ft	In.	
0	Silty Clay	Mottled, grey brown, stiff						
			1	SO	?	5.0		Machine
						7.0	24	Pushed
9.0	Clay	Blue grey, homogeneous, soft to medium						
			2	SO	2	10.0		
						12.0	24	Pushed
				Vane Test		13.5		
			3	SO	2	15.0		
						17.0	24	Pushed
				Vane Test		18.5		
			4	SO	2	20.0		
						21.5	18	Pushed
				Vane Test		23.0		
			5	SO	2	25.0		
						27.0	24	Pushed
				Vane Test		28.5		
32.5	Sand and Gravel	Grey, heterogeneous, medium density with some silt and clay	6	SO	2	30.0		
						31.5		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 - POLYFILM BAG
Z - DISCARDED

INSPECTOR J. MacLeod

LOGGED BY J. MacLeod

APPROVED D. H. MacDonald.

DATE July, 1960

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

DRILLING REPORT

CLIENT: Ontario Department of Highways — JOB No: 892

PROJECT: HF 175-60 — HOLE No: 892-1

SITE: Highway 401 and Forced Road, Lancaster Twp. — SHEET No: 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION (COLOUR, CONSISTENCY OF SOIL, TEXTURE, WATER CONTENT, CLASSIFICATION, FORMATION, MATERIAL, ROCK TYPE, WEATHERING, ETC.)	SAMPLE NO.	SAMPLE TYPE	SAMPLE SIZE	PENETRATION DEPTH	PENETRATION MIN.	PENETRATION Blows *
Feet					In.	Ft	In.	
			7	AQ	2	35.0		
						35.5		10
						36.0		14
						36.5	15	15
			8	AQ	2	40.0		
						40.5		10
						41.0		10
						41.5		15
44.0	Boulders and Gravel	containing some silt and clay, dense	9	DC	1	45 50	2.5	
56.0		End of Hole						

* Penetration Test
The value given in the number of blows of a 140-lb weight falling freely 30 inches required to advance the standard split-spoon sampler 6 inches to the depth indicated.

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

DRILLING REPORT

CLIENT Ontario Department of Highways — JOB No. 892

W.O. No. 175-60 — HOLE No. 892-2

Highway #21 and Forced Road, Lancaster Twp. — SHEET No. 1 OF 1

CONTRACTOR R.S. Johnston Drilling Company Limited — STARTED 9.00 A.M. June 28, 1960 — FINISHED 3.00 P.M. June 29, 1960

METHOD Modified Wash Boring — CASING DIAM. BX and AX

EQUIPMENT Diamond Drill — CORE DIAM. AXT

LOCATION ~~CENTRE~~ Drainage 363+19 — ELEVATIONS: DATUM GSC

DEPARTURE 60 Feet Left — DRILL PLATFORM

BEARING — GROUND SURFACE 162.6

INITIAL DIP 90 Degrees — ROCK SURFACE

OTHER DIPS — BOTTOM OF HOLE 119.6

WATER TABLE

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO	SAMPLE TYPE *	SAMPLE SIZE	SAMPLE DEPTH	SAMPLE RET'D	PENETRATION TEST Blows *
Feet					In.	Ft	In.	
0	Silty Clay	Mottled, grey brown, stiff						
			1	SC	2	5.0		Pushed
						6.0		to Refusal
9.5	Clay	Blue grey, homogeneous, soft to medium						
			2	SC	2	10.0		
						11.5	15	Pushed
				Vane	Test	13.0		
		* Penetration Test						
		The value given in the	3	SC	2	16.0		
		number of blows of a 140-lb				17.5	15	Pushed
		weight falling freely 30 inches						
		required to advance the stan-		Vane	Test	20.0		
		dard split-spoon sampler 6						
		inches to the depth indicated.		Vane	Test	21.8		
			4	SC	2	25.0		
						26.5	18	Pushed
				Vane	Test	30.0		
32.0	Sand and Gravel	Grey, heterogeneous, medium	5	AQ		32.0		
		density with some silt and				32.5		8
		clay				33.0		8
						33.5	6	2
43.0	Boulders	End of Hole						

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 D. H. MacDonald

DATE July, 1960

Plate IIIA

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

DRILLING REPORT

CLIENT: Ontario Department of Highways — JOB No. 892

PROJECT: WP 175-60 — HOLE No. 892-3

SITE: Highway 401 and Forced Road, Lancaster Twp. — SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Company Limited
STARTED 8.00 A.M. July 5. 1960
FINISHED 10.00 A.M. July 6, 1960

METHOD OF DRILLING:
SOIL Modified Wash Boring — CASING DIAM. BX and AX
ROCK Diamond Drill — CORE DIAM. AXT

LOCATION:
~~LATITUDE~~
Chainage 462+92
DEPARTURE 200 Feet Left
BEARING
INITIAL DIP 90 Degrees
OTHER DIPS

ELEVATIONS:
DATUM GSC
DRILL PLATFORM
GROUND SURFACE 162.9
ROCK SURFACE
BOTTOM OF HOLE 116.6
WATER TABLE

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO	SAMPLE TYPE *	SAMPLE SIZE	SAMPLE DEPTH	SAMPLE RET'D	PENETRATION TEST Blows *
Feet					In.	Ft	In.	
0	Silty Clay	Mottled, grey brown, stiff	1	BO	2	6.0		Machine
						7.5	18	Pushed
				Vane Test		9.0		
9.5	Clay	Blue grey, homogeneous, soft to medium						
			2	BO	2	12.0		
						13.5	18	Pushed
				Vane Test		15.0		
			3	BO	2	18.0		
						19.5	18	Pushed
				Vane Test		21.0		
			4	BO	2	24.0		
						25.5	18	Pushed
				Vane Test		27.0		
			5	BO	2	30.0		
						31.5	18	Pushed
				Vane Test		33.0		
40	Sand and Gravel	Grey, heterogeneous, medium density containing some silt and clay with scattered boulders						

SAMPLING METHOD
* A – SPLIT TUBE
– 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 – POLYFILM BAG
Z – DISCARDED

INSPECTOR J. MacLeod
LOGGED BY J. MacLeod

APPROVED D. H. Macdonald.
DATE July, 1960

Plate IVA

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

DRILLING REPORT

CLIENT Ontario Department of Highways — JOB No. 892

PROJECT WP 175-60 — HOLE No. 892-3

SITE Highway 401 and Forced Road, Lancaster Twp. — SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION, COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR FLUSH, ETC.	SAMPLE					PENETRATION TEST
			NO.	TYPE	SIZE	DEPTH	RET'D	Blows *
Feet					In.	Ft	In.	
			6	AQ	2	42.0		
						42.5		15
						43.0		12
						43.5	5	8
46.3		End of Hole						

* Penetration Test
The value given in the number of blows of a 140-lb weight falling freely 30 inches required to advance the standard split-spoon sampler 6 inches to the depth indicated.

Plate IVB

FORM NO. 91-B

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

DRILLING REPORT

CLIENT Ontario Department of Highways — JOB No. 892

PROJECT WP 175-60 — HOLE No. 892-4

SITE Highway 401 and Forced Road, Lancaster Twp. — SHEET No. 1 OF 2

CONTRACTOR: P.E. Johnston Drilling Company Limited — STARTED 9.45 A.M. June 30, 19 60 — FINISHED 5.00 P.M. July 5, 19 60

METHOD OF DRILLING: SOIL Modified Wash Boring — CASING DIAM. BX and AX

ROCK Diamond Drill — CORE DIAM. AXT

LOCATION: ~~LATITUDE~~ Chainage 463+46
DEPARTURE 200 Feet Right
BEARING
INITIAL DIP 90 Degrees
OTHER DIPS

ELEVATIONS: DATUM GSC
DRILL PLATFORM
GROUND SURFACE 162.0
ROCK SURFACE
BOTTOM OF HOLE 94.0
WATER TABLE

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO	SAMPLE TYPE *	SAMPLE SIZE	SAMPLE DEPTH	SAMPLE RET'D	PENETRATION TEST Blows *
Feet					In.	Ft	In.	
0	Silty Clay	Mottled, grey brown, stiff, with some small sand inclusions	1	BO	2	5.0		
						6.5	18	Driven
			2	BO	2	9.0		
10.0	Clay	Blue grey, homogeneous, soft to medium				10.5	18	Pushed
				Vane Test		12.2		
			3	BO	2	13.0		
						15.0	18	Pushed
				Vane Test		16.5		
			4	BO	2	19.0		
						20.5	18	Pushed
				Vane Test		22.0		
		Blending to pinkish grey and then blue grey again from 24 to 25 feet	5	BO	2	23.5		
						25.5	18	Pushed
				Vane Test		27.0		
			6	BO	2	27.5		
						29.0	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 — POLYFILM BAG
Z — DISCARDED

INSPECTOR J. Bateson

LOGGED BY J. MacLeod

APPROVED A. H. MacDonald

DATE July, 1960

Plate VA

FORM NO. B1-A

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

DRILLING REPORT

CLIENT: Ontario Department of Highways — JOB No. 892

PROJECT: WP 175-60 — HOLE No. 892-4

SITE: highway 401 and Forced Road, Lancaster Twp. — SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION, COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, ORIGIN, PACKNESS, WATER CONDITION, ETC.	SAMPLE NO.	SAMPLE TYPE	SAMPLE SIZE	SAMPLE DEPTH	PENETRATION REC.	PENETRATION TEST Blows *
Feet					In.	Ft	In.	
				Vane	Test	30.5		
			7	BO	2	32.0		
						33.5	18	Pushed
				Vane	Test	35.0		
			8	BO	2	36.0		
						37.5	18	Pushed
				Vane	Test	39.5		
40.0	Silt	Grey, soft, with some sand and clay	9	BO	2	40.0		
						42.0	18	Pushed
				Vane	Test	43.5		
			10	B	2	44.0		
45.5	Sand and Gravel	Grey, heterogeneous, containing silt and clay, medium density				45.6	Nil	Pushed
			11	AS	2	46.0		
						46.5		2
						47.0		2
						47.5	Nil	6
			12	FQ		52.0		
						54.0		
			13	FQ		54.0		
						58.0		
			14	FQ		58.0		
64.0	Boulders and Sand and Gravel	Succession of boulders separated by sand and gravel with some silt and clay				64.0		
66		End of Hole						

* Penetration Test
The value given in the number of blows of a 140-lb weight falling freely 30 inches required to advance the standard split-spoon sampler 6 inches to the depth indicated.

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

DRILLING REPORT

CLIENT: Ontario Department of Highways — JOB No.: 892

PROJECT: WP 175-60 — HOLE No.: 892-5

SITE: Highway 401 and Forced Road, Lancaster Twp. — SHEET No.: 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Company Limited
STARTED 8.00 A.M. June 30, 1960
FINISHED 5.30 A.M. July 4, 1960

METHOD OF DRILLING: SOIL Modified Wash Boring — CASING DIAM. BX and AX
ROCK Diamond Drill — CORE DIAM. AXT

LOCATION: ~~LATITUDE~~ Chainage 463+26
DEPARTURE 60 Feet Right
BEARING
INITIAL DIP 90 Degrees
OTHER DIPS

ELEVATIONS: DATUM GSC
DRILL PLATFORM
GROUND SURFACE 162.3
ROCK SURFACE
BOTTOM OF HOLE 112.5
WATER TABLE

DEPTH	SOIL TYPE	DESCRIPTION, COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC	SAMPLE NO	SAMPLE TYPE *	SAMPLE SIZE	SAMPLE DEPTH	SAMPLE RET'D	PENETRATION TEST Blows *
Feet					In.	Ft	In.	
0	Silty Clay	Mottled, grey brown, stiff	1	A2		4.0		Machine
						5.5	18	Pushed
9.5	Clay	Blue grey, homogeneous, soft to medium						
			2	B0	2	10.0		
						11.5	18	Pushed
				Vane	Test	13.0		
			3	B0	2	16.0		
						17.5	18	Pushed
				Vane	Test	19.0		
			4	B0	2	22.0		
						23.5	18	Pushed
				Vane	Test	25.0		
			5	B0	2	28.5		
						30.0	18	Pushed
				Vane	Test	31.5		
			6	B0	2	32.5		
						34.0	18	Pushed
				Vane	Test	35.5		

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 - POLYTHENE BAG
Z - DISCARDED

INSPECTOR J. MacLeod
LOGGED BY J. MacLeod
APPROVED D. H. MacDonald
DATE July, 1960

