#62-F-213-C

W.P. #108-62

HWY. #42 &

BEVERLEY

CREEK CROSSING

Mr. A. M. Toye,

May 31, 1962.

Bridge Engineer.

FOUNDATION INVESTIGATION REPORT
By: Dominion Soil Investigation,
Limited.

Materials & Research Division,

(Foundation Section)

Attention: Mr. S. McCombie.

Re: Proposed Crossing of Revised Hwy. No. 42 and Beverley Creek in Delta, District #8
W.P. 108-62.

We attach herewith, the report on the soil investigation at the above site, submitted by the consultant, Dominion Soil Investigation, Ltd. The report has been reviewed and the following comments may be made:

The interpretation of soil data in the report is open to objections. However, this does not affect the final recommendations made in the report, to which we agree. The predicted settlement of the approach fill is on the high side, but it is certain that some settlement will occur. To minimize any undesirable effect of settlement causing maintenance problems, the fill should be placed for a period of one year before the pavement is placed. If the time schedule does not permit, a temporary pavement may be constructed and the permanent pavement placed a year later.

We believe the data contained in the consultant's report and our foregoing comments, should provide adequate information for your future design work. If further assistance is required, please do not hesitate to contact our Office.

KYL/MdeF
Attach.
cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
J. Ford
E. A. Cash
J. E. Gruspier
J. Roy
T. J. Kovich
E. R. Saint
F. Norman
A. Watt

Foundations Office -- Gen. Files.

KY Lo

K. Y. Lo,
SUPERVISING FOUNDATION ENGR.
For:

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

BA 1432

ONTARIO

DEPARTMENT OF HIGHWAYS

MATERIALS & RESEARCH DIVISION

Proposed Crossing of

REVISED HIGHWAY NO. 42 AND BEVERLEY CREEK IN DELTA

FOUNDATION CONDITIONS

(District #8 - W.P. 108-62)

Submitted by

DOMINION SOIL INVESTIGATION LIMITED
77 Crockford Boulevard,
SCARBOROUGH - ONTARIO

Our Reference: 2-4-10

April 1962

C O N T E N T S

E N C L O S U R E S

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INTRODUCTION

A letter of authorization dated April 13th, 1962 was received from the Ontario Department of Highways, Materials and Research Division, to conduct a foundation investigation at the site of a proposed bridge in Delta, Ont. The existing Highway #42 makes a sharp curve in the village; therefore, the realignment of an entire portion of the highway is under consideration. The present bridge being a part of this modernization is located where the realigned highway will cross the creek.

The structure will consist of three spans according to the Working Plan. However, consideration is given to a one-spanned bridge also.

The site was located with the aid of a site plan (No. E-4071-1) provided to us.

The purpose of the investigation was to reveal the subsurface conditions and determine the necessary soil properties for the design and construction of foundations.

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SUMMARY

(1) The subsoil to the north of the creek consists of:

0 - 5 ft	firm lean clay crust
5 - 16 ft.	normally consolidated silt with clay layers
16 - 36 ft.	fine sand
36 - ft	clay bed and bedrock.

The same or slightly different strata were encountered on the south side but the original topography was changed by filling close to the creek and the bedrock is higher (See subsurface profile, Encl. #1)

(2) On the south shore, the ground water table follows the contour lines and seepage pressures are directed towards the creek. On the north one, artesian pressure in the sand layer was encountered. In the less pervious strata above the sand, the water table is at about 3 ft. below ground level. It is expected that the ground water will lower after a longer, hot, dry period.

(3) The foundations of the bridge should rest upon end bearing piles driven to bedrock. Battered piles should compensate the horizontal earth pressure.

(4) Consideration should be given to the following, probably decisive factors when the general arrangement of the bridge is decided upon:

(i) the south shore should be protected by a retaining wall which would prevent erosion and secure the stability of slope.

(ii) the original ground surface on the south side should be trimmed to a slope not exceeding 6 (horizontal) to 1 (vertical) starting at the top of retaining wall.

