DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31G-162

DIST. 9 REGION

W.P. No.

CONT. No.

W. O. No. 73-F-213 M

STR. SITE No. 27-116

HWY. No.

LOCATION CROSSING OF COUNTY ROAD #5 & CASTER RIVER

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

73-F-213 M

73-F-213 M

GEOL. No 31 G-162

Dist 9

REPORT ON
FOUNDATION CONDITIONS
PROPOSED BRIDGE (STRUCTURE No. 10)
CROSSING OF COUNTY ROAD NO. 5
AND
CASTOR RIVER

STRUCTURE SITE No. 27-116

REPORT

ON

FOUNDATION CONDITIONS

AT

PROPOSED BRIDGE (STRUCTURE NO. 10)

CROSSING OF COUNTY ROAD NO. 5 OVER CASTOR RIVER

CONCESSION VI, TOWNSHIP OF CAMBRIDGE

FOR

UNITED COUNTIES OF PRESCOTT & RUSSELL

BY

F O N D E X L I M I T E D

File No. 3246-S　　　　Ottawa, May 28, 1973.

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SUMMARY

1. The investigation presented in this report was undertaken to assess the foundation conditions for the piers and abutments of a proposed bridge for County Road No. 5 over the Castor River in the Township of Cambridge, United Counties of Prescott and Russell and to appraise the stability of the slopes of the banks of the Castor River at the site of and adjacent to the proposed bridge.

2. The general geology of the area is discussed and the particular geological deposits encountered at the site are described.

3. The field and laboratory investigations which were carried out are described in terms of their methods and objectives.

4. The engineering properties of the various soil types, as observed and determined from the field and laboratory investigations, are discussed in some detail.

5. Based on the above studies, the following recommendations have been made:

   a) The piers and abutments of the bridge can be founded on end bearing, minimum displacement, piles on bedrock.

   b) Steel piles driven to the surface of the bedrock may be designed to carry the maximum load as determined by the pile material and area, i.e., $0.3f_yA_s$

   c) Slopes approximately 30 feet in height in the natural clay stratum should not be steeper than 3 horizontal to 1 vertical.

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d) The south slope of the valley should be flattened to a slope of 3 horizontal to 1 vertical at the bridge site and to a distance of 250 feet upstream and downstream from the bridge site.

e) The sequence of construction operations at the south slope of the bridge site must be planned in order to minimize the risk of slope failure during construction.

f) Piezometers should be installed in and adjacent to pile groups in the south slope in order to allow the stability of the slope to be checked during pile driving operations.

g) Good drainage of surface and subsurface water should be maintained at all times.

REPORT ON FOUNDATION CONDITIONS PROPOSED BRIDGE CROSSING OF COUNTY ROAD NO. 5 AND CASTOR RIVER

1. INTRODUCTION

This report presents an appraisal of the foundation conditions at the site of a proposed bridge for County Road No. 5 over the Castor River on lot 30, Concession VI in the Township of Cambridge, United Counties of Prescott and Russell, Ontario. The appraisal and report were requested by Mr. A.J. Lynch, P. Eng., Counties Engineer, United Counties of Prescott and Russell.

The investigation was carried out to assess the foundation conditions for the piers and abutments of the proposed bridge and to appraise the stability of the slopes of the banks of Castor River at the site of and adjacent to the proposed bridge.

2. DATUM

All elevations quoted in this report refer to a geodetic bench-mark, elevation 201.07 feet, on a nail on the north-east root of a 3 foot diameter maple tree located 65 feet west of station 7+62.

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3. GEOLOGIC CONDITIONS

The site lies within the physiographic region known as the Russell and Prescott Sand Plains. In this area a sand mantle from 5 to 15 feet in thickness occurs with a surface around elevations 210 to 220 feet. At the immediate site of the proposed bridge, the sand mantle was not encountered in the boreholes.

The sand is underlain by a deposit of marine clay which is generally underlain by a stratum of glacial till resting on bedrock. The thickness of the marine clay deposit depends on the elevations of the till and bedrock on which the clay was deposited.

Rivers and streams draining the area have eroded deep valleys into the clay deposit. Lateral erosion of the valleys appears to take place by mass slumping of the clay in the valley walls with large scale flow slides sometimes occurring. The grade of the rivers and streams is generally very flat and the base levels are controlled by bedrock outcrops at several locations in the river bottoms. Where the valleys are wide a flood plain has been developed and an alluvial deposit of fine sand and silt occurs as a terrace on one or both sides of the stream channel.

A stratigraphic profile approximately along the centerline of the existing and proposed bridges shows the location and depth of the materials encountered at the site. The valley varies in width from 300 to 400 feet at this location with the 100 foot wide channel of the Castor River occurring along the south wall of the valley. A terrace of alluvial fine sand and silt occurs in the valley along the north side of the river channel.

The river bottom is at elevation 178 at the bridge site with the surface of the terrace to the north around elevation 195 and the top of the valley slope to the south around elevation 208. The height of the clay slope along the south side of the river is therefore approximately 30 feet and this slope at its steepest section has a present gradient of 2.2 horizontal to 1 vertical.

Numerous indications of shallow slope failures in the valley walls can be seen both upstream and downstream of the bridge site. These slope failures do not appear to have retrogressed back into the valley walls and the slopes have become stable after the initial slumping took place.

4. FIELD INVESTIGATION

The field investigation consisted of four boreholes made to identify the subsoil, obtain samples, and to make field vane shear tests to determine the in-situ undrained shear strength of the undisturbed and disturbed cohesive soils. Standard Penetration tests were carried out on cohesionless soils to determine their relative densities. In addition, one cone penetration test hole was made to determine the depth to dense material.

The borings were made between February 6th and February 20th, 1973. The locations of the boreholes are shown on the appended site plan. The field investigation was made with two rotary drilling washbore machines. Two men crews operated the drilling machines under the full time supervision and direction of Soils Technicians from our staff.

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Samples of cohesionless materials were obtained in a 2 inch O.D. split barrel sampler. Cohesive soils were sampled with 2 inches and 2¾ inches I.D. thin walled Shelby type tubes. Three boreholes penetrated the full depth of the subsoil into bedrock. Samples of the rock were obtained in a BXT size rock core sampler in order to identify the bedrock and to assist in the assessment of its engineering properties.

The borehole records in the appendix give descriptions of the materials encountered together with a stratigraphic plot, the number and type of samples taken and the results of the field vane and the standard and dynamic penetration tests.

The groundwater conditions at the site were determined by recording the water levels in two piezometers sealed in the till and clay strata in borehole no. 3 and in one standpipe placed in borehole no. 4. No piezometers and/or standpipes were installed in boreholes nos. 1 and 2 due to the severe artesian conditions encountered in these boreholes.

Piezometers P1 and P2 were installed in borehole no. 3 in the till and clay strata respectively. Above and below each piezometer a bentonite seal of at least 2 feet was placed. The remainder of the borehole was filled with granular material (crushed stone). Standpipe S1 was placed in borehole no. 4 in the clay stratum and was surrounded by crushed stone.

The location of the piezometers and standpipe are shown on the appropriate borehole logs.

5. LABORATORY INVESTIGATIONS

All samples of cohesive soils were carefully extruded from the

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sampling tubes and, together with the jar samples of cohesionless soils recovered in the field, were identified and their descriptions logged in the laboratory. Portions of selected samples were used to determine certain index properties of the soils in order to further assist in the assessment of their engineering properties.

Other portions were carefully prepared and subjected to one-dimensional consolidation tests and to triaxial compression tests to determine the compressibility and shear strength characteristics of the clayey soil.

The following laboratory tests and determinations were carried out:

a) natural moisture contents
b) Atterberg limits
c) one-dimensional consolidation tests
d) consolidated-drained triaxial compression tests with constant average principal stress loading procedures.

The results of these tests were used to classify the soil with regard to type and consistency, in assessing the compressibility characteristics of the soil, and in analyses of the stability of slopes at the site. The test results are plotted on the borehole records or other appropriate Figures in the Appendix.

6. SOIL CONDITIONS

A stratigraphic plot of the soil types encountered is shown on drawing no. 3246-S-2 in the Appendix. A more detailed description of the soils encountered is shown on the borehole logs. The soils encountered are described and a summary of the field and laboratory work done is given together with an evaluation of their engineering properties in the following paragraphs.

6.1 General

Boreholes nos. 1, 2 & 3 penetrated the full depth of the subsoil into bedrock. Borehole no. 1 encountered, beneath the bottom of the river, a stratum which was composed of a brownish mixture of cobbles, silty sand and organic material. This stratum was 3½ feet thick. Borehole no. 2 initially encountered a 19½ foot thick stratum of alluvium. The upper 18½ feet in borehole no. 3 consisted of brownish-gray sandy earth fill. This loose to compact fill material is part of the approach fill at the south abutment of the existing bridge. Borehole no. 4 encountered 2 feet of gravely top soil followed by about 4 feet of desiccated very stiff, brown, silty clay.

Beneath these upper soil layers all of the boreholes penetrated a stratum of gray and pink banded, firm silty clay. This firm clay stratum extended to elevation 157±2 feet in all boreholes, where there was a marked change in consistency to stiff silty clay.

In boreholes nos. 1, 2 & 3, at elevation 114±2 feet a deposit of dense glacial till of variable thickness was encountered. The glacial till overlays limestone bedrock.

6.2 Alluvial Deposit

The alluvial deposit encountered in borehole no. 2 on the north side of the river channel was composed of silt, fine sand, and scattered pieces of organic matter. It was 19½ feet in thickness, generally brown in colour and very loose.

6.3 Marine Clay

6.3.1 Description of Deposit

The marine clay stratum is the principal soil deposit at the site of the proposed bridge structure.

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In all boreholes, the upper part of the deposit, to elevation 157±2 feet, is a banded pink and gray silty clay, firm in consistency. It contains thin seams and lenses of fine sand and silt and fragments of shells and black streaks occur throughout.

Below elevation 157 the clay is blue-gray in colour and stiff in consistency to its base at elevation 114±2 feet.

6.3.2 Undrained Shear Strength

The field vane shear tests show that the undrained shear strength in the upper banded part of the deposit is about 750 psf. In the lower stiffer part of the deposit the shear strength varies from a low of approximately 1200 psf near its upper boundary, in boreholes nos. 1, 2 & 3, to a maximum of about 2500 psf near the bottom of the deposit in borehole no. 3. The average value of the shear strength of the lower part of the deposit in all three boreholes was about 1500 psf. The field vane shear strengths on the remolded clay varied between approximately 100 psf and 400 psf with the average value in the top half of the deposit being between 100 and 200 psf. The sensitivity of the clay varies between 8 and 10 classifying it as very sensitive.

6.3.3 Index Properties

The water contents and Atterberg limits of the clay show some variation due to the banding and the presence of fine sand and silt seams. The liquidity indices generally have an average value of 1.0 or slightly larger indicating that the soil will lose much of its strength if it is disturbed. The plasticity indices,

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with very few exceptions, have values of above 40 reflecting a highly plastic soil.

6.4 Glacial Till

All three boreholes that penetrated the full depth of the clay deposit encountered a dense mixture of gravel and silty sand with clay traces, generally gray in color. This deposit is considered to be of glacial origin. The thickness of the glacial till varied between 1 and 8 feet in boreholes nos. 1, 2 & 3.

6.5 Bedrock

The bedrock encountered in boreholes nos. 1, 2 and 3 is a gray limestone, generally fine grained, laminated with black shale. The shale content varies between 20% and 50%.

The rock shows vertical joints filled with recrystallized secondary carbonates. Some joint weathering is evident on the joint places.

Core strengths indicate that the rock is moderately strong to strong.

7. GROUNDWATER CONDITIONS

Piezometric elevations were measured in the piezometers in the boreholes during the field investigation and over a period of three weeks following the field investigation in February and March, 1973. The elevations varied little during this time and the maximum elevations found are shown on the borehole logs. The operation of piezometer number P1 was checked in May, found to be still working properly, and its water level was measured to be the same as in February and March.

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A strong artesian upward flow of water was encountered in holes 1, 2 and 5 when the holes reached the sand and gravel till - like stratum at the base of the clay deposit. In borehole number 1 the water level rose to elevation 203.2, 14.8 feet above the river level, in the extended casing. In borehole number 2 the water level rose to elevation 200.1, 5.7 feet above ground level, in the extended casing. The upward flow of water was stopped in all these holes by plugging the holes at several elevations in the clay with bentonite clay seals and wooden plugs. Because of the difficulty in plugging these holes to stop the flow of water, piezometers were not installed in these holes.

In borehole number 3, piezometer P1 was sealed in the sand and gravel stratum below the clay deposit at elevation 110, and piezometer P2 was sealed in the clay stratum at elevation 165. The water level in P1 rose to elevation 205.3 and in P2 rose to elevation 204.0.

In borehole number 4, standpipe S1 was placed at a depth of 36 feet. This standpipe was not sealed in the hole and therefore measured the groundwater table at this location. The water level was found to be at elevation 207.7, 4.6 feet below the ground surface.

From the above measurements, it would appear that the water head in the sand and gravel stratum below the clay deposit reflects the general ground water level of the clay plain in which the Castor River valley is cut. An upward hydraulic gradient of 0.25 exists in the clay stratum below the river channel.

8. FOUNDATIONS FOR BRIDGE

Neither the clay deposit or the alluvial deposit at this site is strong enough or dense enough to support the bridge abutments and piers on spread footings. Piles will therefore have to be used to support the foundations.

8.1 End Bearing Piles on Bedrock

It is recommended that end bearing piles founded on the bedrock surface be used to support the abutments and piers. Minimum displacement piles, such as steel H-piles, should be used for ease in driving through the clay and dense till underlying the clay.

Minimum displacement piles are also required at the locations of the south abutment and south pier because of slope stability considerations. Piezometers should be installed adjacent to or within the location of these pile groups before the piles are driven in order to monitor increases in the pore water pressures from pile driving. Construction progress may have to be controlled by the rate of this pore water pressure dissipation.

Steel piles driven to the surface of the competent bedrock may be designed to carry the maximum load as determined by the pile material and area. The allowable load per pile would therefore be $0.3f_yA_s$ where f_y is the yield stress of the steel and A_s is the cross-sectional area of the steel.

If the north approach fill is increased in width or height from the present dimensions of the fill or if its location is changed then the piles for the north abutment will have to be designed to carry load from negative skin friction caused by consolidation of the underlying alluvium and clay deposits. The magnitude and effective depth of the negative skin friction will depend on the increase in extent of the north approach fill. If consolidation of the clay deposit will be considerable then the pile group for the north

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abutment should include batter piles sloping away from the river to prevent the abutment from moving away from the bridge. The necessity for these batter piles can also be ascertained when the extent of the approach fill has been designed.

It is not anticipated, because of slope stability requirements, that the south approach fill will be of a height to cause consolidation of the underlying clay. The load of this fill on the clay should also be checked however when the height and location of the south abutment has been determined.

8.2 Friction Piles

Although it may be possible to use friction piles to support the abutments and piers, they are not recommended for this site. Because of the very low remolded strength of the clay only very low skin friction values could be assigned to support such piles or a considerable length of time would have to be allowed to elapse after pile driving before the piles could be loaded.

In addition, the best type of friction pile for this clay, tapered timber piles, are not recommended for the south abutment because of slope stability considerations.

The clay is considered to be highly frost susceptible and therefore, a minimum of 5 feet of frost cover is required, that is the bottoms of the pile caps should be at least 5 feet below final grade.

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9. SETTLEMENT OF APPROACH FILLS

Settlement of the north approach fill will take place by consolidation of the underlying loose, silty, sand alluvial deposit and clay deposit if the present approach fill is increased in height or width or if its location is changed.

Figure 1 shows the results of a consolidation test on an undisturbed sample of the clay obtained in borehole number 2 near the top of the clay stratum. This borehole is located north of the present approach fill just beyond the effect of the present approach fill on the underlying deposits. The present effective overburden pressure, p_o', at the elevation of the sample is 1700 psf, assuming the water table at elevation 186. If the overburden pressure is increased beyond the preconsolidation pressure, p_c', of 2300 psf then large settlements will occur in the clay stratum.

As the present approach fill is approximately 15 feet in height, it undoubtably caused large settlements when it was first placed. Any increase of the load of the fill will initiate additional settlements. The magnitude of these settlements can be estimated from the results of the consolidation test when the design of the approach fill has been made.

In addition to causing settlement of the approach fill, consolidation of the clay will add a negative skin friction load to the piles for the north abutment and, if large, may tend to cause the abutment to move north, away from the bridge. This latter movement can be resisted by north sloping batter piles in the pile group for the abutment.

Figure 2 shows the results of a consolidation test on a sample from borehole number 3 at a slightly higher elevation. This borehole was made through the south approach fill behind the south abutment. The present overburden pressure of the fill and clay is close to the preconsolidation pressure of 1900 psf. Any increase in height of this fill will have to be small because of slope stability considerations and loading the clay beyond its preconsolidation pressure cannot be allowed.

The preconsolidation pressures found in the consolidation tests can be approximately checked by assuming that the clay deposit must have been at least at the elevation of the adjacent clay plain before the river valley was eroded. With a surface elevation of 220 feet and the groundwater table at the surface, the effective overburden pressures at the sample elevations would be 2000 psf and 1800 psf in comparison to the measured preconsolidation pressures of 2300 psf and 1900 psf. It would appear therefore that the upper part of this clay deposit is normally consolidated with respect to an original surface elevation slightly higher than 220 feet assuming the groundwater table at the surface.

10. STABILITY OF SLOPES

10.1 Slope Failures

Slope failures have taken place along the Castor River valley both upstream and downstream of the proposed bridge site on both sides of the valley. These slope failures, both old and more recent, are indicated by shallow, circular depressions in the valley slopes. The tops of these depressions generally are at or slightly behind the crest of the slopes. No signs of large, retrogressive failures could be seen along this reach of the Castor River although several of the failures may have retrogressed 10 to 30 feet back from the crest of the slope at the time of failure.

The valley slopes are generally 35 feet or less in height and have slopes flatter than 2 horizontal to 1 vertical. Where the valley is wide, alluvial terraces on one or both sides of the valley reduce the overall slope angles even more.