Plate V1A

FORM NO. R1-A

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

DRILLING REPORT

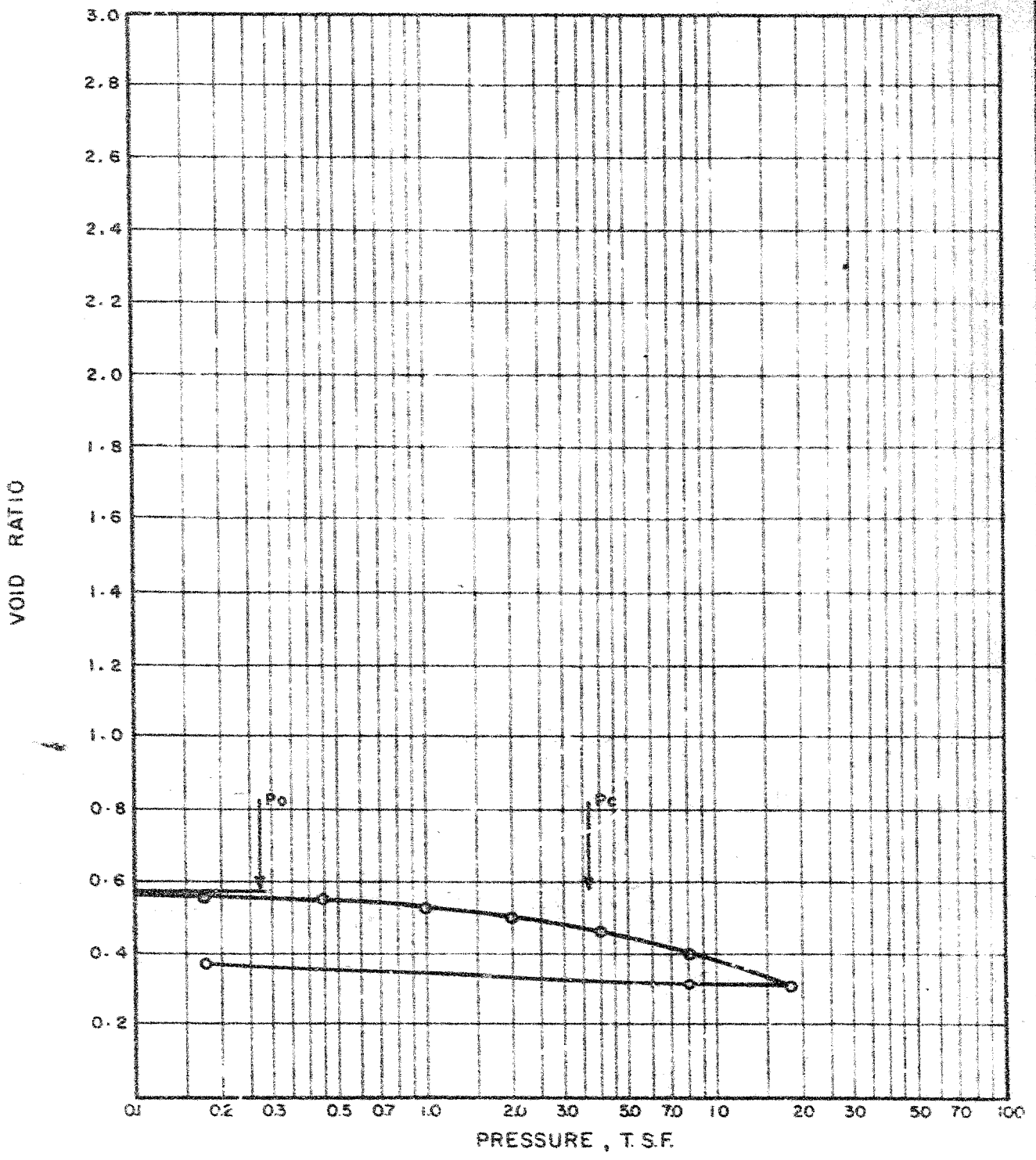
CLIENT Ontario Department of Highways — JOB No. 892

PROJECT WP 175-60 — HOLE No. 892-5

SITE Highway L01 and Forced Road, Lancaster Twp. — SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION COLOUR CONSISTENCY STRUCTURE WATER CONTENT PLASTICITY GRAIN SIZE ANGULARITY WATER LOSS OR GAIN ETC.	SAMPLE					PENETRATION TEST
			NO.	TYPE	SIZE	DEPTH	RET'D	Blows *
Feet					In.	Ft	In.	
37.0	Sand and Gravel	Grey, heterogeneous, with some silt and clay, medium density containing scattered boulders	7	AQ		39.0		
						39.5		4
						40.0		2
						40.5	6	5
			8	AQ		45.0		
						45.5		4
						46.0		6
						46.5	7	4
49.8		End of Hole						

* Penetration Test
The value given in the number of blows of a 140-lb weight falling freely 30 inches required to advance the standard split-spoon sampler 6 inches to the depth indicated.



OVERBURDEN PRESSURE — P_o = 0.28 TSF

CONSOLIDATION PRESSURE — P_c = 3.65 TSF

NATURAL WATER CONTENT 39.9%

LOADING INTERVAL 100% PRIMARY CONSOLIDATION

SAMPLE No. 892-E0-1

TEST No. 892-9-4

TEST DATE AUGUST 10, 1960

TESTED BY B.H.

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

CONSOLIDATION TEST

HOLE No. 892-4 SAMPLE ELEV 156.0'

ONTARIO DEPARTMENT OF HIGHWAYS

APPROVED

D. H. Rochdale

H.G. ACRES & COMPANY LTD.

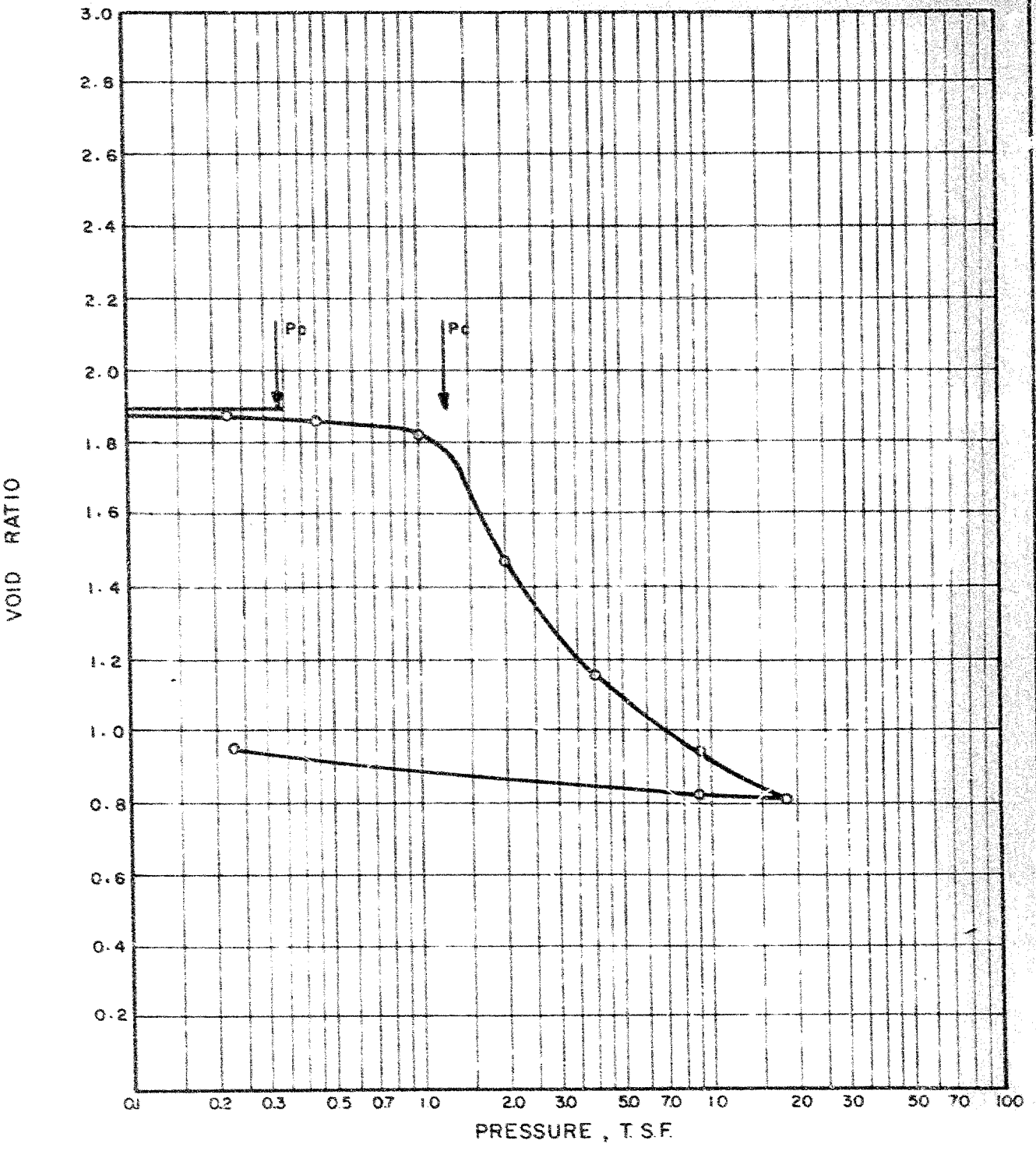
DATE AUGUST, 1960

JOB No. 892

WP 175-60

PLATE - VI

SS-892-1 & 7



OVERBURDEN PRESSURE — P_o = 0.34 TSF

CONSOLIDATION PRESSURE — P_c = 1.25 TSF

NATURAL WATER CONTENT 68.7 %

LOADING INTERVAL 100% PRIMARY CONSOLIDATION

SAMPLE No. 892-BO-2

TEST No. 892-9-2

TEST DATE AUGUST 4, 1960

TESTED BY R.L.

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA F... S CANADA

ONTARIO DEPARTMENT OF HIGHWAYS

W P 175 - 60

CONSOLIDATION TEST

HOLE No. 892 - 1 SAMPLE ELEV. 152.0'

APPROVED

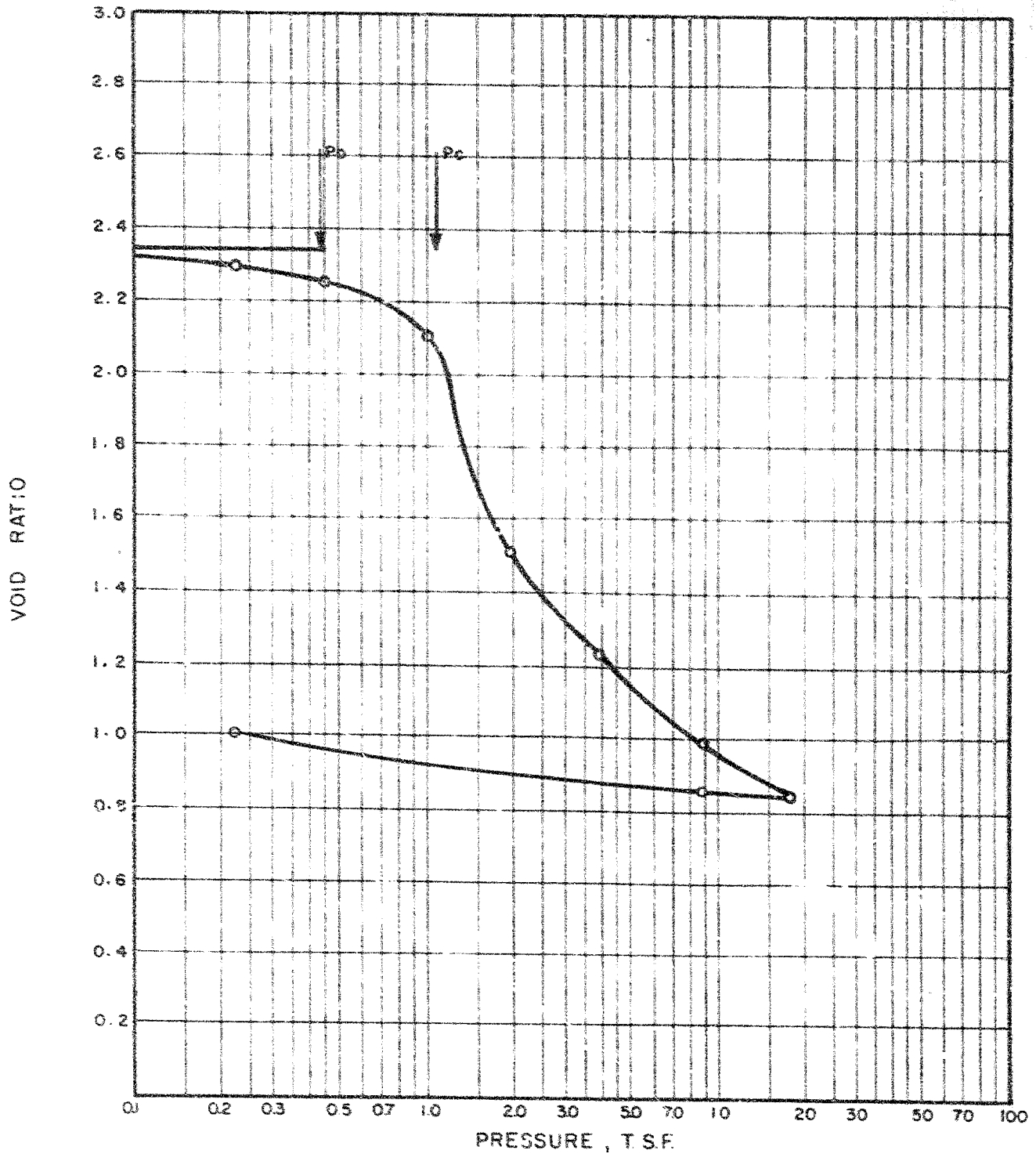
H.G ACRES & COMPANY LTD.