The merits of three proposed arrangements are discussed in paragraph VI.

(5) The dewatering of excavations during construction should be done by pumping from the pits enclosed by sheet piles. These should be driven to a depth below excavation grade equal to about 1½ times the distance between excavation grade and outside water level.

(6) The subsoil is capable of supporting a 10 ft. high embankment and live load with a safety factor of 1.4 or more. The expected settlement of the subsoil will be in the order of half a foot and the consolidation will take some time to complete.

I. DESCRIPTION OF SITE AND GEOLOGY

Delta is a small village located in Leeds County about 20 miles west of Brockville. It is accessible by Highway #42. The population is less than 400 who make their living principally of farming. Owing to the presence of numerous scenie lakes abundant in fish, the people of the village enjoy a considerable income from tourism in certain seasons of the year.

Upper Beverley Lake (to the east) and Lower Beverley Lake (to the west) are connected by a creek. The existing Highway #42 crosses it several hundred feet east from the presently proposed bridge site.

The geographical position of Delta is particularly interesting from the point of view of geology. The area is close to the intersection of the following three different formations:

(i) Precambrian rocks (mostly of the sedimentary and derived metamorphic type, such as greywacke, arkose, slate, etc., containing some narrow bands of volcanics);

(ii) Sedimentary rocks (predominantly sandstone) deposited in the Cambrian period; and

(iii) Sedimentary rocks (sandstone, limestone, shale etc.) deposited in the Devonian period).

This geological fact is probably the explanation of the non-uniform bedrock stratification at the site as revealed by the borings.

The topography of the two banks of the creek differs considerably. Whereas a rather steep slope comprised the southern shore of the creek, the northern one is flat and lies deeper. Rock outcrops are visible all around and the surface features suggest that in the geological past, the two Beverley Lakes may have comprised one big lake and the present creek runs close to the southern limit of the past lake bed.

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The soils were most probably deposited by a postglacial lake. It is a proven fact that upon the retreat of the last Wisconsin glacier, this area was covered by the Lake Iroquois and later by the Champlain Sea. It is therefore not surprising that the soil in this deeper valley (or basin) is a normally or only slightly preconsolidated sediment. The upper crust may have been deposited during a short advance of the ice and its stiffness may be due to subsequent drying out or to the consolidating effect of a relatively thin ice shield.

The area north of the creek is a pastureland because excellent and abundant grass covers the soil surface. There are dwellings near-by at the south side and cracks in the walls of the two-storeyed brick store building are an advance warning of the adverse subsoil conditions.

II. FIELD WORK

Field work was carried out during the period April 17th to May 9th, 1962 and comprised five boreholes and three dynamic cone penetration tests at the locations shown on Enclosure #1. The positions of the test holes were set out on the site with the assistance of a drawing (as referred to in the Introduction) provided to us. Elevations were measured relative to the top of a nail in the south root of a 1.5 ft. diameter ash tree, 88 ft. right of station 81+30 (geodetic datum, = el. 307.57) also indicated on the said drawing.

The boreholes were of 2 7/8 in. diameter. They were lined (or partly lined) with Bx casing advanced to the required sampling depths by the repetitious procedure of alternately driving and washing. One borehole (#7) was 2½" diameter and it was cleaned out with an auger.

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Standard penetration tests were made at frequent intervals using a 2 in. outside diameter split spoon driven into the bottom of the clean borehole by a constant driving energy (140 pound hammer dropping 30 inches). The dynamic cone penetration test is one type of deep sounding in which the Bx rods with a 2 in. diameter 60 degree apex cone driving point are driven into the subsoil without casing and applying the same driving energy as above. The former test provided disturbed samples of the substrata indicating their relative density and consistency and the latter a continuous record of soil density.

Undisturbed samples were taken, whenever possible, with 2 in. diameter thin-walled tubes forced into the subsoil in one rapid continuous movement. Attention is called to the extreme difficulty of obtaining "undisturbed" samples from the grey, slightly or non-plastic silt with soft, silty clay layers stratum. Although the securing of samples was attempted many times, in most cases, this was without success.