The slope failures are a natural phenomenon and are caused by the river eroding the toe of the slope thereby over-steepening the slope and resulting in a shallow shear failure. The failure usually flattens the slope as the sliding mass remains at the toe of the slope.

Slope failures can also be initiated by loading the top of the clay slope with fill. Two of the older bridges in the area show definite movement of their abutments towards the river. Both bridges have fills over 15 feet in height immediately behind their abutments and the movement appears to be a long term creep of the underlying clay carrying the abutment and fill.

At the bridge site, an alluvial terrace occurs along the north side of the valley and this side presents no problems in stability. The river channel, however, lies along the south side of the valley at this location and the valley wall is fairly high and steep. This slope should therefore be designed for stability considerations both at the site of the bridge where the south approach fill will exist and upstream and downstream of the bridge.

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The steepest slope in the vicinity of the bridge site exists approximately 200 feet downstream of the present bridge on the south side of the valley. This slope shown on the Site and Location Plan as Section B-B, has been analyzed to determine its present stability and to determine what slope angle is necessary to give it an adequate factor of safety against slope failure. This section has a height of 30 feet and a present overall slope of approximately 2.2 horizontal to 1 vertical. The results of these analyses can then be used to design the slopes at and adjacent to the bridge. When the location of the south abutment, south pier, and the height of approach fill have been determined, this section must be checked for stability for both construction and long term conditions.

10.2 Shear Strength of Clay

Choosing the values of the shear strength parameters to be used in practice for the prediction of the stability of a slope depends upon the slope conditions being studied and on the previous knowledge of the action of the material when slope failures have occurred. Two different kinds of shear strength parameters can be determined; undrained shear strength and drained or effective stress shear strength. Both of these parameters can be used for the shear strength of soil in the field, each under different well defined sets of field conditions.

10.2.1 Undrained Shear Strength

In saturated clays the undrained shear strength represents the maximum shear stress the clay can sustain when loaded to failure without any change in water content. It can conveniently be determined from field vane tests. In the sensitive clays of eastern Ontario, however, analyses using this shear strength parameter have been found not to be valid for the long term stability of slopes even when the water content does not change with time. The factors of safety calculated with this shear strength parameter appear to be too large. Investigations of slopes have shown that the undrained shear strength analysis gives factors of safety well above one even when applied to slopes which have failed.

The error in this method may lie in using the undrained shear strength found from field vane tests. Because of the possibility of strength anisotrophy and time effects, both of which usually reduce the strength, a reduced value should probably be used. There is some indication at the present time that a reasonable value is approximately two-thirds of the field vane strength.

The average vane strength at the locations and depths to be studied is about 750 pounds per square foot. Slope stability analyses have therefore been made using this value and also using two-thirds of this value, 500 pounds per square foot.

10.2.2 <u>Drained Shear Strength</u>

The drained or effective stress shear strength parameters can be determined in the laboratory by performing a series of drained triaxial tests or consolidated undrained tests with pore pressure measurements. The tests must be performed at stress levels consistent with stress levels in the field.

Seven controlled stress drained triaxial tests were performed to determine the effective stress shear strength parameters. These tests were conducted with a constant average effective principal stress, p'_m, during the shearing stage of the test.

That is, $p'_m = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3}$ was kept constant during shear. This method of test has the advantage that at failure the principal stresses are low and can be kept within the stress level in the field, that volume changes due to changes in the mean principal stress are eliminated and that volume changes due to shear can be measured.

The results of these tests are shown in Figure 3. The tests have been plotted with the horizontal axis equal to $p = \frac{\sigma'_1 + \sigma'_3}{2}$ and the vertical axis equal to $q = \frac{\sigma'_1 - \sigma'_3}{2}$. The change in p and q during the test is shown by the dashed lines and the peak failure point is given by the circles at the end of the test path. A curved line has been drawn through the failure points. Tangents to this line are defined by the angle with the horizontal, α, and the intercept on the vertical axis, a.

The conventional shear strength parameters, ϕ' and c', can be calculated from:

$$\sin\phi' = \tan\alpha \quad \text{and} \quad c' = \frac{a}{\cos\phi'}$$

or the relationship between the shear strength, s, and the normal stress, σ'_n, can be calculated from:

$$\sigma'_n = p - q \sin\phi'$$

and

$$s = q \cos\phi'$$

The relationship between σ_n' and s, that is, the shear strength envelope, is also shown in Figure 3. Tangents to the shear strength envelope are defined by the angle of shearing resistance, ϕ', and the cohesion intercept, c'. It can be seen that for this clay as the effective normal stress decreases, ϕ' increases and c' decreases. The change in ϕ' and c' with σ_n' is shown in Figure 4. The values for σ_n' less than 2 psi are extrapolated and therefore amy be in error but it appears that c' decreases towards zero as σ_n' decreases. The clay exhibited a fissured structure when broken or sheared and it is therefore probable that the strength is predominately frictional in the low stress regions.

A curved shear strength envelope requires that slope stability computations be made using the correct range of shear strength parameters.

10.3 Total Stress Analyses

Section B - B, the steepest natural slope in the vicinity of the bridge, has been analyzed for stability using the undrained shear strength parameters $\phi = 0$, c = 750 psf and $\phi = 0$, c = 500 psf. The river level was taken at elevation 184.

Section B - B, Existing Slope:

Undrained Shear Strength	Factor of Safety
750 psf	1.51
500 psf	1.01

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10.4 Effective Stress Analyses

Section B - B has also been analysed for stability using effective stress shear strength parameters. As the method of analysis uses a constant tanϕ' and c' along the shear surface regardless of the normal stess on the shear surface, the following procedure was used to determine the minimum factor of safety for the slope. The slope was analysed using various combinations of ϕ' and c' over a wide range and the factors of safety for these values of shear strength parameters were then compared to the range of ϕ' and c' for the shear strength envelope obtained from the triaxial tests. The critical failure surface which gave the lowest factor of safety was then analysed using values of tanϕ' and c' which varied with the normal stress, as shown in Figure 4, to determine whether this method of analysis significantly changed the factor of safety.

The groundwater conditions and river level used in the analyses were the most probable highest groundwater level in combination with the most probable lowest river level. The groundwater table was taken at a depth of 4 feet at the top of the slope and at the slope surface as shown in Figure 5. A flownet for this condition resulted in the piezometric surface being approximately 2 feet below the slope surface. The river level was taken at elevation 184. This groundwater condition takes into consideration the upward hydraulic gradient in the clay deposit.

The table on Figure 5 shows the factors of safety calculated with the various combinations of ϕ' and c' They are also plotted in Figure 7. These factors of safety are not meaningful until the shear strength parameters used are compared to the shear strength of the clay.

10.5 Effect of Groundwater Conditions

Because the clay appears to be a predominately frictional material in the low stress range, the groundwater conditions will have a considerable effect on the factor of safety for slope failure. To measure this effect, section B - B was also analysed when the piezometric surface was raised two feet in the slope and the river level was lowered 4 feet to elevation 180.

The table on Figure 6 shows the factors of safety calculated for this groundwater condition and they are also plotted in Figure 7. For the higher values of ϕ' and low values of c', this small change in piezometric surface changes the factor of safety by approximately 0.2. For other values of ϕ' and c' the factor of safety is changed by approximately 0.1.

This comparison shows the importance of measuring the most critical groundwater conditions when making slope stability analyses. The effect of good subsurface and surface drainage on stabilizing slopes is also shown by the changes in the factors of safety with the change in piezometric surface.

10.6 Factor of Safety of Existing Slope

The factors of safety obtained in the slope stability analyses with the most probable maximum groundwater conditions are replotted in Figure 8 showing the variation in factor of safety with various ϕ' and c' values. Also plotted in this figure are the values of ϕ' and c' which define the shear strength envelope of the clay. It can be

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seen that the lowest factor of safety of 1.03 is obtained for the existing slope at section B - B when the shear strength of the clay has a friction angle of approximately 35^{o} and a cohesion value of approximately 80 psf. The factor of safety is also near this value for higher values of the friction angle.

The minimum factor of safety for this existing slope is therefore 1.03 when calculated with a constant value of $\phi' = 35^{o}$ and $c' = 80$ psf for the shear strength on the shear surface.

This shear surface was also analysed by varying the value of ϕ' and c' as the effective normal stress changed using the relationship shown in Figure 4 derived from the shear strength envelope for the clay. The factor of safety calculated with this analysis was 1.01. This agreement was possible because the critical shear surface was shallow and the effective normal stresses on the failure surface varied between 1 and 4 psi as shown in Figure 12. Over this small range the curved shear strength envelope can be represented by the straight line shear strength envelope having $\phi' = 35^{o}$ and $c' = 80$ psf.

10.7 Critical Stress Curve

Another method of presenting the slope stability analyses is by calculating a critical stress curve for the slope conditions.

The average effective normal stress, $\bar{\sigma}_n'$, and the average shear stress, $\bar{\tau}$, on a slip surface are calculated for a number of possible shear surfaces. These stresses are not dependent on the shear strength properties of the soil but are dependent only on the geometry of the slope and the groundwater and river level conditions. These stresses are plotted against one another and a critical stress curve forms the upper limit of all of these average stress points.

A critical stress curve represents the maximum shear stresses in the slope for the specified slope conditions and may only be compared with the shear strength of the soil when the shear strength has been measured by laboratory tests carried out in the appropriate range of normal stresses.

Figure 9 shows the critical stress curve for the existing slope at section B - B and also shows the shear strength envelope of the clay. For shear surfaces with average normal stresses of less than 2 psi, the shear stresses are almost as large as the shear strength of the clay. The factor of safety of these shear surfaces is approximately 1.02.

The similar values for the minimum factor of safety obtained by the two methods of analysis occur because the average normal stress on the critical failure surface for $\phi' = 35^{\circ}$ and $c' = 80$ psf is slightly lower than 2 psi.

10.8 Factor of Safety of 3:1 Slope

In order to achieve a reasonable factor of safety for slope stability for the south slope of the valley, it must be flattened. Section B - B has been analysed for a slope cut to 3 horizontal to 1 vertical. The water table used in the analysis is shown on Figure 10. A flow net indicated that the piezometric surface would average about 1 foot below the slope surface. The river level was taken at elevation 184.

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The factors of safety obtained for a range of effective stress shear strength parameters is shown in the table on Figure 10. These factors of safety have been plotted on Figure 11 along with the values of ϕ' and c' which define the shear strength envelope of the clay.

A minimum factor of safety of approximately 1.33 is obtained with a shear strength of $\phi' = 35^0$ and c' = 80 psf. This factor of safety is considered to be adequate for the long term stability of the slope.

10.9 Residual Shear Strength Analysis

The method used in this report to calculate the factor of safety against slope failure has been to assume that the peak shear strength of the clay can exist at the same time along the entire shear surface. This method is not considered valid with some soil types because large shear stresses and strains along part of the shear surface may have reduced the shear strength to a residual value which may be considerably lower than the peak value. The brittle, sensitive clays of eastern Ontario are soils which exhibit residual strengths lower than the peak strengths.

The method used, however, appears to be valid because if lower values than the peak strengths are used, the factor of safety of existing slopes will be less than one. Therefore, peak strengths are used in design with the provision that the design factor of safety must be large enough to compensate for any errors in the method of analysis.

With the controlled stress method of conducting the triaxial tests, it was not possible to measure the residual strength of the clay. Triaxial tests performed on similar clays in which the structural bonds have been broken down have indicated, however, effective stress shear strength parameters of $\phi' = 20^{\circ}$ and $c' = 0$. These values have been used to analyse the stability of the critical failure surfaces found earlier.

Figure 12 shows the variation in the normal stress and in the shear stress along the critical failure surface for the existing slope at section B - B. The peak shear strengths for the existing normal stresses are also shown. It can be seen that the shear stress for the upper half of the failure surface is larger than the shear strength. If the shear strength of the clay along this portion of the shear surface has been reduced to the residual value, the factor of safety of the slope would be 0.72.

Figure 13 shows the same stresses for the 3:1 slope. The shear stress on the upper half of the most critical surface is larger than the shear strength. Using the residual shear strength over this portion, the factor of safety comes to 0.92.

This method of estimating what portion of the failure surface may be at the residual strength value is not analytically correct as it does not take into account where the slope may strain sufficiently to allow the peak shear strength to be exceeded. The calculations however do give an indication of the magnitude of the change in the factor of safety if the peak shear strength is exceeded along some portion of the shear surface.

10.10 Recommendations for Slope Stability

1. It is recommended that slopes approximately 30 feet in height in the natural clay stratum be no steeper than 3 horizontal to 1 vertical.

2. The south slope of the river valley for 250 feet upstream and 250 feet downstream from the centerline of the bridge should be flattened to a slope of 3 horizontal to 1 vertical.

3. The finished slope at the south abutment should be 3 horizontal to 1 vertical from the bottom of the slope to the top of the approach fill at the abutment. If the height of this slope is greater than 30 feet, its stability should be analysed.

4. Drainage tile leading to a frost free outlet should be installed at the base of the pile cap of the south abutment on both the upslope and downslope faces.

5. If the south pier is founded in the south slope, drainage tile leading to a frost free outlet should be installed at the base of its pile cap on the upslope face.

6. If the southern most pier is founded in the south slope or close to its toe, the slope should be analysed for construction conditions when the excavation for the pier is made. The excavation should be sheetpiled and the bracing prestressed.

7. Piezometers should be installed in or adjacent to the location of the pile groups for the south abutment and south pier before the piles are driven. These piezometers are necessary to monitor the pore water pressure buildup and dissipation during pile driving in order that the stability of the slope during construction can be assessed.

10.11 Construction Sequence

The sequence of construction operations at the south slope of the bridge site must be planned in order to minimize the risk of slope failure during construction. The following sequence is suggested, but changes may be necessary when the locations of the south abutment and pier are finalized.

1. The south slope at the bridge site and upstream and downstream should be cut to its final grade.

2. The excavation for the south abutment should be made.

3. The excavation for the south pier should be made inside a sheetpile cofferdam with prestressed bracing.

4. Piles for the south pier should be driven, pile cap and pier constructed, and the cofferdam backfilled with compacted fill.

5. Piles for the south abutment may now be driven, the abutment constructed, and backfill placed.

6. Good drainage of surface and subsurface water should be maintained at all times.

... 27

11. PERSONNEL

The field work was carried out by Messrs. B. Hopkins and N. Kantartzis under the supervision of Dr. J.D. Scott.

The report was prepared by Mr. N. Kantartzis and Dr. J.D. Scott.

We are at your disposal for any additional information you may require concerning the contents of this report.

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J.D. Scott

J.D. Scott, P. Eng.

APPENDIX

- Explanatory Notes
- Borehole Logs
- Figures 1 to 13
- Site & Location Plan
- Stratigraphic Profile

EXPLANATORY NOTES
ON THE
RECORDS OF BOREHOLES

The purpose of borehole records is to assemble on a single sheet all of the field and laboratory data obtained during the investigation regarding the soil, bedrock, and groundwater conditions at the location of the borehole.

SOIL PROFILE

Elevation: This column gives the elevations of boundaries between various geological strata. The elevation refers to the datum shown in the heading of the borehole record. The corresponding depths below the ground surface are also shown.

Description: Each geological stratum is described, using standard terminology, from examination and analyses of samples.

The relative density of granular soils is defined on the basis of the Standard Penetration Test. The consistency of cohesive soils is referred to in terms of either shear strength or unconfined compressive strength. The proportion of each constituent part as defined by the grain size is denoted by the following terms.

Relative Density (granular soils)	Standard Penetration Test Value "N" (Blows per foot)
Very loose	0 to 4
Loose	4 to 10
Compact or Medium	10 to 30
Dense	30 to 50
Very dense	over 50

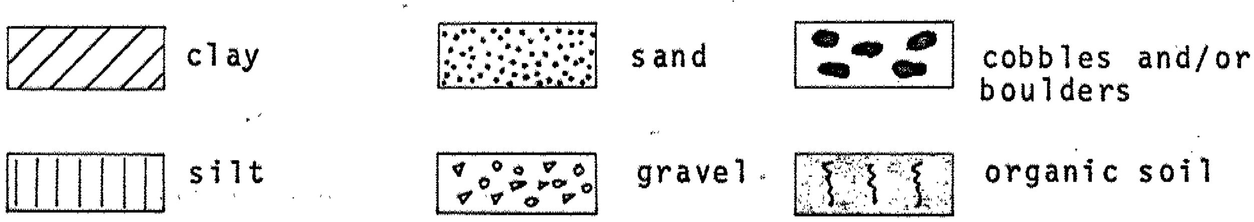
Consistency (cohesive soils)	Undrained Shear Strength (c) (lbs/ sq. ft.)
Very soft	under 250
Soft	250 to 500
Medium or firm	500 to 1000
Stiff	1000 to 2000
Very stiff	2000 to 4000
Hard	over 4000

Plasticity of Cohesive Soils	Liquid Limit
Low	under 30%
Medium	30 to 50%
High	over 50%

Descriptive Terms	Range of Proportion
"Trace"	1 to 10%
"Some"	10 to 20%
Adjective (c.g. sandy, silty)	20 to 35%
"and" (c.g. sand and gravel)	35 to 50%

STRATIGRAPHIC PLOT

The following stratigraphical symbols are used to denote main soils types:



GROUNDWATER CONDITIONS

The groundwater level as observed in the borehole is shown by the symbol $\underline{\nabla}$

SAMPLES

Number: Each sample taken from the borehole is numbered as shown in this column; the exact location and the length of each sample are also shown.

Type: The symbols shown are referred to the following sample types:

- AS : auger sample
- SS : split spoon sample
- ST : Shelby tube sample
- WS : washed sample
- RC : rock core sample

Blows/ft (N): Standard Penetration Test values "N" are shown in this column. This value corresponds to the number of blows required for a 140 pound hammer dropping 30 inches to drive a standard 2 inch outside diameter split spoon sampler a distance of 1 foot into the soil.

Recovery: Soil sample and rock core recoveries are given in percentages.

DYNAMIC PENETRATION RESISTANCE

When dynamic penetration tests are carried out on the casing or on a cone, the results are shown graphically in the "Dynamic Penetration Resistance" column. These tests differ from the standard penetration test, and the diameter of the casing or the cone, together with the driving energy, are shown.