DATE AUGUST, 1960

JOB No. 892

PLATE - VIII

GK-892-LS-B



OVERBURDEN PRESSURE — P_o = 0.14 TSF
CONSOLIDATION PRESSURE — P_c = 1.09 TSF

NATURAL WATER CONTENT 81.9 %
LOADING INTERVAL 100% PRIMARY CONSOLIDATION

SAMPLE No 892-BO-3
TEST No 892-9-1

TEST DATE JULY 29, 1960
TESTED BY R.G

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

ONTARIO DEPARTMENT OF HIGHWAYS

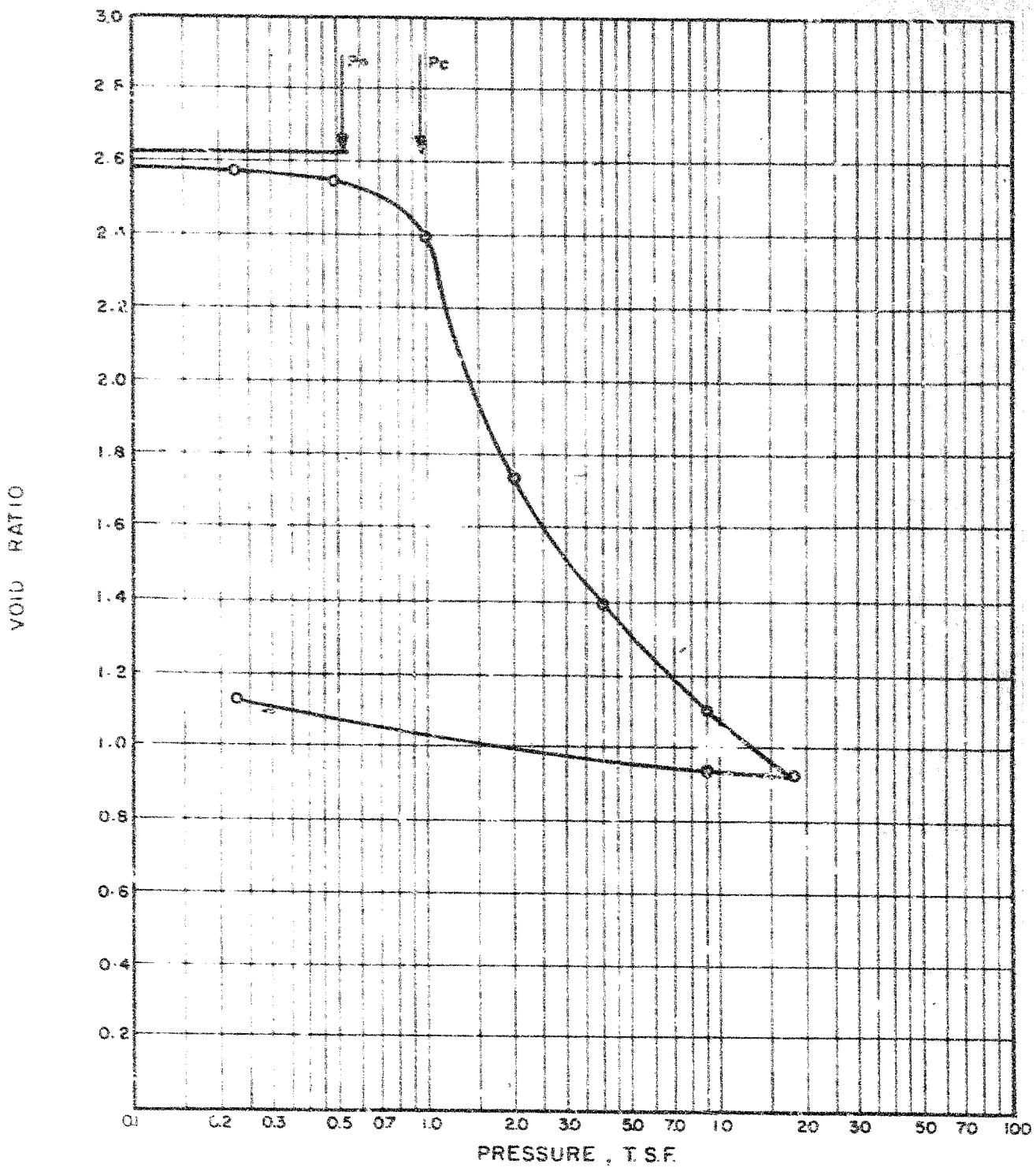
W P 175-60

CONSOLIDATION TEST

HOLE No. 892-1 SAMPLE ELEV. 147.0'

APPROVED
H.G. ACRES & COMPANY LTD.

DATE: AUGUST, 1960
JOB No. 892
PLATE - IX



OVERBURDEN PRESSURE — P_o = 0.53 TSF

CONSOLIDATION PRESSURE — P_c = 0.96 TSF

NATURAL WATER CONTENT 93.9 %

LOADING INTERVAL 100% PRIMARY CONSOLIDATION

SAMPLE No. 892-BO-4

TEST No 892-9-3

TEST DATE AUGUST 5, 1960

TESTED BY R.L.

H G ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS CANADA

ONTARIO DEPARTMENT OF HIGHWAYS

W P 175-60

CONSOLIDATION TEST

HOLE No. 892-1 SAMPLE ELEV. 142.0'

APPROVED

D. H. Lockwood

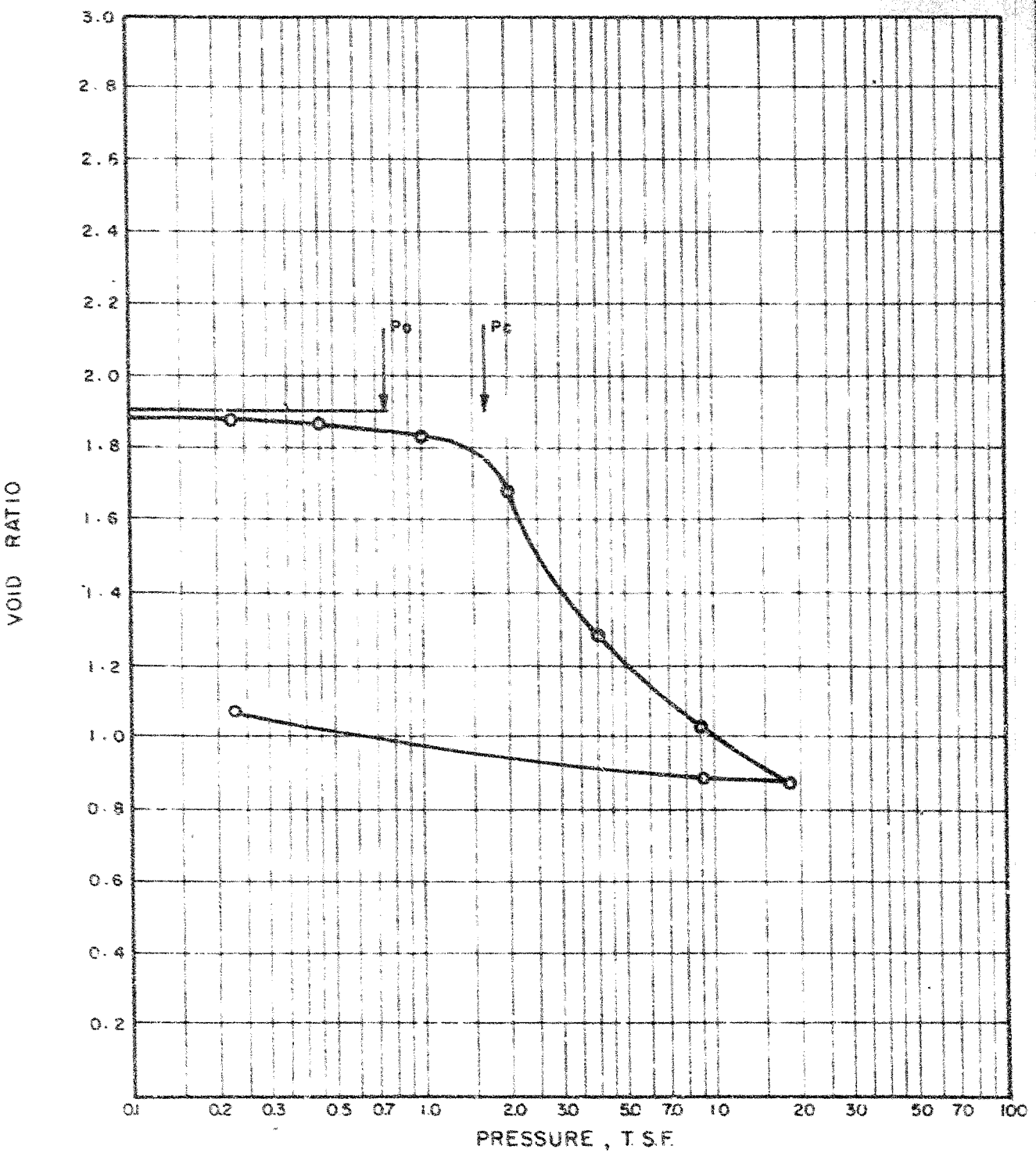
H G ACRES & COMPANY LTD.

DATE: AUGUST, 1960

JOB No. 892

PLATE - X

SK-892-LS-10



OVERBURDEN PRESSURE — P_o = 0.76 TSF

CONSOLIDATION PRESSURE — P_c = 1.73 TSF

NATURAL WATER CONTENT 68.8 %

LOADING INTERVAL 100% PRIMARY CONSOLIDATION

SAMPLE No. 892-80-7

TEST No. 892-9-5

TEST DATE AUGUST 11, 1960

TESTED BY R.G.

