Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Division,

(Foundation Section).

November 8, 1961.

FOUNDATION INVESTIGATION REPO[illegible]

By: W.A. Trow & Associates, [illegible]

Attention: Mr. [illegible]urbin.

Re: [illegible]-60 and W.P. 116-59,
[illegible] 401 [illegible] #2, District #9.

We have [illegible]eviewed the Consultants' report for the above projects, and submit the following comments:-

(1) Structure Foundations:

The Consultant has recommended Frankl type caisson piles for the Hwy. #2 structure, and timber piles for the Hwy. #401 structure. He has raised some objections to the use of 'H' piles on the grounds that during the process of driving through the dense boulder stratum located above the bedrock, serious distortion and buckling of the piles may result.

In view of the fact that the Department has, on several occasi[illegible], driven 'H' piles under similar conditions and obtained [illegible]tisfactory subsequent performance, we feel that heavy section 'H' piles with reinforced tips should be used for both these structures. A design load of 65 tons per pile may be used. Refusal may occur either on the bedrock or in the boulder stratum.

(2) Embankment Stability:

The Consultant has recommended the installation of sand drains for fills greater than 25' on the Hwy. #2 bridge approaches. The maximum height here, is 28'. This involves a section about 150' long on the East approach. We have discussed this matter with the Consultant and he is in agreement with our

cont'd. /2 ...

If a critical condition is anticipated from these pore pressure readings, the stability of the approach fill can be ensured by installing vertical sand drains at the toe of the embankment. No drilling through the embankment fill is therefore necessary.

From a theoretical approach, the effect of the proposed stage construction is difficult to assess. The computed initial time of 4 months, for a minimum factor of safety to develop, is believed to be high and the probable field time may be as low as 1 to 2 months. After this period, the average strength of this clayey silt, and hence the factor of safety, will slowly increase. This rise in average effective stress with time can be deduced from the pore pressure migration curves*, quoted in the report. However, even after 6 months, this increase is relatively small. An equal or even larger increase in effective stress may be possible due to vertical dissipation of pore pressure over the 6 month period into the clay and till surrounding the questionable stratum. Inclusion of the effects of vertical dissipation makes the problem complex and difficult to analyse, and, accordingly, no computations have been made. However, it is believed that the effective stresses will increase sufficiently at this time to safely support the final 3 feet of fill.

As indicated earlier in this letter, a direct estimate of the stability of the approach fill will be available from the measured pore pressures values. It is recommended that at least 4 piezometers be installed under each side of the embankment at two or three locations as indicated on the accompanyir ketch.

It is understood that Geonor piezometer tips will be used. They should, of course, be installed before placement of the fill commences, so that a complete pattern of readings during and subsequent to load application may be obtained.

Vertical standpipes carried up through the fill are vulnerable to damage by construction equipment and are not recommended. If normal procedures are followed, therefore, polyethylene tubing will be led laterally from the piezometer locations to beyond the toe of the embankment, where the pressure measuring device will be positioned. Measurements of the pore pressures can be recorded either by gauges or by water level observations in an open vertical length of tubing. If this latter alternative is used, the tubing under the embankment will have to be laid in trenches approximately 4 to 5 feet below ground level, so as to ensure that the initial ground water level can be measured in the vertical portion of the tube. The replacement of the above depth of natural clay along the line of this trench with relatively permeable fill material, may result in excess vertical dissipation of pore pressure in this zone. This condition, reflected in the piezometric readings, would not be representative of the site. In addition, the provision of a suitable

* "Stability of a Bank On A Thin Peat Layer" - Ward, Penman & Gibson Geotechnique, Vol. V, No. 2, P.163

height of standpipe above ground level to accommodate the anticipated pore pressures, presents some difficulty. Accordingly, the use of pressure guages is recommended.

If pressure gauges are used, a satisfactory grade can be achieved with a 2½ feet deep trench, rising to 1 foot at the toe of the embankment. This shallow trench will not significantly affect the vertical permeability characteristics of the site. If the gauge is to be positioned at ground level, it should be capable of registering a negative pressure of at least 2 psi. Otherwise a positive reading gauge should be installed in a small pit at water table level. Entrapped air bubbles must be bled from the plastic tubing and gauge before embankment construction begins and pore pressures begin to register.

We shall be pleased to further discuss this matter if necessary after you have reviewed this letter, and also to cooperate with you in analysing any pore pressure measurements at this site.

Yours very truly,

P.G.M. Imrie, P.Eng.

PGMI/gc
Encls.- Sketch J542A/1a

WILLIAM A. TROW AND ASSOCIATES LTD.
SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.

1850 JANE ST.,
WESTON, ONT.
CH. 1-4644

Project: J542A

November 17, 1961

Mr. A. Rutka,
Acting Materials & Research Engineer,
Department of Highways of Ontario,
Parliament Buildings, Toronto, Ontario

Attention: Mr. N.D. Stermac, P.Eng.,
Principal Soils and Foundations Engineer

Re: Stage Construction of Embankment
Proposed Highway 2 over Sutherland Creek
WP 116-59 & WP 177-60

Dear Sirs:

In our recently submitted report on the above project, we recommended that vertical sand drains be installed under the slopes of the east approach fill, where the height of the embankment exceeded 25 feet. The purpose of these sand drains was to relieve the pore pressures migrating laterally from the centre to the edge of the embankment, along the more permeable clayey silt stratum at a depth of about 11 feet. This precaution would prevent the development of a critical stability condition at a computed time of approximately 4 months after completion of construction.

It is understood that initial construction of the embankment to a height of 25 feet, with the addition of the remaining 3 feet of fill after a period of 6 months, can be incorporated in the construction programme. In addition, the site is to be fully instrumented with piezometers to determine the magnitude and extent of this lateral pore pressure migration. The question arises, therefore, of the necessity for the sand drains.

As indicated in the report, conservative assumptions of the nature of the pore pressure migration have had to be made. The piezometers, however, will provide direct measurements of pore pressure under the embankment. As this information becomes available, the possibility that the factor of safety will reduce to the low values quoted in the report, can be examined more exactly.

The installation of sand drains is therefore recommended to control these pore pressures, and thereby ensure a steady increase in safety after construction. The engineering computations which form the basis for these opinions are outlined in the report.

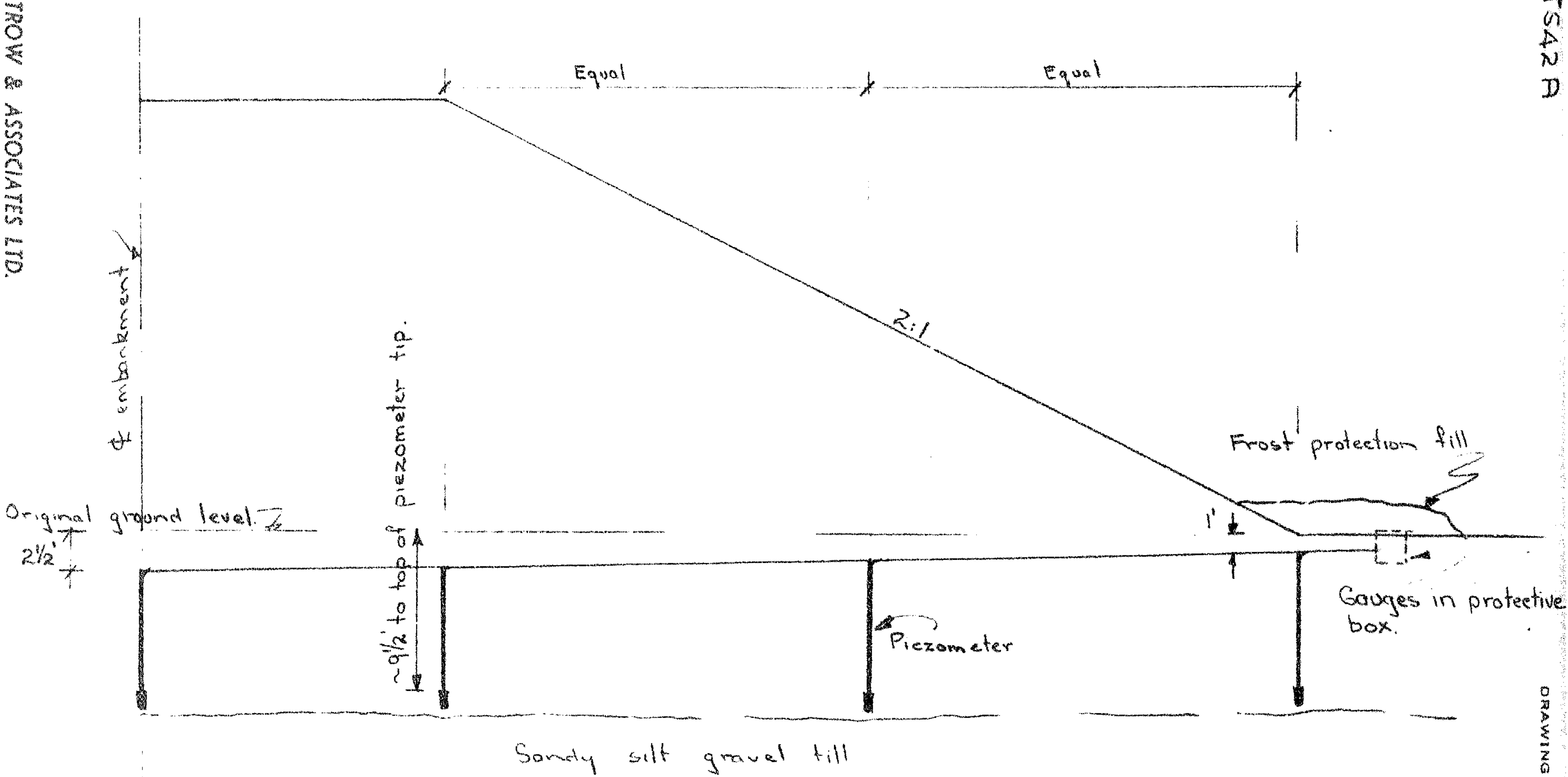
We shall be pleased to discuss any outstanding matter pertaining to this investigation after your review of this report.

Yours very truly,

W. Trow

William A. Trow, P.Eng.

WAT/gc
Enc.



SKETCH SHOWING SUGGESTED PIEZOMETER LOCATIONS

July 12 1962.

Mr. John Ford requested possibility of constructing abutment and driving piles without waiting for 6 months after placing critical height of fill. Solution recommended is to place fill up to bottom of foundation level, drive piles construct abutment but 35 feet clearance measured from back of foundation slab is required as sketched in drawing. The fill behind the abutment should be placed as soon as possible up to 7' below final grade. At least 6 months lapse is req'd before construction to final grade.

Department of Highways

COPY

For the information of

Mr. A. Stermac

Mr. L. E. Walker,
District Engineer,
~~Ottawa, Ont.~~

J. Ford,
Sr. Project Design Engineer
Downsview.

July 16th, 1962.

Contract 62-82
(W.P. 52-59-2)
Quebec Boundary Westerly - Hwy. #401
District 9

This memorandum is to confirm our telephone conversation regarding treatment at the Sutherland Creek approaches on Highway #2 relocation. As you know, it is stipulated in the contract that the approaches be constructed to a maximum height of 25-ft. and then be allowed to settle for six months before completing ~~the~~ to profile grade. I discussed this situation further with the Foundation Section and the following method of stage construction has been proposed:

1. Construct the fills to the level of the top of the abutment footings with a bench of at least 35-ft. in length at the back of the footings.

2. Drive the piles and construct the abutments.

3. Immediately after the construction of the abutments, place fill in the bench to a height not exceeding 7-ft. from profile grade.

4. Allow the fill to settle for a period of four months before bringing the fill up to final grade.

Attached please find sketches to explain the foregoing suggestions. I trust this information is satisfactory but if clarification required, please advise.

J. Ford

J. FORD,
SR. PROJECT DESIGN ENGINEER.

JF:gc.
c.c. S. J. Markiewicz
A. Stermac

20/7/62

O

OTTA DOWN 2 JUNE 5/62 900A VR

G METCALFE CONSTRN ENGR

RE: CONTRACT 62-82

INSTRUMENTATION OF APPROACH GRADES FOR LANCASTER TWP BRIDGE NO. 11

HWY 2 AT SUTHERLAND CREEK

WOULD YOU KINDLY ADVISE US OF THE DATE WHEN YOU EXPECT THE CONTRACTOR TO START CONSTRUCTION OF THE APPROACH GRADES OF THIS SUBJECT STRUCTURE. WE WOULD LIKE TO KNOW APPROXIMATELY TWO WEEKS AHEAD OF TIME IN ORDER THAT ARRANGEMENTS CAN BE MADE TO INSTRUMENT THE APPROACH FILLS

J B CURTIS BRIDGE LOCATION ENGR

Sr

Not to be instrumented — A. G. Sterma

Oct. 15th 196

DEPARTMENT OF HIGHWAYS OF ONTARIO
MATERIALS AND RESEARCH BRANCH
PARLIAMENT BUILDINGS, TORONTO, ONT.

FOUNDATION INVESTIGATION
PROPOSED CROSSING, SUTHERLAND CREEK
W.P. 177-60 & W.P. 116-59
HWY. 401 AND HWY. 2

Project: J542A

October, 1961

William A. Trow & Associates Ltd.

TABLE OF CONTENTS

ENCLOSURES

FOUNDATION INVESTIGATION
PROPOSED CROSSING, SUTHERLAND CREEK
HWYS. 401 AND HWY. 2

Project

The proposed construction of Highway 401 and associated service roads in the above area, involves a realignment of Highway 2 to the north. The existing route of Highway 2 is to be retained as a service road, but a replacement bridge over Sutherland Creek is envisaged. The field investigation for this structure is the subject of a separate report.

These proposals incorporate the simultaneous crossing of Highway 401 and Sutherland Creek by an overhead skew bridge carrying Highway 2. At the time of the field investigation, preliminary plans for this overhead structure indicated two spans of about 120 feet. This layout has since been modified to include two central spans of 140 feet and two outer spans of about 120 feet. Maximum embankment heights associated with this bridge are about 28 feet, including superelevation.

The Highway 401 crossing of Sutherland Creek initially incorporated a 40 feet single span structure. The present proposals are for a 110 feet long three span structure.

A previous field investigation of twelve borings was performed in August, 1960, before the location of the bridges was finalised.

This report presents the results of this work, together with the findings of a more recent field investigation, and it discusses the foundation problems at this site. Specifically these problems are:

a) embankment stability on the east bank, and

b) the choice of a suitable pile for bridge support.

Site Description

In the vicinity of the proposed bridges, Sutherland Creek is a sluggish stream about 120 feet wide. Considerable swamp growth of reeds and rushes extends some distance out from each bank. Maximum depth of the creek is about 6 feet. The water level in the creek remained level at El 152.6 feet during the investigation.

East of the creek, the ground is a typical very flat lake bed deposit. This area is in pasture with isolated groups of trees. The west bank, supporting a heavy growth of long grass and numerous trees, gradually rises away from the river. This gently undulating topography is indicative of a glacial till soil.

Soil Types Encountered

In the initial investigation, twelve borings, 1 to 12, were put down at positions indicated on drawing 2 of this report. The results of these borings are presented on the borehole logs as drawings 3 to 14. Of these twelve boreholes, only borings 3, 4 and 5 are now applicable to the project.

Four borings, 13 to 16, were made in the field investigation just completed, and the results presented in the borehole logs as drawings 15 to 18. Two of these borings, 13 and 14, located at the central pier and the east abutment of the then proposed structure, were taken to bedrock which was proved. Bedrock was also proved in boring 5, located by the west abutment. Boreholes 15 and 16, located under the proposed embankment, were put down to obtain continuous samples of the surface deposit of clay in this area, and were terminated in the underlying till.

In view of the modifications to the bridge layout subsequent to the field investigation, several borings are now wrongly located. However, in view of the relative uniform soil stratigraphy encountered at the site, further borings are considered to be unnecessary.

The relevant information from the borehole logs has been summarized and presented in drawing 1 in the form of an estimated subsoil stratigraphical profile.

Reference to this drawing shows that a slightly plastic sandy silt till, with numerous gravel sizes is the predominant foundation material at this site. According to the field penetration measurements, it exists in a loose to medium dense state for the first 23 to 32 feet. At approximate El 133 feet under the west bank decreasing to El 122 feet under the east bank, it becomes very dense. A heavy concentration of large stones, in a matrix of sandy silt or silty sand and gravel is encountered after a penetration of 1 foot into this dense till. There is a slight artesian pressure in this bouldery deposit. The hydrostatic head is El 154.5 feet approximately, which is at or just above ground level on the east bank.

Bedrock, consisting of dense slightly argillaceous limestone was encountered at about El 114 feet across the site. Farther to the south, under the existing bridge, it was encountered at El 118 to 120 feet, and to the north of the project site in boring 10, it was encountered at El 124.3 feet.

The till on the east approaches to the bridge is overlain by a very stiff to stiff fissured marine clay. Below about 9 feet, the clay becomes softer and layered with the silt phase predominating between 10 and 12 feet, where the till contact occurs. There is no clay of consequence in the west bank or in the creek bed. About 1 foot of organic muck overlies the till in this latter location.

Undrained triaxial tests indicate a shear strength of about 400 psf in the layered deposits just above the till contact. Atterberg limit tests indicate a plasticity index of about 6 for the silt with a corresponding value of 50 to 60 for the upper clay.

A series of consolidated drained tests on samples from depths between 10 and 12 feet indicated an effective angle of internal friction of 32°. A single test on a sample of the till immediately below, indicated a probable angle of about 42°. These results confirm the fairly obvious conclusion that any potential failure surface will be confined to the soft layered clay.

Measurements of the horizontal coefficients of consolidation were made during the consolidation process of the above tests, and compared to values obtained from silt and clay samples subjected to vertical drainage in the oedometer. The purpose of these tests was to show that the horizontal permeability of the silt was much greater than the vertical permeability of the clay surrounding it. Unfortunately these tests are not conclusive and it is assumed that the filter paper drains surrounding the triaxial specimens possess a comparable permeability to the sample when subjected to cell pressure. Similarly, reliable values of the ratio of the coefficients of consolidation and swelling for the silt, as required for the analysis in the following section of this report, could not be obtained.

Grain size analyses were also performed on samples of silt and till. A lower permeability of the silt is indicated by the relative D_{15} sizes of the two materials. Conversely, a higher silt permeability is indicated by comparison of the uniformity coefficients. Hence the similar consolidation coefficient values obtained for these two materials is not unreasonable. The results of a similar test on a sample of clay, however, would indicate a much lower consolidation coefficient and lower permeability, for the clay, than obtained from the tests.