The in situ shear strength of the cohesive strata was measured wherever the undrained shear strength was less than 3000 pounds per square ft. One hole (#7) consisted entirely of successive vane tests along its depth. A four bladed vane 4 ins. long and 2 ins. in diameter with a blade thickness of 1/8 in. was used. The remoulded shear strength was also measured, thus providing the sensitivity index of the subsoil. The method suggested by Skempton (Ref: 8) was followed in carrying out these vane tests, i.e. the disturbance of the material was minimized by allowing a distance of 1'-6" between the bottom of borehole and top of vane blades.

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Where bedrock was encountered, the holes were advanced by diamond drilling. Axt size 1 1/8 in. diameter core was recovered.

The stratification of the subsoil, sampling depths, the results of the penetration and vane tests together with percentages of core recovery are recorded on geotechnical data sheets comprising Enclosures #2 to #6 inclusive.

III. LABORATORY WORK

The properties of the subsurface material were determined by laboratory methods. The purpose of some tests was the proper classification of the soil only; others were performed in order to evaluate the shear strength.

The grain size distribution of six materials was obtained by sieve and, if needed, hydrometer analyses. They are presented on Enclosures #7 and #8. Attention is called to the results of the two specimens taken from the very same sample. (B.H. #4, Samples #3A and #3B). The difference in shape of the curves is conspicuous and truly illustrates the non-uniform subsurface conditions.

The liquid and plastic limits together with the natural moisture contents indicate the state of the material in connection with water. This relationship is expressed by the liquidity index (=LI). The principal error inherently committed by computing this index is that it compares the moisture contents in the remolded and natural states. However, regarding the fact that the LI is generally accepted and used in soil mechanics, I included a tabulated summary of results.

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B.H.	SA.	LL %	PL %	PI%	W %	LI
1	1	23	18.6	4.4	23.4	1.09
	2	24	19.4	4.6	29.0	2.09
	3	19	11.5	7.5	20.7	1.23
4	1	38.5	19.4	19.1	25.8	0.34
	2	36.3	19.1	17.2	28.8	0.57
	3B	25.6	13.9	11.7	38.1	2.07
5	1	31.0	17.9	13.1	26.1	0.63
	2A	28.8	14.0	14.8	40.6	1.8
	2B	22.5	12.0	10.5	27.1	1.44
7	3	19.2	13.0	6.2	23.0	1.61

(All the above data are also recorded graphically on the corresponding Geotechnical Data Sheets).

An unconfined compression test was performed on a sample representing the stiffer, upper lean clay stratum (B.H.#7 - Sample #2, depth 3 ft.). The specimen failed under 0.834 TSF compressive stress at a strain of 8.6%. The unit weight of the sample was 123 pcf, average moisture content 23.9%, giving a void ratio of 0.72 if we assume the specific gravity of solids equal to 2.72.

Undrained, quick triaxial compression tests were performed on several samples. Only two resulted in acceptable values, namely, two specimens taken from B.H. #5 - Sample #1. The details of the tests are shown on Enclosure #9.

Attention is called to the fact that no specimens of any regular shape could have been obtained from the oversaturated silt. This material has a consistency similar to partly melted soft ice cream,and the performance of compression tests was literally impossible by ordinary laboratory methods. Therefore, we have to accept the in situ field vane tests as the only indicators of the shear strength of the stratum.

IV. SUBSURFACE CONDITIONS

The two banks of the creek will be treated separately because of the different topography.

The Northern Shore

(i) A brown, firm, lean clay with sand and silt layers and pockets extending to a depth of an average 5 ft. was encountered first. (The upper 8 to 12 ins. thick part is the topsoil proper, containing roots, etc.). The shear strength as measured by the unconfined compression test (= 834 psf) fits well into the shear strength profile as determined by two vane tests. (See Encl. #6). Several very high results were obtained by the latter method and they can probably be explained by the presence of sand pockets. The stratum is preconsolidated and the consistency varies between the following limits:

LL:	38.5 to 31 %
PI:	19.1 to 13.1 %
W:	28.8 to 25.8 %
LI:	0.34 to 0.63

The appropriate group symbols: CL & CI.