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

ONTARIO DEPARTMENT OF HIGHWAYS

WP 175-60

CONSOLIDATION TEST

HOLE No. 892-4 SAMPLE ELEV. 129.0'

APPROVED

D. H. Macdonald

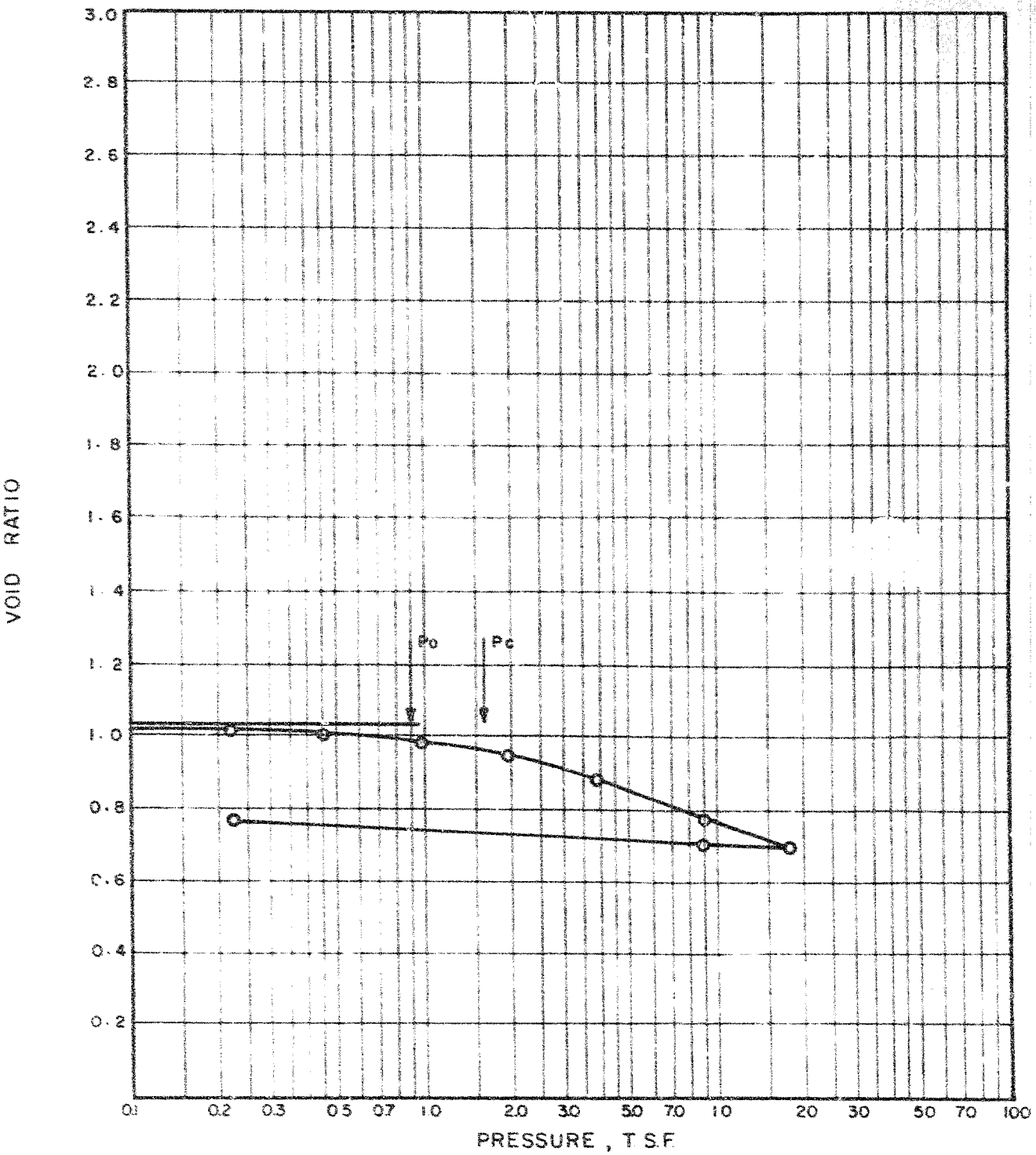
H.G. ACRES & COMPANY LTD.

DATE: AUGUST, 1960

JOB No. 892

PLATE - XI

SK-892-LS-11



OVERBURDEN PRESSURE — P_o = 0.89 TSF

CONSOLIDATION PRESSURE — P_c = 1.61 TSF

NATURAL WATER CONTENT 39.6 %

LOADING INTERVAL 100% PRIMARY CONSOLIDATION

SAMPLE No 892-80-9

TEST No 892-9-6

TEST DATE AUGUST 29, 1960

TESTED BY B.H.

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

ONTARIO DEPARTMENT OF HIGHWAYS

WP 175 - 60

CONSOLIDATION TEST

HOLE No. 892-4 SAMPLE ELEV. 122.0'

APPROVED

D. H. Mackenzie

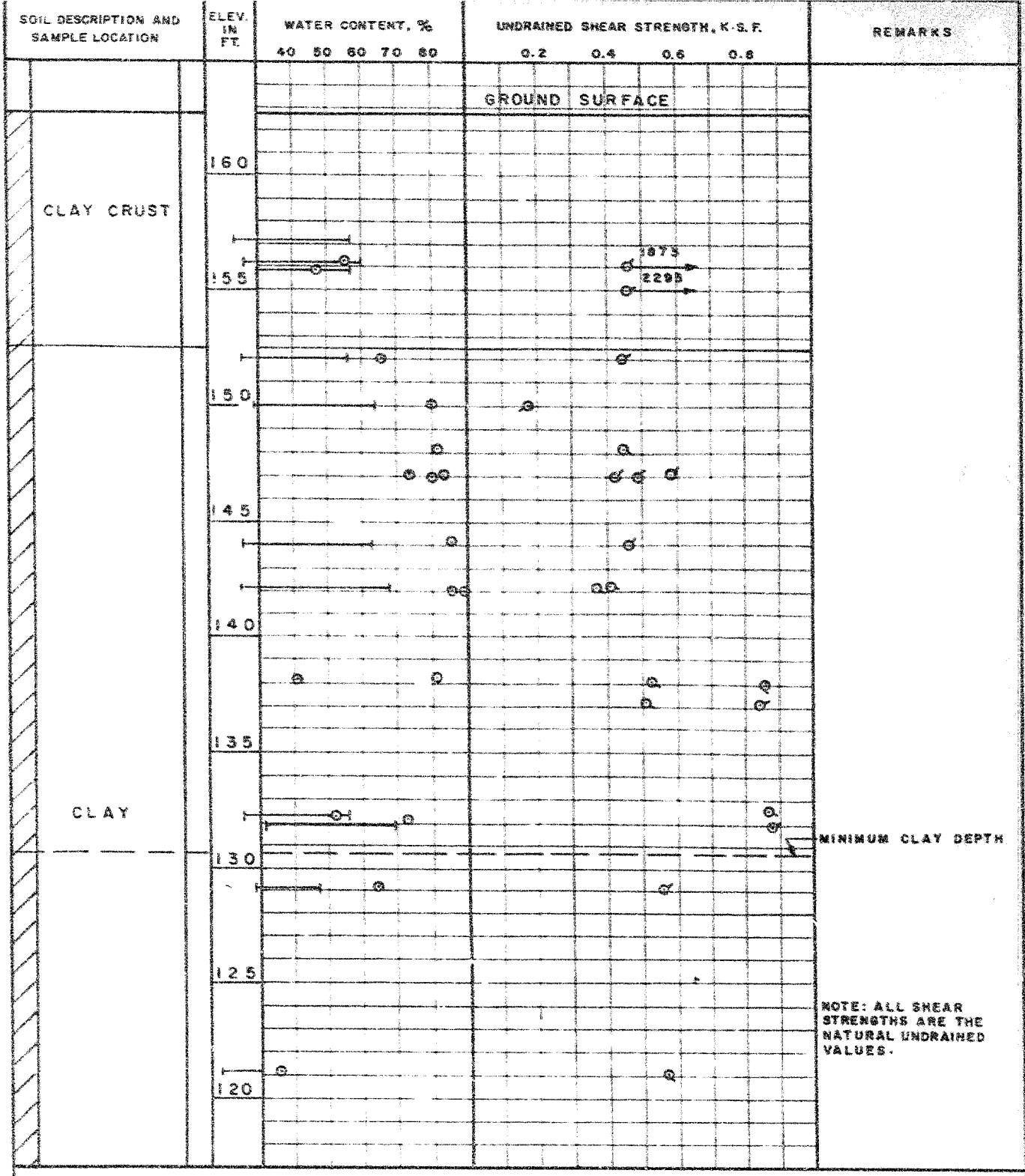
H G ACRES & COMPANY LTD.

DATE. AUGUST, 1960

JOB No. 892

PLATE - XII

SK-892-LS-12



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

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

ONTARIO DEPARTMENT OF HIGHWAYS

WP 175 - 60

SUMMARY OF DRILLING AND TEST RESULTS

LABORATORY TESTS

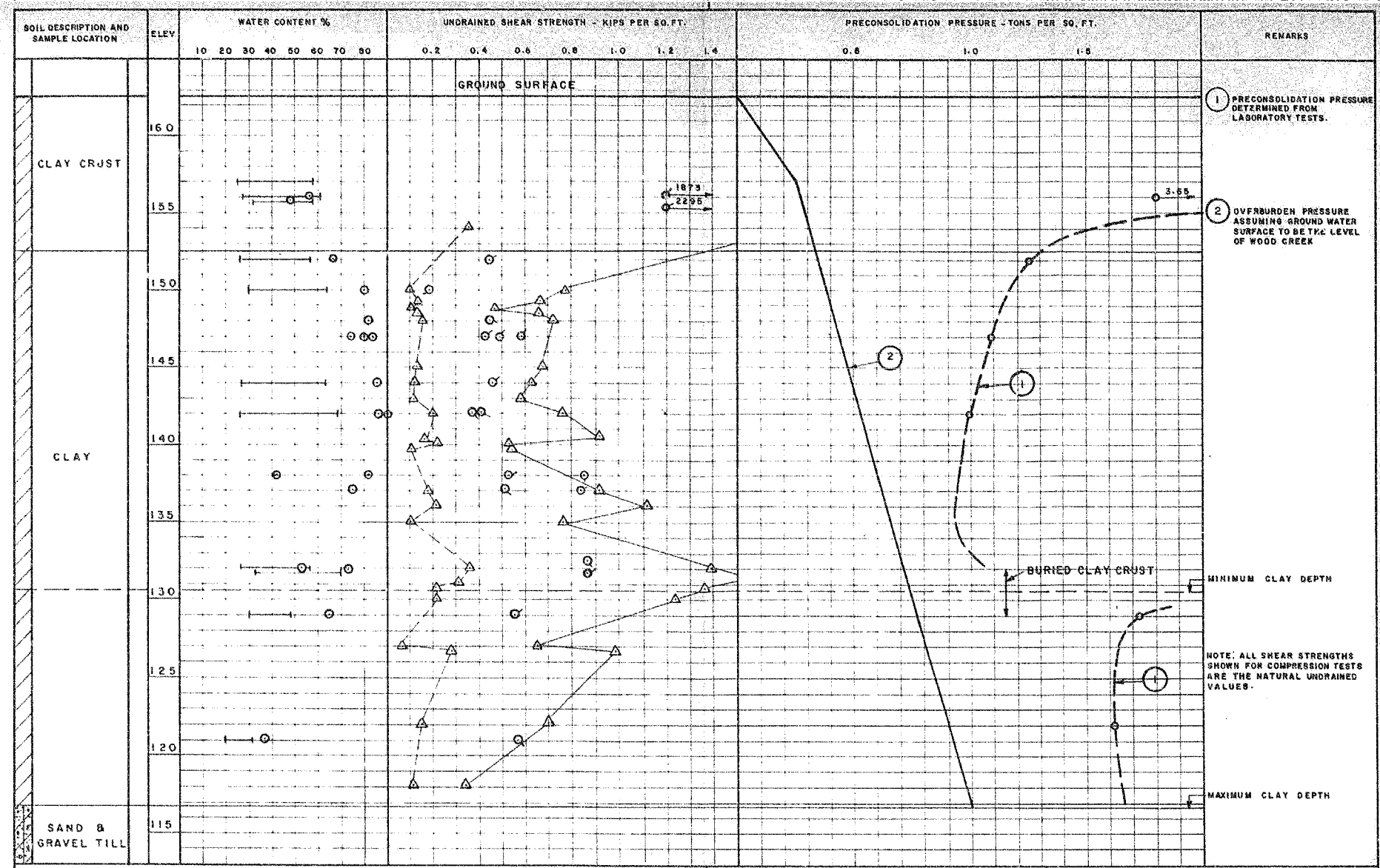
APPROVED

H.G. ACRES & COMPANY LTD.