The results of these various laboratory tests are presented on drawings 19 to 29 of this report.

Foundation Requirements

Embankment stability over the surface deposits of the layered silty clay, and the selection of the most suitable type of bridge support, represent the major foundation problems at this site.

a) Embankment Stability

The soft condition and layered nature of the lower 2 to 3 feet of the deposit of silty clay on the east bank of Sutherland Creek have been described in the previous section of this report. In view of these questionable soil characteristics, the stability of the proposed 28 feet high embankment is discussed in the following paragraphs.

It is reasonable to assume that a horizontal displacement of a block of soil, along a plane in the clay just above the till contact, is the most probable mode of failure. This displacement is assumed to be caused by the active thrust on the rear face of the block, and resisted by the sum of the passive resistance at the front of the block, and the shear resistance along its base. Other methods of analysis using composite sliding surfaces must lead to similar results to the above method and have not been investigated.

Computations outlined in Appendix B show the maximum active thrust, P_A, on a vertical plane through the top of the embankment slope and the minimum passive resistance, P_p, at the toe of the embankment to be 35,120 and 22,700 plf respectively.

Immediately after the embankment construction is complete, the shear resistance developed in the soft clay is 400 psf or 22,400 plf. Hence the corresponding factor of safety at this stage of the project is 1.28.

However, a more critical state may exist at some time subsequent to the application of the surcharge. This undesirable condition could develop during the horizontal transfer of pore pressure along the relatively more permeable silt layers from under the centre portion of the embankment towards and beyond the toe. Hence the effective stresses in the soil in this latter area may be decreased considerably.

The computations in Appendix B indicate that the factor of safety could decrease to 1.10 under these conditions, assuming a triangular bank. It is also shown in these computations that an even lower factor of safety is conceivable when the embankment contains a central section of constant height. Offsetting this latter probability, however, is the slight increase in effective pressures which occurs as a result of the very gradual pore pressure dissipation in the vertical direction through the clay.

Since neither of these two conditions can be evaluated with any degree of accuracy, it is assumed that they approximately cancel out each other. The resultant factor of safety, therefore, is about 1.10, as quoted above.

It is computed that this critical condition occurs about 4 months after placing the fill, but will probably develop sooner due to higher field rates of drainage.

It is recommended, therefore, that additional measures be taken to increase the factor of safety. These measures incorporate the installation of two rows of vertical sand drains, under either embankment slope, extending down to the till contact, to intersect the silt layers. These drains should be located, 8 feet apart, under the midpoint of the embankment slopes. This position is the approximate transition point beyond which pore pressures theoretically increase with time. The presence of the highly permeable sand drains, therefore, prevent any build-up in pore pressures near the toe. Consequently the effective pressures and hence the factor of safety will steadily increase after construction is complete.

It is suggested that these vertical drains be drilled with an 18 inch diameter highway-type auger down to the gravel till contact. Immediately upon withdrawal of the auger, the hole should be filled with concrete sand conforming to ASTM C33-46. If any significant time is allowed to elapse between auger withdrawal and placing the sand, there is a danger of the silt sloughing into the hole.

Computations in Appendix B show that the critical factor of safety of 1.10 has increased to 1.28 when the embankment height is reduced to 25 feet. Hence the sand drains should extend eastwards to this location or about 150 feet from the abutments.

The positions and details of these sand drains are indicated on drawing 1.

In view of the short embankment length requiring sand drains, it might be argued that they are not entirely necessary. It is considered, however, that they offer a relatively inexpensive insurance against failure near the abutment, which, if occurring, could seriously distort the abutment.

The possibility of a failure towards the creek, where no passive resistance can be developed, was also considered. Calculations in Appendix B show that a factor of safety of 1.5 is developed if the toe of the slope is at least 76 feet from the creek bank. It is believed that this requirement is satisfied under the present proposals. A certain decrease in the safety factor may occur as previously described. However, on the basis of the reduction quoted above, it should not decrease below about 1.3.

b) <u>Bridge Foundations</u>

Preliminary plans, based on the results of the initial site investigation had envisaged steel H piles to bedrock as support for the bridge structures.

The recently completed field investigation, however, has revealed the presence of a dense boulder till stratum below El 122 to El 133 feet, overlying bedrock. It is possible that some H piles may penetrate this stratum to reach bedrock without undue deflection or damage. However, it is believed that the majority of such piles will be considerably impeded by the boulder matrix, with consequent distortion and damage. Several piles may encounter refusal some distance above the bedrock.

While a satisfactory load capacity is implied if piles are driven to refusal, some concern is felt about the possible effects of buckling and damage of the pile. It is conceivable that the long term yielding of a buckled pile and the surrounding ground could result in a potential localised settlement of one pile.

It is considered that a more satisfactory solution, in the engineering sense, can be achieved by the use of a caisson-type pile bearing in the top of the dense boulder till located about 22 to 30 feet below the present ground surface. A pile of the Franki type is recommended. The compacted bulb of such a pile would serve two purposes;

a) the load will be spread out over a large area and therefore the danger of load concentration on the side of a boulder is avoided, and

b) the compaction process will ensure that the boulders assume a dense condition.

A safe load capacity of 100 tons should be achieved with these Franki type piles, but a load test should be performed on one pile in the pier location.

These piles should be installed in prebored holes about 18 inches in diameter, stabilised with a temporary liner to hold back loose pockets of wet sandy till. This procedure will avoid both the danger of disturbance to the soft clayey silt and the tendency for the displaced relatively impermeable till to squeeze in against the green concrete as the temporary liner is withdrawn. After this operation is complete, the bulb of the Franki pile should be compacted in place in the usual manner, followed by the installation of the concrete shaft.

Since augering equipment will be on the site, the additional cost of preboring through embankment fill for the abutment piles should not be excessive. Since the soft clay and silt will be partially consolidated under the embankment weight, and since the fill will not be at full height, the danger of a temporary instability condition occuring in the soft clay during the driving of the Franki bulb appears unlikely.

The same type of pile could be used to support the Hwy. 401 crossing of the creek. However, since this is to be a much lighter structure, it may not be possible to efficiently utilize the full capacity of Franki piles. Therefore it may be more economic to use untreated Class B timber piles for this bridge. Because of the boulders, there will be uncertainty regarding the end-bearing capacit of these units, and therefore additional piles should be used to provide a factor of safety. It is recommended that the piles be designed for a maximum load of 15 tons. The criterion for refusal should be 8 blows per inch under 8750 ft.lbs. of driving energy. Refusal should be encountered at the top of the dense boulder deposit. It may be necessary to redrive some of the timber piles because they may be pushed up during driving of adjacent members.

In the preceeding paragraph, caisson-type and timber piles have been recommended because of the uncertainty regarding the reliability of H piles driven through this dense boulder till. It is understood, however, that considerable economy would be achieved by the use of steel H piles of 60 ton capacity throughout the entire project including the downstream Hwy. 2 replacement bridge.

If, therefore, the decision is made to use steel H piles, it is recommended that they be driven to a refusal criterion of 12 to 15 blows per inch penetration under a driving energy of 15000 ft.lbs. per blow. A heavy section pile should be used to minimize the danger of distortion. The pile caps should be designed so as to adequately distribute the loads to other piles should any one pile yield excessively at some future date.

Pile load tests would be desirable on about four or five piles which met refusal in the dense boulder till.

Conclusions and Recommendations

1) A slightly plastic loose to medium dense sandy silt till overlain by a surface deposit of marine clay is the predominant soil at this site. At depths of about 23 to 32 feet below ground level it becomes very dense with a large concentration of boulders. Slight artesian pressure exists in this stratum. Dense limestone bedrock is encountered at a uniform depth equal to El 114 feet approximately. The surface deposits of stiff fissured clay extend to a depth of 12 feet on the east bank; they become soft and layered with silt below 9 feet. The undrained strength of the silt and clay just above the till contact is about 400 psf.

2) Computations show that a 28 feet high embankment on the east approaches is just safe immediately after construction. At some subsequent period, however, an insufficient margin of safety exists due to build up of pore pressures under the embankment toe.

3) In order to raise the factor of safety to acceptable values, the installation of vertical sand drains to intercept the silt layers is recommended. Details of these drains are given in the report and summarized on drawing 1.

4) Caisson piles are recommended for support of the overhead structure. Prebored Franki piles are suitable, with an estimated safe capacity of 100 tons. Timber piles, with a conservative capacity of 15 tons are suggested as a more economic alternative for the light Highway 401 bridge structure.

5) Some objections to the use of steel H piles from an engineering point of view are outlined in the report. Nevertheless, the use of steel H piles throughout the whole project would result in considerable economy. Certain precautions and recommendations covering the use of these piles are presented in the report.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
P. G. M. IMRIE
PROVINCE OF ONTARIO

P.G.M. Imrie, P.Eng.

PGMI/gc
J542A
Oct., 1961

APPENDIX A

Field and Laboratory Investigation Methods

Twelve borings were made at this site in August, 1960, and a further four boreholes put down in September, 1961. In both instances, the work was accomplished using two conventional diamond drills equipped for soil sampling purposes. Four holes were taken to bedrock, which was proved by recovering AX core. Except for two shallow borings in the surface, clay deposits, the other holes were terminated in the very dense boulder till overlying bedrock. Progress through these boulders, in the four borings to bedrock, was very slow and was achieved by alternately drilling ahead with BX casing and AX core barrel. Dynamic cones were driven adjacent to most borings.

Measurements of the slight artesian flow encountered in this latter stratum were made in the normal manner. After completing flow measurements just above ground level, a short length of casing was added and the stabilised water level noted. On completion, the borings were sealed with bentonite, just above the artesian zone.

Disturbed samples of the various soil strata were obtained by driving a standard 2 inch outside diameter split spoon sampler into the soil ahead of the boring. The number of 350 ft.lb. hammer blows required to extend the penetration of the split spoon from 6 inches to 18 inches was recorded as the penetration resistance of the soil. Upon recovery, the samples were inspected, and retained in moisture proof plastic bags.

Undisturbed samples of the clay deposit were obtained by pressing a 2 inch inside diameter shelby tube sampler into the soil ahead of the boring. Upon recovery, the tubes were sealed with low melting point wax for transportation to the laboratory.

In-situ strength measurements of the clay were determined by field vane tests. Upon completion of the initial test, the remoulded strength was determined after six complete revolutions of the vane.

In the laboratory, conventional undrained triaxial tests were performed on undisturbed samples of the clay. Particular care was taken when handling and setting up the samples with high silt content, to minimise disturbance. The results of these tests are presented in drawings 21- 23.

Consolidated drained tests were performed on three samples of the layered clay to determine the effective angle of internal friction. A similar test was performed on a sample of the immediately adjacent till. The results of these tests are presented on drawings 24 to 27 and summarized on drawing 28.

The samples in these above tests were set up with filter paper perimeter drainage. Theoretically, the coefficient of consolidation determined for the consolidation stage of the above tests would be indicative of the horizontal permeability. Results of these tests are also presented on drawings 24 to 27. In order to compare the values so obtained with those for drainage in a vertical direction, samples of both the silt and clay phase of the layered clay were subjected to consolidation in the oedometer. Drainage of the silt specimen was permitted at one end only to slow the consolidation to a measureable rate. The results of these tests are presented on drawings 19 and 20.

Grain size analyses were carried out on samples of the clay, silt, and till. Sizes passing the No. 14 sieve were subjected to hydrometer analyses in the usual manner. Drawing 29 shows these test results.

Atterberg limit and moisture content tests were performed on several samples of the silty clay.

The elevations of all boreholes were referred to a bench mark in a maple tree on the east bank. The level of this bench mark was taken as El 159.03 feet.

APPENDIX B

Stability Analysis of Embankment Fill

The assumed physical dimensions and soil properties are shown on Fig. 1. It is assumed that the soft soil just above the till contact at about El 145.5 feet represents the potential failure plane, under the active pressure of a central wedge of soil in the embankment. This active lateral thrust, P_A, is resisted by the shear strength, R, along the failure surface and the passive resistance P_p, developed at the toe of the embankment.

The factor of safety, F, is therefore given by the expression:

$$F = (P_p + R)/P_A$$

The active and passive pressure distribution at the top and toe of the embankment slope respectively are shown in Fig. 2.

i) At Completion of Construction

$$P_A = \tfrac{1}{2}30.1125 + 1125 \times 2.5 + (1125 + 3610)/2 \times 6.5$$

$$= \underline{35,120 \text{ plf}}$$

$$P_p = 9x(3600 + 1460)/2$$

$$= \underline{22,700 \text{ plf}}$$

$$R = 400 \times 56$$

$$= \underline{22,400 \text{ plf}}$$

$$F = \underline{1.28}$$

ii) A Short Time After Construction is Complete

After partial dissipation of pore pressure, the value of R is given by the expression:

$$R = 22400 + \tfrac{1}{2}\gamma H.L. \sin \phi' \ (1 - 2I)^*$$

* "Stability of A Bank On A Thin Peat Layer" - Ward, Penman & Gibson. Geotechnique, Vol. V, No. 2, P. 162. (In first term, undrained strength used to allow for pore pressures set up due to horizontal shear stresses - see Bishop & Skempton - "Gain in Stability Due to Pore Pressure Dissipation" - 5th Congress on Large Dams - 1955)

where: $2I$ is the ratio of the total pore pressure at any time to the total weight of embankment above the sliding surface.

The second term represents the change in strength of the soft clayey silt due to the horizontal redistribution of pore pressure.

Assuming that the ratio of the coefficients of consolidation and swelling for this material is about $\frac{1}{4}$, the maximum value of I for a triangular embankment is 0.53 as quoted.**

Hence: $R = 22400 - 125x28x56x \sin 32° \times 0.03$

$= \underline{19,280 \text{ plf}}$

The above value of I occurs approximately at $\sqrt{c_v t/a^2} \sim 0.11$. At this time, the pore pressure at the toe of the embankment is $\mu \sim 0.11\gamma H = 385$ psf. The reduced value of P_p is therefore:

$$P_p = 22,700 - 385x9$$

$$= \underline{19,240 \text{ plf}}$$

Change in P_A due to decrease in pore pressure will be negligible.

The factor of safety, F is therefore

$$F = \frac{19280 + 19,240}{35,120}$$

$$= \underline{1.10}$$

However it should be noted that the above pore pressure redistribution value I = 0.53 is applicable for a triangular embankment. An even more unfavourable redistribution is conceivable when the embankment contains a central section of constant height. This possible condition is indicated in Fig. 3.

iii) <u>Time of Critical Condition</u>

This unfavourable condition occurs when $\sqrt{c_v t/a^2} \sim 0.11$.

If an average value of $c_v = 0.3$ sq.ft./day is assumed, then

$$t = \frac{0.0121 \times 56^2}{0.3}$$

$$= \underline{126 \text{ days}}$$

** "Stability of A Bank On A Thin Peat Layer" - Ward, Penman & Gibson. Geotechnique, Vol. V, No. 2, P. 163

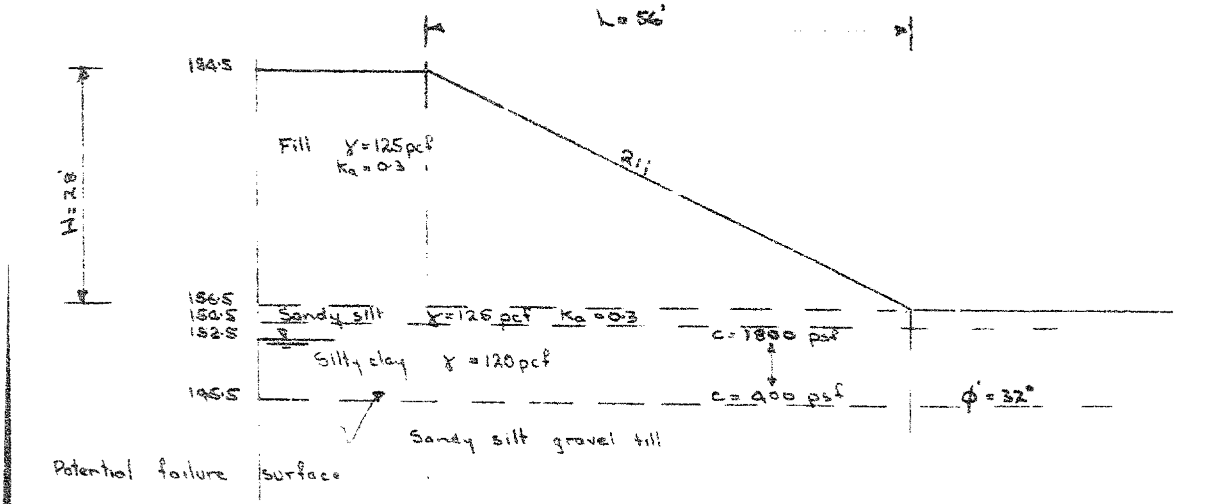


FIG. 1

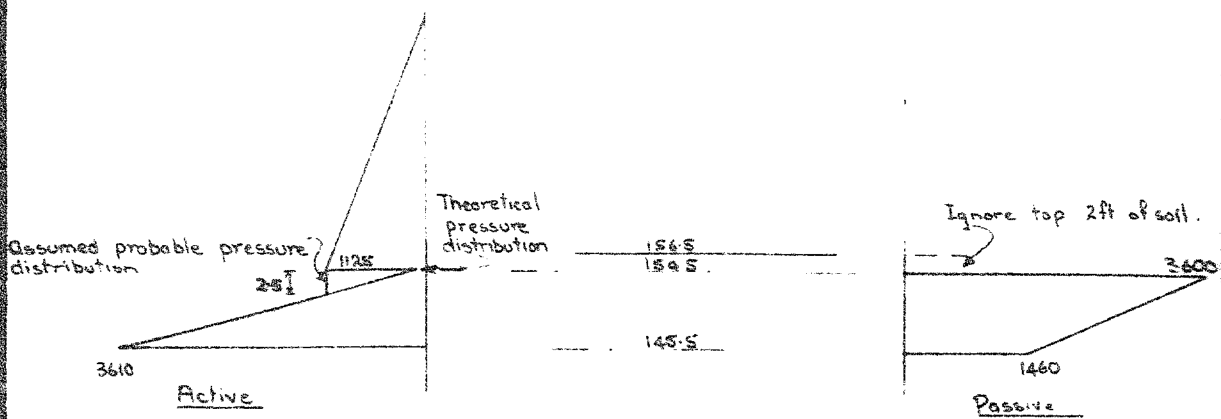
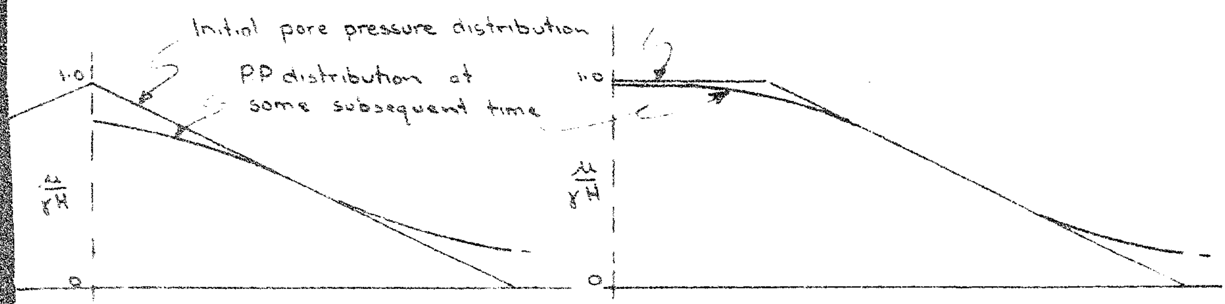


FIG. 2



Sketch indicating how a higher total pore pressure is probable under an embankment with central portion at constant height.