The unit weight is around 123 pcf and the void ratio: 0.72. The modulus of compressibility, based on test results obtained on similar materials, can be taken as 40 TSF.

(ii) Grey silt, with layers of lean clay and fine sand. The stratum was found between five and sixteen ft. depths. The silt is the main material, and the grain size analysis performed on B.H. #4, SA #3A is representative. 90% of the particles passes the #200 sieve but is larger than 2 microns. It is non-plastic. The clay is represented by the grain size distribution curve of B.H. #4, SA #4B in which 30% of the particles is smaller than 2 microns. It exhibits low plasticity and the thickness of layers varies between 1/8 and 3 ins. The sand appeared either in the form of thin beds between the clay and silt or as a layer of several inches thickness. It is emphasized, however, that the layered structure does not prevail throughout the entire depth of the stratum, but appears irregularly. The silt is the principal material.

The plasticity of the strata varies between the following limits:

LL:	28.8 to 19 %
PI:	14.8 to 4.4 %
W:	40.6 to 20.7 %
LI:	2.09 to 1.09 %

The appropriate group symbols: CL-ML,
CL and CI.

The unit weight of the material is around 112 pcf. (it was measured on a small, geometrical-shaped specimen) and taking 30% as average moisture content and 2.70 as specific gravity, a void ratio of 0.95 is obtained.

The evaluation of shear strength is a more difficult problem. We already mentioned before that no triaxial compression tests could have been performed because of the nature of the material. (See III - Laboratory Tests). Therefore, the field test data are the only measures of shear strength. The dynamic penetration tests are not too reliable; on the other hand, the vane shear tests are extremely useful in deposits such as this. (Ref: 9).

The field vane tests show a shear strength increasing with depth. Whether this strength is derived from cohesion or from friction cannot be clearly seen. It is known, however, that the soil does possess a strength - otherwise what is the cause of the resistance against turning the vane blades? It is also possible that the soil drained partially while the vane test was carried out and this naturally caused an increase in shear strength. But if the soil drains during the relatively quick vane test - it is only logical to suppose that it will also drain during the application of surcharge which takes place at a much slower rate. Consequently, an increase in shear strength can be expected under the influence of applied loads also.

Assuming that the c/p ratio for this normally consolidated stratum is 0.2, the shear strength profile can be readily plotted (see Enclosure #6). The average shear strength is 350 psf. The very high shear strength as indicated by one vane test at a greater depth is probably due to the presence of a sand pocket.

One more question should be cleared before the discussion of the properties of this stratum is closed; is there any contradiction between the high liquidity index and the shear strength? According to one Author (Ref: 9) the shear strength of clays whose water content is at the liquid limit, is 150 to 290 psf. Would it not be reasonable to expect that a material having a liquidity index greater than one, will have even less shear strength ? The answer is "No". According to published data (Ref. 10 and 12) clays having a natural water content greater than the liquid limit may have undrained shear strength much higher than the above values. (N.B. This fact was also confirmed by laboratory tests which were performed by this company on materials obtained from similar deposits found in Northern Ontario). Although the present material is not a clay proper, we still believe that the above negative statement is applicable in the present case.

The modulus of compressibility based on results obtained on similar materials is around 20 TSF.

(iii) Compact, grey, fine sand, between a depth of 16 and 36 ft. The relative density of the material (N av. = 8) including the effect of the overburden pressure (see Ref: 2) is around 50%. Two grain size distribution curves indicate the composition of the material (B.H. #2 - SA. #3 and B.H. #4 - SA #4 - see Encls. #7 & #8). The modulus of compressibility of this material can be obtained by the following empirical statistical relationship (Ref:

$$K = C_1 + C_2 \cdot N$$

Where $C_1 = 71$ $C_2 = 4.9$ and N is 8.