STRENGTH

Results of field or laboratory strength tests on cohesive soils are shown graphically in the "Shear Strength" column using the indicated symbols.

CONSISTENCY

Results of moisture content, liquid limit, and plastic limit tests as determined in the laboratory are shown under "Consistency".

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RECORD OF BOREHOLE 1

PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5
DRILLING DATE 6-7 8/2/73

LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO
REPORT DATE 28/5/73

DATUM GEODETIC BOREHOLE TYPE WASH
DRAWN BY N.K.

SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA. NX
CHECKED BY J.D.S.

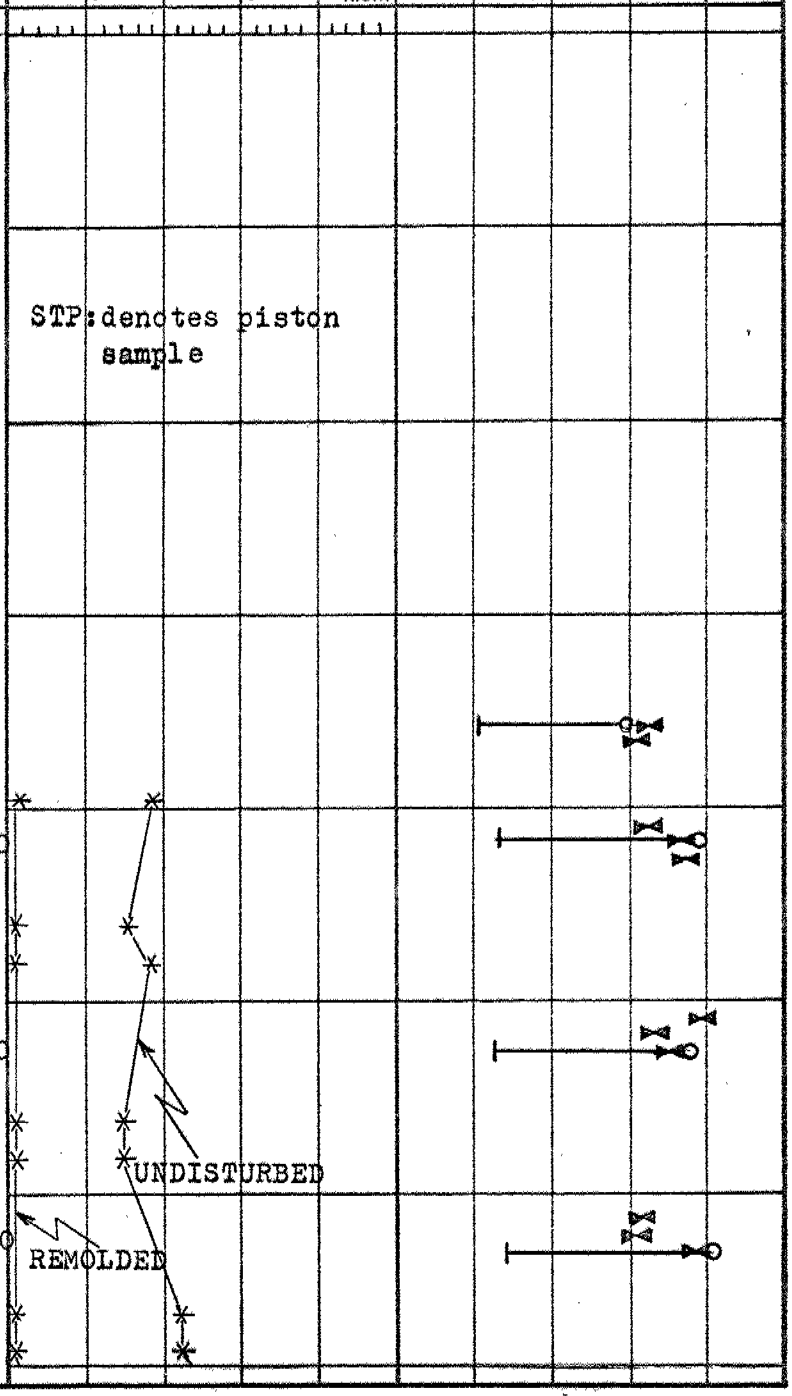
Elev. / Depth	SOILS PROFILE – DESCRIPTION	STRAT. PLOT	SAMPLES – NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY
188.3' / 0'0"	ICE SURFACE 6/2/73					
5'0"						
178.0' / 10'3"	RIVER BOTTOM MIXTURE OF COBBLES & SILTY SAND WITH CLAY TRACES; PRESENCE OF ORGANIC MATERIAL (WOOD PIECES); BROWN		1	WS		
174.6' / 13'8"	SILTY CLAY PINKISH-GRAY; PRESENCE OF SHELLS; DARK STREAKS; - FIRM -					
15'0"			2	ST		0
			3	STP		85
20'0"			4	STP		100
25'0"			5	STP		100
158.3' / 30'0"			6	STP		100

DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

STRENGTH
FIELD VANE SHEAR *
LAB VANE SHEAR X
UNCONF COMP STRENGTH (q_u)

CONSISTENCY:
NATURAL MOISTURE CONTENT (W)
LIQUID LIMIT (W_l)
PLASTIC LIMIT (W_p)

STP: denotes piston sample



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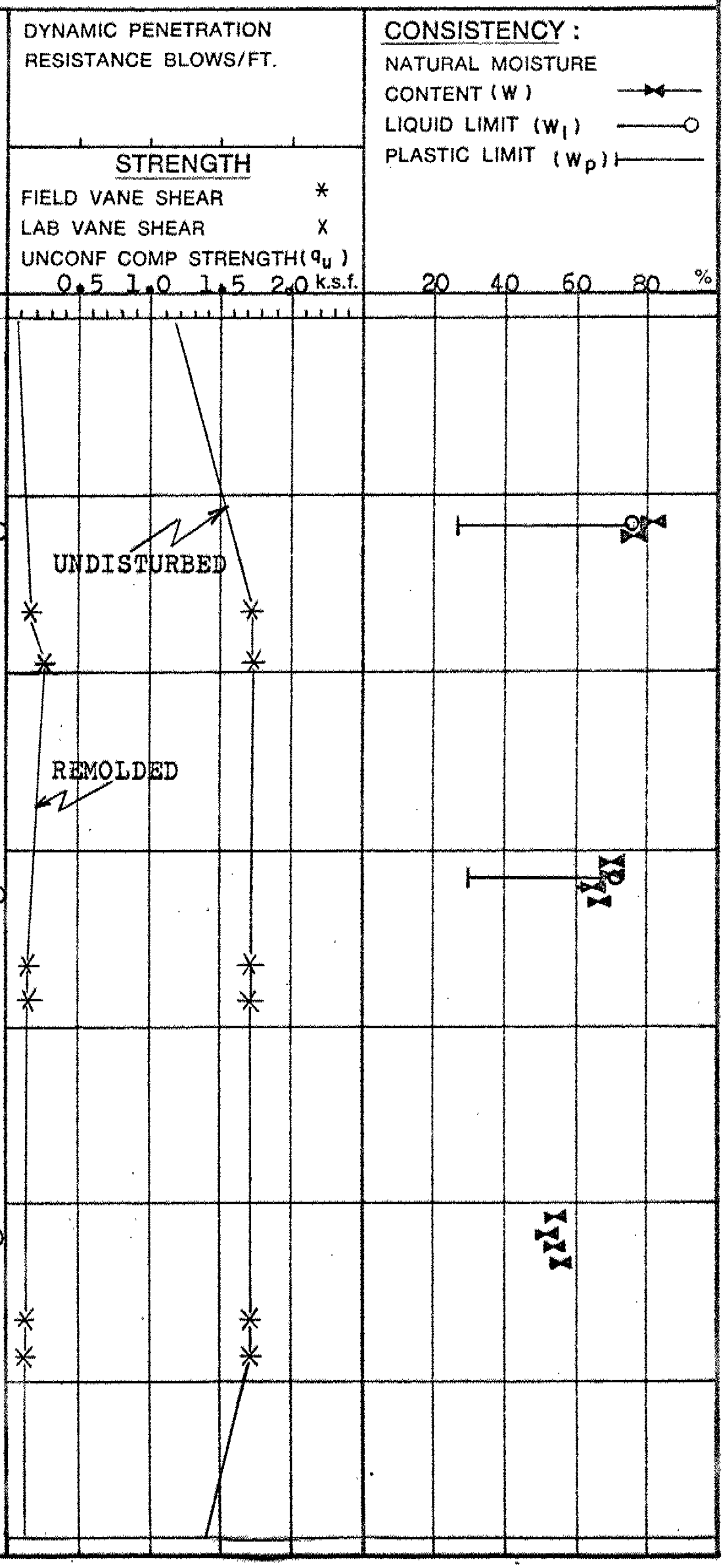
RECORD OF BOREHOLE 1

PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5
LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO
DATUM GEODETIC BOREHOLE TYPE WASH
SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA. NX

DRILLING DATE 6-7 8/2/73
REPORT DATE 28/5/73
DRAWN BY N.K.
CHECKED BY J.D.S.

Elev. / Depth	SOILS PROFILE — DESCRIPTION	STRAT. PLOT	SAMPLES — NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY
35'0"						
40'0"			7	STP		100
45'0"	SILTY CLAY GRAY; PRESENCE OF SHELLS; OCCASIONAL SILT AND FINE SAND SEAMS; DARK STREAKS; - STIFF -		8	WS		
50'0"			9	ST		100
55'0"			10	WS		
60'0"			11	ST		100
65'0"			12	WS		



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RECORD OF BOREHOLE 1

PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5

LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO

DATUM GEODETIC BOREHOLE TYPE WASH

SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA. NX

DRILLING DATE 6-7 8/2/73

REPORT DATE 28/5/73

DRAWN BY N.K.

CHECKED BY J.D.S.

SOILS PROFILE / SAMPLES / DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

STRENGTH: FIELD VANE SHEAR *; LAB VANE SHEAR X; UNCONF COMP STRENGTH(q_u) 0.5 1.0 1.5 2.0 k.s.f.

CONSISTENCY: NATURAL MOISTURE CONTENT (W); LIQUID LIMIT (W_l); PLASTIC LIMIT (W_p) 20 40 60 80 %

Elev. / Depth	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY
70'0"	AS ABOVE		13	ST		100
115.8' / 72'6"	MIXTURE OF GRAVEL AND SAND WITH TRACES OF SILT AND CLAY; GENERALLY GRAY; - DENSE -		14	WS		
75'0"			15	SS	47	25
			16	WS		
108.1' / 80'2"	BEDROCK: GREY LIMESTONE - MEDIUM STRONG TO STRONG -		17	RC		100
			18	RC		92
85'0"			19	RC		100
99.1' / 89'2"	END OF BOREHOLE					

ROCK DESCRIPTION

Limestone, grey, fine grained, laminated with black shale, (shale content 50%).

The rock shows vertical joints filled with recrystallized secondary carbonates, down to about 87' where fractures tend to occur parallel to these joints, some faint weathering on the joint planes.

From 87' to 89', the bedding is inclined and irregular, fractures tend to be horizontal in this portion.

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RECORD OF BOREHOLE 2

PROJECT: BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5	DRILLING DATE: 14-15 16/2/73
LOCATION: CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO	REPORT DATE: 28/5/73
DATUM: GEODETIC BOREHOLE TYPE: WASH	DRAWN BY: N.K.
SAMPLER HAMMER WEIGHT: 140 lb. DROP: 30 inches BOREHOLE DIA.: NX	CHECKED BY: J.D.S.

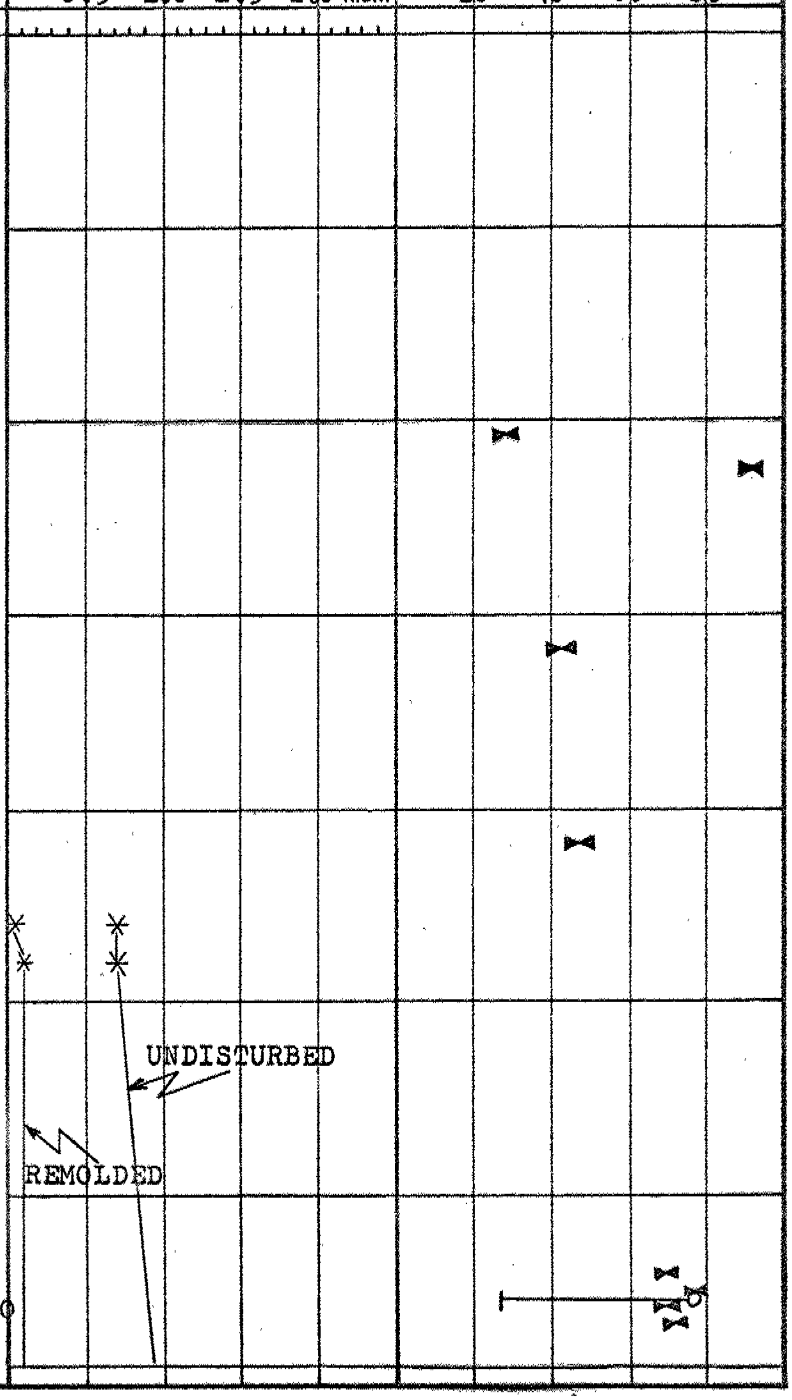
SOILS PROFILE / SAMPLES

Elev. Depth	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY
194.4'	GROUND SURFACE					
0'0"	ORGANIC TOPSOIL					
1'0"	SILTY SAND BROWN; PRESENCE OF ORGANIC MATERIAL; - LOOSE -					
5'0"			1	SS	2	97
10'0"			2	ST		63
15'0"			3	ST		73
174.9' 19'6"	SILTY CLAY BANDED PINK AND GRAY; DARK STREAKS; - FIRM -		4	STP		100
25'0"						
30'0"			5	STP		100

DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

STRENGTH
FIELD VANE SHEAR *
LAB VANE SHEAR X
UNCONF COMP STRENGTH(q_u)
0.5 1.0 1.5 2.0 k.s.f.

CONSISTENCY:
NATURAL MOISTURE CONTENT (W)
LIQUID LIMIT (W_l)
PLASTIC LIMIT (W_p)
20 40 60 80 %



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RECORD OF BOREHOLE 2

PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5

LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO

DATUM GEODETIC BOREHOLE TYPE WASH

SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA. NX

DRILLING DATE 14-15 16/2/73

REPORT DATE 28/5/73

DRAWN BY N.K.

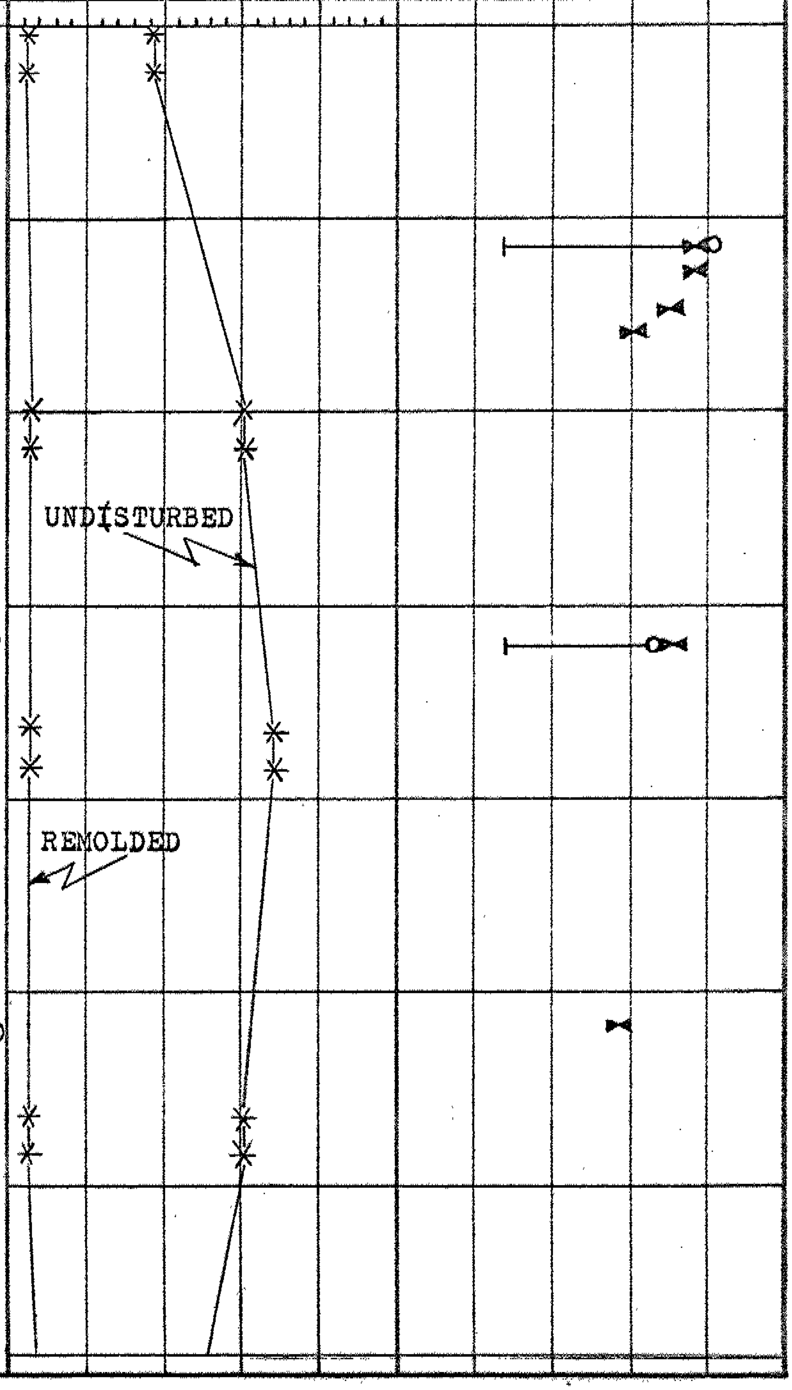
CHECKED BY J.D.S.