FIG. 3.

iv) Factor of Safety for H = 25 feet

In a similar manner as before but with H = 25, L = 50, and I = 0.53

P_A = 28960 plf

R = 20000 - 2500 = 17,500 plf

P_p = 22700 - 3090 = 19610 plf

therefore F = $\frac{37110}{28960}$

= 1.28

v) Failure Towards Creek

Immediately after construction

P_p = 0

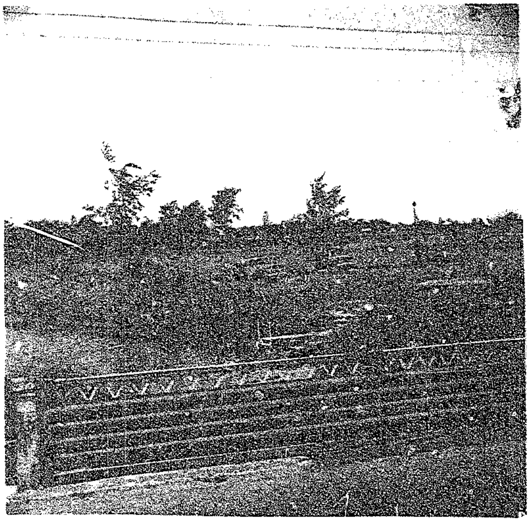
R = 400 x L

P_A = 35120 plf

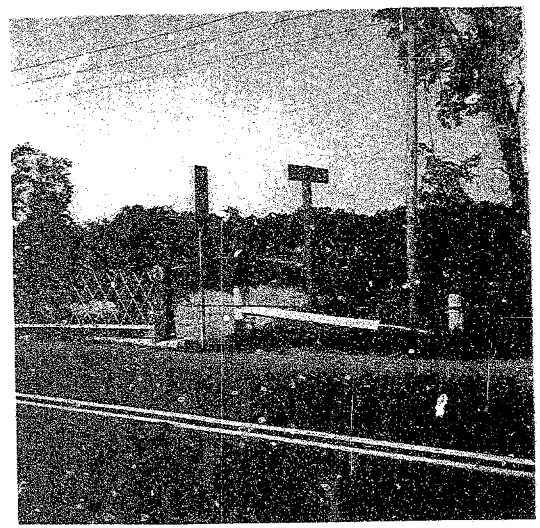
For F = 1.5, L = 35120/400

= 132 feet

or the toe of the fill must not extend nearer than (132 - 56) = 76 feet to the bank of the creek.

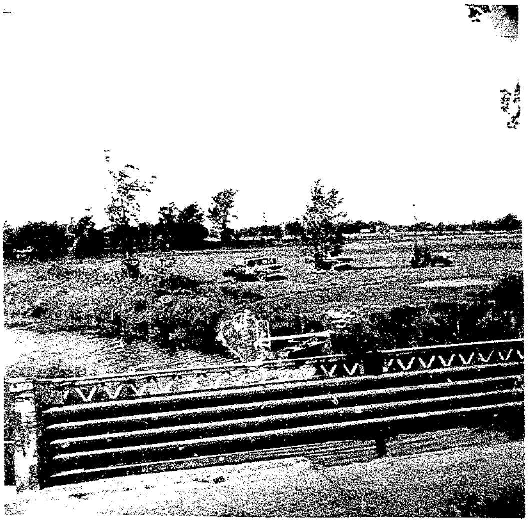


View looking North
Left drill on BH 14
Right drill on BH 15

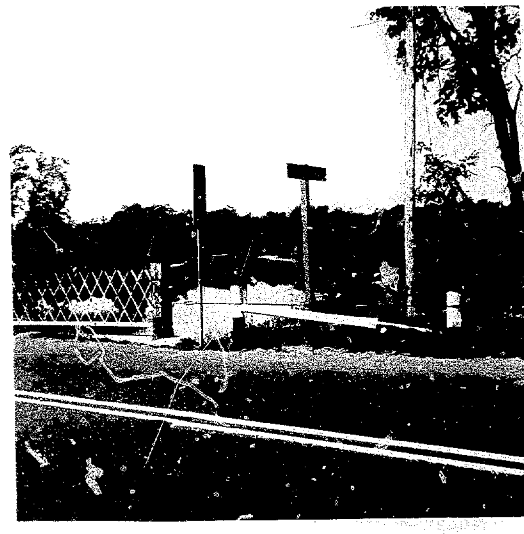


View looking Northwest (Upstream)
Drill on BH 13

View looking West along Line "C"
Drill on BH 13



View looking North
Left drill on BH #4
Right drill on BH #5



View looking Northwest (Upstream)
Drill on BH #3

View looking West along Line "C"
Drill on BH #3

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

Dwg. 3
J542A

DRAWING NO. 2
PROJECT NO. J542

BOREHOLE NO. 1
PROJECT Sutherland Creek and Hwy. No. 2 Crossing-Hwy.401
LOCATION East of Lancaster, Ont.
HOLE LOCATION See Dwg. No. 1
HOLE ELEVATION 156.7 ft.
DATUM Top North Guard Rail Existing Bridge = 163.3

LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —o—o—o—
2" I.D. SHELBY TUBE —x—x—x—x—
2" DIA. CONE ————

SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI

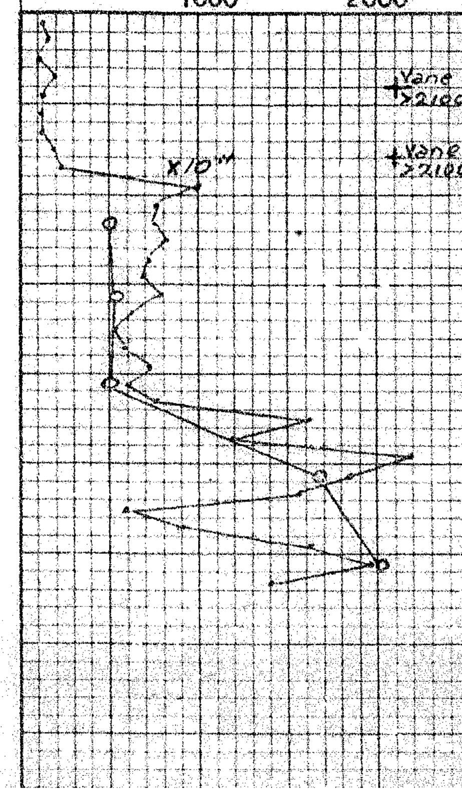
ATTERBERG LIMITS
LIQUID LIMIT —o
PLASTIC LIMIT ⊢—

SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.
	Ground Surface	156.7	0		
	Very stiff brown fissured clay. Changing to wet brown silt about 8½ft.			1 Pushed; 2 Levered; 3 Levered 8"	
	Dense cohesive silty sand with numerous fine gravel very dense below 22 feet	148.3	10	4; 5; 6; 7; 8	
			20		
			30		
	End of Bore	125.3			
			40		

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT. 20 40 60 80
SHEAR STRENGTH P.S.F. 1000 2000

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT



Notes:
1) Hole cased to 30 ft.
2) Cone 5 ft. west of hole

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

Dwg. 4
J542A

DRAWING NO. 3
PROJECT NO. J542

LEGEND

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 2" DIA. CONE

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X[LI]

ATTERBERG LIMITS
- LIQUID LIMIT
- PLASTIC LIMIT

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

BOREHOLE NO. 2

PROJECT Sutherland Creek and Hwy. No. 2 Crossing-Hwy.401

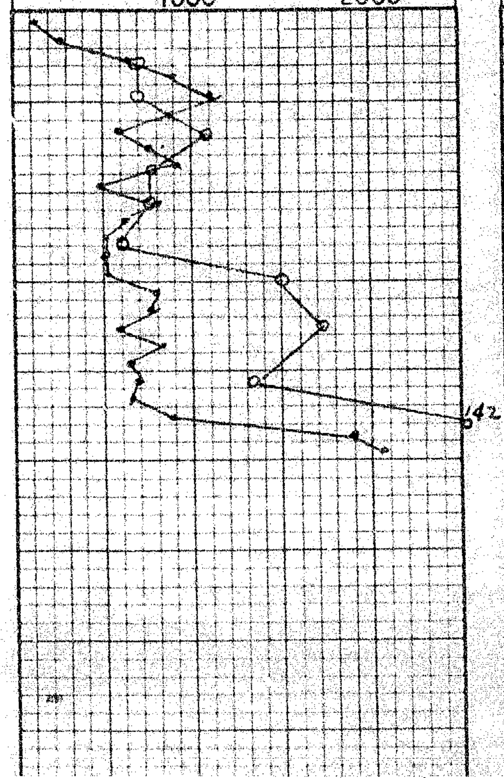
LOCATION East of Lancaster, Ont.

HOLE LOCATION See Dwg. No. 1

HOLE ELEVATION 157.2 ft.

DATUM Top North Guard Rail Existing Bridge = 163.3

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT. 20 40 60 80 / SHEAR STRENGTH P.S.F. 1000 2000	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.
	Ground Surface	157.2	0				
	Hard dry brown clayey-sandy silt with gravel sizes					1, 2	
	becoming dense brown cohesive silty sand with numerous shale gravel below about 8 feet.		10			3, 4, 5, 6, 7, 8	
			20	142		9, 10	
	End of Bore	132.3					
			30				
			40				



Notes:

1) Hole cased to 24 ft.

2) Cone driven 5 feet east of hole.

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

Dwg. 5
J542A

DRAWING No. 4
PROJECT No. J542

LEGEND

BOREHOLE No. 3
PROJECT Sutherland Creek and Hwy. No. 2 Crossing-Hwy.401
LOCATION East of Lancaster, Ont.
HOLE LOCATION See Dwg. No. 1
HOLE ELEVATION 154.4 ft.
DATUM Top North Guard Rail Existing Bridge = 163.3

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 2" DIA. CONE

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI

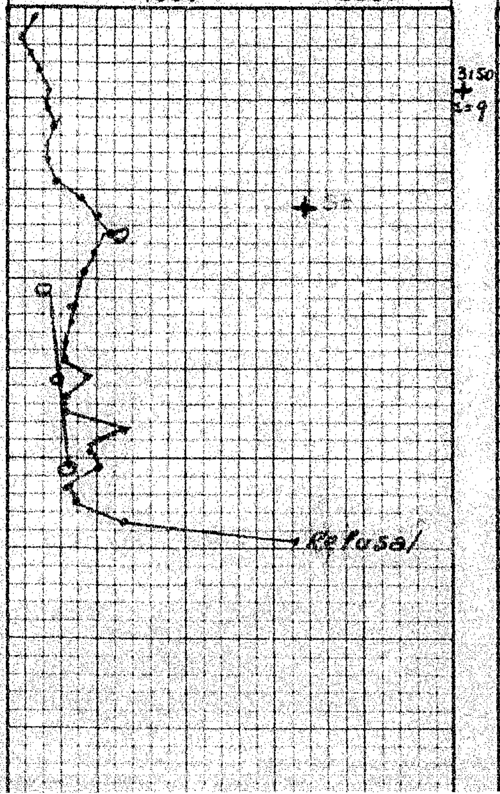
ATTERBERG LIMITS
- LIQUID LIMIT
- PLASTIC LIMIT

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
	Ground Surface	154.4	0		
	Very stiff grey brown fissured clay becoming stiff below about 5 feet and soft below 8 feet. Wet clayey silt below about 9 feet.			1 Levered 2 Levered 3 Levered	
	Refusal to Shelby Tube 12.3 ft.	142.3	10		
	Dense angular gravel in a matrix of slightly cohesive silty sand.		20	4 5 6 7	
	Casing bouncing at 29½ ft. End of Bore	124.8	30		
			40		

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT. 20 40 60 80
SHEAR STRENGTH P.S.F. 1000 2000

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT



Notes:

1) Hole cased to 25 ft.

2) Cone driven 20 ft. north

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

Dwg. 6
J542A

DRAWING NO. 5
PROJECT NO. J542

LEGEND

BOREHOLE NO. 4
PROJECT Sutherland Creek and Hwy. No. 2 Crossing-Hwy.401
LOCATION East of Lancaster, Ont.
HOLE LOCATION See Dwg. No. 1
HOLE ELEVATION 154.4 ft.
DATUM Top North Guard Rail Existing Bridge = 163.3

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE —o—o—o—
- 2" I.D. SHELBY TUBE —x—x—x—x—
- 2" DIA. CONE ———

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) +S

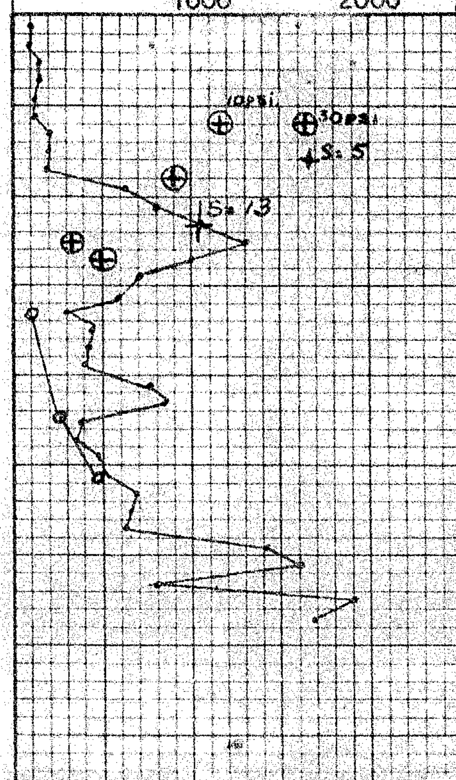
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI

ATTERBERG LIMITS
- LIQUID LIMIT —o
- PLASTIC LIMIT ⊢—

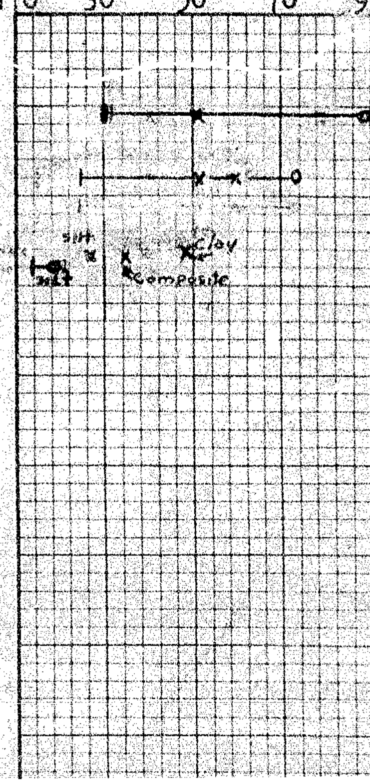
SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
	Ground Surface	154.4	0		
	Very stiff grey brown fissured clay becoming less stiff with depth; varved below about 8 feet 8-10 ft.-clay about 1/4 inch thick, silty clay about 1/4 inch thick 12-13-clay 1/4 to 1/2 inch; silt about 3/4 to 1 inch.			1 Levered 2 Levered 3 Levered 4 Levered	
		141.3	10		
	Soft to medium stiff wet clayey sandy silt with gravel.			5	
		135.3	20		
	Dense grey cohesive silty sand with considerable gravel.			6 7	
		124.3	30		
	End of bore on bedrock or boulder				
			40		

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT. 20 40 60 80
SHEAR STRENGTH P.S.F. 1000 2000



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT
10 30 50 70 90



Notes:
1) Hole cased to 30 ft.
2) Cone driven 10 ft. south of hole (according to blows gravel indicated below 10 ft.)

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

Dwg. 7
J542A

DRAWING NO. 6
PROJECT NO. J542

LEGEND

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE —o—o—o—
- 2" I.D. SHELBY TUBE —x—x—x—
- 2" DIA. CONE ————

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

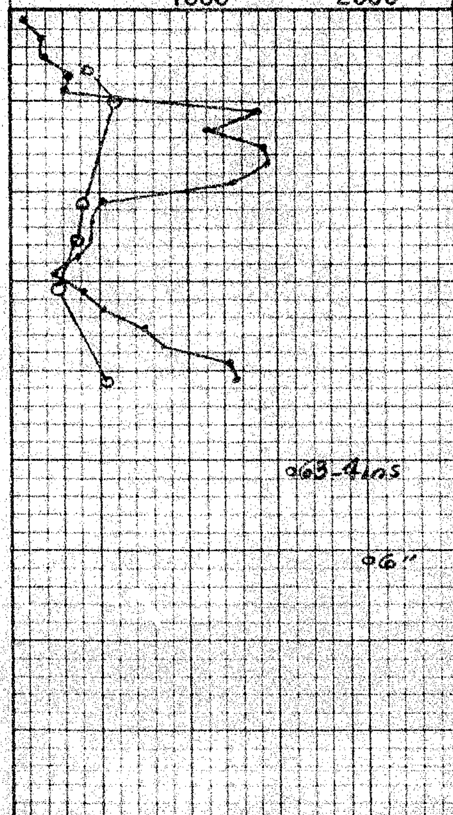
ATTERBERG LIMITS
- LIQUID LIMIT —o
- PLASTIC LIMIT |—

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

BOREHOLE NO. 5
PROJECT Sutherland Creek & Hwy. No. 2 Crossing Hwy. 401
LOCATION East of Lancaster
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 154.6 ft.
DATUM Top north Guard Rail existing bridge 163.3

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET
	Ground surface	154.6	0
	Stiff brown clay with roots hairs & numerous shale gravel.	149.6	
	Medium dense to dense cohesive sandy silt with gravel sizes (larger stones between 5 to 10 ft.)	133.3	10, 20
	Very dense mixture of fine sand and gravel with numerous cobbles.	124.3	30
	Very dense angular shale gravel in a matrix of slightly cohesive silty sand.	114.3	40
	Shaley limestone bedrock. Shale increases with depth – 100% recovery to 51.5 ft.		