Hence K = 71 + 4.9 x 8 = 110.2 - say 110 TSF.

(iv) A thin layer of soft clay covers the bedrock surface. It is of interest to note that red clay layers were found in the predominantly grey stuff. The properties of the material are similar to those represented by the test results of B.H. #4 - SA #3B.

(v) The bedrock surface is around elevation 268 ft. It is mostly limestone of varying colours. (See the corresponding geotechnical data sheets).

The Southern Shore

A retaining wall securing the stability of the slope and gravel-sand fill encountered in Borehole #2 suggest that the natural topography has been altered. Otherwise, the stratigraphy on this side is not quite dissimilar from that on the northern one.

The uppermost stiffer clay stratum is substituted here by a less plastic, firm silt, which contains soft clay layers. The standard penetration resistances indicate the higher density of the deposit. The brownish colour changes to greyish at around 4 ft. and at greater depths, its properties are very much the same as those of the "grey silt with clay and sand layers" on the northern shore. The upper strata consist of sand-gravel fill along the creek, resting on the grey, slightly plastic silt deposit. The fine sand was encountered in both boreholes.

The bedrock surface is higher here and a grey, soft, silty clay (the material is about the same as represented by B.H. #4 - SA 3B) layer was found above it in Borehole #1.

The stratigraphy can be the best visualized with the aid of a subsurface profile presented on Enclosure #1.

V. WATER CONDITIONS

At the time of field work, the elevation of the water level in the creek was 301.6 ft, and due to the nature of the season, the water table is close to its highest position. The same applies to ground water conditions.

In Borehole #1, it is about 2'5" higher than in the creek. The water follows the surface contours and exerts seepage pressures on the soil grains. In Borehole #2, the water level corresponds to that in the creek.

On the northern shore, the subsurface water conditions are more complicated. As long as the boreholes did not penetrate the relatively impervious clay and silt with clay stratum, the phreatic level was around elevation 302.8 (i.e. 1'8" below surface). Judging from the fact that the colour changed at a depth of about 5 ft. and that on May 9th when the seventh hole was drilled, the water table was standing around 3 ft. below ground level, the latter is accepted as the true one. (It is very likely that the water will subside another 1 to 2 ft. after a longer, hot dry period).

As the boreholes advanced deeper and their bottom reached the sand stratum, a rise in water level was observable. Artesian pressure with a head of 3' 10" above ground level (i.e. at elevation 308.4 ft.) prevailed in the granular material. Upward flow (with a head of 4" above ground level) was experienced even from the hole made by the dynamic cone penetration test #3.

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Thus, summarizing the observations, water under artesian pressure was encountered in the sand layer. The phreatic level, however, is around 3 ft. below surface on the northern shore, because the water loses energy by creeping upwards through the impervious strata. On the northern shore, the water table follows more or less the contour lines.

VI. DISCUSSION AND RECOMMENDATIONS

(a) Foundations of the Bridge

In view of the fact that the subsoil has a low bearing capacity and that the bedrock at the site is within an economical depth, bearing piles will be the most suitable to support the piers and abutments of the proposed bridge.

Timber, reinforced concrete or steel piles may be used. If the cut-off elevation is below the permanent ground water table (i.e. about 5 ft. below ground level), timber piles may be untreated. All types of piles should be driven to refusal at the bedrock surface and the anticipated bearing capacity of timber piles will be 18 to 28 tons - that of concrete piles 20 to 50 tons - depending on size. The safe bearing value of steel piles is 6,000 psi of point area, that is, it also depends on the size.

No measurable settlements are anticipated.

Three types of superstructure should be given consideration. These are:

(i) Three span, open type structure in which the end of the embankment finishes off in a slope and no retaining wall type abutment is required. In this case, a protective retaining wall should be built along the creek and the ground surface should be trimmed sloping upwards, not exceeding a slope

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of 6 horizontal to 1 vertical, starting from the top of retaining walls. (This measure deems to be necessary to prevent slip failure of the subgrade which prevails owing to the presence of soft clay layers and seepage forces resulting from ground water movements).