DATE AUGUST, 1960

JOB No 892

PLATE - XIII



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

ONTARIO DEPARTMENT OF HIGHWAYS

WP 175-60

SUMMARY OF DRILLING AND TEST RESULTS

COMPARISON OF ALL TESTS

APPROVED

H.G. ACRES & COMPANY LIMITED

DATE: AUGUST, 1960

JOB No. 892

PLATE - XIV

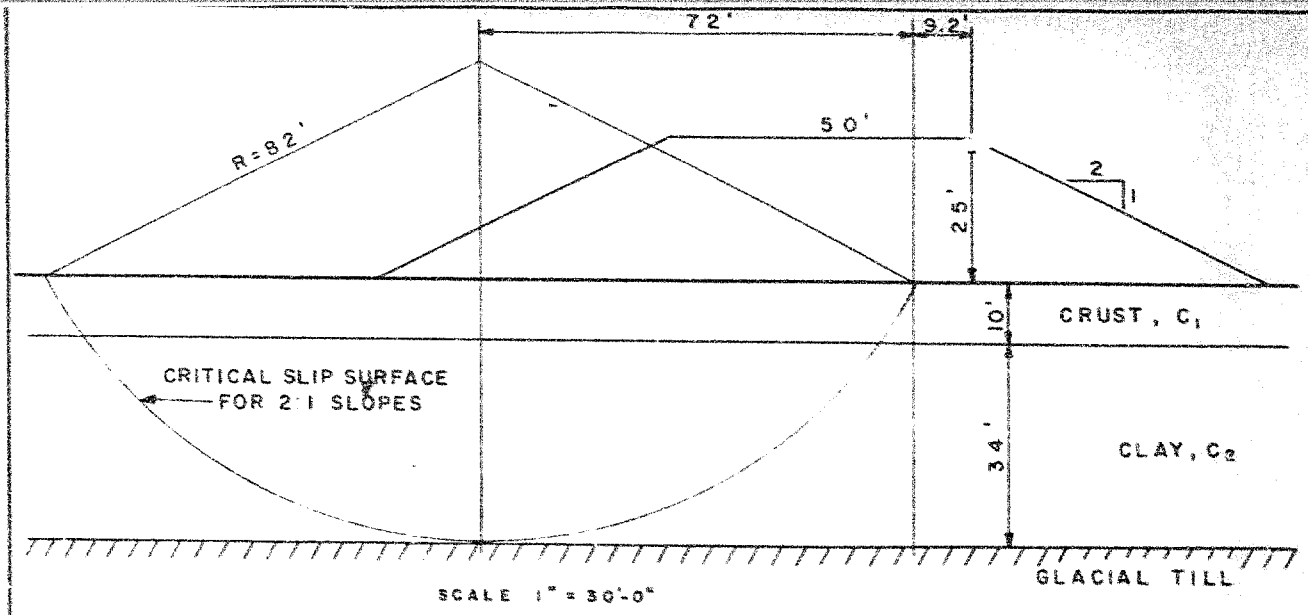


TABLE I

FACTOR OF SAFETY FOR 2:1 SLOPES

C_1 (PSF)	550	680	2000	2000	1300
C_2 (PSF)	550	680	550	680	680
FACTOR OF SAFETY	1.04	1.28	1.38	1.59	1.42

TABLE II

FACTOR OF SAFETY FOR 3:1 SLOPES

C_1 (PSF)	550	680	1300
C_2 (PSF)	550	680	680
FACTOR OF SAFETY	1.10	1.36	1.53

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

DEPARTMENT OF HIGHWAYS OF ONTARIO

WP 175-60

SUMMARY OF STABILITY ANALYSES FOR THE CASE OF AN EMBANKMENT WITHOUT BERMS

APPROVED

D. H. MacDonald

H.G.ACRES & COMPANY LIMITED

DATE: AUGUST, 1960

SCALE

JOB No. 892

PLATE - XV

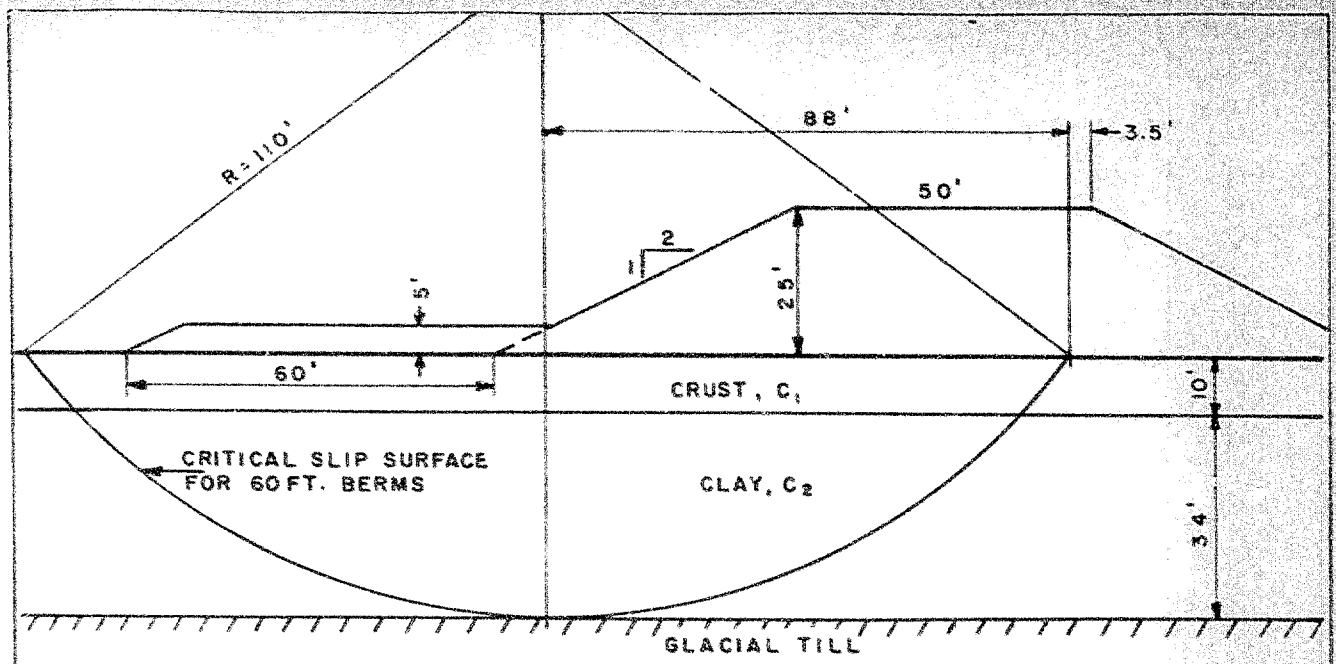


TABLE III

BERM LENGTH	60'	60'	40'	40'
C_1 (PSF)	680	1300	680	1300
C_2 (PSF)	680	680	680	680
FACTOR OF SAFETY	1.49	1.67	1.32	1.48

NOTE: THE DEPTH OF BERM PROPOSED IS THAT REQUIRED TO FORCE FAILURE SURFACE TO CUT BEYOND THE END OF THE BERM. DEEPER BERMS CAN BE USED BUT ARE LESS EFFICIENT.

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

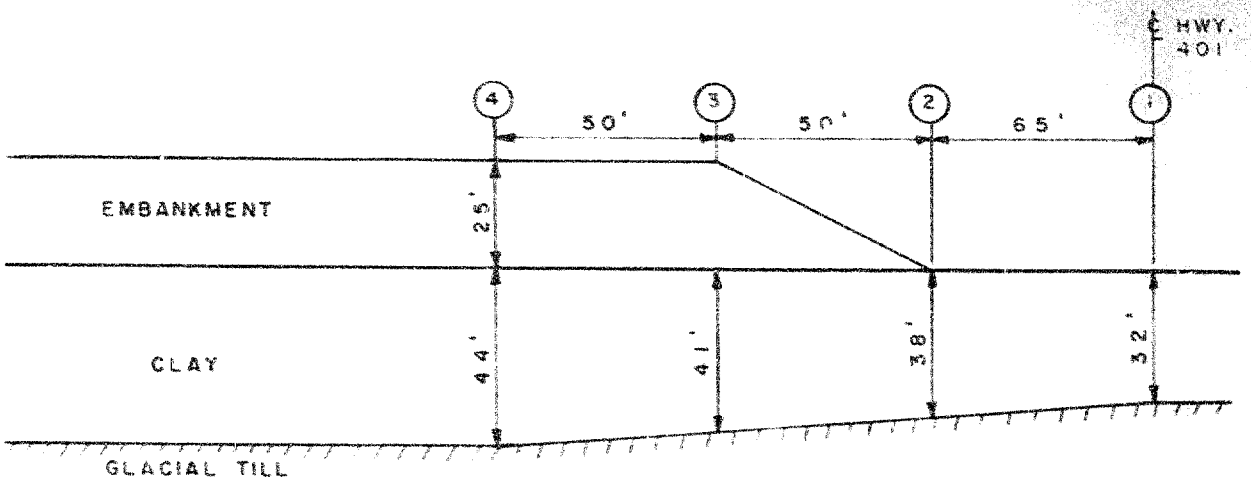
ONTARIO DEPARTMENT OF HIGHWAYS

WP 175-60

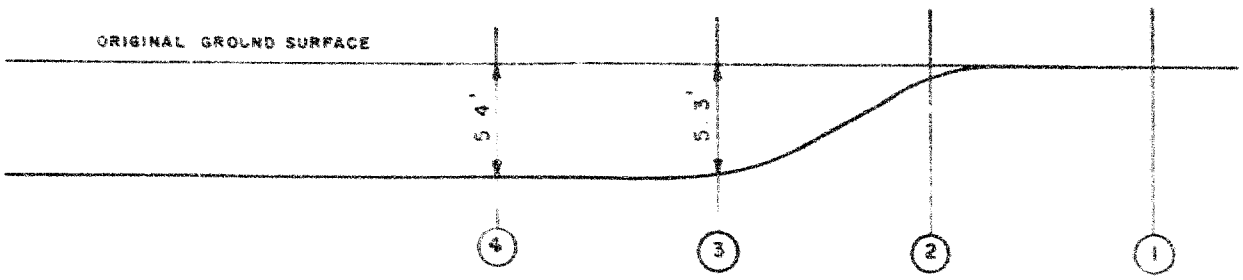
SUMMARY OF STABILITY ANALYSES FOR THE CASE OF AN EMBANKMENT WITH BERMS

APPROVED	DATE: AUGUST, 1960
	SCALE / JOB No. 892
H.G ACRES & COMPANY LIMITED	PLATE - XVI

SK-892-LS-16



SECTION ALONG ℄ ROAD ALLOWANCE



ELASTIC SETTLEMENT IN FEET	0.4	0.4	0	0
CONSOLIDATION SETTLEMENT IN FEET	5.0	4.9	0.2	0
ULTIMATE SETTLEMENT IN FEET	5.4	5.3	0.2	0

RESULTS OF SETTLEMENT PREDICTIONS

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

ONTARIO DEPARTMENT OF HIGHWAYS

WP 175 - 60

FOUNDATION SETTLEMENTS DUE TO EMBANKMENT LOAD

APPROVED

H. G. ACRES & COMPANY LIMITED

DATE. AUGUST, 1960

SCALE —

JOB No 892

PLATE - XVII

SK-892-LS-17

#60-F-210

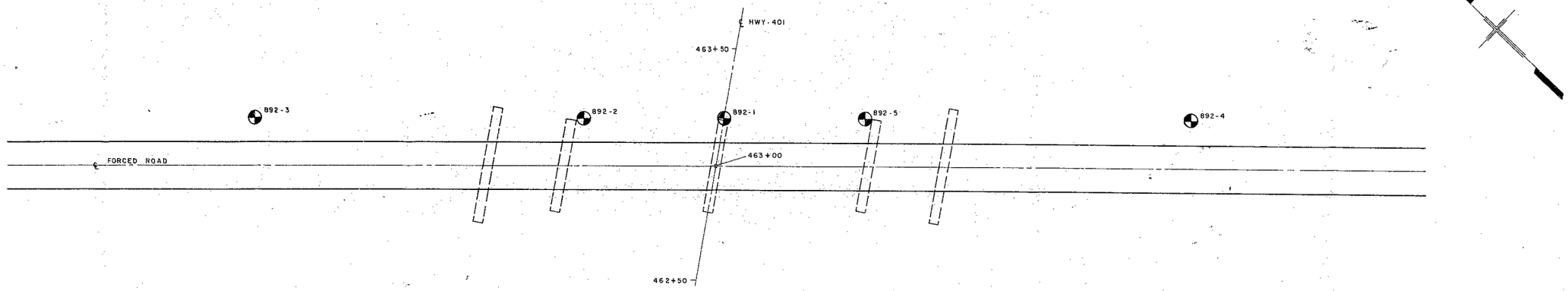
W.P.# 175-60

Hwy. # 401

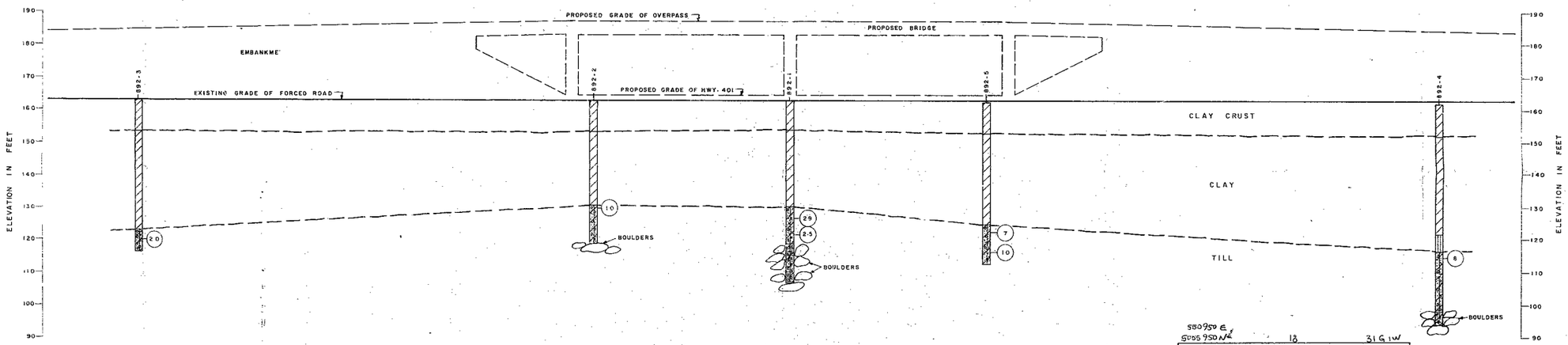
PROP. CROSSING

CON. # 1

LANCASTER



PLAN
SCALE: 1"= 20'-0"



SECTION ALONG ℄ OF FORCED ROAD
SCALE: 1"= 15'-0"

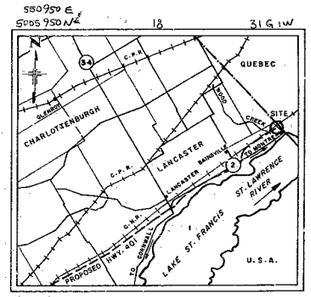
LEGEND

- ORGANIC SOIL
- CLAY
- SILT
- SAND
- GRAVEL
- BEDROCK

▽ WATER TABLE

EXPLORATORY DRILL HOLE

(20) BLOWS PER FOOT FOR STANDARD PENETRATION TEST



KEY PLAN
SCALE: 1 IN. = 4 MI.