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT: 20, 40, 60, 80
SHEAR STRENGTH P.S.F.: 1000, 2000



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

SAMPLE TYPE AND No: 1, 2, 3, 4, 5, 6, 7, 8

NATURAL UNIT WEIGHT P.C.F.

Notes: 1) Hole cased to 35 ft. 2) Cone 5 ft. north.

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

Dwg. 8
J542A

DRAWING No. 7
PROJECT No. J542

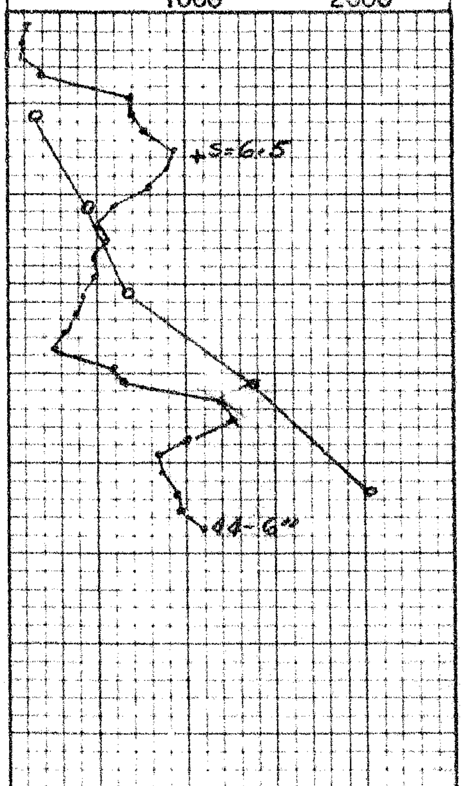
LEGEND

BOREHOLE No. 6
PROJECT Sutherland Creek & Hwy. No. 2 Crossing Hwy. 401
LOCATION East of Lancaster, Ont.
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 156.7 ft.
DATUM Top north Guard Rail existing Bridge 163.3

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —o—o—o—
2" I.D. SHELBY TUBE —x—x—x—x—
2" DIA. CONE ————
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}
ATTERBERG LIMITS
LIQUID LIMIT —o
PLASTIC LIMIT ├—
SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT (20, 40, 60, 80) / SHEAR STRENGTH P.S.F. (1000, 2000)	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
	Ground surface	156.7	0				
	Approx. 8 ins. topsoil. Med. stiff to stiff light grey brown clay appears fissured and reworked.	148.3				1	
	Med. dense to dense cohesive sandy silt with numerous limestone and shale gravel sizes. Large stones at 8½ ft., 14 ft.		10			2, 3	
			20			4	
	End of Bore	129.3				5	
			30				
			40				



Notes:
1) Hole cased to 25 ft.
2) Cone driven 21 ft. east of hole.

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

Dwg. 9
J542A

DRAWING NO 8
PROJECT NO. J542

LEGEND

BOREHOLE NO. 7
PROJECT Sutherland Creek & Hwy. No. 2 Crossing Hwy. 401
LOCATION East of Lancaster, Ont.
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 155.4 ft.
DATUM Top north Guard Rail existing bridge = 163.3

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —o—o—o—
2" I.D. SHELBY TUBE —x—x—x—
2" DIA. CONE ———

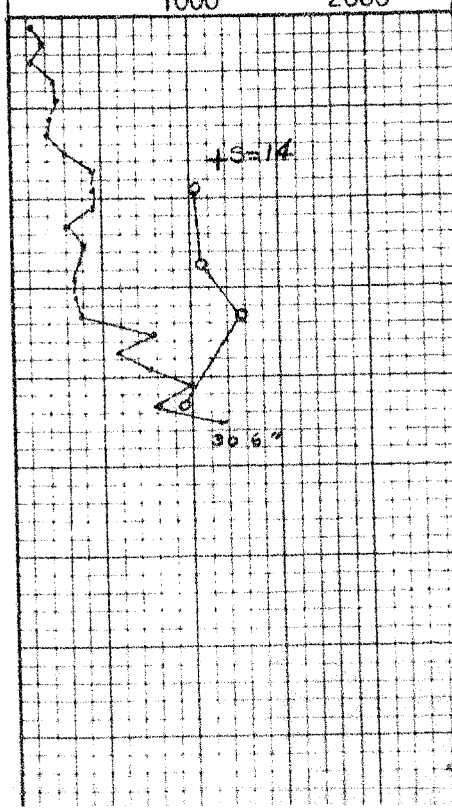
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS
LIQUID LIMIT —o
PLASTIC LIMIT |—

SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE 350 FT. LB BLOWS/FT (20, 40, 60, 80); SHEAR STRENGTH PSF (1000, 2000)	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
	Ground surface	155.4	0				
	Very stiff grey brown fissured clay becoming stiff below about 5 ft.; stiff grey reworked clay with some gravel at 8 ft.	146.3	10	+S=14		1 Layered 2 Layered 3	
	Dense cohesive silty sand with numerous gravel sizes.		20	30.6"		4 5 6	
	End of Bore	132.3	30 40				



Notes:

1) Hole cased to 20 ft.

2) Cone driven 10 ft. south of hole.

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

Dwg. 10
J542A

DRAWING NO. 9
PROJECT NO. J542

LEGEND

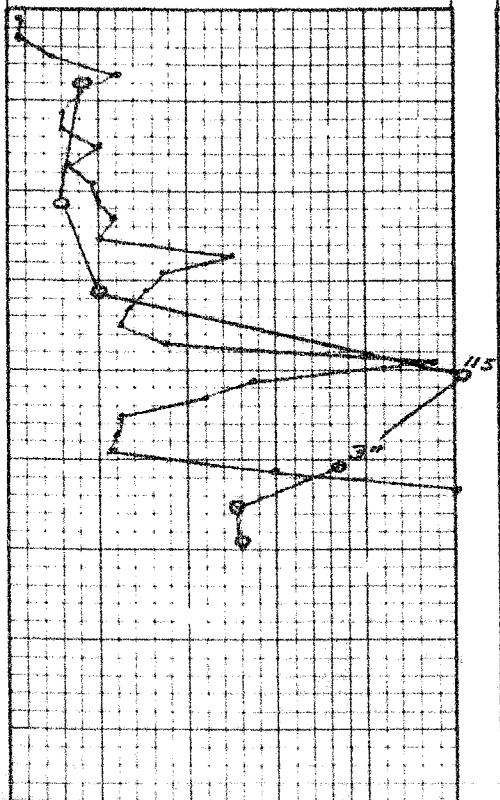
BOREHOLE NO. 8
PROJECT Sutherland Creek & Hwy. No. 2 Crossing Hwy. 401
LOCATION East of Lancaster
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 157.20 ft.
DATUM Top north Guard Rail existing bridge 163.3

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —o—o—o—
2" I.D. SHELBY TUBE —x—x—x—
2" DIA. CONE ————
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}
ATTERBERG LIMITS
LIQUID LIMIT —o
PLASTIC LIMIT ⊢—
SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.
	Ground surface	157.2	0		
	Stiff brown clay numerous gravel sizes	154.3		1	
	Dense brown clayey sandy silt with numerous fine to medium gravel sizes, becoming larger gravel below 15 ft.; large stones below 25 feet. Lost water at 2.7 feet in sand and gravel.		10	2	
			20	3, 4, 5, 6, 7	
	End of Bore	127.3	30		
			40		

PENETRATION RESISTANCE 350 FT. LB BLOWS/FT. 20 40 60 80
SHEAR STRENGTH P S F



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

Notes:
1) Hole cased to 25 ft.
2) Cone driven 8 ft. east of hole.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

Dwg. 11
J542A

DRAWING NO. 10
PROJECT NO. J542

LEGEND

BOREHOLE NO. 9
PROJECT Sutherland Creek & Hwy. No. 2 Crossing Hwy. 401
LOCATION East of Lancaster, Ont.
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 157.0 ft.
DATUM Top north Guard Rail existing bridge 163.3

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
2" DIA. CONE

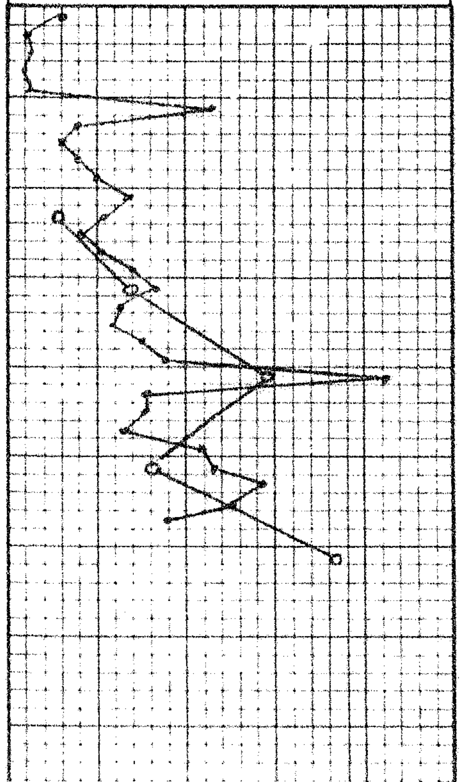
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT

SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE 350 FT. LB BLOWS/FT. (20, 40, 60, 80) / SHEAR STRENGTH P.S.F.	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.
	Ground surface	157.0	0				
	Stiff brown slightly cohesive coarse silt with then layers of clay at about 1 inch intervals.	151.1				1 Levered	
	Dense grey slightly cohesive silty sand with numerous gravel sizes; numerous large gravel sizes or stones below 20 ft.		10			2	
						3	
			20			4	
						5	
	End of Bore	127.3	30			6	
			40				



Notes:

1) Hole cased to 30 ft.

2) Cone driven 5 ft. east of hole.

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

Dwg. 12
J542A

DRAWING NO. 11
PROJECT NO. J54

LEGEND

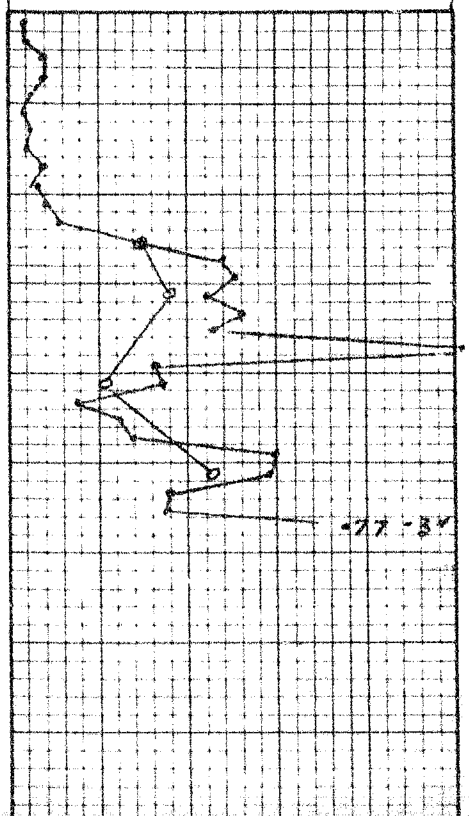
PENETRATION RESISTANCE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
2" DIA. CONE
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT
SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

BOREHOLE NO. 10
PROJECT Sutherland Creek & Hwy. No. 2 Crossing Hwy. 401
LOCATION East of Lancaster, Ont.
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 155.4 ft.
DATUM Top north Guard Rail existing bridge 163.3

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F
	Ground surface	155.4	0		
	Very stiff brown grey fissured clay becoming stiff at 5 ft. and soft below about 8 ft. Numerous silt layers and some gravel 8½ - 10 ft.	146.4	10	1 Levered 2 Levered 3 Levered	
	Dense clayey silty sand with numerous angular gravel.	124.3	20 30	4 5 6 7	
	Bedrock Layers of limestone and shale; shale more prominent with depth. End of Bore	118.3	40		

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT — 20 40 60 80 — SHEAR STRENGTH PSF



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

Notes:
1) Hole cased to 29 ft.
2) Cone driven 5 ft. north of hole.

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

Dwg. 13
J542A

DRAWING No. 12
PROJECT No. J542

LEGEND

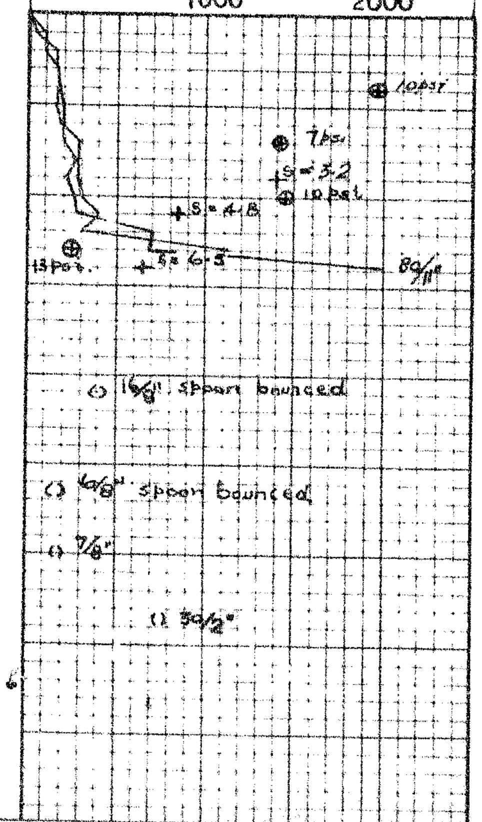
BOREHOLE No. 11
PROJECT Sutherland Creek & Hwy. No. 2 Crossing Hwy. 401
LOCATION East of Lancaster, Ont.
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 156.0 ft.
DATUM Water level Sutherland Creek = 152.13
On August 8, 1960

PENETRATION RESISTANCE	
2" O.D. SPLIT TUBE	—o—o—o—
2" I.D. SHELBY TUBE	—x—x—x—
2" DIA. CONE	———
SHEAR STRENGTH	
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE	⊕
UNCONFINED COMPRESSION	⊗
VANE TEST AND SENSITIVITY (S)	$+^{S}$

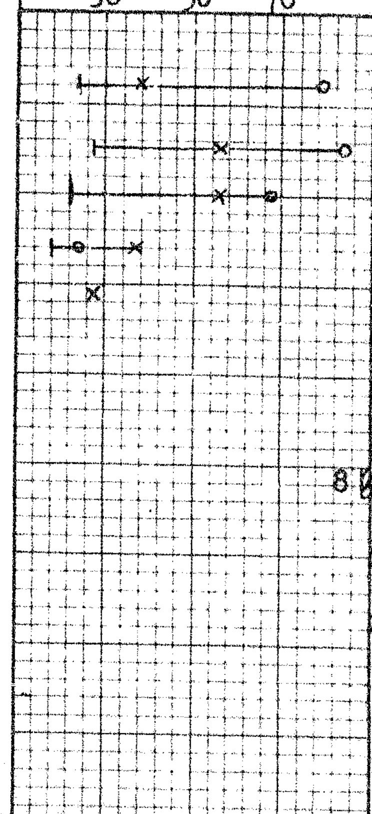
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX	X^{LI}
ATTERBERG LIMITS	
LIQUID LIMIT	—o
PLASTIC LIMIT	⊢—
SAMPLE TYPE	
2" O.D. SPLIT TUBE	
2" I.D. SHELBY TUBE	
3" O.D. SHELBY TUBE	

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET
	Ground surface	156.0	0
	About 8 ins. topsoil		
	Fissured very stiff grey brown clay becoming less stiff with depth. ▽	152.3	
	Quite silty below about 13 ft.		10
	Shelby difficult to lever at 16 ft.		
		140.0	
	Dense angular gravel in a matrix of slightly cohesive silty sand.		20
			30
	End of Hole	121.0	
			40

PENETRATION RESISTANCE 350 FT. LB BLOWS/FT: 20, 40, 60, 80
SHEAR STRENGTH P.S.F.: 1000, 2000



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT: 30, 50, 70



SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
1	Levered 10" Driven rest for 23 blow
2	Levered
3	Levered
4	Levered
5	Levered Difficult at 16'
6	
7	Lost 7 & 8
8	
9	
10	
11	

Notes: 1) Carboloy bit on pressure at 35 ft. (Probable bedrock)
2) Hole cased to 33 ft.
3) Cones driven 5 ft. & 10 ft. south of hole refusal ~~at 13 and 13ft. 11 ins. respectively.~~

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SITE INVESTIGATIONS   SOIL MECHANICS CONSULTATION

Dwg. 14
J542A

DRAWING NO 13
PROJECT NO J542

LEGEND

PENETRATION RESISTANCE

- 2" O.D. SPLIT TUBE —o—o—o—
- 2" I.D. SHELBY TUBE —x—x—x—
- 2" DIA CONE ————

SHEAR STRENGTH

- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) +S

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI

ATTERBERG LIMITS

- LIQUID LIMIT ——o
- PLASTIC LIMIT |——

SAMPLE TYPE

- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

BOREHOLE NO. 12

PROJECT Sutherland Creek & Hwy. No.2 Crossing Hwy. 401

LOCATION East of Lancaster, Ont.

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 156.4 ft.