The approach embankment can be made of granular material and the slope will depend on the properties of this fill. The rise should start at the top of retaining wall and it should be protected by rip-rap against erosion.

(ii) One span structure, with retaining wall type abutments. This will resist any potential movement of the embankment, and considering the subsurface conditions, preference should be given to this arrangement. Adequate drainage of the soil behind the abutment must be provided for.

(iii) Three span structure with piers and retaining wall type abutments. This arrangement is a combination of (i)and (ii). The retaining wall at the creek should still be built and the ground level should slope away from it with a batter not exceeding 6 to 1.

In any event, battered end bearing piles should completely compensate the horizontal component of earth pressures.

The best method for dewatering the pits for the duration of construction of pile caps would be by enclosing them with sheet piles driven to a depth below the bottom of the excavation one and a half times the distance between outside water level and bottom of excavation. This precaution will ascertain the stability of subgrade against hydraulic uplift. Water can then be removed by pumping.

(The successful use of wellpoints is doubtful because of the presence of the clay layers in the substrata).

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(b) The Stability of Subsoil under the Approach Fill

Access to the proposed bridge will be secured by an approach embankment whose maximum height will be around 10 ft. as indicated on the Working Plan. In the following paragraphs, consideration is given to the stability of the subsoil(on the northem shore in particular)under this fill. The simplifications and assumptions on which the further calculations are based are outlined herein:

(i) The subsoil is a homogeneous material, which derives its undrained shear strength from cohesion only.

(ii) The shear strength is independent from depth and the higher strength of the upper crust is not taken into account. A uniform undrained shear strength of 350 psf is assumed to prevail throughout the entire stratum to a depth of 16 ft. i.e. to the top of sand layer.

(iii) The loading is infinite in the direction of the longitudinal axis of the embankment and the height of the fill is taken as constant. Thus, the treatment of the problem as two dimensional is possible.

(iv) The live load on the embankment is taken into account by assuming that this is equivalent to one and a half foot additional height of embankment through its entire top width (=180 lbs/sq.ft. loading).

(v) The ground level is horizontal.

(vi) The slip surface terminates at the ground level - i.e. the own strength of the embankment is neglected.

(vii) The sand layer beginning at 16 ft. depth is the "firm stratum" which is tangential to the slip circles.

Jakobson's paper (see Ref: 7) was used as a guide in the analysis. For notations and details, see Enclosure #10.

The safety factor for slip circles not extending to firm stratum is more than 1.41. (The allowable surcharge is:

$$\frac{C}{0.18} = \frac{350}{0.18} = 1950 \text{ psf}$$

in this case, and not considering the trapezoidal load distribution, the maximum weight of the fill is 1380 psf).

Several slip circles tangential to the sand stratum were analyzed with the aid of Jakobson's diagrams. The locus of the centres together with the factors of safety is shown on Enclosure #10. The minimum value is 1.45; therefore, the subsoil is capable of supporting the ten ft. high fill plus live load.

(Note: a similar favourable safety factor is obtained if the load bearing capacity of the subsoil with the Meyerhof formula (Ref: 6) is analyzed. This gives a uniformly distributed maximum load of 1800 psf, hence, the safety factor is 1.3 which of course is also a necessarily low value).

The real safety factor is probably much higher because of the greater shearing resistance of the upper crust and the trapezoidal load distribution. Furthermore, the shear strength of the silt was deliberately chosen on the low side.

(c) The Settlement of the Embankment.

An attempt is made to predict the probable settling of the fill, assuming a 10 ft. high fill with 180 psf additional surcharge. The stress distribution can be taken as uniform throughout the entire depth and the conditions on the north shore are scrutinized.

The moduli of compressibility (K) are chosen on the basis of published statistical relationships or from test results performed on similar materials (See IV - Subsoil Conditions).