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

ONTARIO DEPARTMENT OF HIGHWAYS

WP-175-60

EXPLORATORY HOLES
PLAN AND SECTION

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

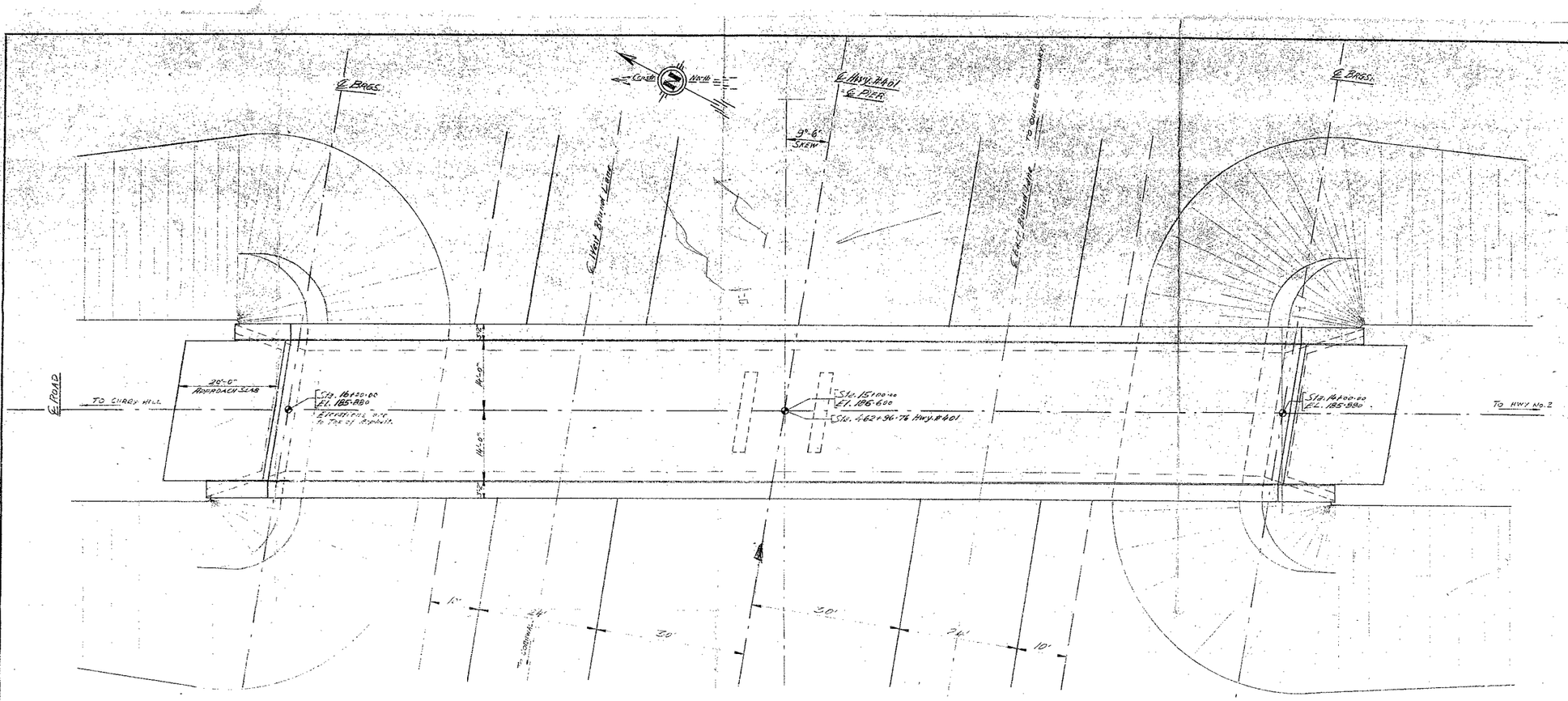
DATE: AUGUST, 1960

SCALE AS NOTED

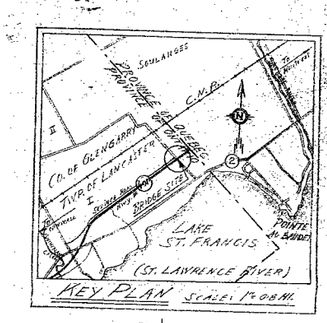
JOB No. 832

PLATE-I

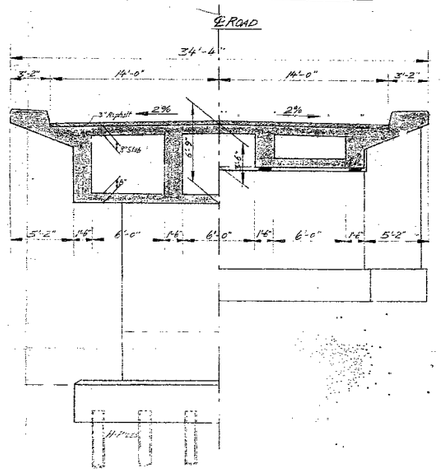
SK-892-C-2



PLAN SCALE: 1"=10'-0"

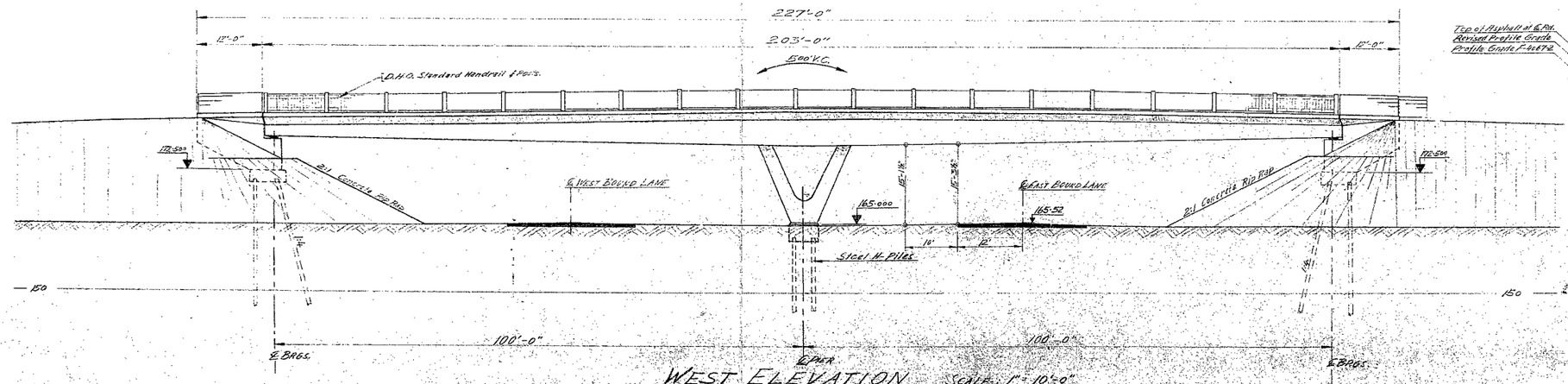


KEY PLAN SCALE: 1"=0.8 MI.

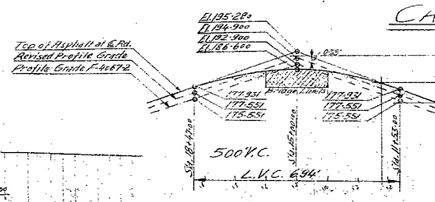


AT PIER AT ABUTMENT

CROSS SECTION SCALE: 1"=5'-0"



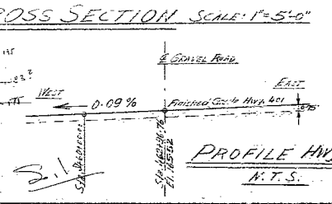
WEST ELEVATION SCALE: 1"=10'-0"



VERTICAL CURVE DIAGRAM N.T.S.

[2'-0" Raise in Grade proposed by the Bridge Office Nov. 18, 1960]

NOTE: APPROACH FILL TO BE PLACED WELL AHEAD OF THE CONSTRUCTION OF THE STRUCTURE AS RECOMMENDED BY THE SOILS ENGINEER.



PROFILE HWY. #401 N.T.S.

W.P. 175-60

DEPARTMENT OF HIGHWAYS:-ONTARIO-

BRIDGE OFFICE:- TORONTO

Apvd. '60

LANCASTER TWP. BRIDGE No 15

THE KING'S HIGHWAY No. 401 DIST. No. 9

CO. GLENGARRY

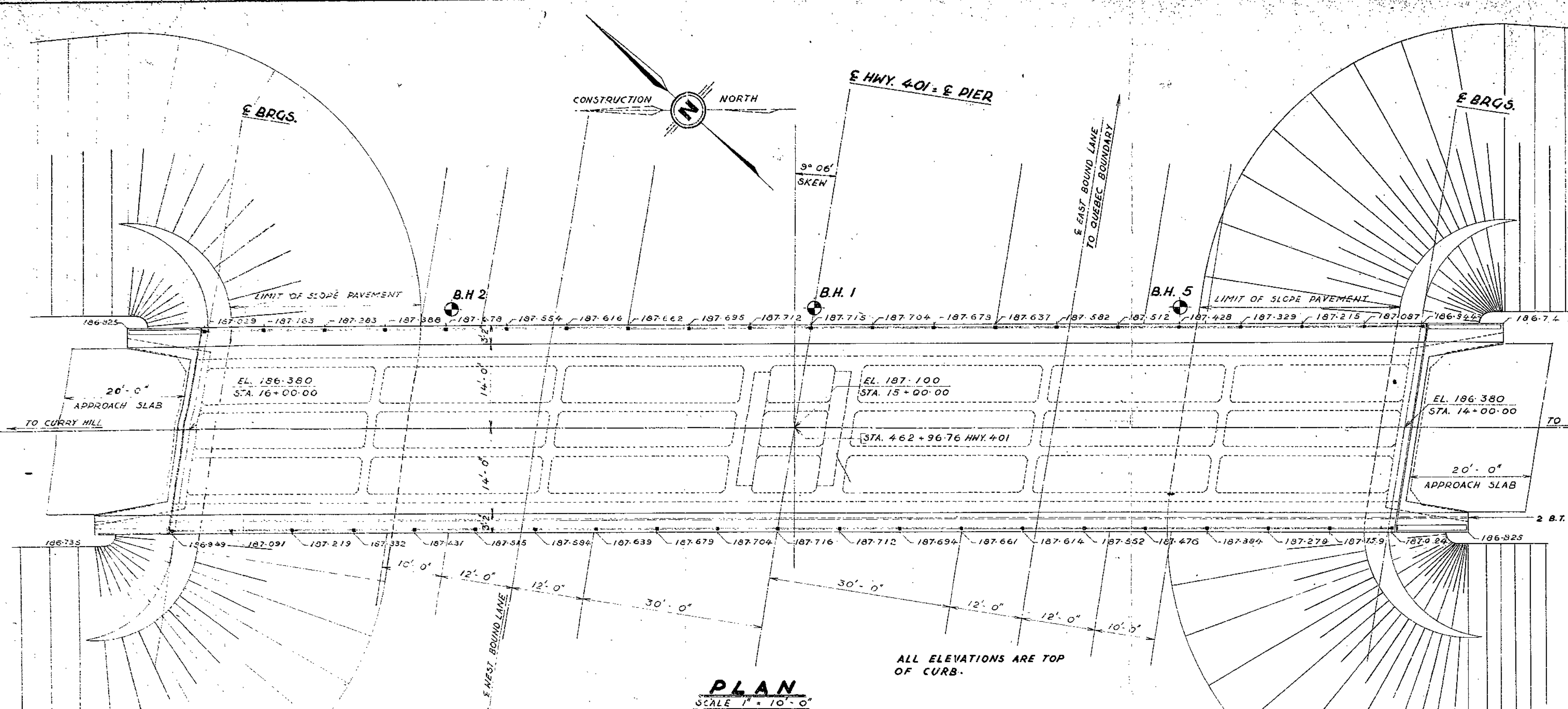
TWP. LANCASTER LOT 182 CON. I

PRELIMINARY PLAN

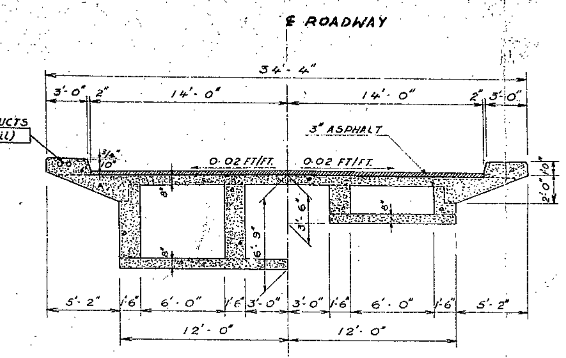
APPROVED

	BRIDGE ENGINEER		DESIGN ENGINEER
DESIGN	W.L.L.	CHECK	CONTRACT NUMBERS
DRAWING	K.B.J.	CHECK	
TRACING		CHECK	LOADING 1020 S16
DATE	NOV. 1960		DRAWING NUMBER D-4652-P