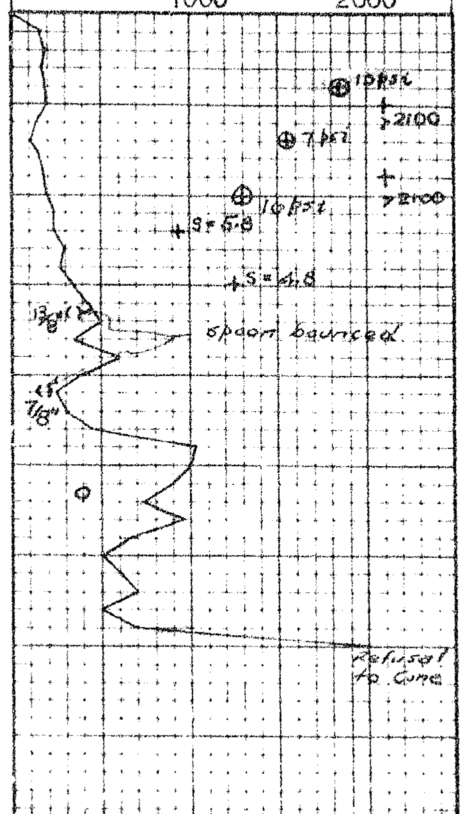
DATUM Water level Sutherland Creek = 152.13 On August 8, 1960

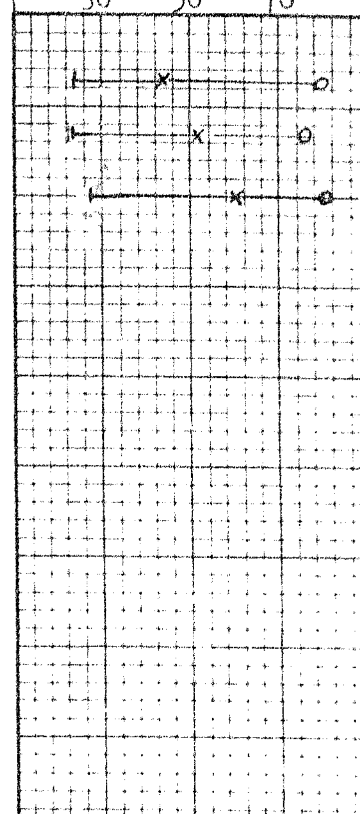
SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET
	Ground surface	156.4	0
	About 8 ins. of dark brown topsoil		
	Very stiff grey fissured clay becoming stiff below 9 ft. and medium stiff at approximately 12 ft. Refusal to open end Shelby at 15.3 ft.		10
		141.1	
	Dense angular gravel in a matrix of slightly cohesive silty sand.		20
	End of Hole	129.4	
			30

Notes:

1) Hole cased to 25 ft.

2) Cone driven 8 ft. south of hole.





SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.
1 Levered	
2 Levered	
3 Levered	
4 Levered	
+ 140 lb.wt.	
5 Driven	
6 Refusal 15'.	
7	
8	

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SITE INVESTIGATIONS   SOIL MECHANICS CONSULTATION

DRAWING NO. 15

PROJECT NO. 542A

LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
2" DIA. CONE

SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
UNCONFINED COMPRESSION
VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT

SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

BOREHOLE NO. 13

PROJECT Proposed Crossing Hwys. 401 & 2

LOCATION Sutherland Creek, Lancaster, Ont.

HOLE LOCATION See Dwg. 1

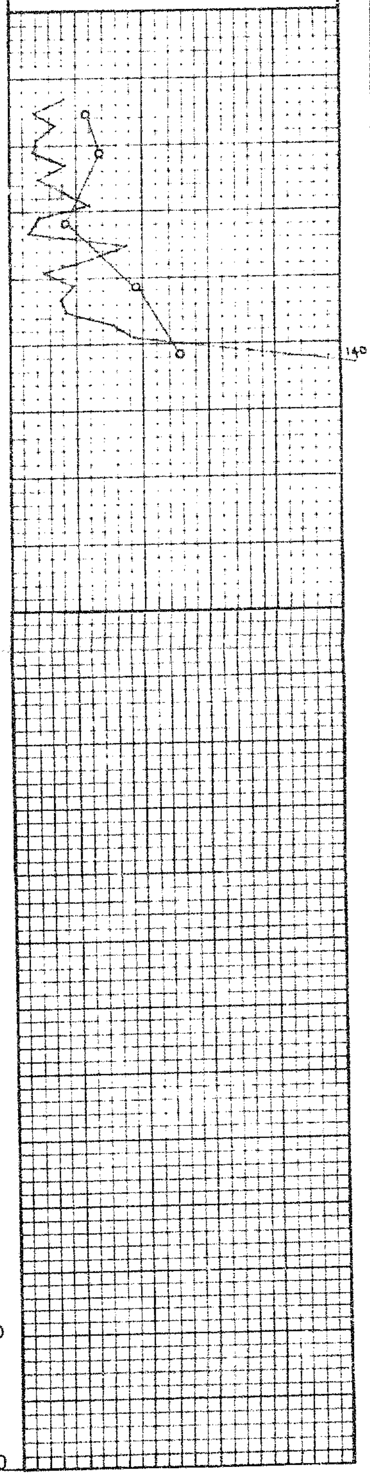
HOLE ELEVATION 152.6 feet

DATUM See Dwg. 1

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
	Water Surface	152.6	0		
	Water				
		146.6			
	Organic muck, some sand	145.4		1	
	SANDY SILT GRAVEL TILL - -loose to mediume dense fine to coarse angular gravel, predominantly shale, in a slightly cohesive sandy silt matrix		10	2	
				3	
			20	4	
	-more cohesive at ~ 25 ft.			5	
		125.0			
	Boulders below $27\frac{3}{4}$ feet-16 in. boulder from $27\frac{3}{4}$ ft. -almost continuous boulders with sandy silt till matrix		30		
	-3 ft. boulder from 33 ft.				
		113.4	40		
	BEDROCK -slightly argillaceous limestone. 100% Recov.				
	End of Hole 99% Recov.	103.5	50		

PENETRATION RESISTANCE 350 FT. LB BLOWS/FT 20 40 60 80; SHEAR STRENGTH PSF

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT



NOTES:
1) Hole advanced to $27\frac{3}{4}$ ft. by alternately driving BX casing and washing clean. Hole continued to bedrock by running AX casing or AX core barrel as necessary to progress through boulders.
2) Bedrock recovered in AX core barrel. Drill on continuous pressure with full water return.
3) Artesian condition encountered at ~ 33 ft. Rose to 154.8 ft. Flow about 1 g.p.m. Sealed on withdrawal of casing.
4) Cone driven 6 ft. west of boring.

Depth scale (feet): 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No. 16
PROJECT No. J542A

LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
2" DIA. CONE

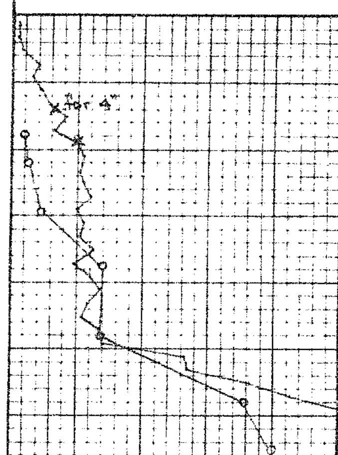
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
UNCONFINED COMPRESSION
VANE TEST AND SENSITIVITY (S)

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX
ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT

SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

BOREHOLE No. 14
PROJECT Proposed crossing Hwys. 401 and 2.
LOCATION Sutherland Creek, Lancaster, Ont.
HOLE LOCATION See Dwg. 1
HOLE ELEVATION 152.9
DATUM See Dwg. 1

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No.
	Ground surface	152.9	0	
	~12 ins. black peaty topsoil. SILTY CLAY -med. stiff with thin sand seams and occasional gravel sizes.			1 L
	SANDY SILT GRAVEL TILL -loose to med.dense fine to coarse gravel in slightly cohesive sandy silt binder.	145.6	10	2 L-8" D-4"; 3; 3A D; 4; 5
	-higher sand content and non-cohesive at 20-22 ft.		20	6
	-dense below ~ 25 ft.			7
		121.9	30	8
	-boulders below 31 ft. -4 boulders between 6 ins.and 11 ins. cored. Probable sandy matric.			9 No recov.
	BEDROCK -slightly argillaceous limestone. 100% Recov. " "	113.5	40	
	End of hole	100.1	50	



NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

NATURAL UNIT WEIGHT P.C.F.

* L-Levered
D-Driven

NOTES:
1) Hole advanced to 31 ft. by alternately driving EX casing and washing clean. Hole continued to bedrock by running AX casing or AX core barrel as necessary to progress through boulders.
2) Bedrock recovered in AX core barrel Drill on continuous pressure with full water recovery.
3) Slight artesian condition encountered at ~ 33 ft.; rose to El. 154.6 in casing. Boring sealed with bentonite as casing withdrawn.
4) Cone driven 5 ft. east of boring.

Depth scale: 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110

LEGEND

BOREHOLE NO 15

PROJECT Proposed Crossing, Hwys. 401 and 2

LOCATION Sutherland Creek, Lancaster, Ont.

HOLE LOCATION See Dwg. 1

HOLE ELEVATION 156.3 ft.

DATUM See Dwg. 1

PENETRATION RESISTANCE

- 2" O.D. SPLIT TUBE —o—o—o—
- 2" I.D. SHELBY TUBE —x—x—x—
- 2" DIA CONE ———

SHEAR STRENGTH

- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) $+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

ATTERBERG LIMITS

- LIQUID LIMIT —o
- PLASTIC LIMIT |—

SAMPLE TYPE

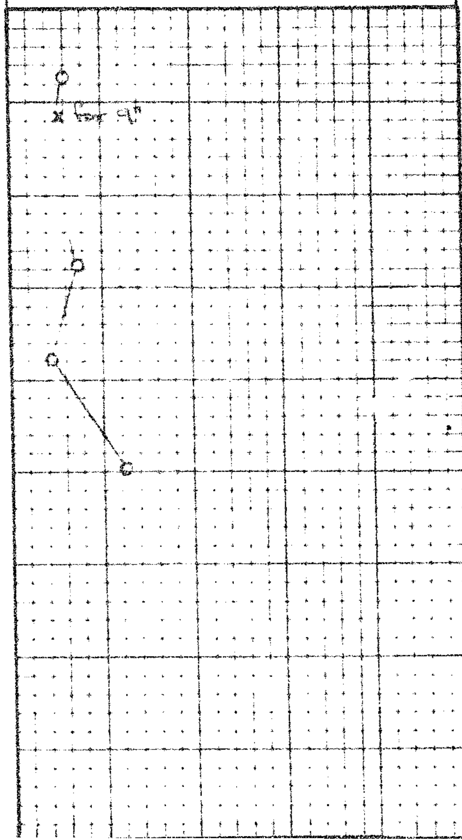
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND NO	NATURAL UNIT WEIGHT P.C.F.
	Ground surface	156.3	0		
	8" topsoil, ~ 2 ft. dry sandy silt.				
	SILTY CLAY - very stiff mottled brown and desiccated	153.3		1	
	-odd fine gravel sizes.			2-L -9" D- 9"	
	-becomes less stiff with depth			3-L	
	-varved with clayey silt below ~ 9 ft.		10	4-L	
	-soft moderately cohesive clayey silt below 11 ft.	144.2		5-L-8" D-9"	
				6	
	SANDY SILT GRAVEL TILL -loose to med.dense fine to coarse gravel sizes in moderately cohesive sandy silt binder.		20	7	
	-slightly higher sand content with depth			8	
	End of hole	130.8			
			30		

PENETRATION RESISTANCE 350 FT LB BLOWS/FT 20 40 60 80

SHEAR STRENGTH P.S.F.

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT



NOTES:

1) Hole advanced by alternately driving EX casing and washing clean.
2) Water level after 28 hrs.-4½ ft.
   " 34 " 3 ft.
   After removal of casing -3 ft.

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 18

PROJECT No. J542

LEGEND

PENETRATION RESISTANCE	
2" O.D. SPLIT TUBE	—o—o—o—
2" I.D. SHELBY TUBE	—x—x—x—x—
2" DIA. CONE	———
SHEAR STRENGTH	
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE	⊕
UNCONFINED COMPRESSION	⊗
VANE TEST AND SENSITIVITY (S)	$+^{S}$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX	X^{LI}
ATTERBERG LIMITS	
LIQUID LIMIT	—o
PLASTIC LIMIT	├—
SAMPLE TYPE	
2" O.D. SPLIT TUBE	
2" I.D. SHELBY TUBE	
3" O.D. SHELBY TUBE	

BOREHOLE No. 16

PROJECT Proposed Crossing Hwys.401 & 2

LOCATION Sutherland Creek, Lancaster, Ont.

HOLE LOCATION See Dwg.1

HOLE ELEVATION 156.4

DATUM See Dwg.1

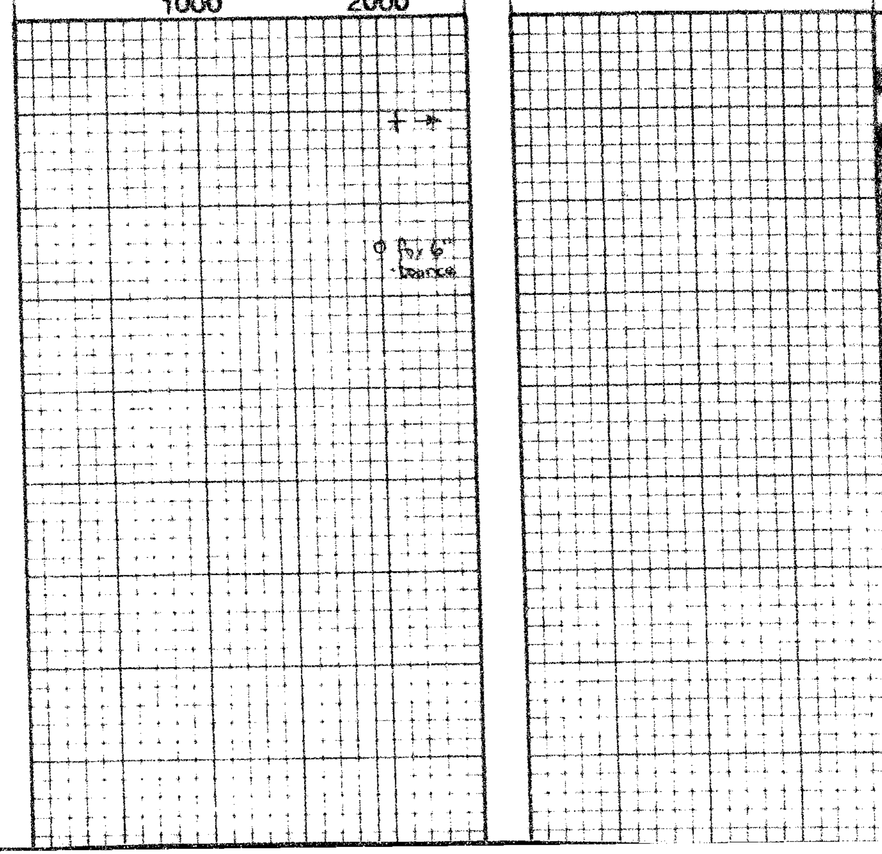
SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
	Ground surface	156.4	0		
	~7" topsoil, 2 ft. dry sandy silt				
	SILTY CLAY -stiff mottled brown and desiccated	153.0		1 -L	
	-becomes less stiff with depth.			2 -L	
				3 -L	
	-varved with clayey silt below ~9ft.		10	4 -L	
		144.3		5 -L	
	End of hole	143.7		6 -no recov.	
	-refusal on probable boulder in glacial till				
			20	L - Levered	
			30		
			40		

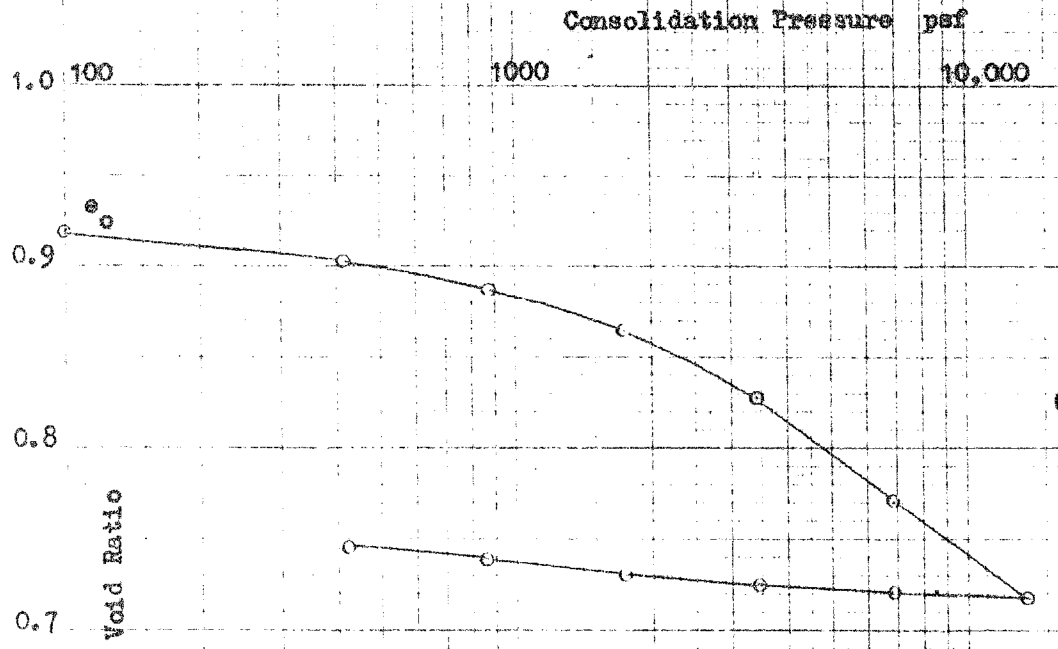
NOTES: 1) Hole advanced by driving and withdrawing BX casing. Below 9¼ ft. samples recovered with piston sampler.