Elev. Depth	SOILS PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY
35'0"	AS ABOVE					
156.4' 38'0"						
40'0"			6	STP		43
			7	STP		50
45'0"						
50'0"	SILTY CLAY GRAY; PRESENCE OF SHELLS; DARK STREAKS - STIFF -		8	ST		100
55'0"						
60'0"			9	ST		100
65'0"						

DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

STRENGTH
FIELD VANE SHEAR *
LAB VANE SHEAR X
UNCONF COMP STRENGTH(q_u)
0.5 1.0 1.5 2.0 k.s.f.

CONSISTENCY:
NATURAL MOISTURE CONTENT (W)
LIQUID LIMIT (W_l)
PLASTIC LIMIT (W_p)
20 40 60 80 %



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RECORD OF BOREHOLE 2

PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5

LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO

DATUM GEODETIC BOREHOLE TYPE WASH

SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA. NX

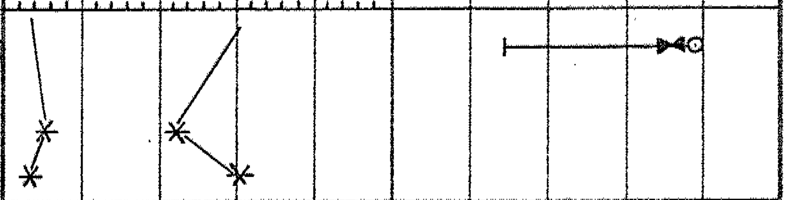
DRILLING DATE 14-15 16/2/73

REPORT DATE 28/5/73

DRAWN BY N.K.

CHECKED BY J.D.S.

SOILS PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS/FT. / STRENGTH	CONSISTENCY
Elev. / Depth	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY	FIELD VANE SHEAR * ; LAB VANE SHEAR X ; UNCONF COMP STRENGTH (q_u) 0.5 1.0 1.5 2.0 k.s.f.	NATURAL MOISTURE CONTENT (W) ; LIQUID LIMIT (W_l) ; PLASTIC LIMIT (W_p) 20 40 60 80 %
70'0"			10	ST		100		
75'0"	AS ABOVE							
113.4' / 81'0"	MIXTURE OF GRAVEL & SILTY SAND; DENSE		11	SS	36	75		
112.3' / 82'1"	BEDROCK: GREY LIMESTONE		12	RC		100		
85'0"								
107.3' / 87'1"	END OF BOREHOLE							



ROCK DESCRIPTION

Limestone, grey, fine grained laminated with black shale (shale content 40%). The rock is sound, showing very few thin joints; fractures tend to occur horizontally parallel to shale/limestone interfaces. One small vertical fracture (3") at 84'6" shows no sign of weathering.

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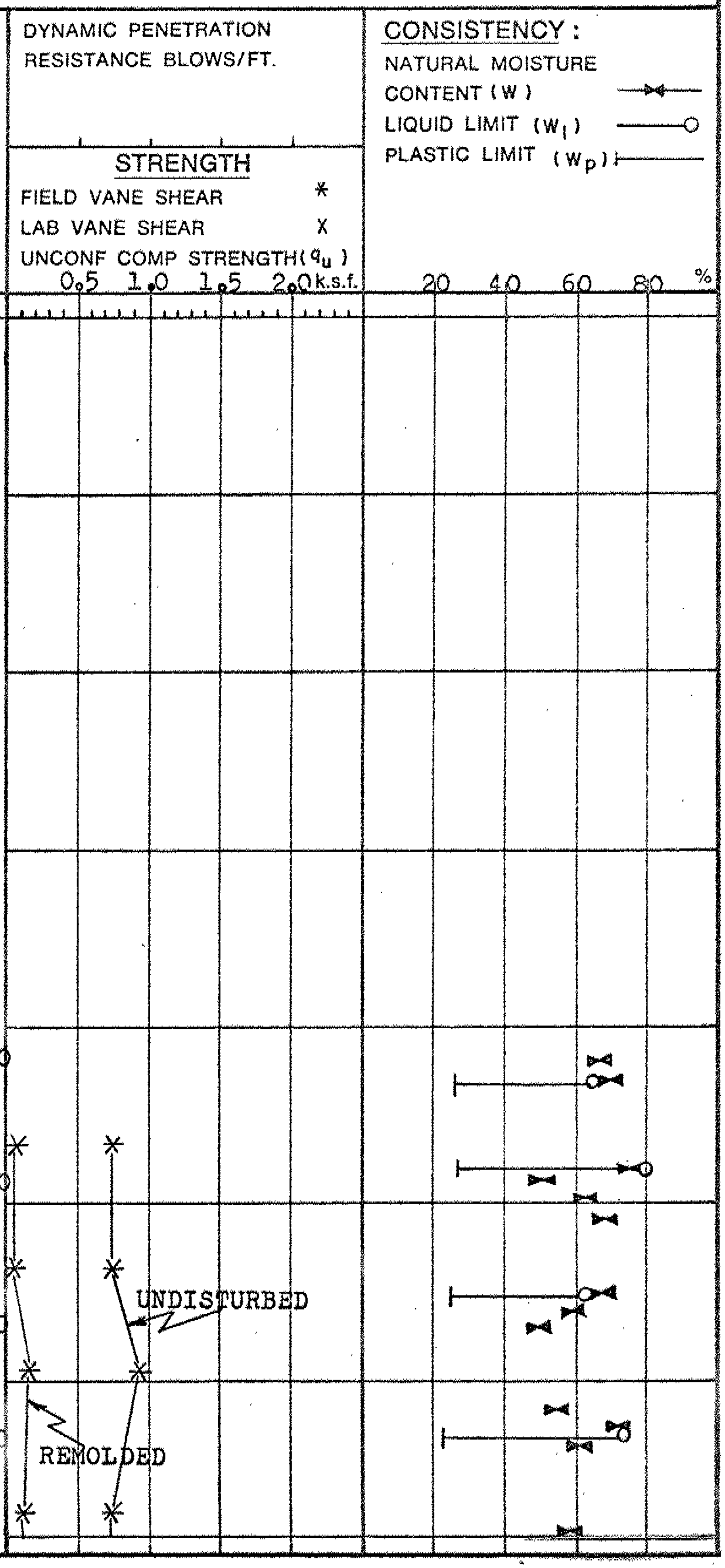
RECORD OF BOREHOLE 3

PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5
LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO
DATUM GEODETIC BOREHOLE TYPE WASH
SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA. NX

DRILLING DATE 14-15 16/2/73
REPORT DATE 28/5/73
DRAWN BY N.K.
CHECKED BY J.D.S.

Elev. Depth	DESCRIPTION	NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY
209.5'	GROUND SURFACE				
0'0"	∇ P1 (8/3/73)	1	SS	9	75
5'0"	∇ P2 (8/3/73) FILL MATERIAL SILTY FINE SAND; BROWNISH-GRAY; - LOOSE TO COMPACT -	2	SS	5	75
		3	SS	8	0
10'0"		4	SS	9	42
		5	SS	9	75
15'0"		6	SS	12	42
		7	SS	7	63
191.0' 18'6"	SILTY CLAY BANDED PINK AND GRAY; DARK STREAKS; OCCASIONAL SILT SEAMS; - FIRM -				
20'0"		8	ST		100
		9	ST		100
25'0"		10	ST		100
30'0"		11	ST		100
		12	ST		



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RECORD OF BOREHOLE 3

PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5	DRILLING DATE 14-15 16/2/73
LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO	REPORT DATE 28/5/73
DATUM GEODETIC BOREHOLE TYPE WASH	DRAWN BY N.K.
SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA. NX	CHECKED BY J.D.S.

SOILS PROFILE

Elev. Depth	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY
35'0"	AS ABOVE		12	ST		100
			13	ST		100
40'0"			14	ST		100
		P2				
45'0"			15	ST		100
50'0"			16	ST		100
157.5' 52'0"	SILTY CLAY GRAY; PRESENCE OF SHELLS; OCCASIONAL SILT SEAMS; DARK STREAKS; - STIFF -					
55'0"						
60'0"			17	ST		100
65'0"						

DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

STRENGTH

FIELD VANE SHEAR	*
LAB VANE SHEAR	X
UNCONF COMP STRENGTH(q_u)	

0.5 1.0 1.5 2.0 k.s.f.

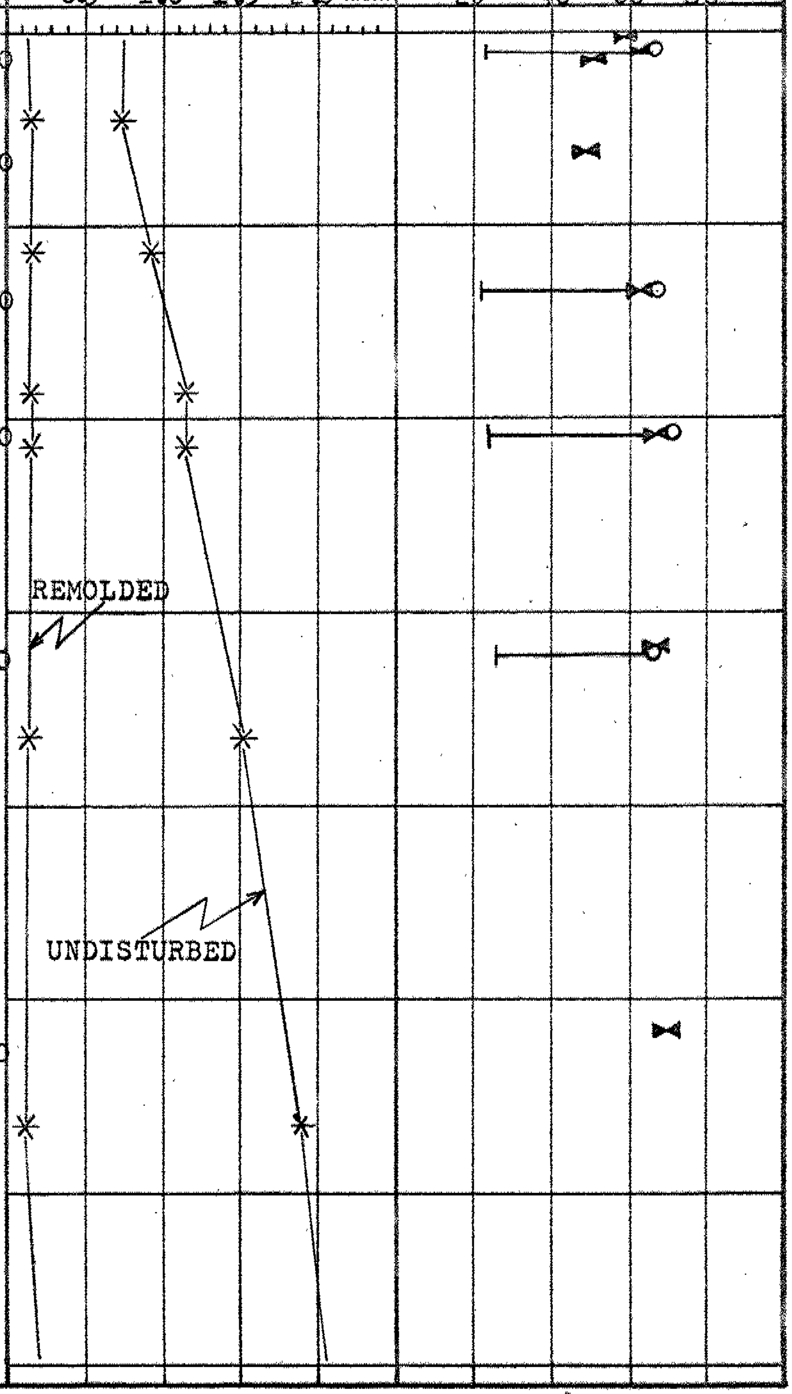
CONSISTENCY:

NATURAL MOISTURE CONTENT (W)

LIQUID LIMIT (w_l)

PLASTIC LIMIT (w_p)

20 40 60 80 %



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RECORD OF BOREHOLE 3

PROJECT: BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5
LOCATION: CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO
DATUM: GEODETIC   BOREHOLE TYPE: WASH
SAMPLER HAMMER WEIGHT: 140 lb.   DROP: 30 inches   BOREHOLE DIA.: NX

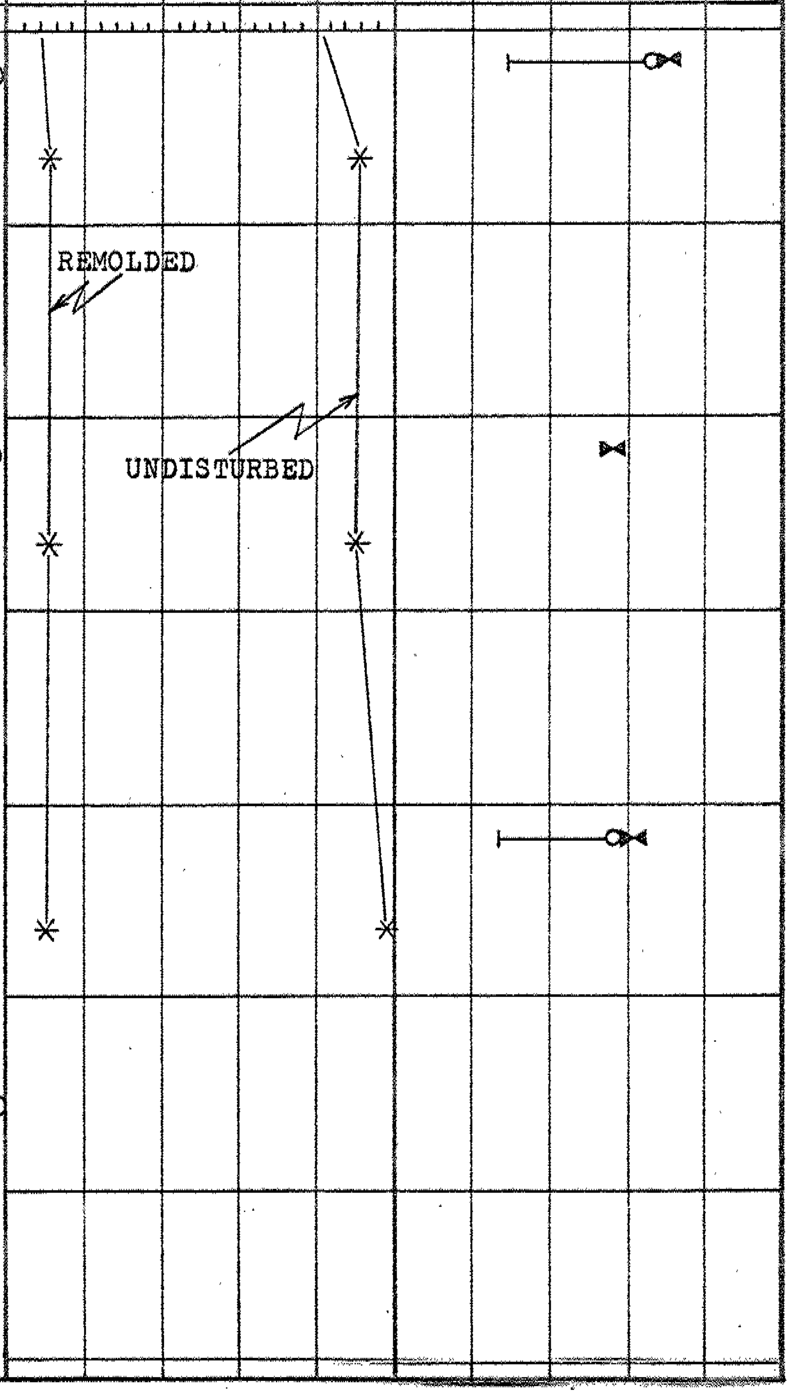
DRILLING DATE: 14-15/2/73
REPORT DATE: 28/5/73
DRAWN BY: N.K.
CHECKED BY: J.D.S.

Elev. / Depth	SOILS PROFILE: DESCRIPTION	NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY
70'0"	AS ABOVE	18	ST		100
75'0"					
80'0"		19	ST		100
85'0"					
90'0"		20	ST		100
95'0"					
112.8' / 96'8"	MIXTURE OF GRAVEL AND SILTY SAND; GENERALLY GRAY; - DENSE -	21	SS	41	100
100'0"					
106.2' / 103'3"	BEDROCK: GREY LIMESTONE	22	RC		

DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

STRENGTH
FIELD VANE SHEAR *
LAB VANE SHEAR X
UNCONF COMP STRENGTH(q_u)
0.5 1.0 1.5 2.0 k.s.f.