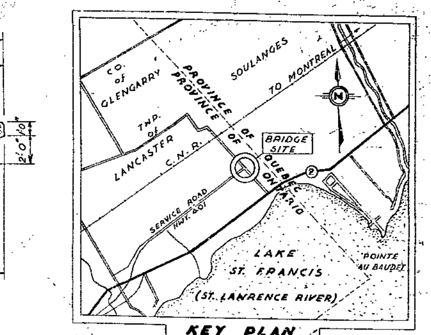
REVISIONS	DATE	BY	DESCRIPTION	REFERENCE PLANS
				L-3182-1
				F-40087-2
				1835
				1137



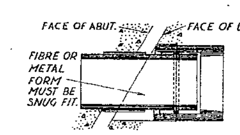
PLAN
SCALE 1" = 10'-0"



SECTION THROUGH GIRDER AT ℄ OF PIER **SECTION THROUGH GIRDER AT ABUTMENT**
SCALE 3/16" = 1'-0"

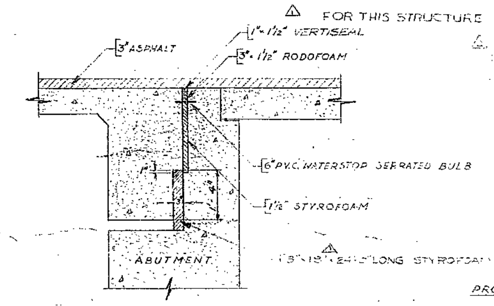


KEY PLAN
SCALE 1" = 0·8 MI.



DETAIL OF FIBRE EXPANSION JOINT CONCRETE ENCASEMENT TYPE FOR BELL TELEPHONE CONDUIT.

DUCTS SUPPLIED BY THE B.T. CO AND TO BE PLACED BY GENERAL CONTRACTOR.



EXPANSION JOINT DETAIL
SCALE ½" = 1'-0"

NOTES

TO DISTRICT ENGINEER :

CONCRETE WORK ON THIS STRUCTURE MUST NOT BE COMMENCED UNTIL MONUMENTS TO FIX CONTROL POINTS HAVE BEEN ERECTED AND CHECKED BY THE DISTRICT ENGINEER.

TO CONTRACTOR :

STRUCTURE TO BE BUILT IN ACCORDANCE WITH FORM Nº 9 AND THE SPECIAL PROVISIONS, EXTRA COPIES OF WHICH MAY BE OBTAINED FROM THE DISTRICT ENGINEER.

CONCRETE MIX	MINIMUM STRENGTH AT 28 DAYS	MAXIMUM SIZE OF AGGREGATE
THROUGHOUT	3000 P.S.I.	3/4"

APPROVED ADMIXTURES SUPPLIED BY THE CONTRACTOR WILL BE ADDED TO ALL CONCRETE AS SPECIFIED BY THE ENGINEER.

BORING DATA :

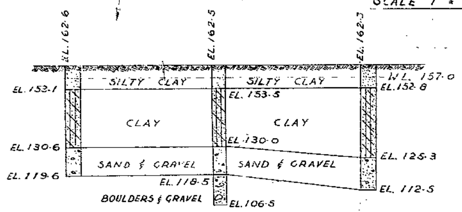
THE COMPLETE SOIL INVESTIGATION REPORT MAY BE EXAMINED AT THE BRIDGE OFFICE, DOWNSVIEW. ... REGIONAL ... GUARANTEE THE ACCURACY OF THIS ... OF THE ABRIDGED VERSION SHOWN ON THESE PLANS.

CLEAR COVER ON REINFORCING STEEL :

FOOTINGS : 3" PIER : 2"
ABUTMENT : 3"
DECK : 1"
HANDRAILS : 1½"

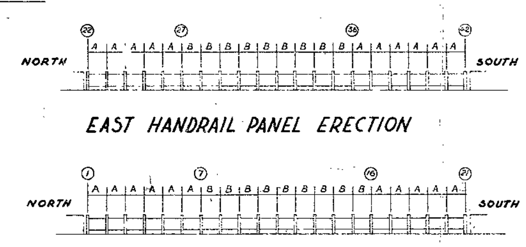
CONSTRUCTION NOTES :

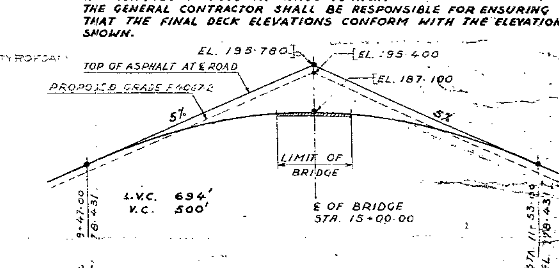
ALL EXPOSED EDGES TO BE CHAMFERED 1"×1" EXCEPT AS NOTED.
ALL CONSTRUCTION JOINTS MUST BE APPROVED BY THE BRIDGE ENGINEER. THE GENERAL CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BRIDGE SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF PLUS OR MINUS 1/8 INCH.
THE GENERAL CONTRACTOR SHALL BE RESPONSIBLE FOR ENSURING THAT THE FINAL DECK ELEVATIONS CONFORM WITH THE ELEVATIONS SHOWN.



SECTION OF BOREHOLES

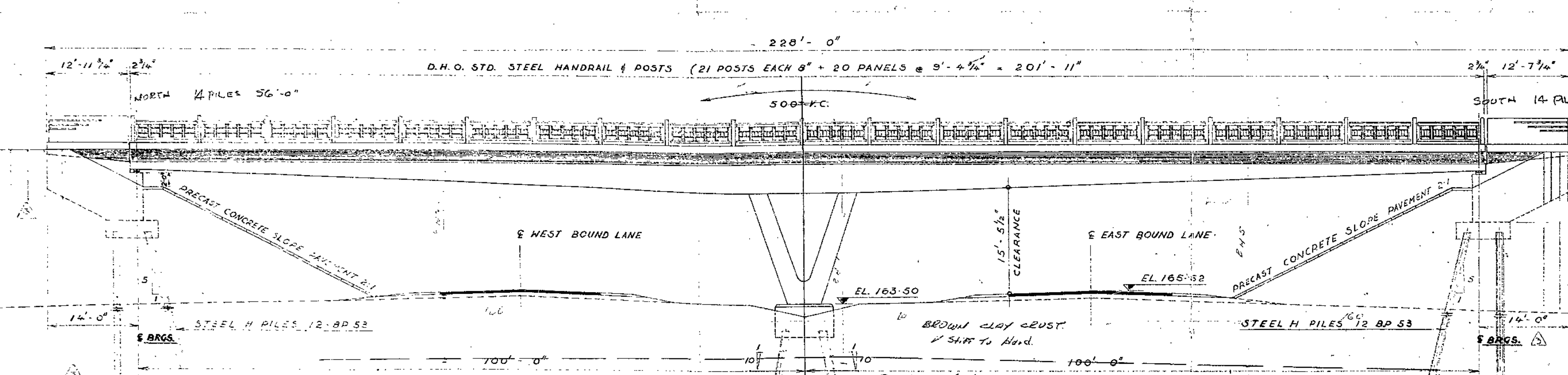
SKEW ANGLE DATA α = 9° 06'	
SIN. α =	0·1581581
COS. α =	0·9874138
TAN. α =	0·1601740





VERTICAL CURVE DIAGRAM
(NOT TO SCALE)

B.M. ELEVATION = 163·15
Geodetic Datum:
N. & W. IN N/W. ROOT OF 3'0 OAK
216'0 RT. OF STA. 458+66 HWY Nº 401 LINE 'A'



WEST ELEVATION
SCALE 1" = 10'-0"

LIST OF DRAWINGS	
D4652-1	GENERAL PLAN
D4652-2	FOOTINGS
D4652-3	DECK PLAN - DIMENSIONS
D4652-4	DECK REINFORCEMENT
D4652-5	ABUTMENT - APPROACH SLAB
D4652-6	WING WALLS & BRG DET.S
D4652-7	PIER - GIRDER & DIAPHRAGMS
[illegible]	[illegible]
D4652-9	CONCRETE RIP-RAP
D4652-10	STEEL HANDRAIL PANELS
D4652-10	INTERIOR STEEL POSTS
D4652-10	STEEL END POSTS
D4652-11	REINFORCING STEEL
D4652-12	REINFORCING STEEL

REVISIONS: DATE	BY	DESCRIPTION	REFERENCE PLANS
OCT 1/62		DELETE BRG DETS D4652-2 - ADD TO D4652-6 ④	E3782-1
MAR 6/63	JGG	OMIT - NOTE - EXISTING FILL ELEVATION ③	F4067-2
MAR 6/62	JGG	NOTE CHANGED (BORING DATA) ②	7833
27-2-62	JGG	△ EXPANSION JOINT FILLERS REVISED	8A 1137

W.P. 175-60

DEPARTMENT OF HIGHWAYS:-ONTARIO-
BRIDGE OFFICE: - TORONTO

TWP. RD. UNDERPASS
(0·2 MI. WEST OF QUEBEC BDRY.)