PENETRATION RESISTANCE 350 FT. LB. BLOWS/FT: 20 40 60 80

SHEAR STRENGTH P.S.F: 1000 2000

NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT





Coefficients of Consolidation

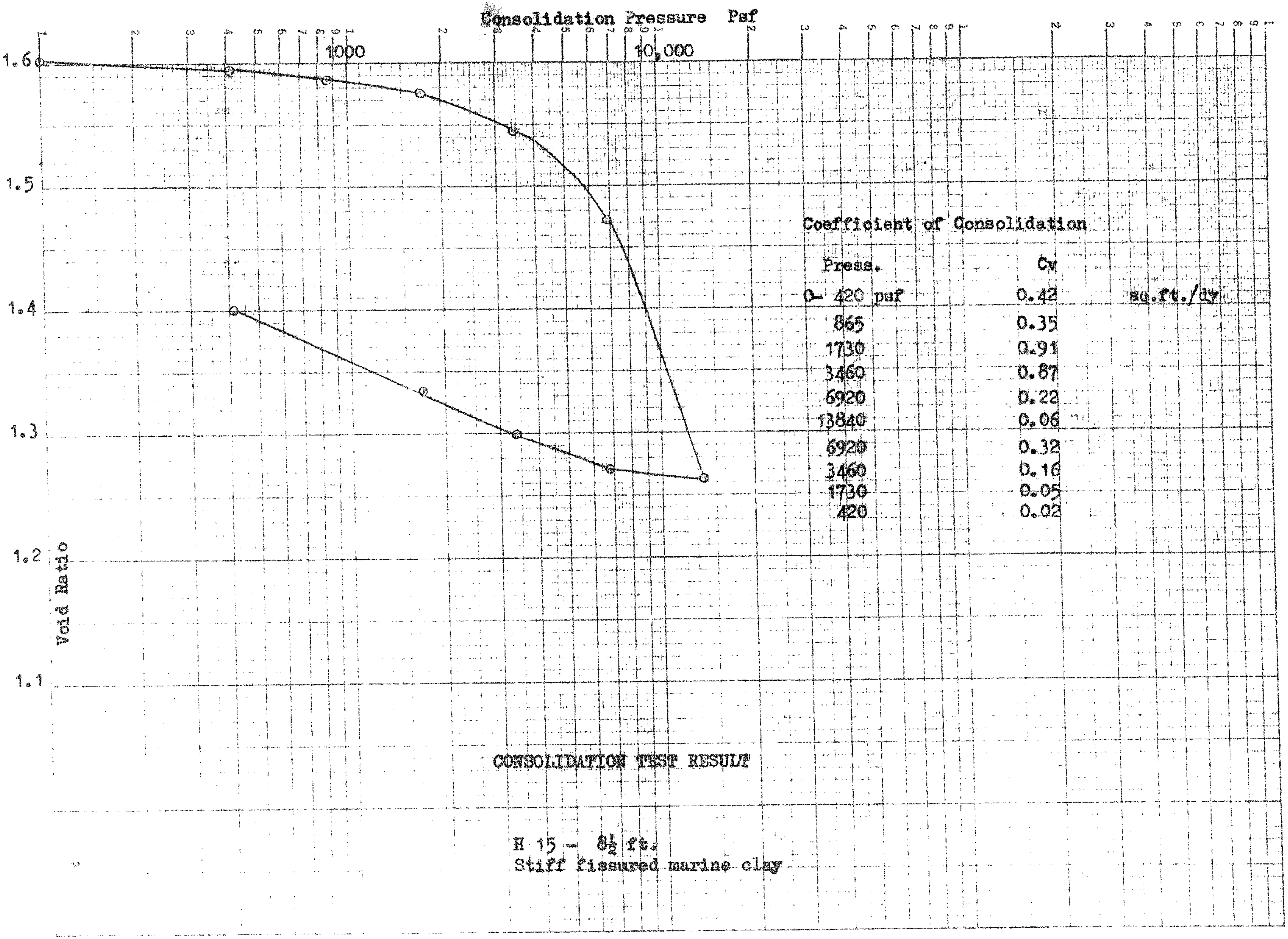
0 - 420 psf	0.44	sq.ft./dy.
865	0.44	
1730	0.93	
3460	0.89	
6920	0.45	
13840	0.67	
6920		
3460	3.90	
1730	1.07	
865	0.57	
420	0.24	

CONSOLIDATION TEST RESULTS

- BH16 - 11'6" - slightly clayey silt

J 542A Dwg. 19

K+E SEMI-LOGARITHMIC 359-81G
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS



Coefficient of Consolidation

Press.	Cv	
0- 420 psf	0.42	sq.ft./dy
865	0.35	
1730	0.91	
3460	0.87	
6920	0.22	
13840	0.06	
6920	0.32	
3460	0.16	
1730	0.05	
420	0.02	