CONSISTENCY:
NATURAL MOISTURE CONTENT (W)
LIQUID LIMIT (w_l)
PLASTIC LIMIT (w_p)
20 40 60 80 %



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RECORD OF BOREHOLE 3

PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5	14-15 DRILLING DATE 16/2/73
LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO	REPORT DATE 28/5/73
DATUM GEODETIC BOREHOLE TYPE WASH	DRAWN BY N.K.
SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA. NX	CHECKED BY J.D.S.

SOILS PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS/FT. / STRENGTH: FIELD VANE SHEAR *, LAB VANE SHEAR X, UNCONF COMP STRENGTH (q_u) k.s.f.	CONSISTENCY: NATURAL MOISTURE CONTENT (W), LIQUID LIMIT (w_l), PLASTIC LIMIT (w_p) %
Elev. Depth	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY		
103.4'			22	RC		81		
106'1"	END OF BOREHOLE							

ROCK DESCRIPTION

Limestone, grey, fine to medium grained, laminated with black shale(shale content 20%).

The rock is fractured vertically throughout, parallel to numerous thin joints filled with recrystallized secondary carbonates. Horizontal fractures occur parallel to the shale/limestone interfaces.

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RECORD OF BOREHOLE 4

PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5 — DRILLING DATE 19/2/73

LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO — REPORT DATE 28/5/73

DATUM GEODETIC — BOREHOLE TYPE WASH — DRAWN BY N.K.

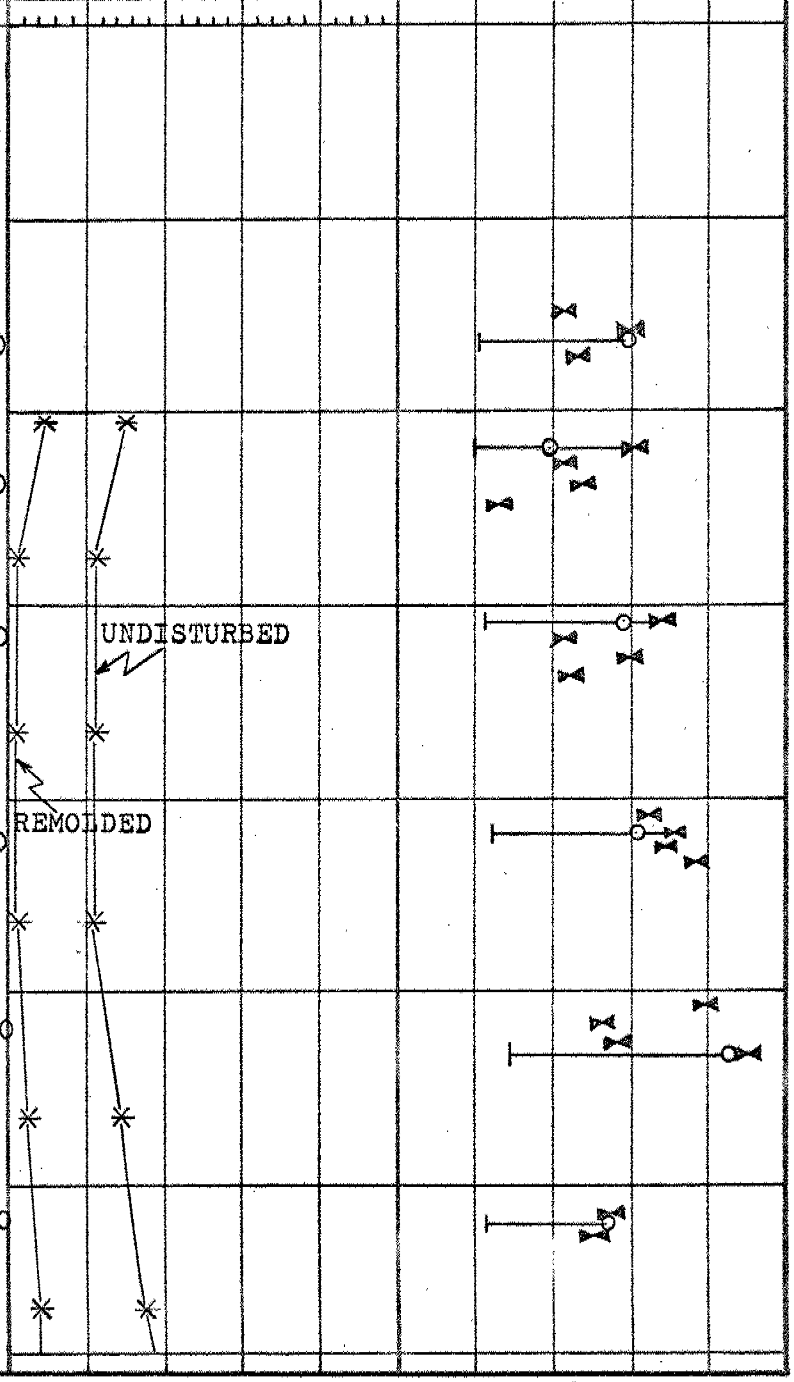
SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA. NX — CHECKED BY J.D.S.

Elev. / Depth	SOILS PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY
212.3'	GROUND SURFACE					
0'0"	ORGANIC TOPSOIL CONTAINING GRAVEL					
210.3' / 2'0"	SILTY CLAY DESICCATED; BROWN; - VERY STIFF -		1	SS	21	67
∇ (8/3/73)						
206.3' / 6'0"	SILTY CLAY BANDED PINK AND GRAY; SEAMS OF SILTY VERY FINE SAND; - FIRM -		2	ST		100
10'0"			3	ST		100
15'0"			4	ST		100
20'0"			5	ST		100
25'0"			6	ST		100
30'0"			7	ST		100

DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

STRENGTH
FIELD VANE SHEAR *
LAB VANE SHEAR X
UNCONF COMP STRENGTH(q_u)
0.5 1.0 1.5 2.0 k.s.f.

CONSISTENCY:
NATURAL MOISTURE CONTENT (W)
LIQUID LIMIT (W_l)
PLASTIC LIMIT (W_p)
20 40 60 80 %



FONDEX LTD.
FOUNDATION ENGINEERS

RECORD OF BOREHOLE 4

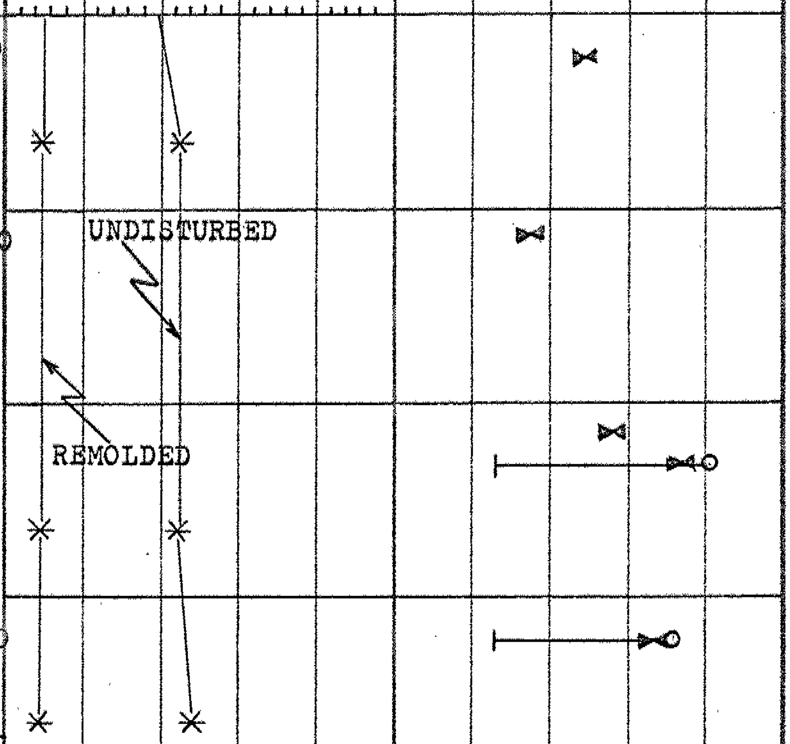
PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5 — DRILLING DATE 19/2/73
LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO — REPORT DATE 28/5/73
DATUM GEODETIC — BOREHOLE TYPE WASH — DRAWN BY N.K.
SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA. NX — CHECKED BY J.D.S.

SOILS PROFILE			SAMPLES				
Elev. / Depth	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY
35'0"			8	ST		100
40'0"			9	ST		100
			10	SS		56
45'0"	AS ABOVE		11	ST		100
50'0"			12	ST		100
158.8' / 53'6"	END OF BOREHOLE					

DYNAMIC PENETRATION RESISTANCE BLOWS/FT.

STRENGTH
FIELD VANE SHEAR *
LAB VANE SHEAR x
UNCONF COMP STRENGTH (q_u)
0.5 1.0 1.5 2.0 k.s.f.

CONSISTENCY:
NATURAL MOISTURE CONTENT (W)
LIQUID LIMIT (W_L)
PLASTIC LIMIT (W_P)
20 40 60 80 %



FONDEX LTD.
FOUNDATION ENGINEERS

RECORD OF BOREHOLE 5

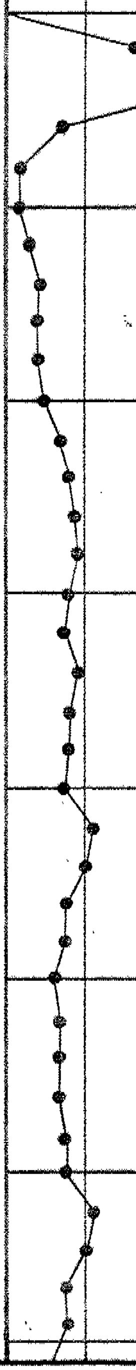
PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5
DRILLING DATE 19-20/2/73

LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO
REPORT DATE 28/5/73

DATUM GEODETIC BOREHOLE TYPE DYNAMIC PENETRATION
DRAWN BY N.K.

SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA.
CHECKED BY J.D.S.

SOILS PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS/FT. 2 " ϕ CONE 20 40 60 80 / STRENGTH: FIELD VANE SHEAR *, LAB VANE SHEAR X, UNCONF COMP STRENGTH(q_u) k.s.f.	CONSISTENCY: NATURAL MOISTURE CONTENT (W), LIQUID LIMIT (W_l), PLASTIC LIMIT (W_p) %
Elev. / Depth	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY		
193.4	GROUND SURFACE							
0'0"								
5'0"								
10'0"								
15'0"								
20'0"								
25'0"								
30'0"								



FONDEX LTD.
FOUNDATION ENGINEERS

RECORD OF BOREHOLE 5

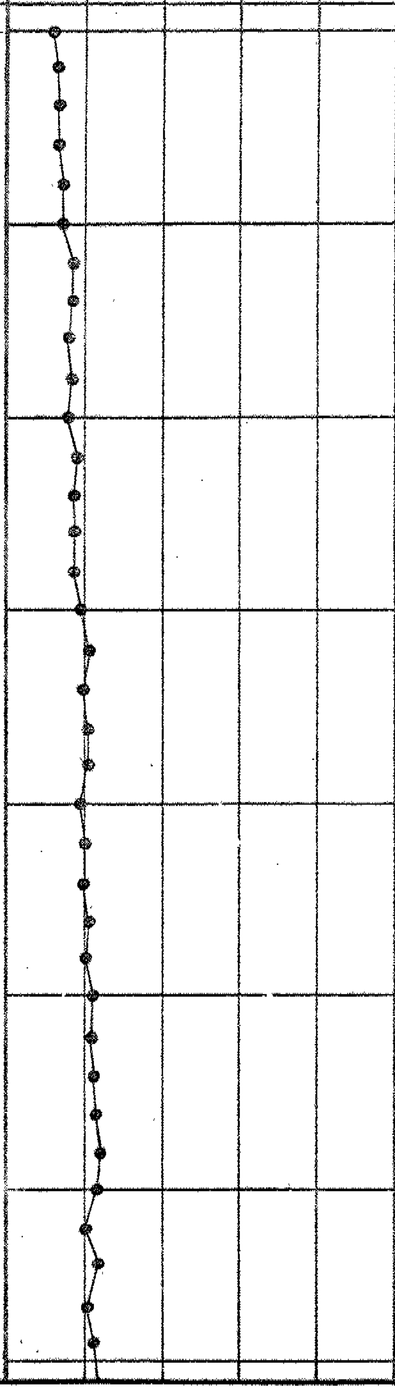
PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5 | DRILLING DATE 19-20/2/73

LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO | REPORT DATE 28/5/73

DATUM GEODETIC | BOREHOLE TYPE DYNAMIC PENETRATION | DRAWN BY N.K.

SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA. | CHECKED BY J.D.S.

SOILS PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS/FT. 2 " Ø CONE 20 40 60 80 / STRENGTH: FIELD VANE SHEAR *, LAB VANE SHEAR X, UNCONF COMP STRENGTH(q_u) k.s.f.	CONSISTENCY: NATURAL MOISTURE CONTENT (W), LIQUID LIMIT (W_l), PLASTIC LIMIT (W_p) %
Elev. Depth	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY		
35'0								
40'0								
45'0								
50'0								
55'0								
60'0								
65'0								



FONDEX LTD.
FOUNDATION ENGINEERS

RECORD OF BOREHOLE 5

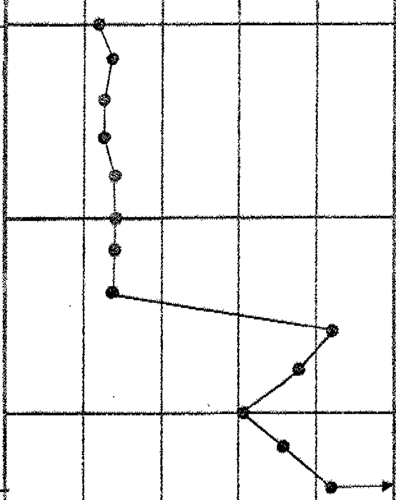
PROJECT BRIDGE STRUCTURE No. 10, COUNTY ROAD No. 5
DRILLING DATE 19-20/2/73

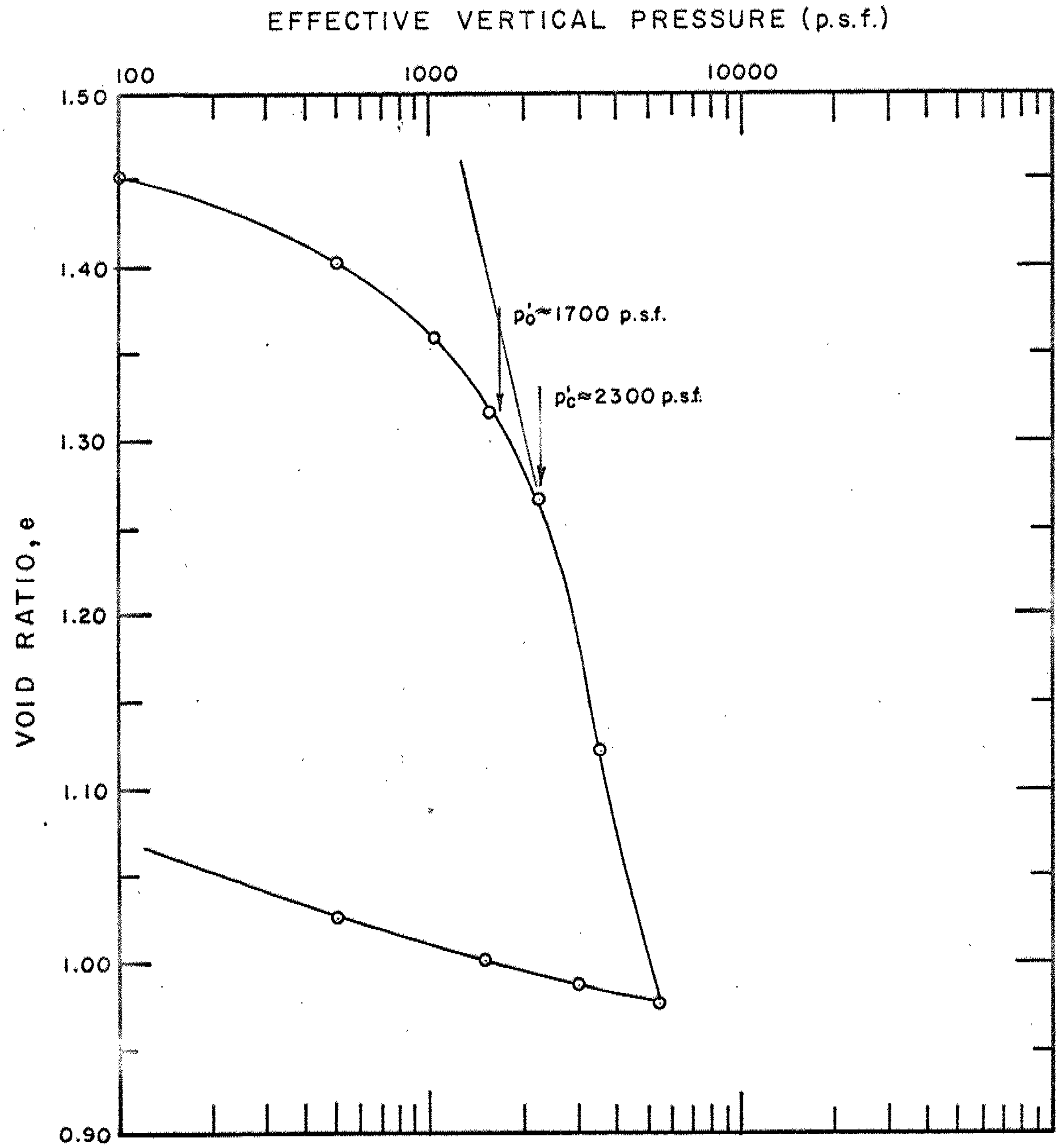
LOCATION CONCESSION VI, TOWNSHIP OF CAMBRIDGE, ONTARIO
REPORT DATE 28/5/73

DATUM GEODETIC BOREHOLE TYPE DYNAMIC PENETRATION
DRAWN BY N.K.