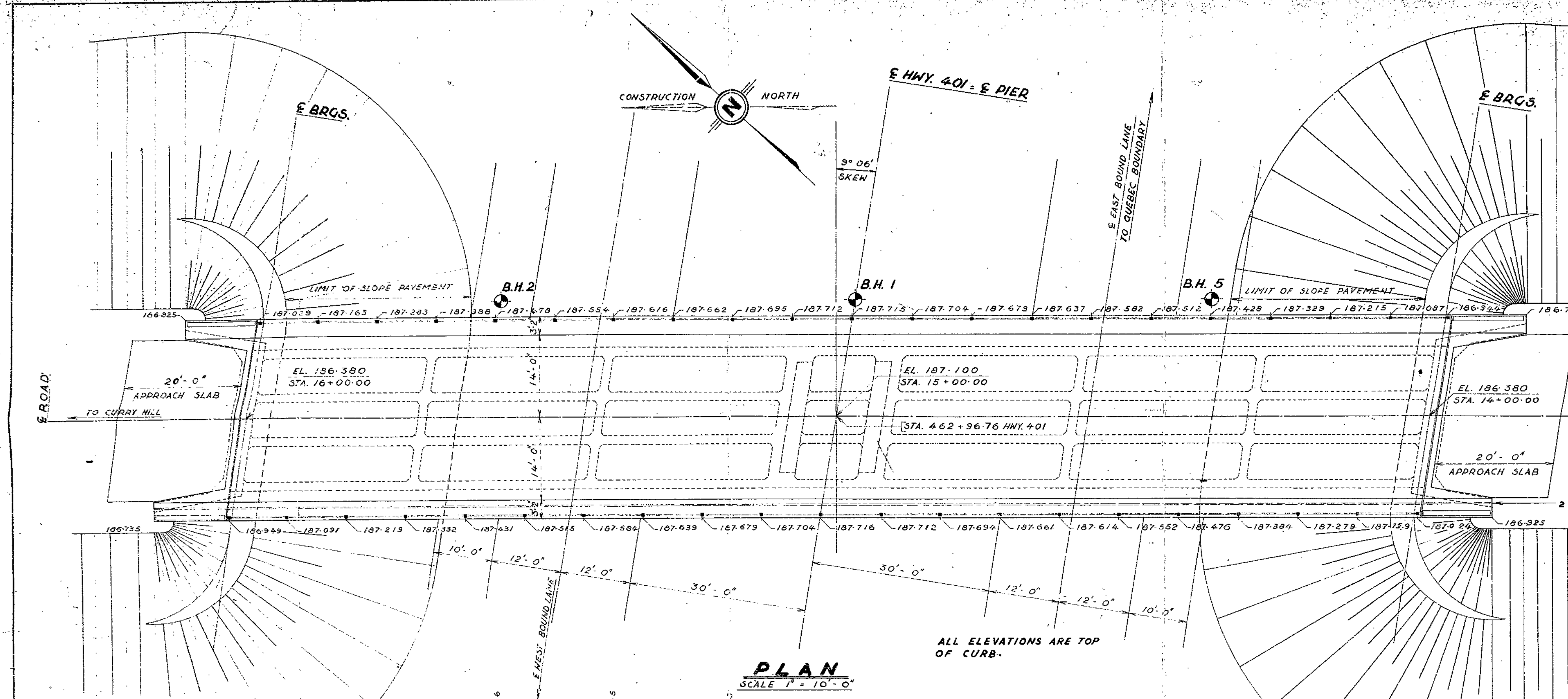
THE KING'S HIGHWAY NO. 401 DIST. NO. 9
CO. GLENGARRY LANCASTER TWP. BRIDGE No. [illegible]
TWP. LANCASTER LOT 1 & 2 CON. 1

GENERAL PLAN

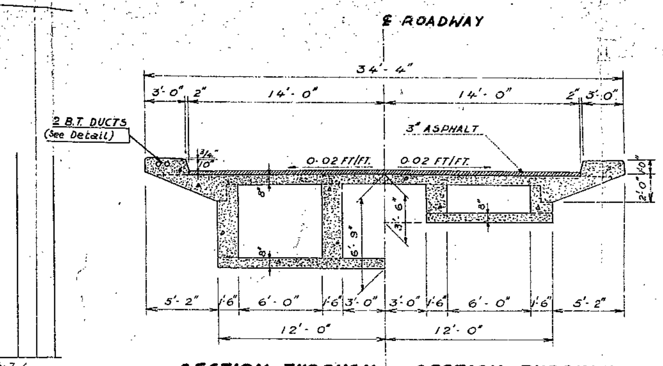
APPROVED
BRIDGE ENGINEER DESIGN ENGINEER

DESIGN	[illegible]	CHECK	D.G.	CONTRACT NUMBERS	
DRAWING	V.D.P.	CHECK	D.G.		
TRACING		CHECK		LOADING	DRAWING NUMBER D-4652-1
DATE		JUNE	1961	H20-S16	

②

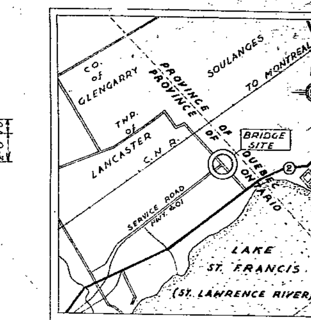


PLAN
SCALE 1" = 10'-0"

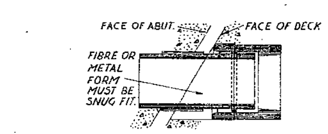


SECTION THROUGH GIRDER AT ℄ OF PIER | SECTION THROUGH GIRDER AT ABUTMENT

SCALE 3/16" = 1'-0"

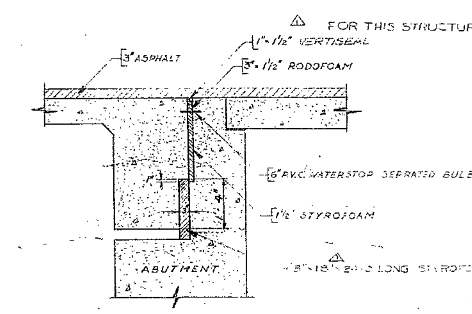


KEY PLAN
SCALE 1" = 0·8 MI.



DETAIL OF FIBRE EXPANSION JOINT CONCRETE ENCASEMENT TYPE FOR BELL TELEPHONE CONDUIT.

DUCTS SUPPLIED BY THE B.T.CO AND TO BE PLACED BY GENERAL CONTRACTOR.



EXPANSION JOINT DETAIL
SCALE 1/2" = 1'-0"

NOTES

TO DISTRICT ENGINEER :
CONCRETE WORK ON THIS S... MUST NOT BE COMMENCED UNTIL MONUMENTS TO FIX... POINTS HAVE BEEN ERECTED AND CHECKED BY THE ... ENGINEER.

TO CONTRACTOR :
STRUCTURE TO BE BUILT IN ACCO... FORM N° 9 AND THE SPECIAL PROVISIONS, EXTRA... WHICH MAY BE OBTAINED FROM THE DISTRICT ENG...

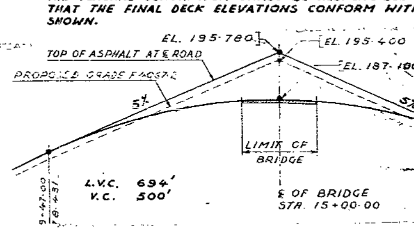
CONCRETE MIX	MINIMUM STRENGTH AT 28 DAYS	MAXIMUM ... OF AGGREG...
THROUGHOUT	3000 P.S.I.	3/4...

APPROVED ADMIXTURES SUPPLIED BY THE CONTRACTOR ... ADDED TO ALL CONCRETE AS SPECIFIED BY THE ENGI...

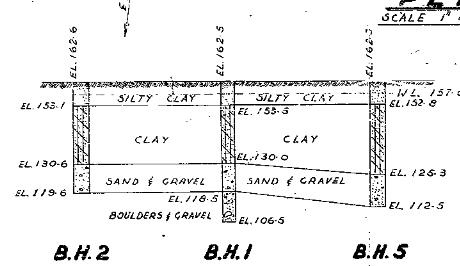
BORING DATA :
THE COMPLETE SOIL INVESTIGATION ... MAY BE EXAMINED AT THE BRIDGE OFFICE, DOWNSVIEW ...

CLEAR COVER ON REINFORCING STEEL :
FOOTINGS : 3" PIER : 2"
ABUTMENT : 3"
DECK : 1"
HANDRAILS : 1½"

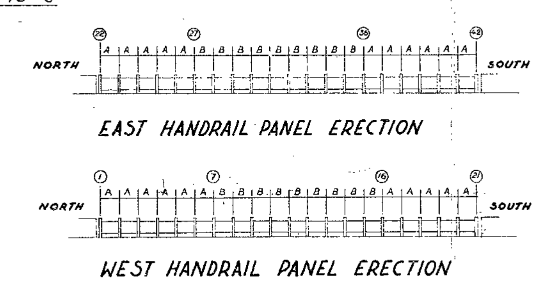
CONSTRUCTION NOTES :
ALL EXPOSED EDGES TO BE CHAMFERED 1"×1" EXCEPT AS ... ALL CONSTRUCTION JOINTS MUST BE APPROVED BY T... ENGINEER. THE GENERAL CONTRACTOR IS RESPONSIBL... THE BRIDGE SEATS DEAD LEVEL TO THE SPECIFIED ELE... A TOLERANCE OF PLUS OR MINUS 1/8 INCH.
THE GENERAL CONTRACTOR SHALL BE RESPONSIBLE F... THAT THE FINAL DECK ELEVATIONS CONFORM WITH ... SHOWN.



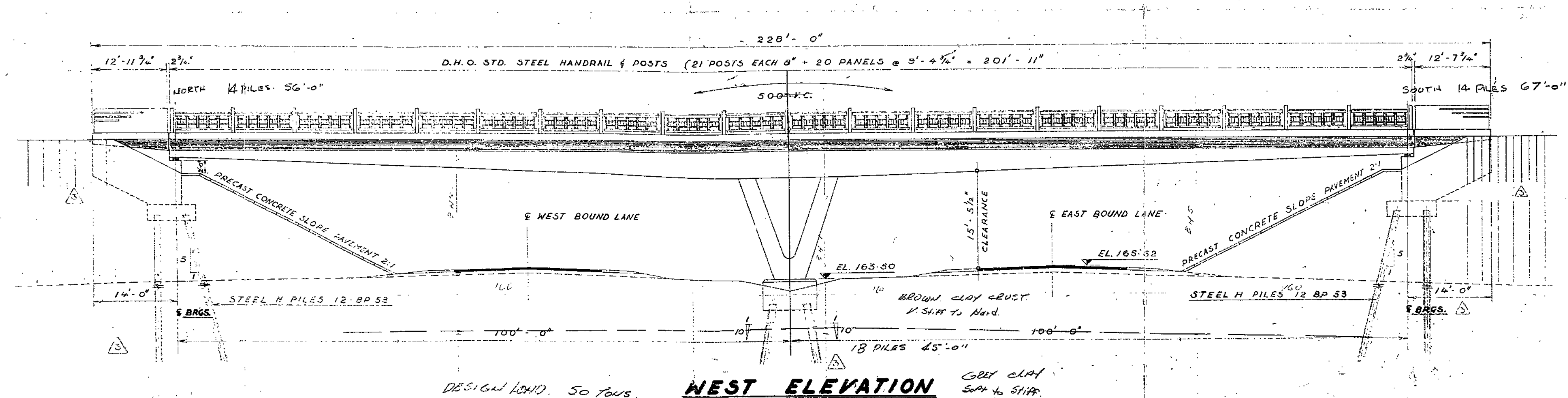
VERTICAL CURVE DIAGRA...
(NOT TO SCALE)



SECTION OF BOREHOLES



SKEW ANGLE DATA α = 9° 06'	
SIN. α =	0·1581581
COS. α =	0·9874138
TAN. α =	0·1601740



DESIGN LOAD. 50 TONS.

WEST ELEVATION
SCALE 1" = 10'-0"

GREY CLAY Soft to Stiff.

PRINT RECORD		
NO.	FOR	DATE
2	W.S.	9.10.61

B.M. ELEVATION = 163·15
Geodetic Datum:
N. & W. IN N/W. ROOT OF 3'0 OAK
216'0 R.F. OF STA. 458+66 HWY Nº 401 LINE A

LIST OF DRAWINGS	
D 4652-1	GENERAL PLAN
D 4652-2	FOOTINGS
D 4652-3	DECK PLAN - DIMENSIONS
D 4652-4	DECK REINFORCEMENT
D 4652-5	ABUTMENT - APPROACH SLAB
D 4652-6	WING WALLS & BRG. DET.S
D 4652-7	PIER - GIRDER & DIAPHRAGMS
[redacted]	
D 4652-9	CONCRETE RIP-RAP
D 4652-10	STEEL HANDRAIL PANELS
D 4652-10	INTERIOR STEEL POSTS
D 4652-10	STEEL END POSTS
D 4652-11	REINFORCING STEEL
D 4652-12	REINFORCING STEEL

REVISIONS: DATE	BY	DESCRIPTION	REFERENCE PLANS
OCT 1/62		DELETE BRG DETS D4652-2, ADD TO D4652-6 △4	E 3782 - 1
MAR 6/62	JSG	OMIT - NOTE - EXISTING FILL ELEVATION △3	F 4067 - 2
MAR 6/62	JSG	NOTE CHANGED (BORING DATA) △2	7833
27-2-62	JSG	△ EXPANSION JOINT FILLERS REVISED	8A 1137

W.P. 1...

DEPARTMENT OF HIGHWAYS:-O...
BRIDGE OFFICE: - TORONTO

TWP. RD. UNDERP...
(0·2 MI. WEST OF QUEBE...

THE KING'S HIGHWAY NO. 401
CO. GLENGARRY — LANCASTER TWP.
TWP. LANCASTER — LOT 1 & 2

GENERAL PLAN

APPROVED
BRIDGE ENGINEER

DESIGN	W.O.	CHECK	D.G.	CONTRACT NUMBERS	
DRAWING	V.D.P.	CHECK	D.G.		
TRACING		CHECK		LOADING	DRAWING NUMBER D-...
DATE		JUNE	1961	#20-516	