Load	Layer	Thickness	K TSF	Deformation
1380 psf = 0.69 TSF	Crust	5 ft.	40 TSF	0.086 ft.
= Constant Vertical Stress	Grey silt	11 ft.	20 TSF	0.38
	Grey sand	20 ft.	110 TSF	0.126
				0.592 = approx. 7"

The above 7 in. settlement, of course, is only a guiding value and its exactness is not claimed. However, it is believed the order of expectable settling can be obtained by the above method. (The 7 ins. considers only the subsoil deformation. The settlement of embankment surface owing to densification of fill material depends mainly on the effectiveness of compaction).

The longer the time between the end of construction of the embankment and pouring of pavement, the less additional settlement will be experienced, because the consolidation of the subgrade will take a longer period to complete. This is principally caused by the practically impermeable upper crust.

Settlements will be somewhat less on the south shore because of the different subsoil conditions.

DOMINION SOIL INVESTIGATION LIMITED

L R Szalatkay

L. R. Szalatkay, P.Eng.,
Senior Soils Engineer.

LRS/oed

Encls.

VII. REFERENCES

(1) Procedures for Testing Soils - ASTM, April 1958, pp 186 to 198. (Unified Soil Classification System - by A. A. Wagner).

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(3) The Physiography of Southern Ontario by L.J.Chapman and D. F. Putnam of the Ontario Research Foundation - University of Toronto Press - 1951.

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(8) Vane Tests in the Alluvial Plain of the River Forth Near Grangemouth - Geotechnique, Vol. I - 1948 - page 111 & seq. by A. W. Skempton.

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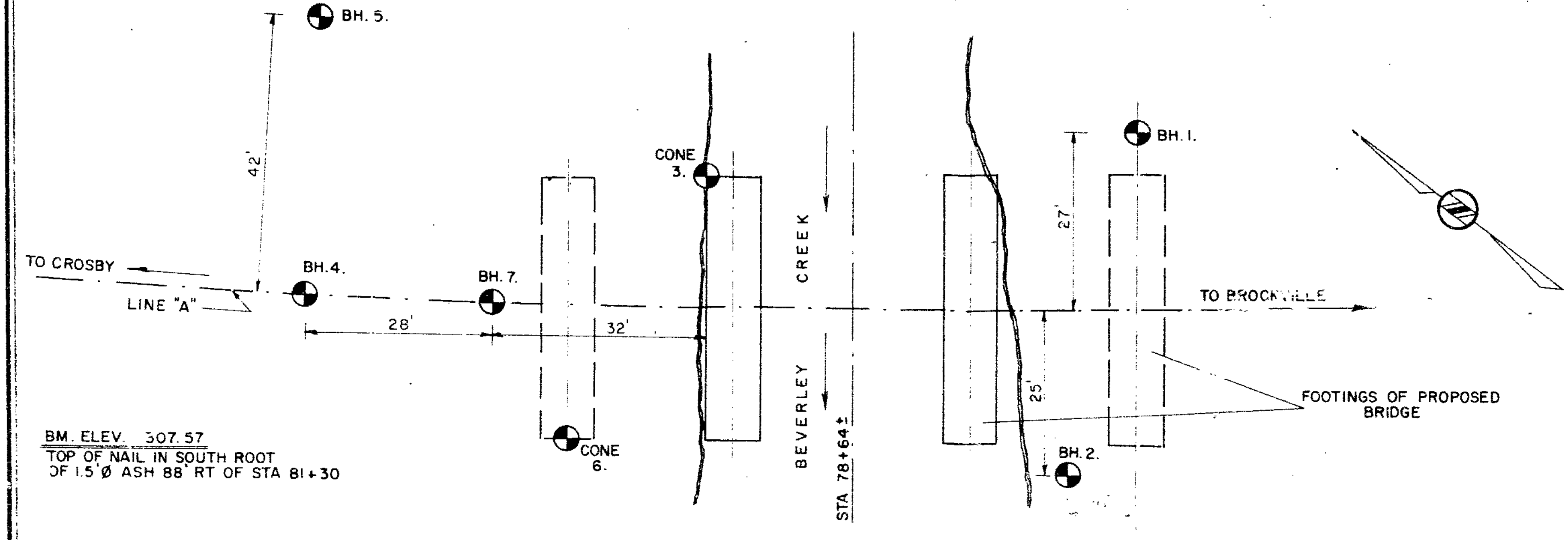
(10) Geotechnical Properties of Leda Clay in the Ottawa Area - by Eden & Crawford, Proceedings of the 4th International Conference - 1957, London,England.