SAMPLER HAMMER WEIGHT 140 lb. DROP 30 inches BOREHOLE DIA.
CHECKED BY J.D.S.

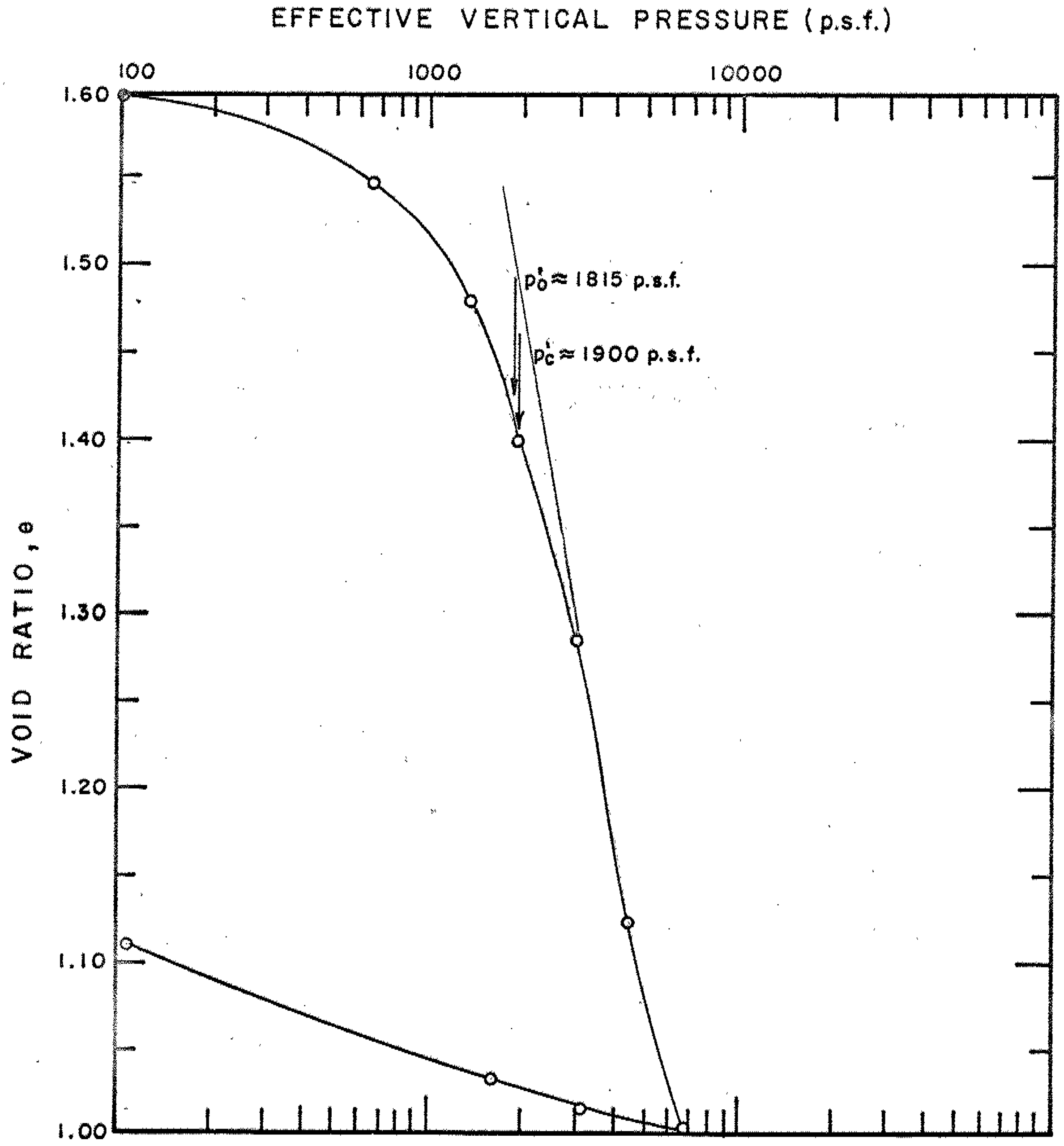
SOILS PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS/FT. 2" Ø CONE 20 40 60 80 / STRENGTH: FIELD VANE SHEAR *, LAB VANE SHEAR X, UNCONF COMP STRENGTH(q_u) k.s.f.	CONSISTENCY: NATURAL MOISTURE CONTENT (W), LIQUID LIMIT (W_l), PLASTIC LIMIT (W_p) %
Elev. Depth	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT (N)	% RECOVERY		
70'0"								
75'0"								
80'0"								
111.4' 81'11"	END OF PENETRATION						REFUSAL	





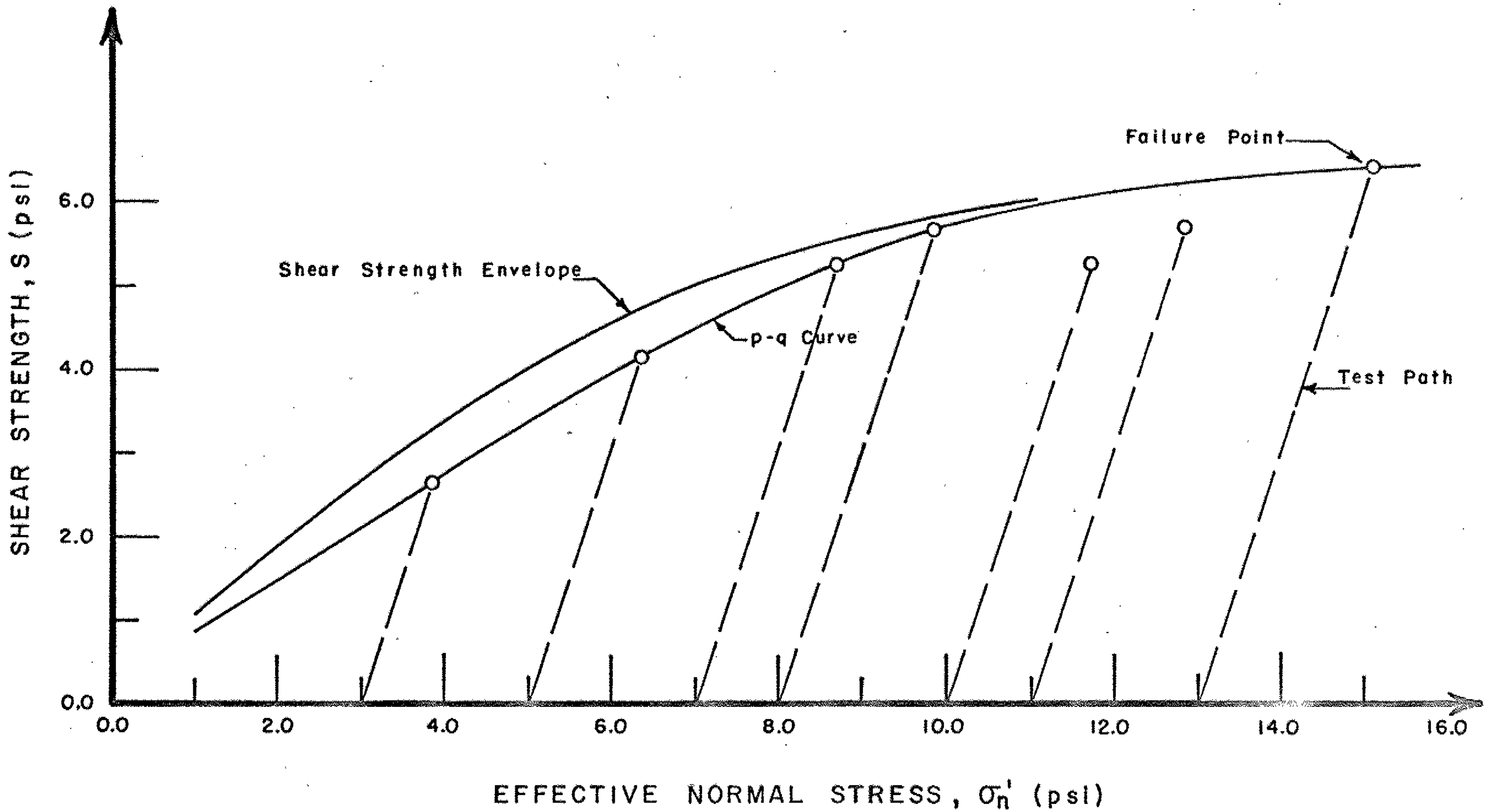
CONSOLIDATION TEST RESULTS
B.H. No.2, SAMPLE No.4, DEPTH 20'-5"

FIGURE 1



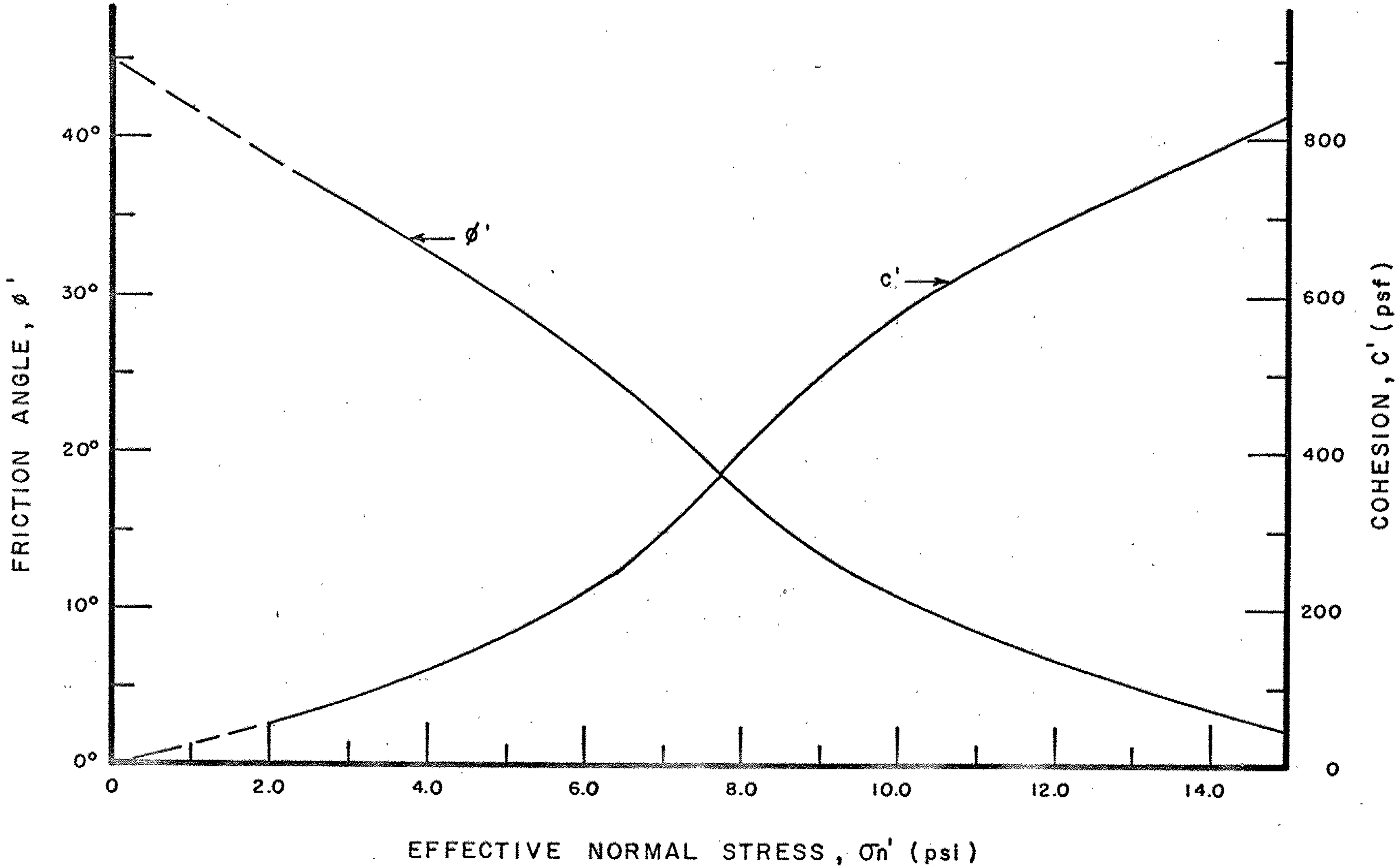
CONSOLIDATION TEST RESULTS
B. H. No. 3, SAMPLE No. 15, DEPTH 31'-2"

FIGURE 2



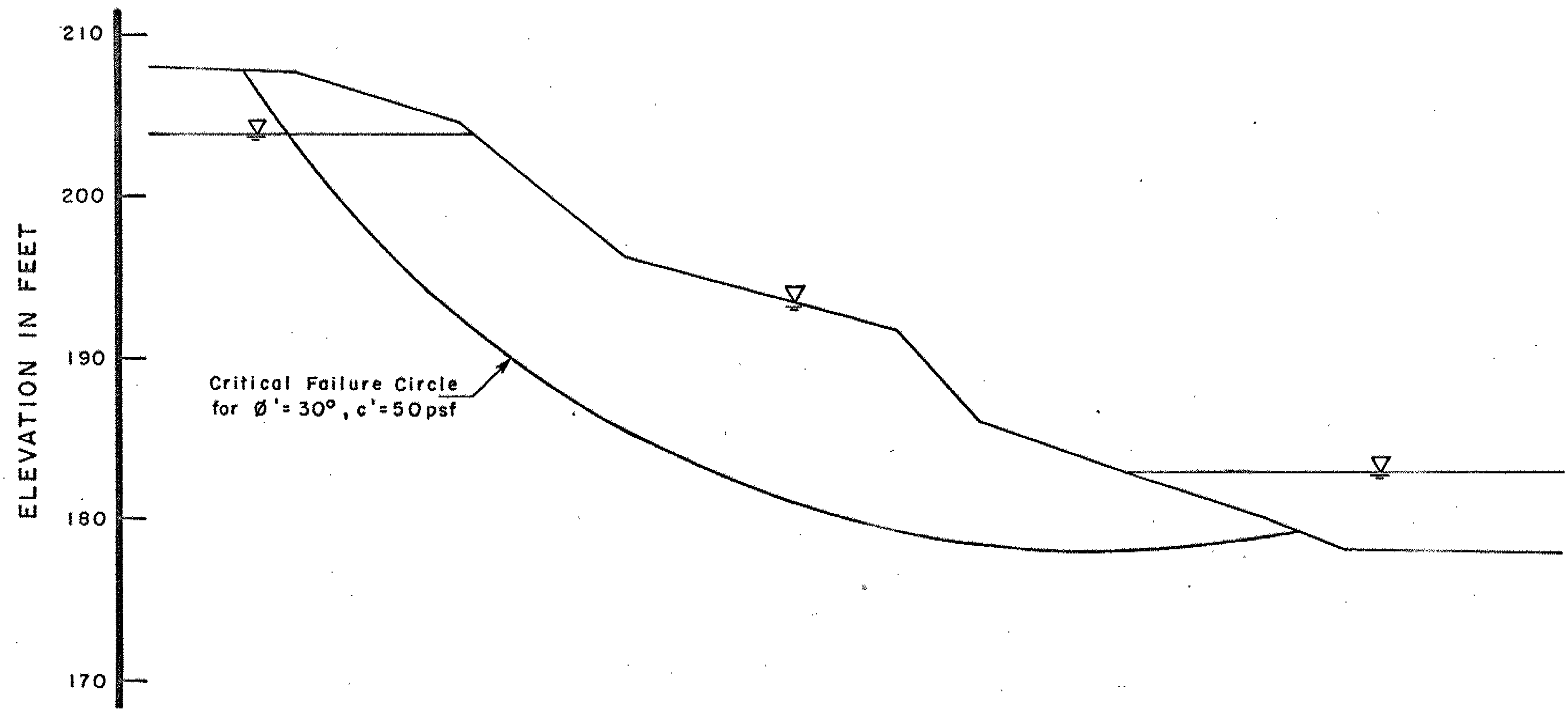
EFFECTIVE STRESS SHEAR STRENGTH

FIGURE 3



EFFECTIVE STRESS SHEAR STRENGTH PARAMETERS

FIGURE 4

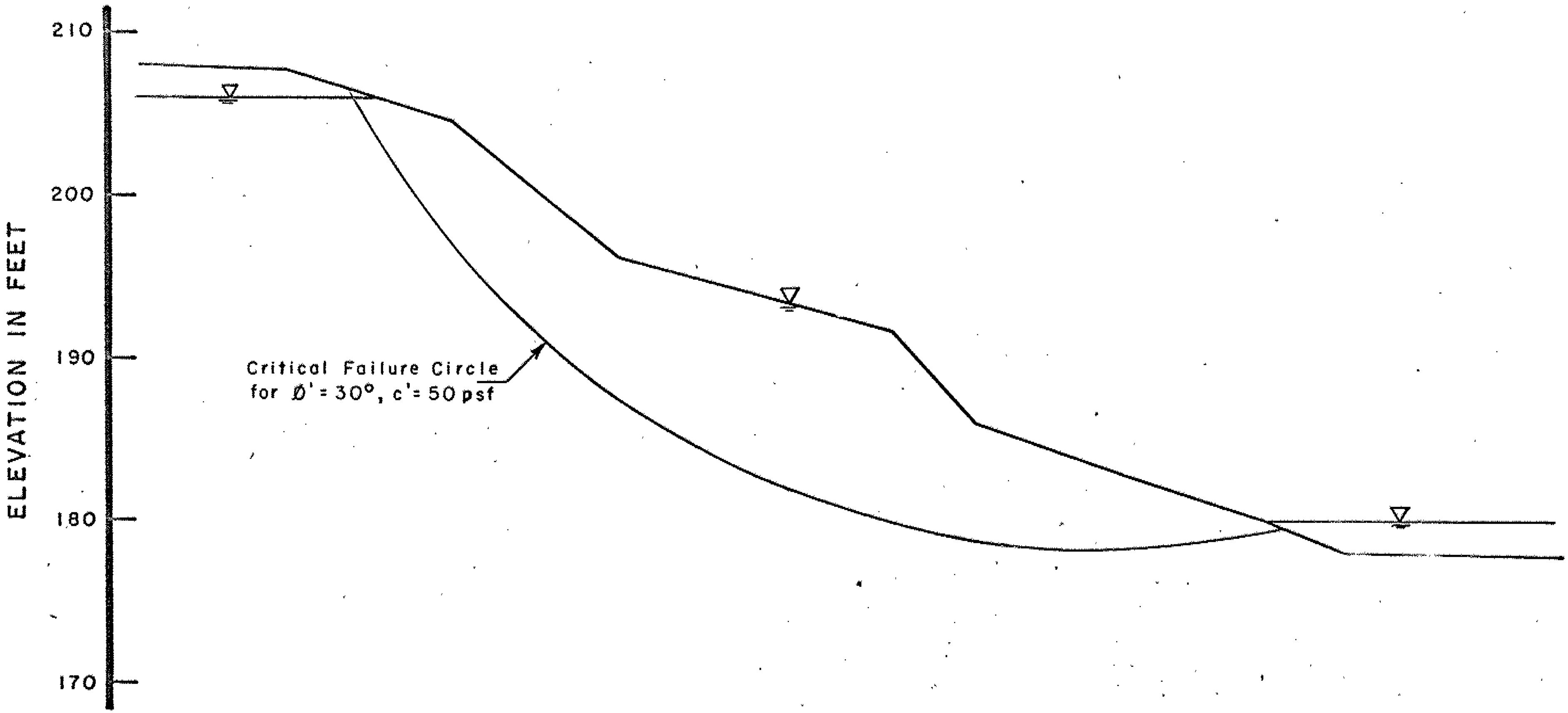


FACTORS OF SAFETY

c' / ø'	50 (psf)	200 (psf)	350 (psf)
20°	0.57	0.97	1.33
30°	0.80	1.26	1.62
40°	1.09	1.57	1.97