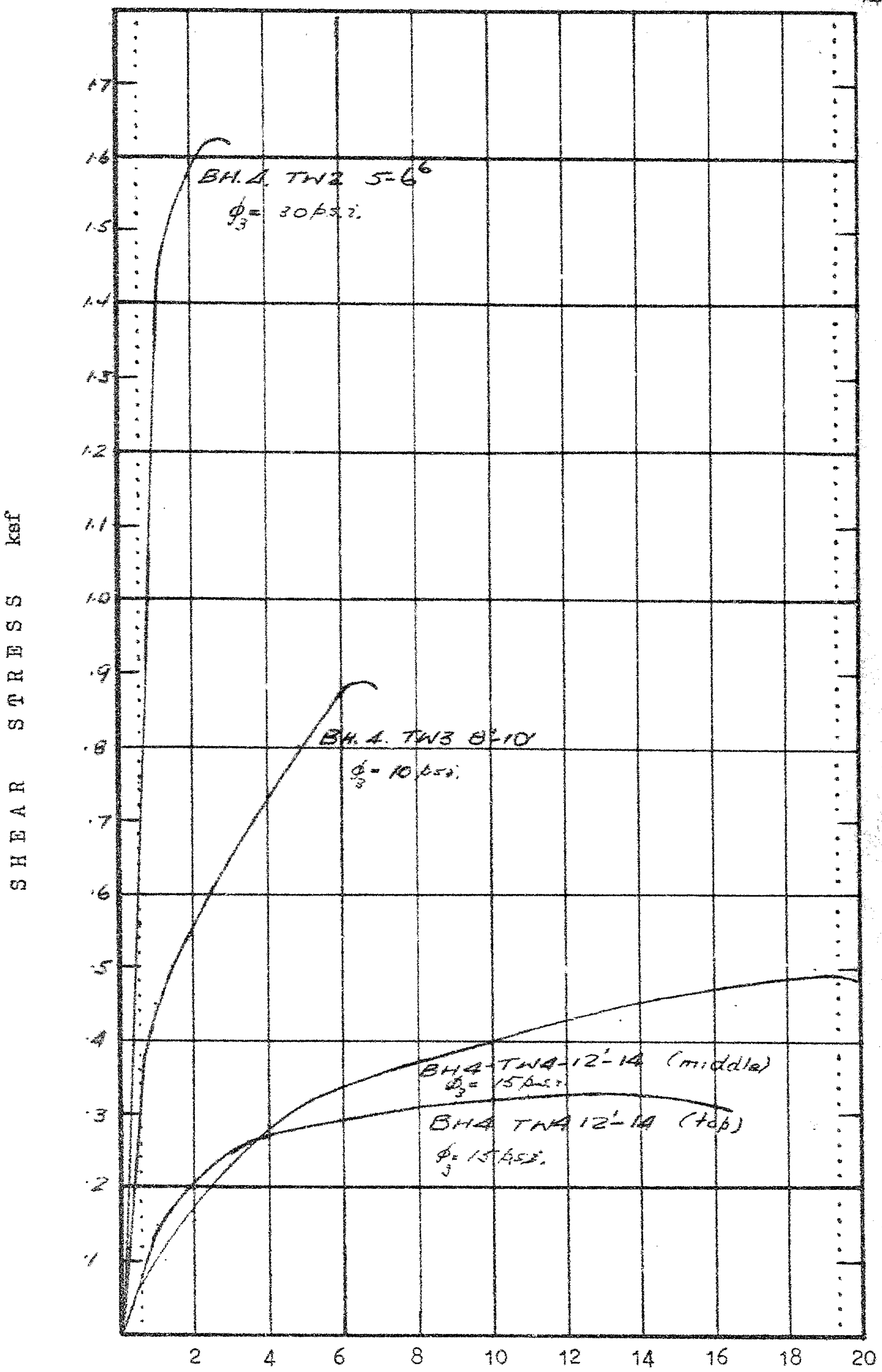
CONSOLIDATION TEST RESULT

H 15 — 8½ ft.
Stiff fissured marine clay

J 542A

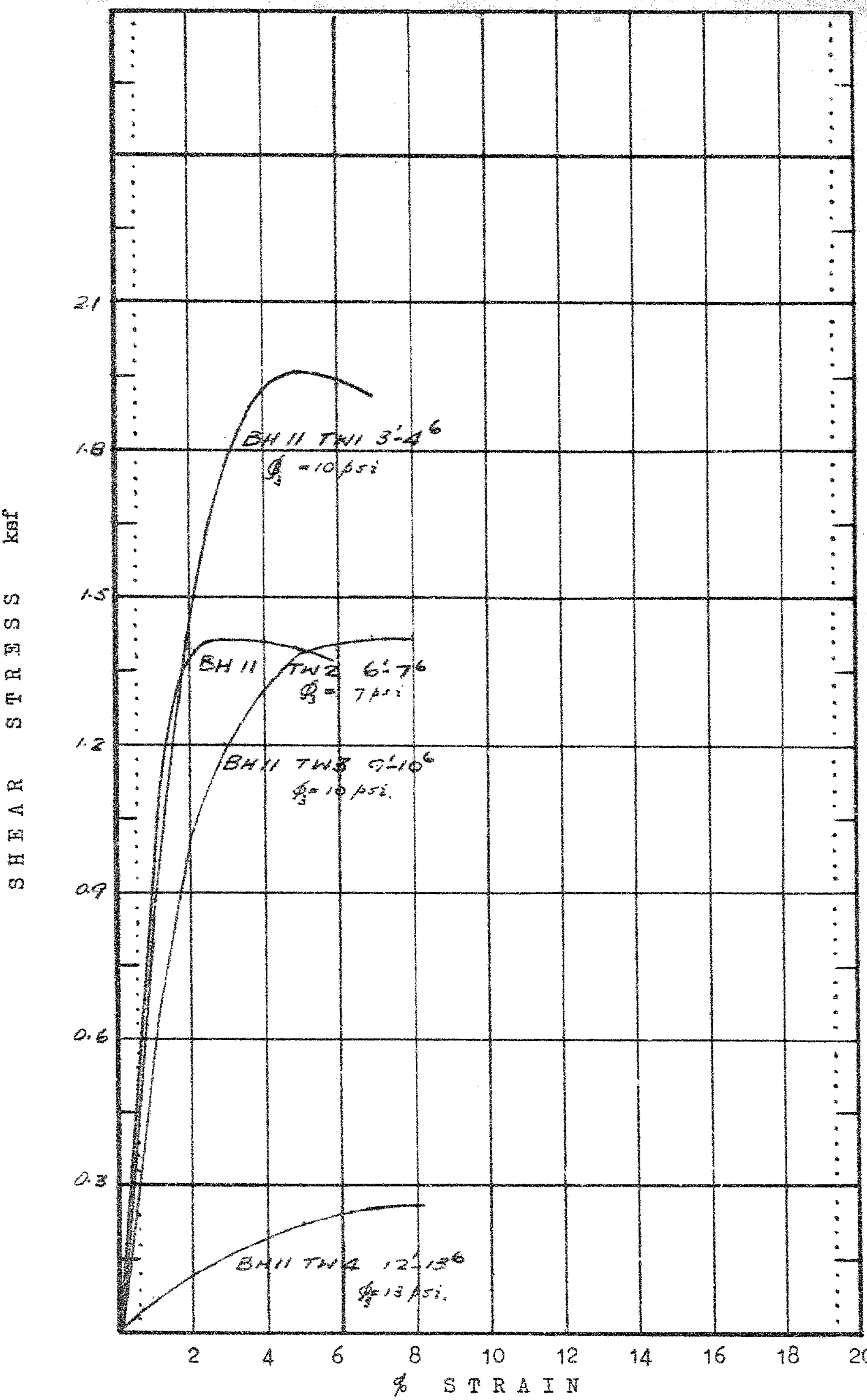
Dwg. 20

PROJECT NO. J 542 DRAWING NO. 14

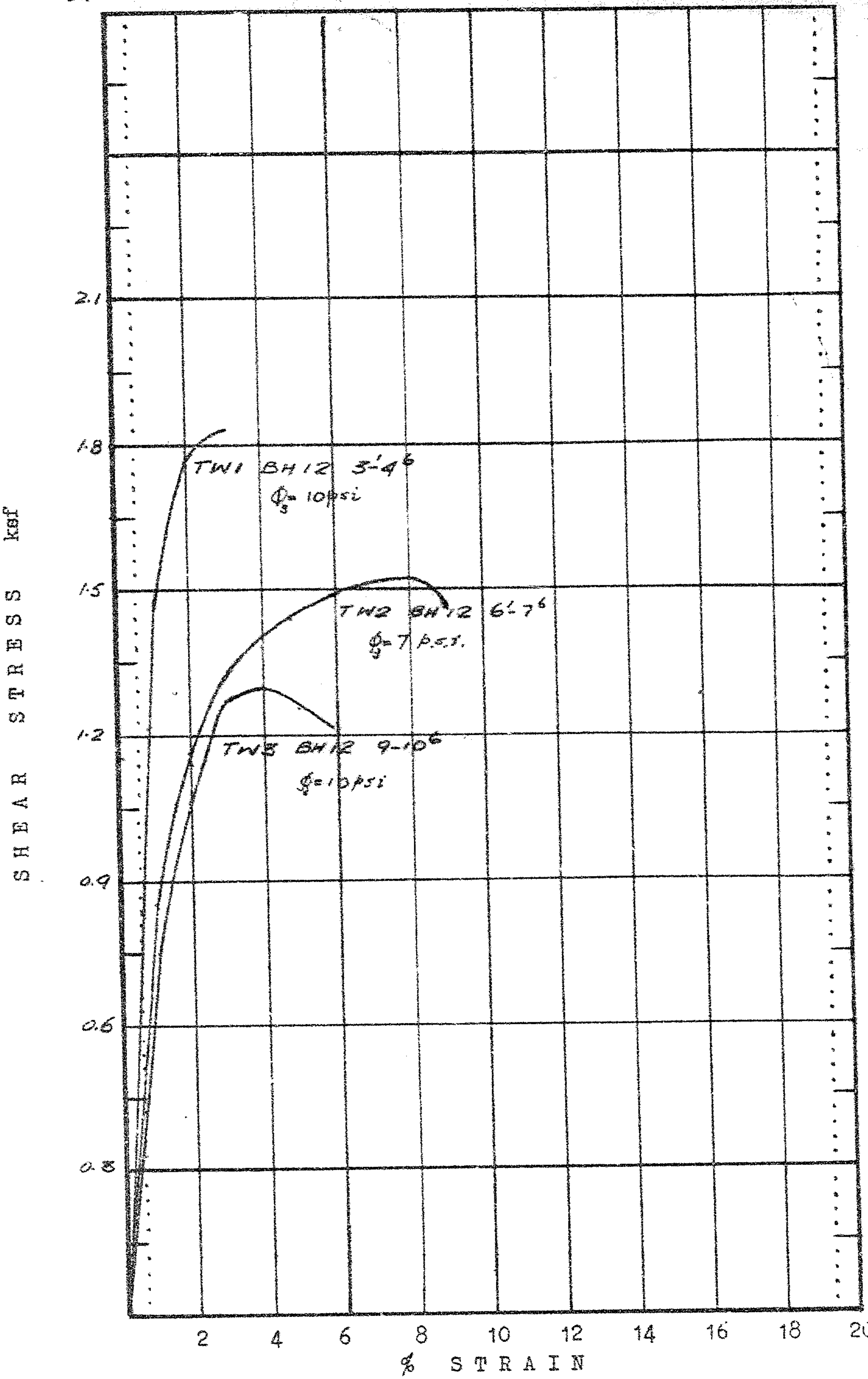


STRESS STRAIN CURVES - UNDRAINED TRIAXIAL TEST RESULTS

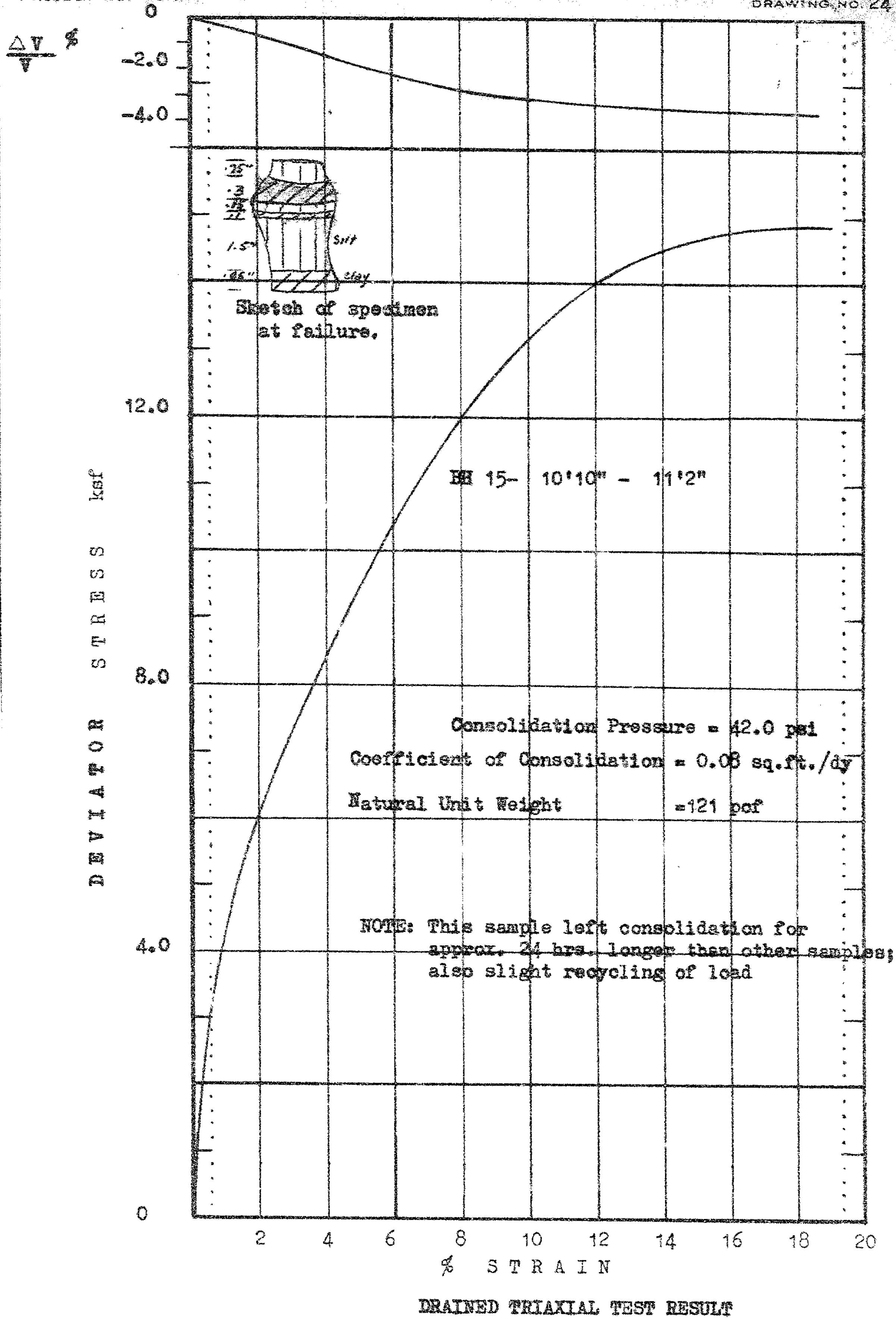
WILLIAM A. TROW AND ASSOCIATES



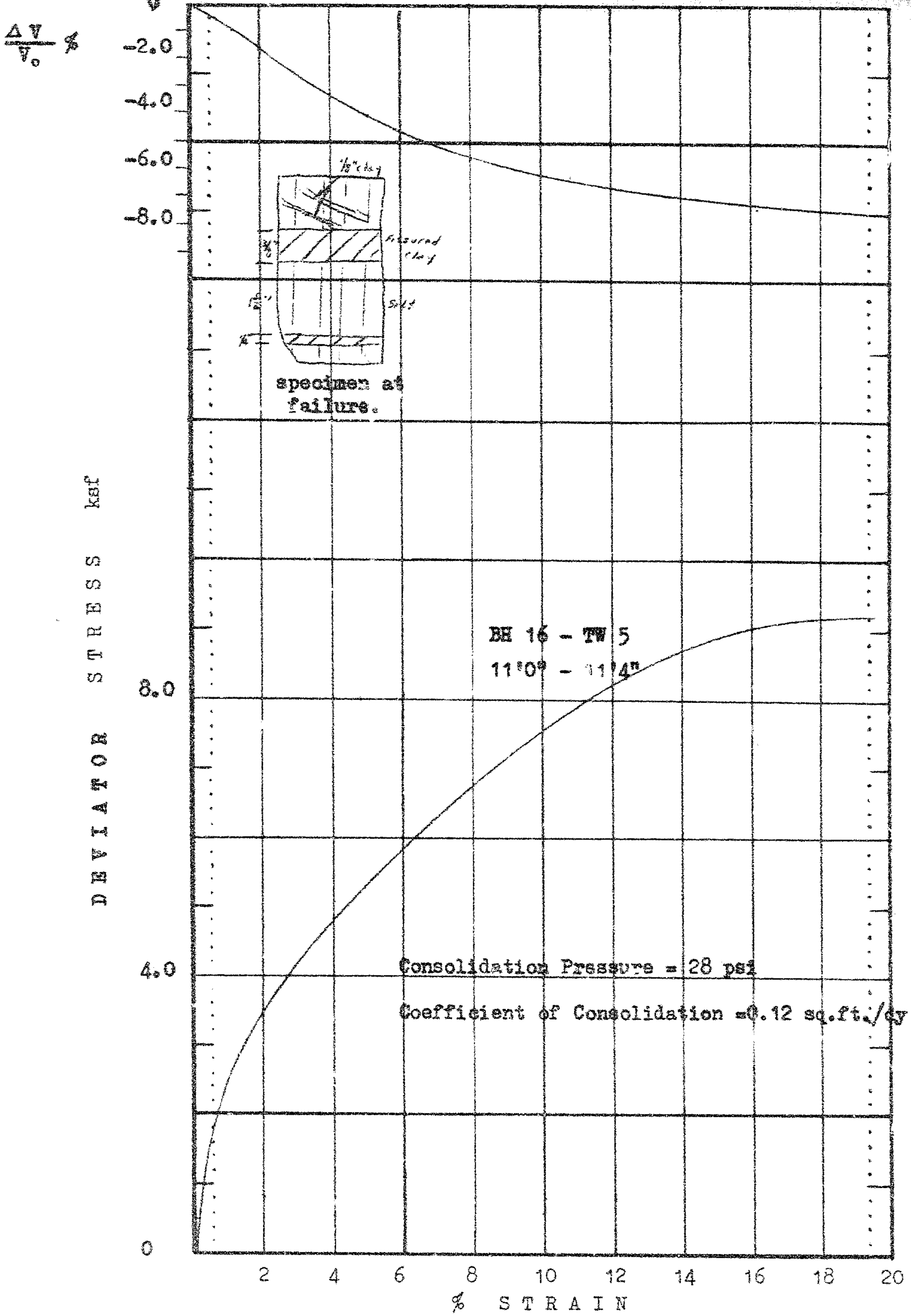
STRESS STRAIN CURVES - UNDRAINED TRIAXIAL TEST RESULTS



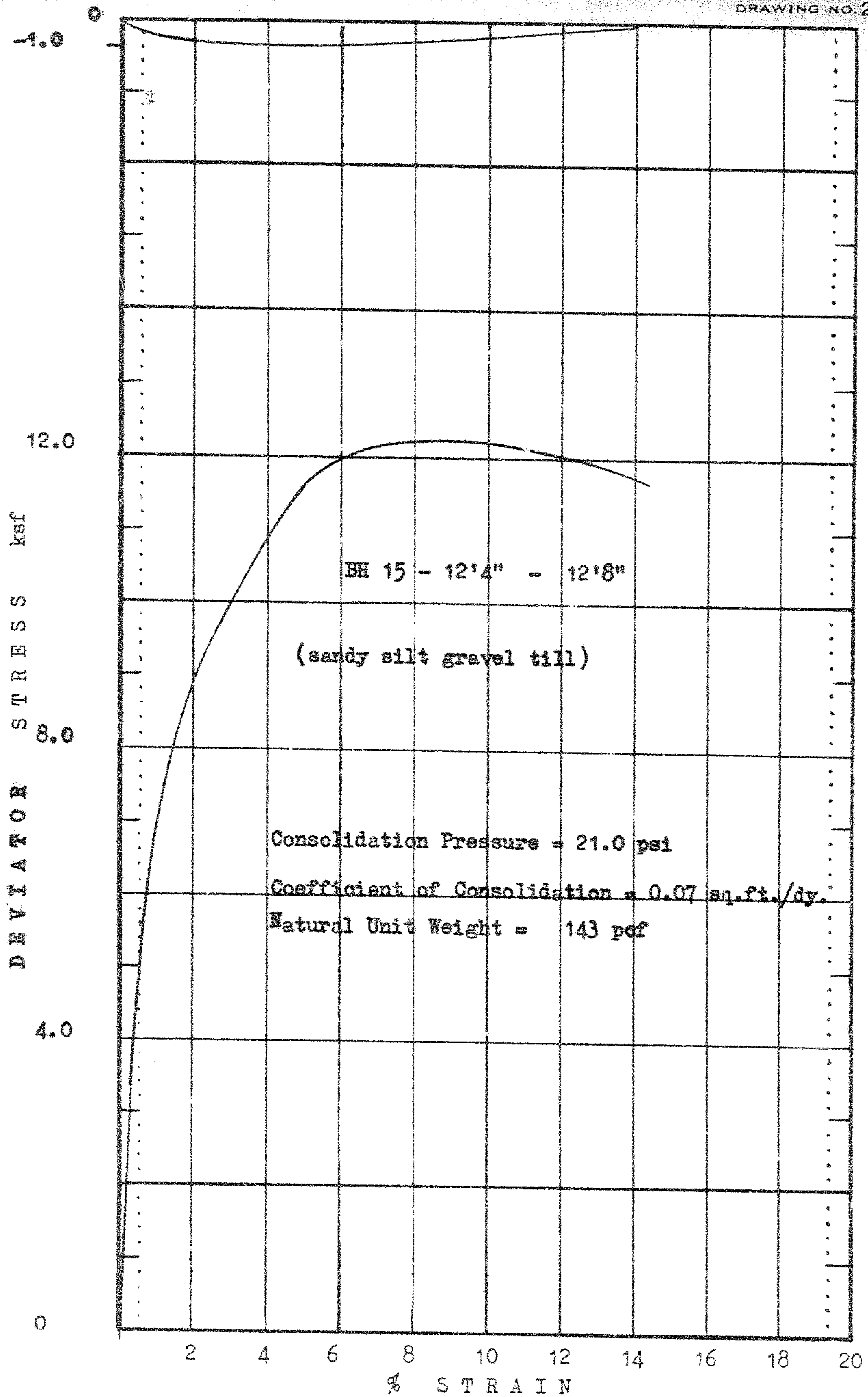
STRESS STRAIN CURVES - UNDRAINED TRIAXIAL TEST RESULTS



DRAINED TRIAXIAL TEST RESULT

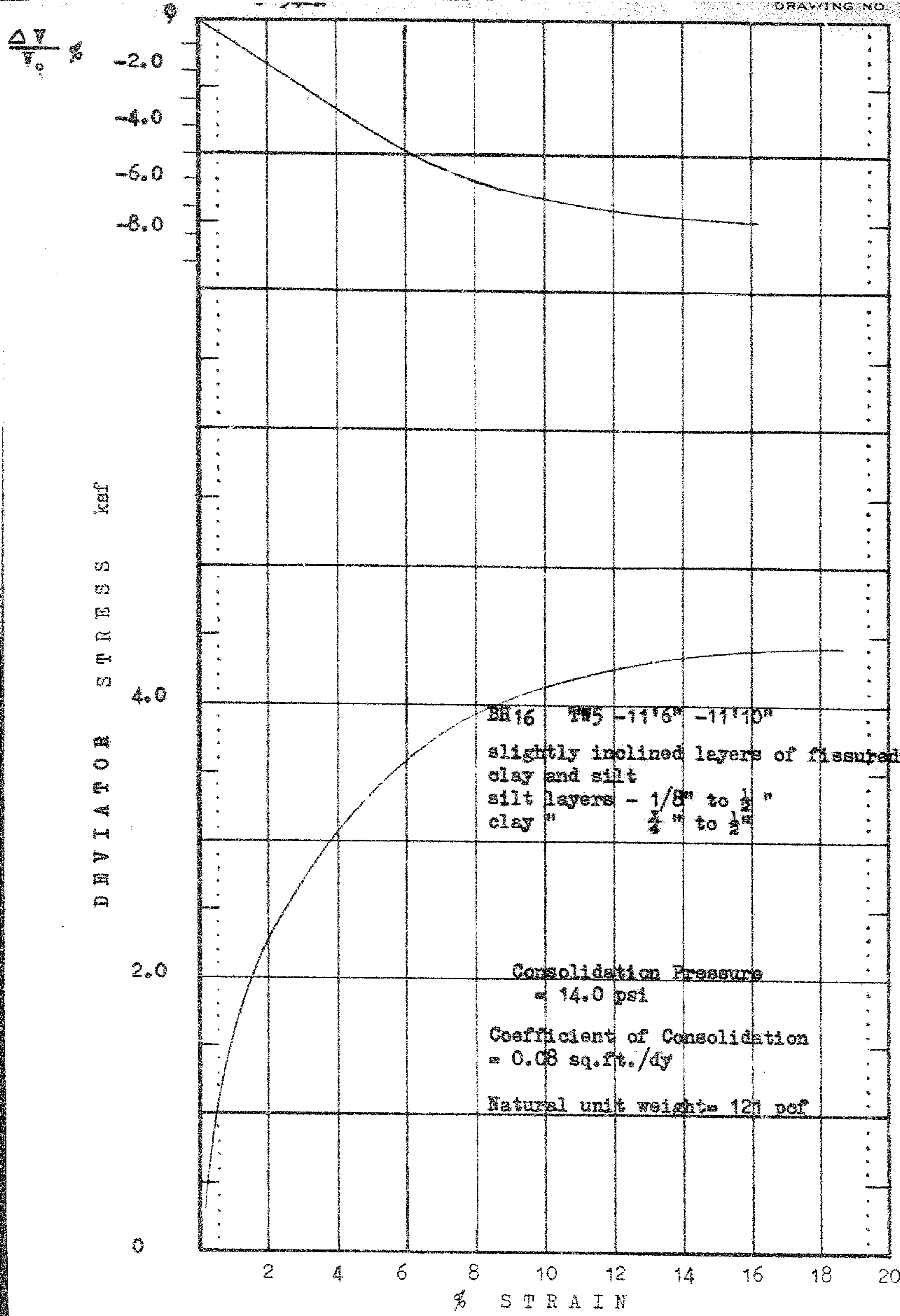


DRAINED TRIAXIAL TEST RESULT



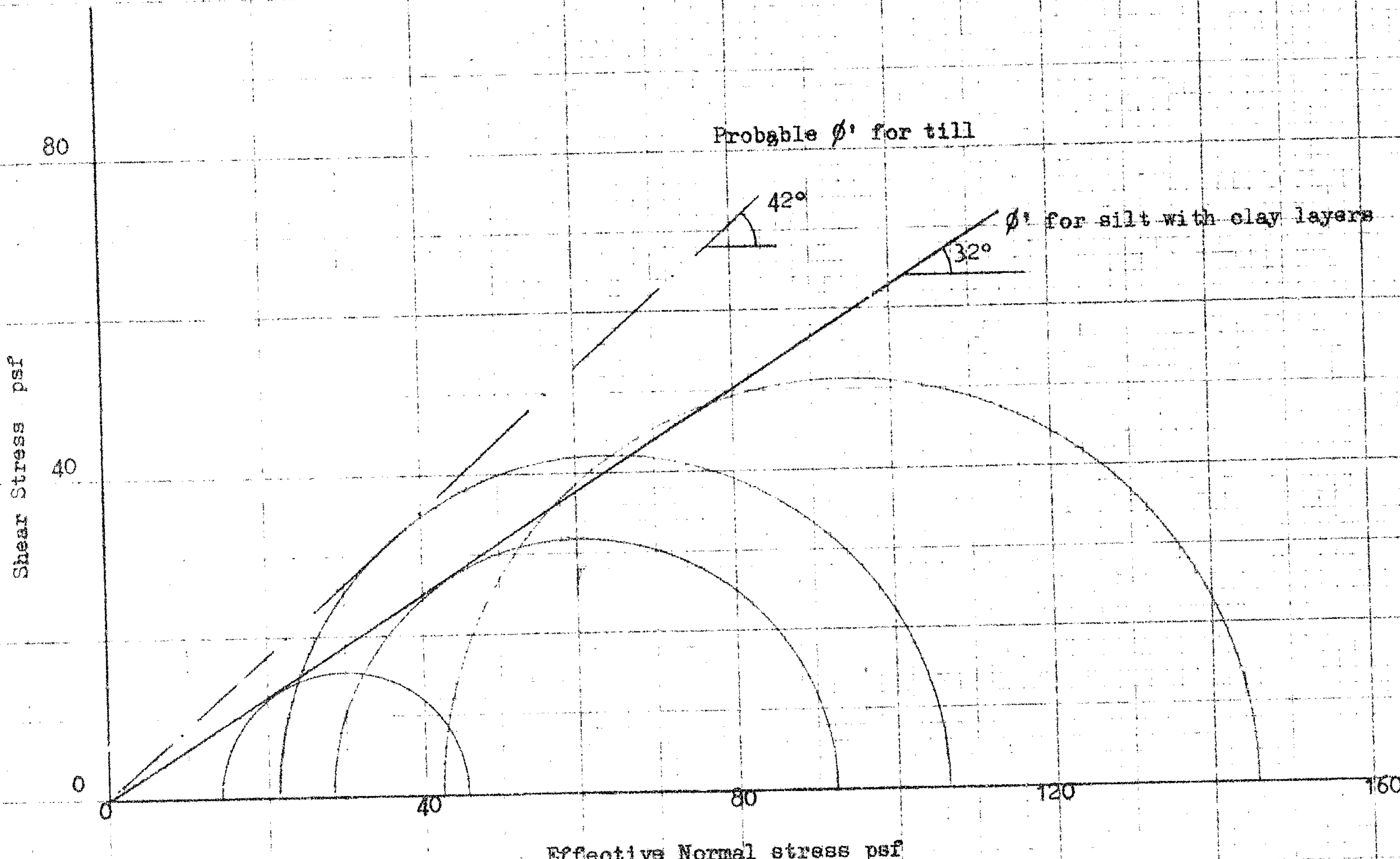
DRAINED TRIAXIAL TEST RESULT

WILLIAM A. TROW AND ASSOCIATES



DRAINED TRIAXIAL TEST RESULT

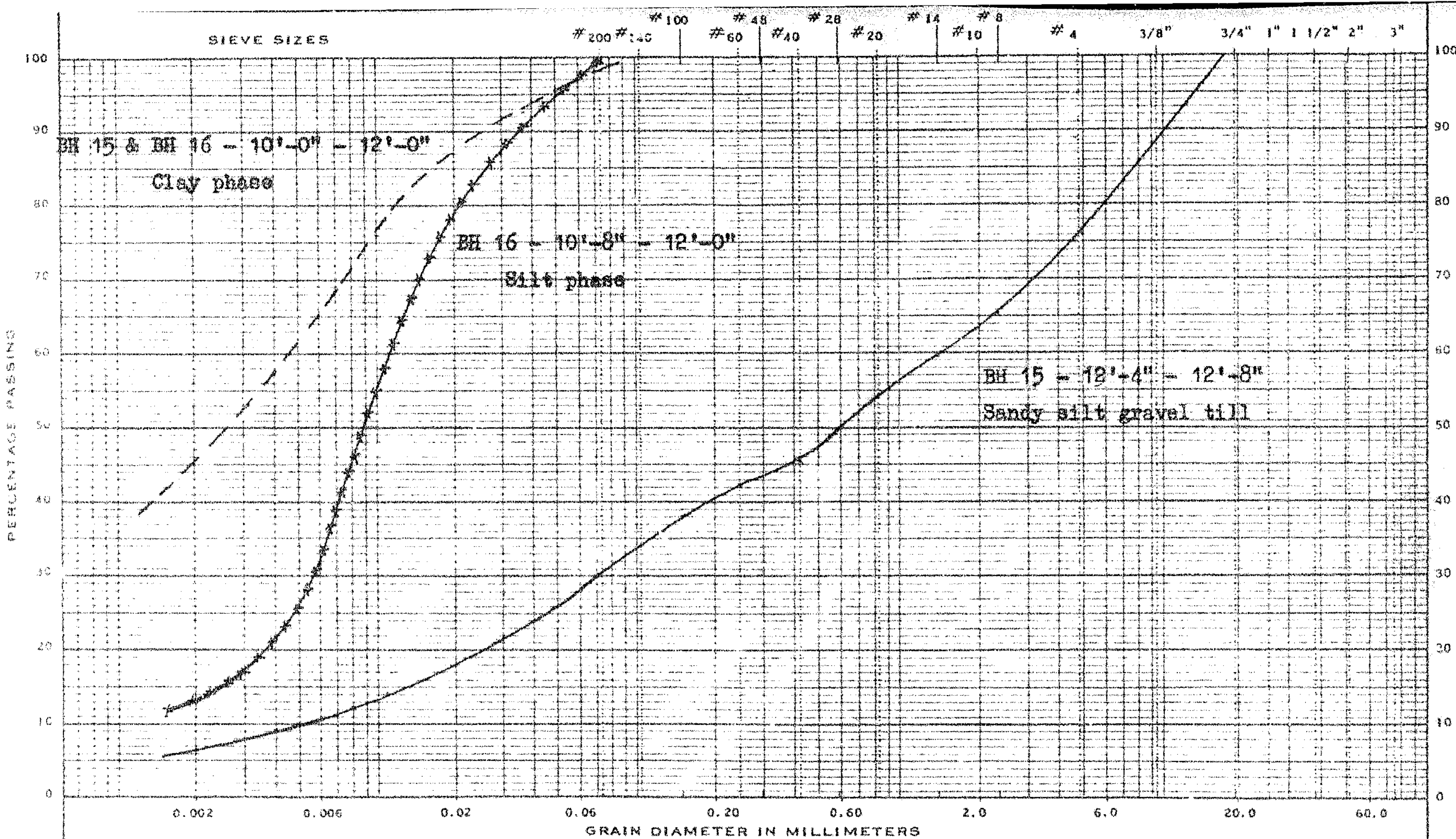
WILLIAM A. TROW AND ASSOCIATES



RESULTS OF CONSOLIDATED UNDRAINED TESTS

J542A Dwg. 28

J542A



CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT			SAND			GRAVEL		

MODIFIED M.I.T. CLASSIFICATION

GRAIN SIZE DISTRIBUTION OF RESULTS

WILLIAM A. TROW AND ASSOCIATES LTD.

Dwg. 29

ONTARIO

DEPARTMENT OF HIGHWAYS

Memo to Mr. A. M. Toye, | *Date* November 8, 1961.

Bridge Engineer. | *Subject* FOUNDATION INVESTIGATION REPO

From Materials & Research Division, | By: W.A. Trow & Associates,

(Foundation Section).

Attention: Mr. S. McCombie.

Re: W.P. 177-60 and W.P. 116-59,
Hwys. #401 and #2, District #9.

We have reviewed the Consultants' report for the above projects, and submit the following comments:-

(1) Structure Foundations:

The Consultant has recommended Franki type caisson piles for the Hwy. #2 structure, and timber piles for the Hwy. #401 structure. He has raised some objections to the use of 'H' piles on the grounds that during the process of driving through the dense boulder stratum located above the bedrock, serious distortion and buckling of the piles may result.

In view of the fact that the Department has, on several occasions, driven 'H' piles under similar conditions and obtained satisfactory subsequent performance, we feel that heavy section 'H' piles with reinforced tips should be used for both these structures. A design load of 65 tons per pile may be used. Refusal may occur either on the bedrock or in the boulder stratum.

(2) Embankment Stability:

The Consultant has recommended the installation of sand drains for fills greater than 25' on the Hwy. #2 bridge approaches. The maximum height here, is 28'. This involves a section about 150' long on the East approach. We have discussed this matter with the Consultant and he is in agreement with our

cont'd. /2 ...

Comments cont'd. .....

(2) Embankment Stability: (cont'd.) ...

suggestion that the installation of these drains will not be necessary provided that the section in question, is constructed to a height of 25', then left in place for a period of about 6 months prior to adding the additional material to complete the section to profile grade. We have also discussed this matter with the Road Design Section who are of the opinion that this procedure will cause only minor inconvenience and expense.

If further assistance is required in connection with these projects, please do not hesitate to contact our Office.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.
Per:

K. G. Selby

(K. G. Selby,
SR. PROJECT FOUNDATION ENGR.)

KGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
J. Ford
J. E. Gruspier
L. E. Walker
J. Roy
T. J. Kovich
E. R. Saint
F. Norman
A. Watt
Foundations Office
Gen. Files.

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.

1850 JANE ST.,
WESTON, ONT.
CH. 1-4644

BA 1271

Project: J542A

October 27, 1961

Mr. A. Rutka,
Acting Materials and Research Engineer,
Department of Highways of Ontario,
Parliament Buildings,
Toronto, Ontario

Attention: Mr. N.D. Stermac, P.Eng.,
Principal Soils and Foundations Engineer

Re: Foundation Investigation
Proposed Crossing, Sutherland Creek
Hwy. 401 and Hwy. 2

Dear Sirs:

This report is our formal submission on the foundation conditions at the site of the proposed bridge structures for Hwys. 2 and 401 across Sutherland Creek.

Dense limestone bedrock is encountered about 40 feet below ground surface at this site. It is overlain by about 10 to 19 feet of very dense boulder till. It is understood that H piles to bedrock would be the most economical bridge foundation proposal. While there is no doubt that refusal will be obtained with these piles, some concern is felt about the effects of the possible distortion and damage suffered by them after driving through the dense boulder till deposit.

Because of this uncertainty, we have recommended that the Hwy. 2 overhead structure be founded on the dense boulder till utilizing caisson piles, similar to the Franki type. Timber piles are believed to be more economical for the lighter Hwy. 401 bridge.

Under the east approaches of Hwy. 2, the lower 3 feet of a 12 feet thick surface deposit of stiff fissured marine clay becomes soft and layered, with the silt phase predominating. We have examined the stability of the proposed embankment over this stratum, and it was found to be just safe immediately after construction. However, an insufficient margin of safety exists subsequent to completion of the embankment, due to lateral transfer of pore pressures towards the toe of the embankment.

The installation of sand drains is therefore recommended to control these pore pressures, and thereby ensure a steady increase in safety after construction. The engineering computations which form the basis for these opinions are outlined in the report.

We shall be pleased to discuss any outstanding matter pertaining to this investigation after your review of this report.

Yours very truly,

W. Trow

William A. Trow, P.Eng.

WAT/gc
Enc.

#61-F-206-C

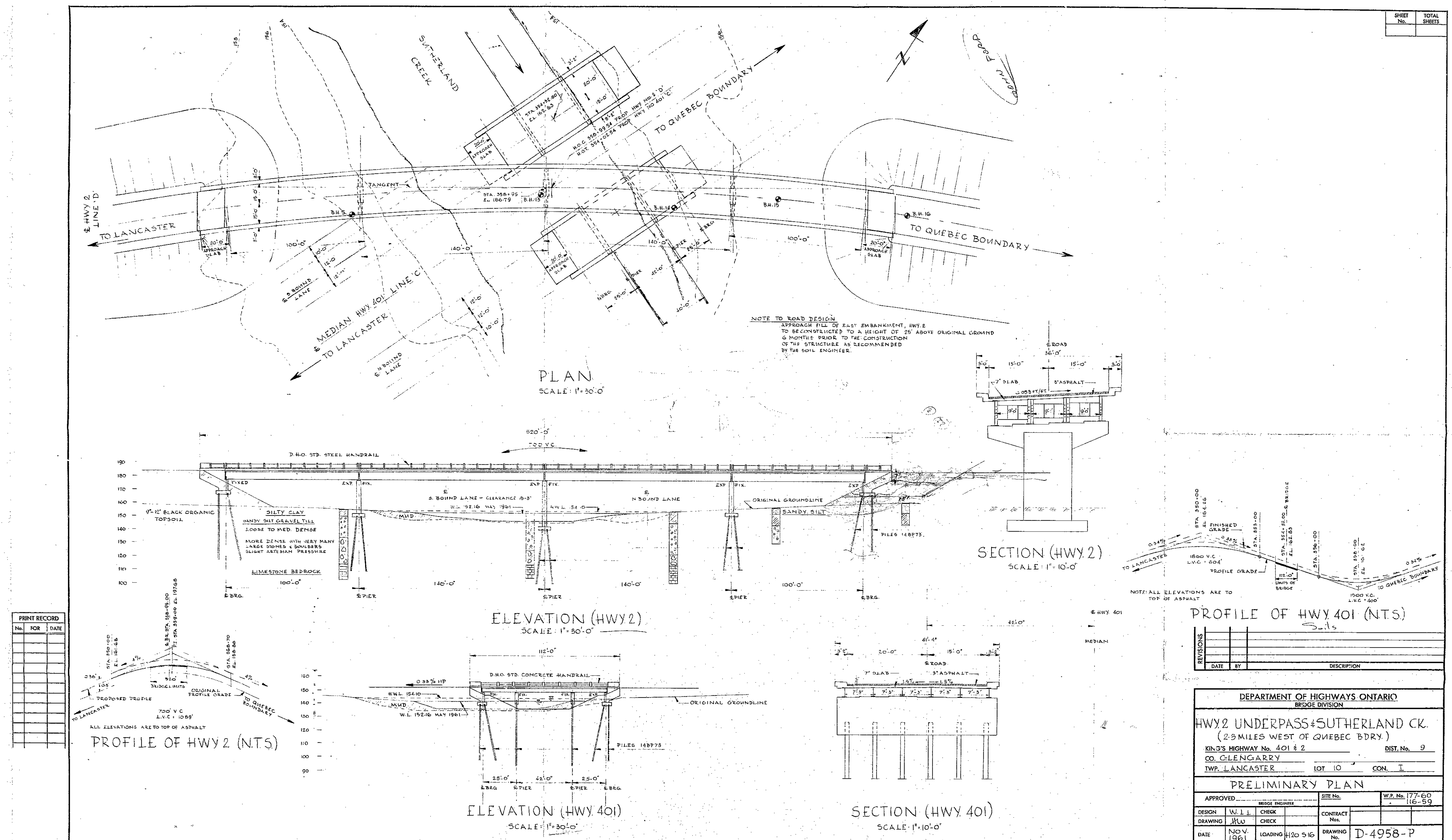
W.P.#116-59

& 177-60

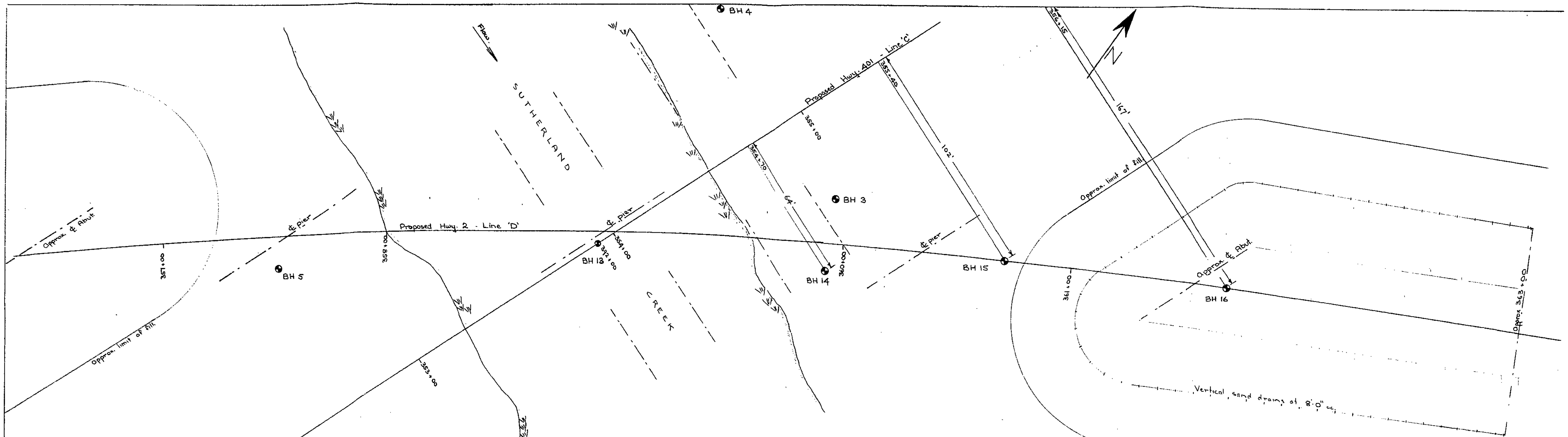
HWY.#401 &

SUTHERLAND

CREEK



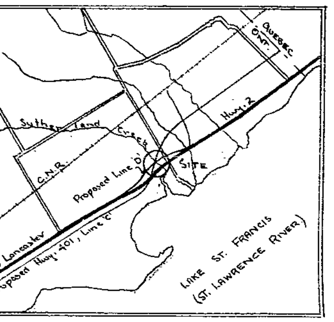
PLAN
SCALE: 1"=30'-0"
SUTHERLAND CREEK
TO QUEBEC BOUNDARY
TO LANCASTER
℄ HWY 2 LINE 'D'
℄ MEDIAN HWY 401 LINE 'C'
TO LANCASTER
℄ S. BOUND LANE
℄ N. BOUND LANE
APPROACH SLAB
B.H.5
B.H.13
B.H.14
B.H.15
B.H.16
TANGENT
NOTE TO ROAD DESIGN
APPROACH FILL OF EAST EMBANKMENT, HWY. 2
TO BE CONSTRUCTED TO A HEIGHT OF 25' ABOVE ORIGINAL GROUND
6 MONTHS PRIOR TO THE CONSTRUCTION
OF THE STRUCTURE AS RECOMMENDED
BY THE SOIL ENGINEER.
D.H.O. STD. STEEL HANDRAIL
700' V.C.
9"-12" BLACK ORGANIC TOPSOIL
SILTY CLAY
SANDY SILT GRAVEL TILL
LOOSE TO MED. DENSE
MORE DENSE WITH VERY MANY LARGE STONES & BOULDERS SLIGHT ARTESIAN PRESSURE
LIMESTONE BEDROCK
℄ S. BOUND LANE – CLEARANCE 16'-3"
℄ N. BOUND LANE
ORIGINAL GROUNDLINE
SANDY SILT
PILES 14BP73
℄ BRG.
℄ PIER
ELEVATION (HWY 2)
SCALE: 1"=30'-0"
SECTION (HWY. 2)
SCALE: 1"=10'-0"
7" SLAB
3" ASPHALT
PROFILE OF HWY. 401 (N.T.S.)
NOTE: ALL ELEVATIONS ARE TO TOP OF ASPHALT
PROFILE GRADE
FINISHED GRADE
TO LANCASTER
TO QUEBEC BOUNDARY
1800 V.C.
1500 V.C.
PROFILE OF HWY 2 (N.T.S.)
PROPOSED PROFILE
ORIGINAL PROFILE GRADE
700' V.C.
ALL ELEVATIONS ARE TO TOP OF ASPHALT
D.H.O. STD. CONCRETE HANDRAIL
0.33% UP
H.W.L. 154.10
MUD
W.L. 152.16 MAY 1961
ORIGINAL GROUNDLINE
PILES 14BP73
ELEVATION (HWY. 401)
SCALE: 1"=30'-0"
SECTION (HWY. 401)
SCALE: 1"=10'-0"
PRINT RECORD
No. | FOR | DATE
SHEET No. | TOTAL SHEETS
REVISIONS | DATE | BY | DESCRIPTION
DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION
HWY. 2 UNDERPASS & SUTHERLAND CK.
(2.5 MILES WEST OF QUEBEC BDRY.)
KING'S HIGHWAY No. 401 & 2
DIST. No. 9
CO. GLENGARRY
TWP. LANCASTER
LOT 10
CON. I
PRELIMINARY PLAN
APPROVED
BRIDGE ENGINEER
SITE No.
W.P. No. 177-60, 116-59
DESIGN W.I.L.
DRAWING
CHECK
CONTRACT Nos.
DATE NOV. 1961
LOADING H20 S16
DRAWING No. D-4958-P



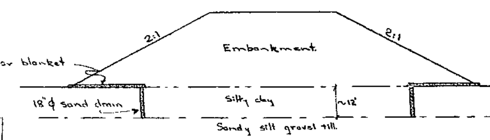
Notes:
1) Boreholes 3, 4 and 5 put down August 1960. Positions approximate only.
2) Boreholes 13, 14, 15 and 16 made September 1961. Positions related to Line 'C' only; Line 'D' was not pegged out.
3) Additional spans proposed for bridges, as shown above after field work complete.

B.M. - EL. 159.03'
GEODETIC DATUM
N. & W. in N.W. root of 15" Maple, 220' Rt. of Sta. 354+55, Line 'C'.

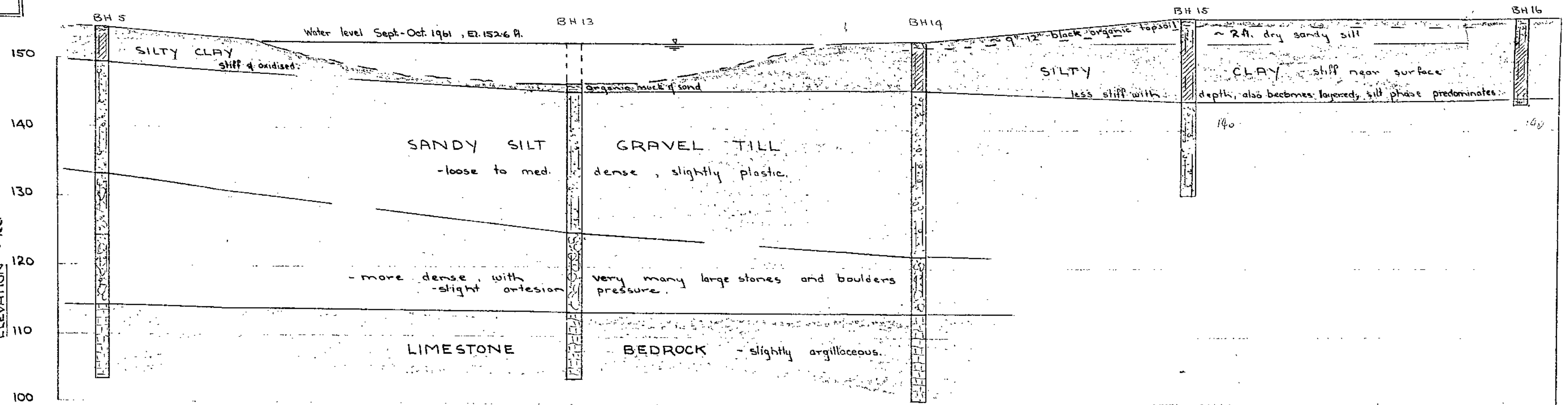
PLAN OF SITE - SHOWING BORING LOCATIONS.
(Overlay of plan E4003-1)
SCALE: 1" = 20'



KEY PLAN
1in. = 1 mile.



CROSS-SECTION THROUGH EMBANKMENT
- SHOWING PROPOSED SAND DRAINS



ESTIMATED SUBSOIL STRATIGRAPHICAL PROFILE
SCALES: VERT. 1" = 10'
HORIZ. 1" = 20'

FOUNDATION INVESTIGATION.
PROPOSED CROSSING, SUTHERLAND CREEK
HWY. 401 AND HWY. 2

WILLIAM A. TROW & ASSOCIATES LTD.

J542A OCT. 1961 DWG. 1