FIGURE 5

CASTOR RIVER BRIDGE
COUNTY ROAD No. 5
SLOPE STABILITY ANALYSIS
GROUNDWATER SURFACE No. 1
SECTION B-B, EXISTING SLOPE
SCALE: 1"= 10'

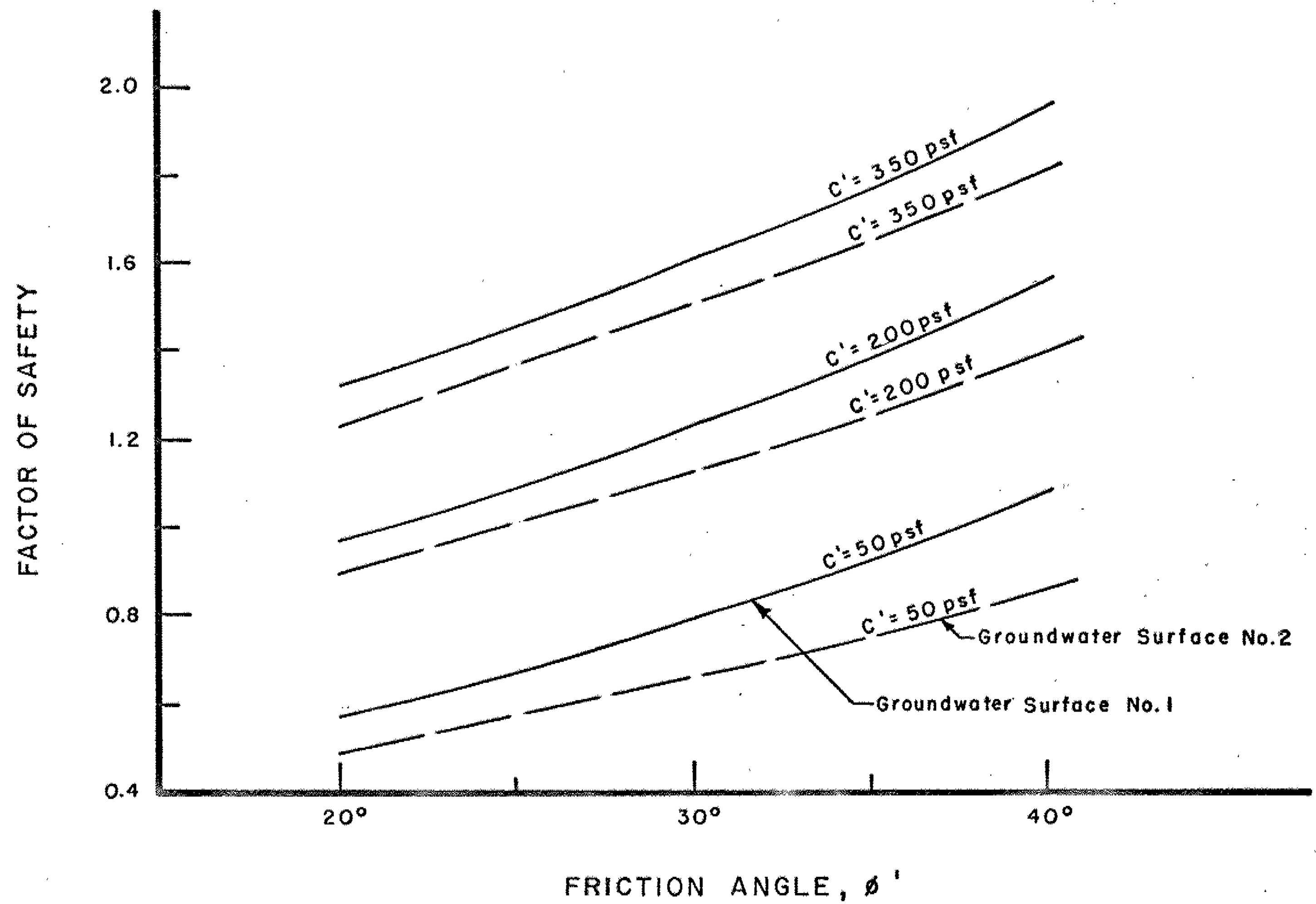


FACTORS OF SAFETY

c' / Ø'	50 (psf)	200 (psf)	350 (psf)
20°	0.49	0.91	1.26
30°	0.67	1.16	1.51
40°	0.88	1.42	1.83

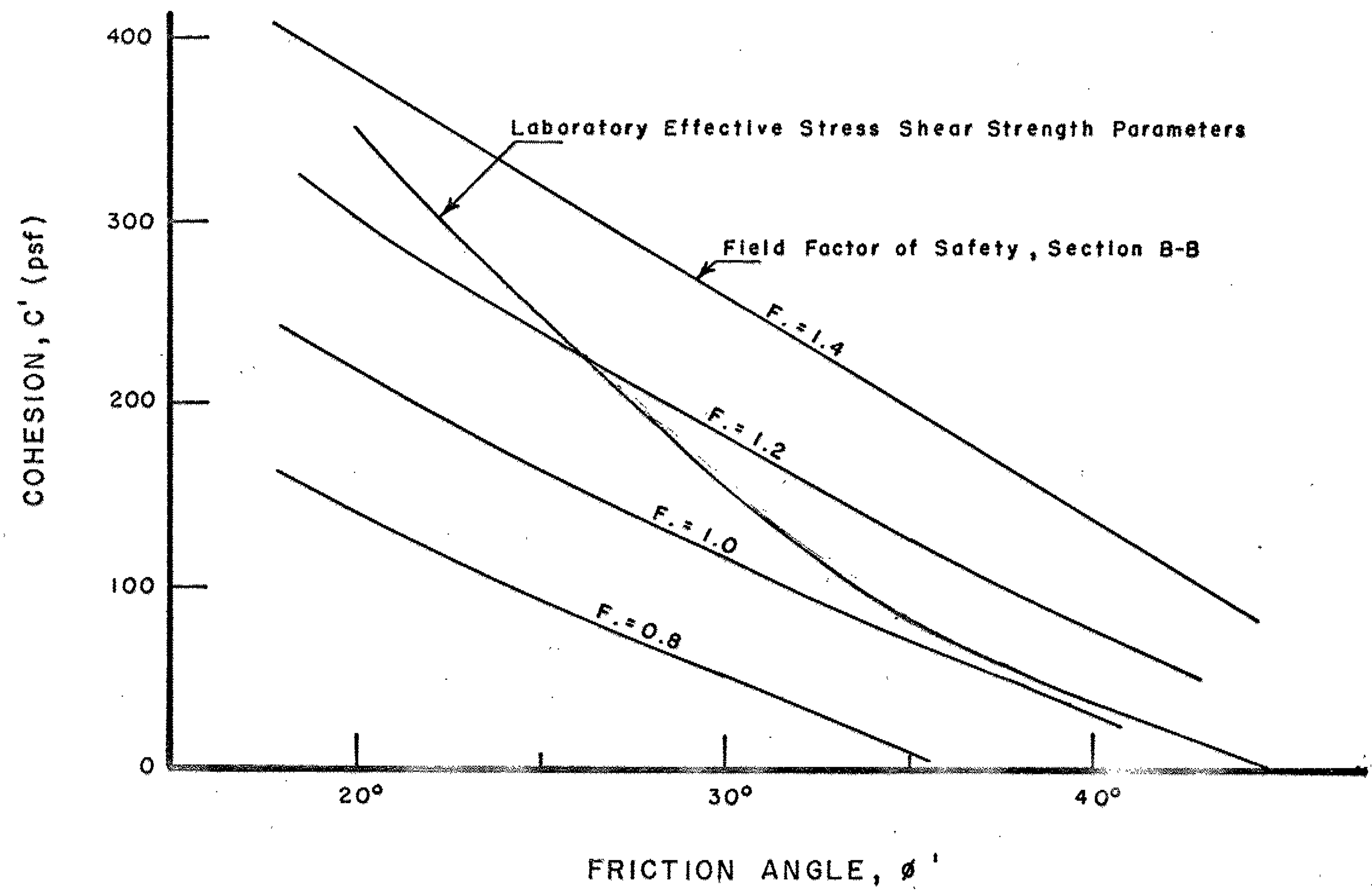
FIGURE 6

CASTOR RIVER BRIDGE
COUNTY ROAD No. 5
SLOPE STABILITY ANALYSIS
GROUNDWATER SURFACE No. 2
SECTION B-B, EXISTING SLOPE
SCALE: 1" = 10'



SLOPE STABILITY ANALYSIS
FACTOR OF SAFETY, SECTION B-B, EXISTING SLOPE

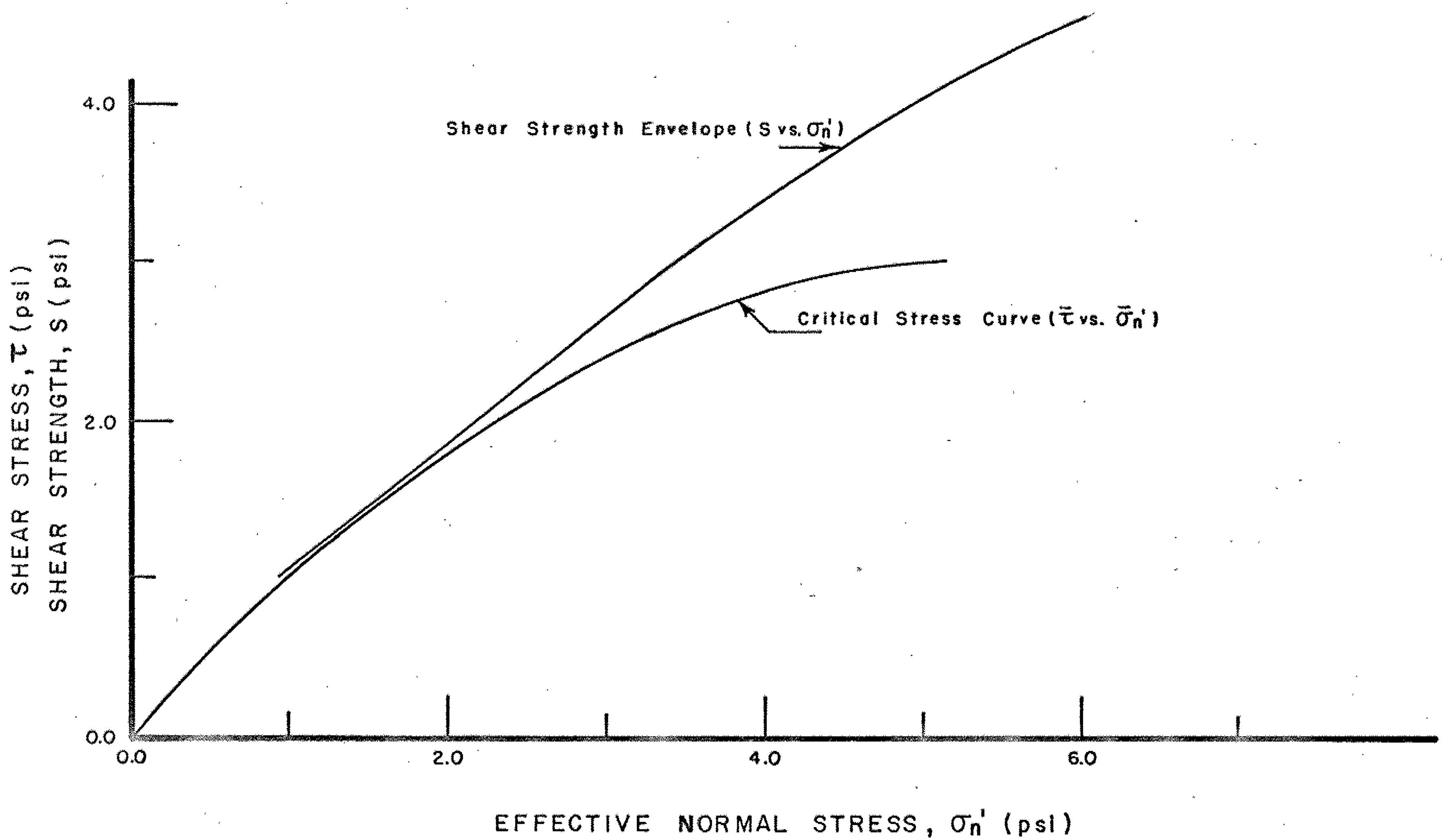
FIGURE 7



SLOPE STABILITY ANALYSIS

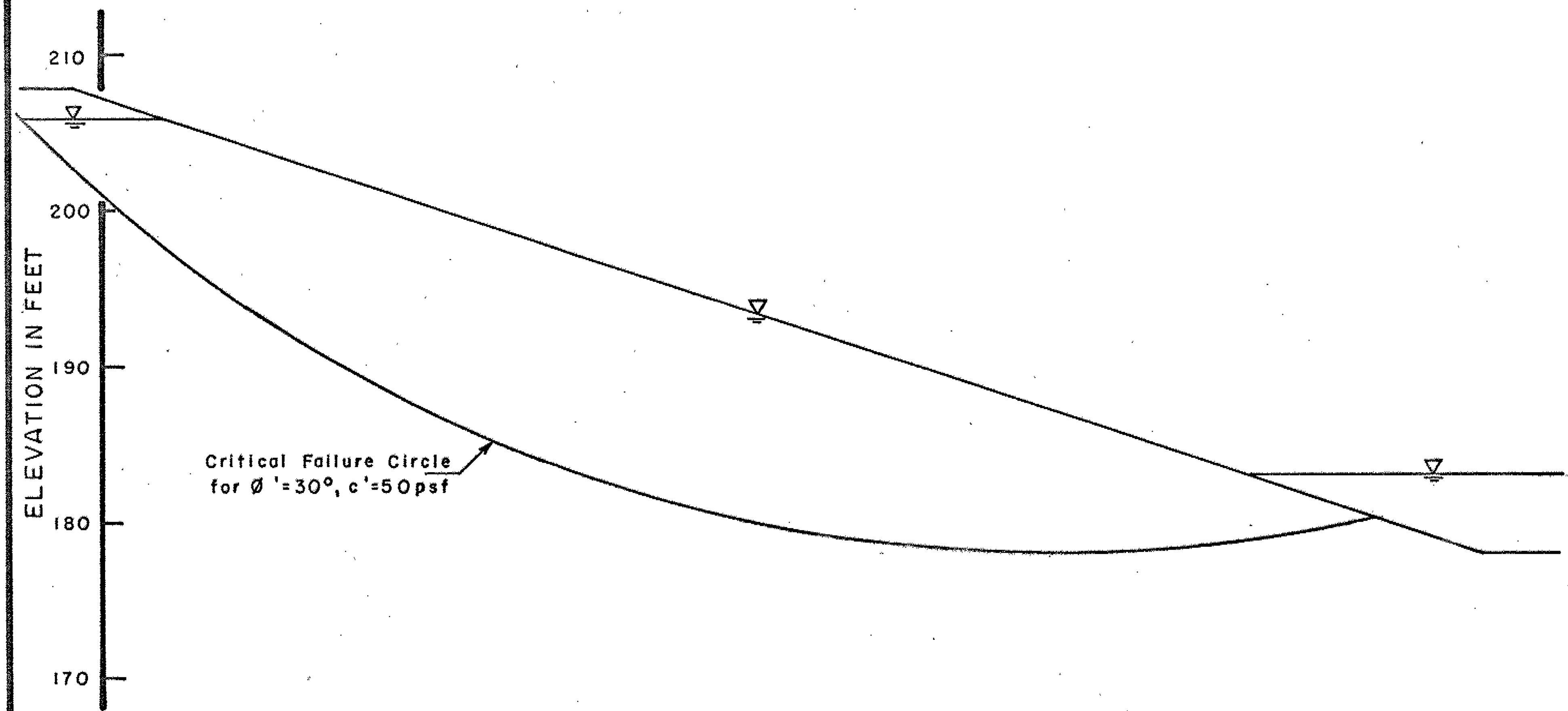
SHEAR STRENGTH - FACTOR OF SAFETY, SECTION B-B, EXISTING SLOPE

FIGURE 8



CRITICAL STRESS CURVE, SECTION B-B, EXISTING SLOPE

FIGURE 9

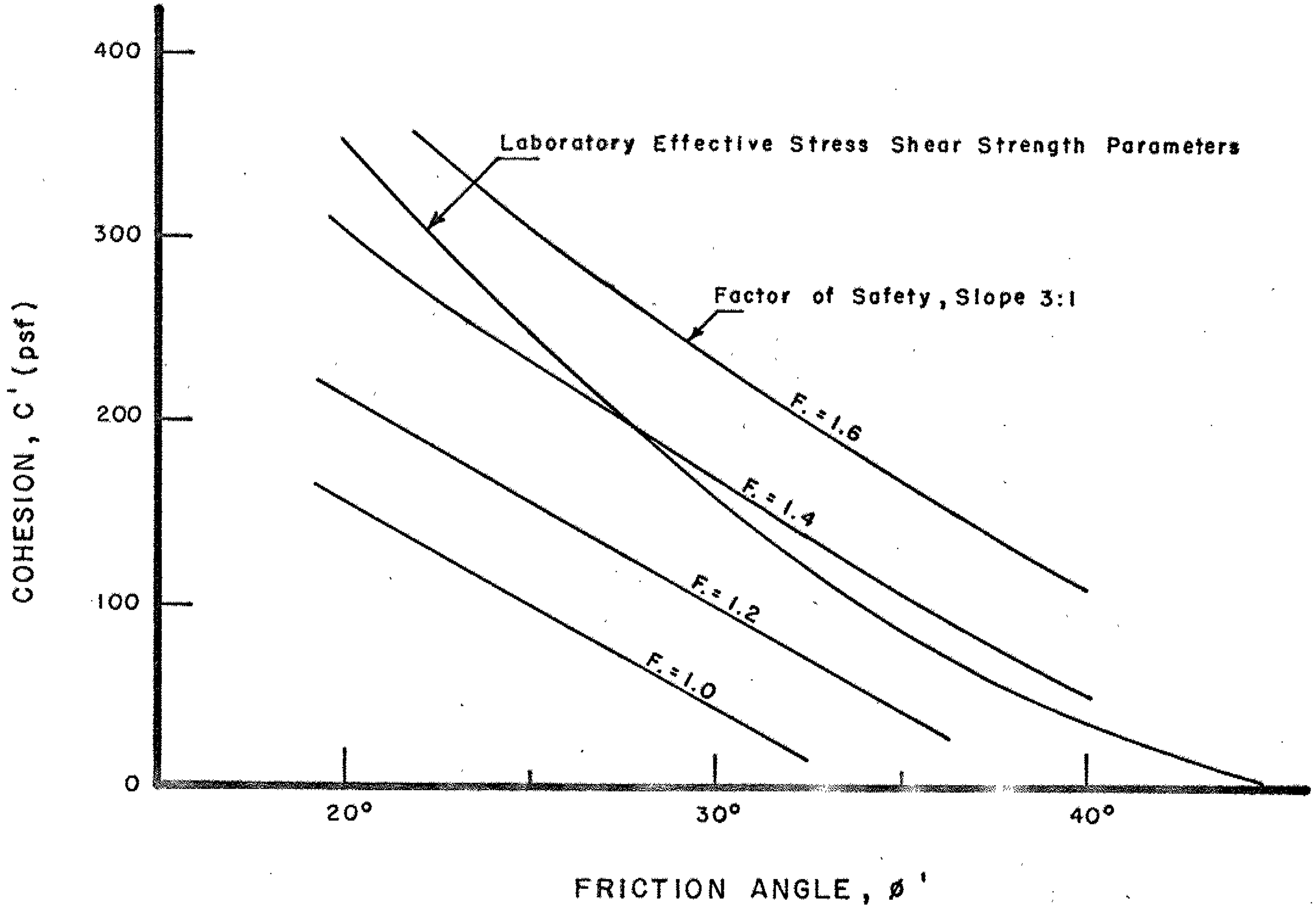


FACTORS OF SAFETY

C' / Ø'	50 (psf)	200 (psf)	350 (psf)
20°	0.72	1.15	1.53
30°	1.03	1.51	1.91
40°	1.41	1.93	2.37

FIGURE 10

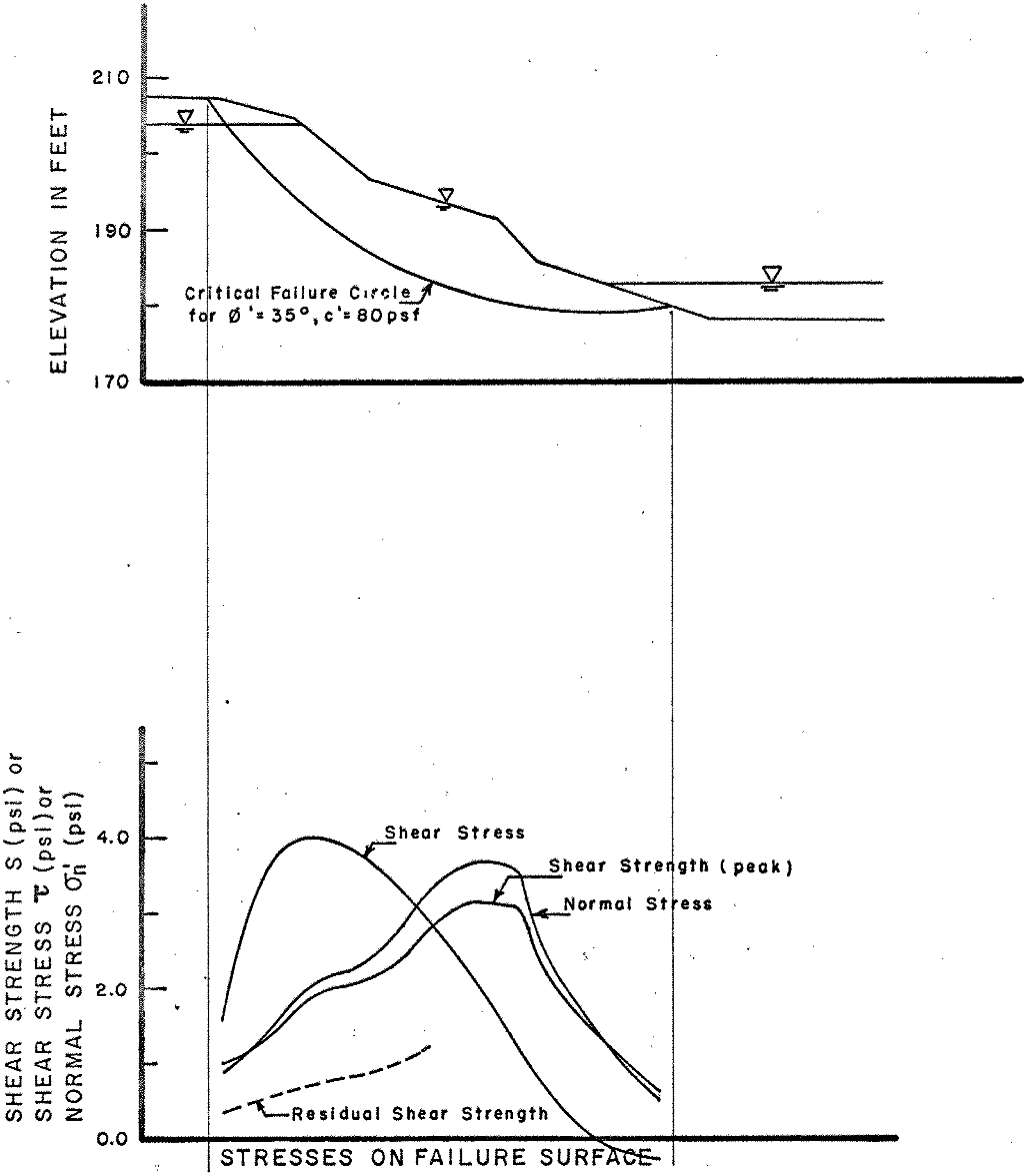
CASTOR RIVER BRIDGE
COUNTY ROAD No. 5
SLOPE STABILITY ANALYSIS
GROUNDWATER SURFACE No. 1
SLOPE CUT 3 HORIZONTAL
TO 1 VERTICAL
SCALE: 1" = 10'



SLOPE STABILITY ANALYSIS

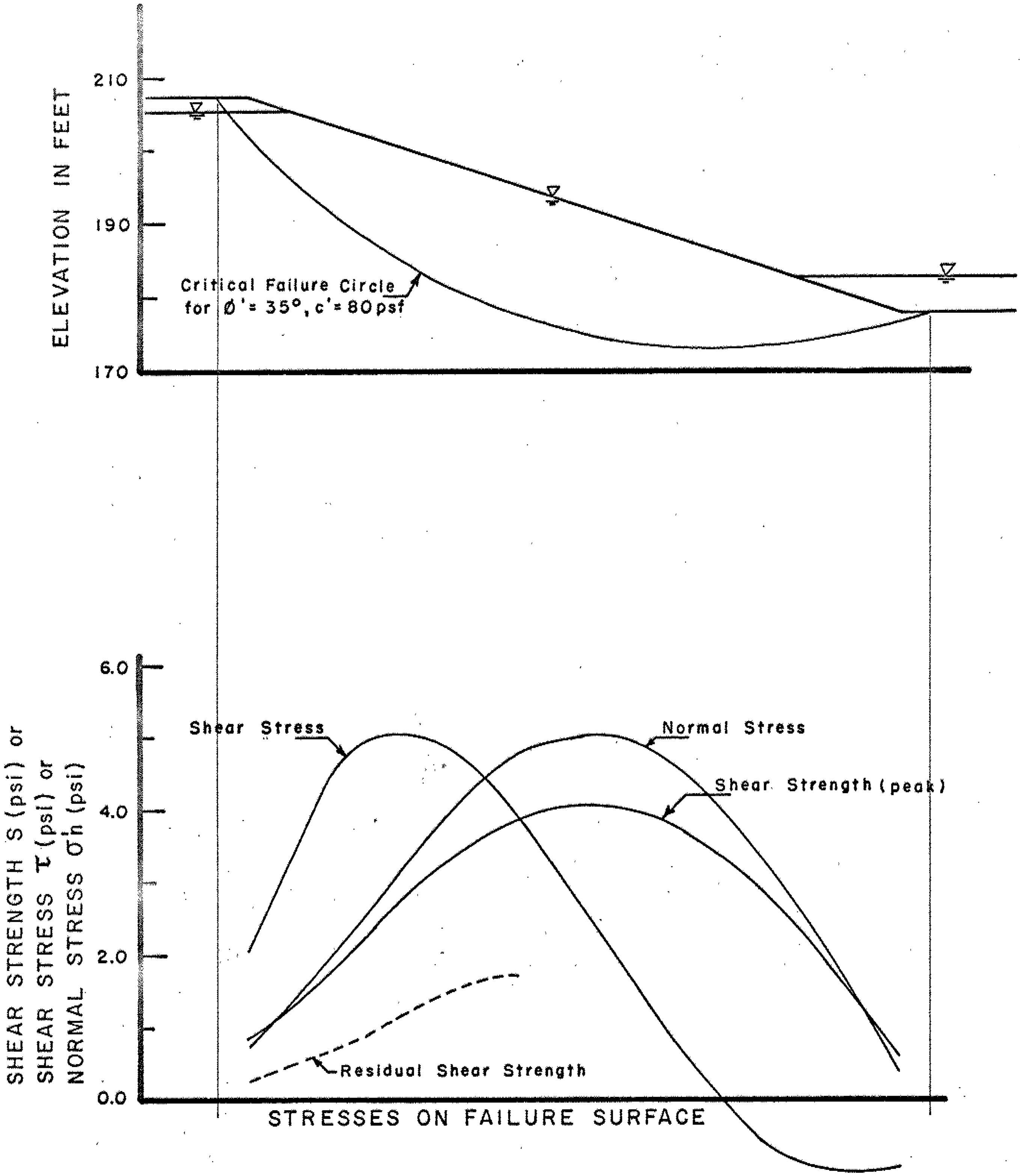
SHEAR STRENGTH - FACTOR OF SAFETY, SLOPE 3:1

FIGURE 11



SECTION B-B
EXISTING SLOPE
STRESSES ON FAILURE SURFACE

FIGURE 12

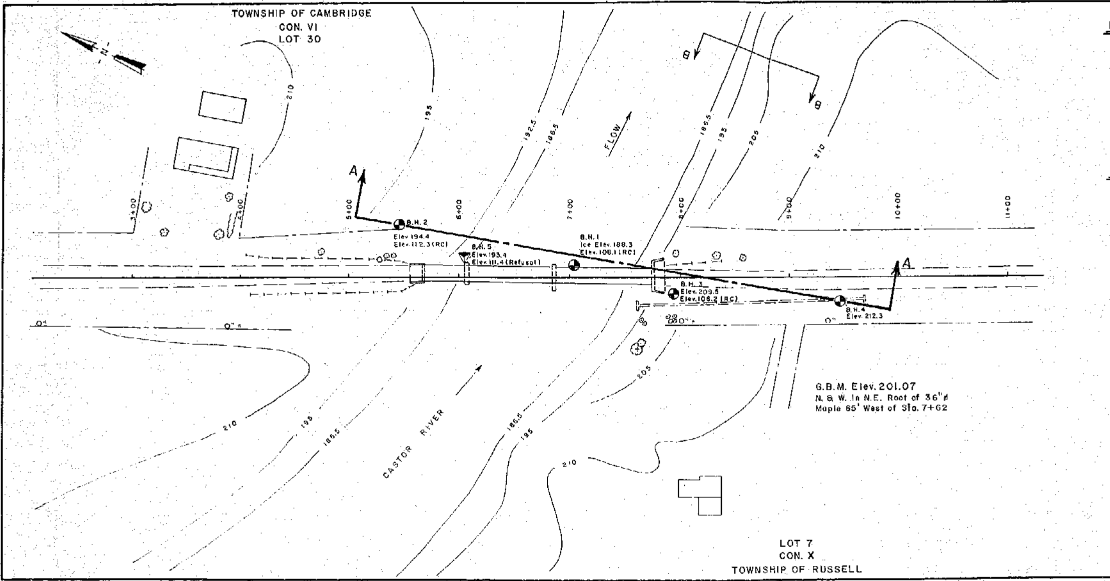


SLOPE 3:1

STRESSES ON FAILURE SURFACE

FIGURE 13

OVERSIZE DRAWING



LEGEND

BOREHOLE IN PLAN

CONE PENETRATION IN PLAN

Elev. 194.4 GROUND SURFACE ELEVATION

Elev. 112.3 (RC) BEDROCK SURFACE ELEVATION

NOTES

1- A DETAILED DESCRIPTION OF THE SOIL CONDITIONS IS GIVEN IN THE REPORT BY FONDEX LTD.

2- ALL ELEVATIONS REFER TO GEODETIC DATUM.

UNITED COUNTIES OF PRESCOTT & RUSSELL

PROPOSED BRIDGE AT CROSSING
OF COUNTY ROAD No. 5 OVER CASTOR RIVER
CONCESSION VI, TOWNSHIP OF CAMBRIDGE

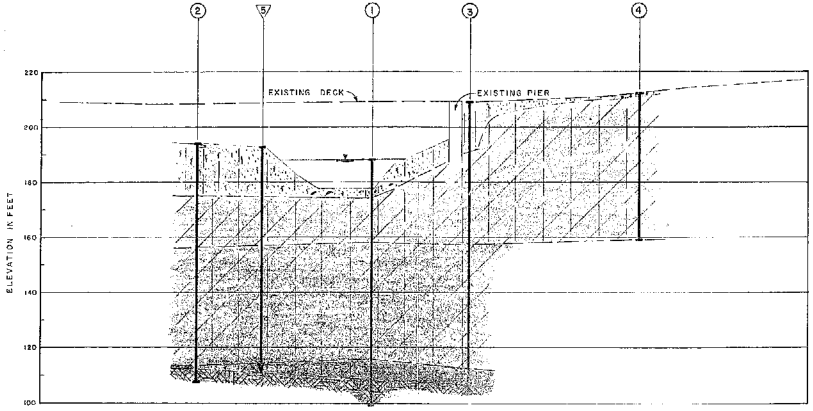
SITE & LOCATION PLAN

DRAWN BY : B.D.
CHECKED BY : [illegible]

DRAWING N° 3246-5-1

FONDEX LTD.
OTTAWA GATINEAU

SCALE : 1"=50'
DATE : MAY 15, 1973



SECTION "A-A"

LEGEND

②	BOREHOLE IN SECTION
▽	CONE PENETRATION IN SECTION
	SILTY CLAY: BANDED PINK & GRAY -FIRM-
	SILTY CLAY: GRAY; SILT SEAMS. -STIFF-
	GLACIAL TILL: -DENSE-
	ALLUVIUM: SILTY SAND; BROWN; PRESENCE OF ORGANIC. -LOOSE-
	FILL MATERIAL: SILTY FINE SAND; BROWNISH-GRAY. -LOOSE TO COMPACT-
	BEDROCK: GRAY LIMESTONE

NOTES

1 - THE GEOLOGICAL BOUNDARIES HAVE BEEN DETERMINED AT THE BOREHOLE LOCATIONS ONLY. THE STRATIGRAPHY GIVEN BETWEEN BOREHOLES IS INFERRED FROM GEOLOGICAL EVIDENCE AND DOES NOT NECESSARILY CORRESPOND TO THE TRUE STRATIGRAPHY.

2 - A DETAILED DESCRIPTION OF THE SOIL CONDITIONS IS GIVEN IN THE REPORT BY FONDEX LTD.

3 - ALL ELEVATIONS REFER TO GEODETIC DATUM.

UNITED COUNTIES OF PRESCOTT & RUSSELL

PROPOSED BRIDGE AT CROSSING OF COUNTY ROAD No. 5 OVER CASTOR RIVER CONCESSION VI, TOWNSHIP OF CAMBRIDGE

STRATIGRAPHIC PROFILE

DRAWN BY : B.D. CHECKED BY : N.K.	DRAWING N° 3246-S-2
FONDEX LTD. OTTAWA GATINEAU	SCALE : HOR. 1"=50' VERT. 1"=20' DATE : MAY 15, 1973