(11) Standard Penetration Tests and Compressibility of Soils by E. Schultze and E. Menzenboch, Proceedings of the 5th International Conference - 1961 - Volume I, page 527.

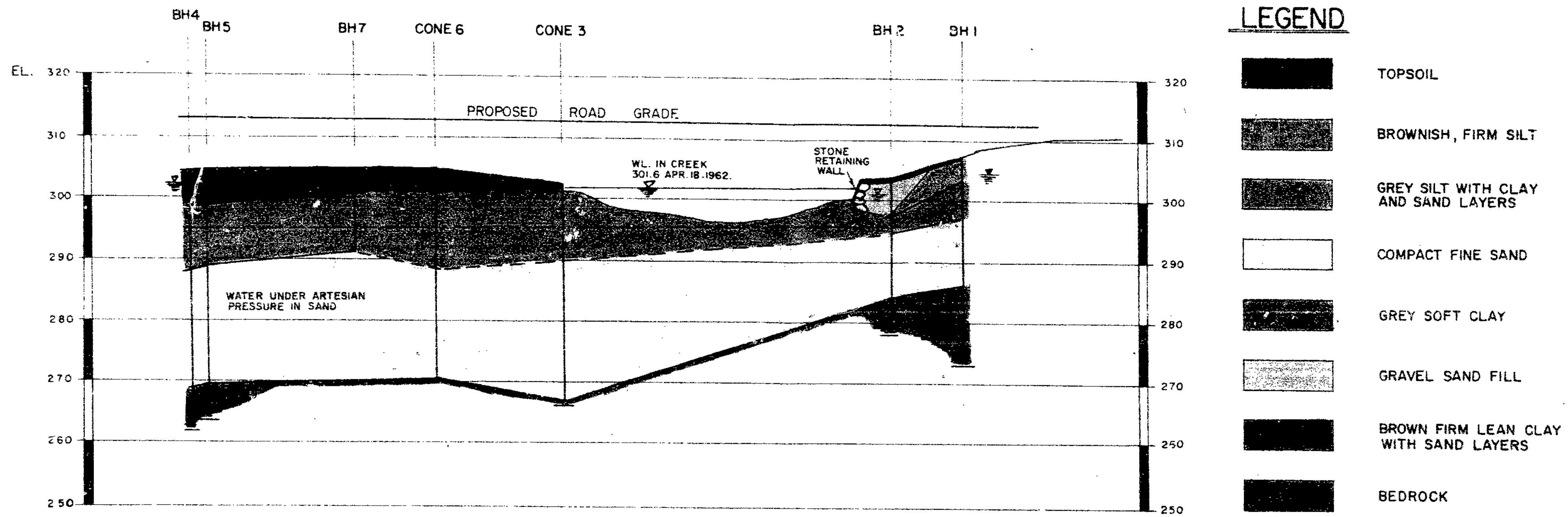
(12) An Investigation of Slide in a Test Trench Excavated in Fissured Sensitive Marine Clay - by Bazett Adams and Matyas - Proceedings of the 5th International Conference - 1961 - Volume I, page 431.

E n c l o s u r e s



LOCATION OF BOREHOLES

SCALE : 1" TO 20'



SUBSURFACE PROFILE

SCALE : 1" TO 20'

GEOTECHNICAL DATA SHEET FOR BOREHOLE . .1. . . .

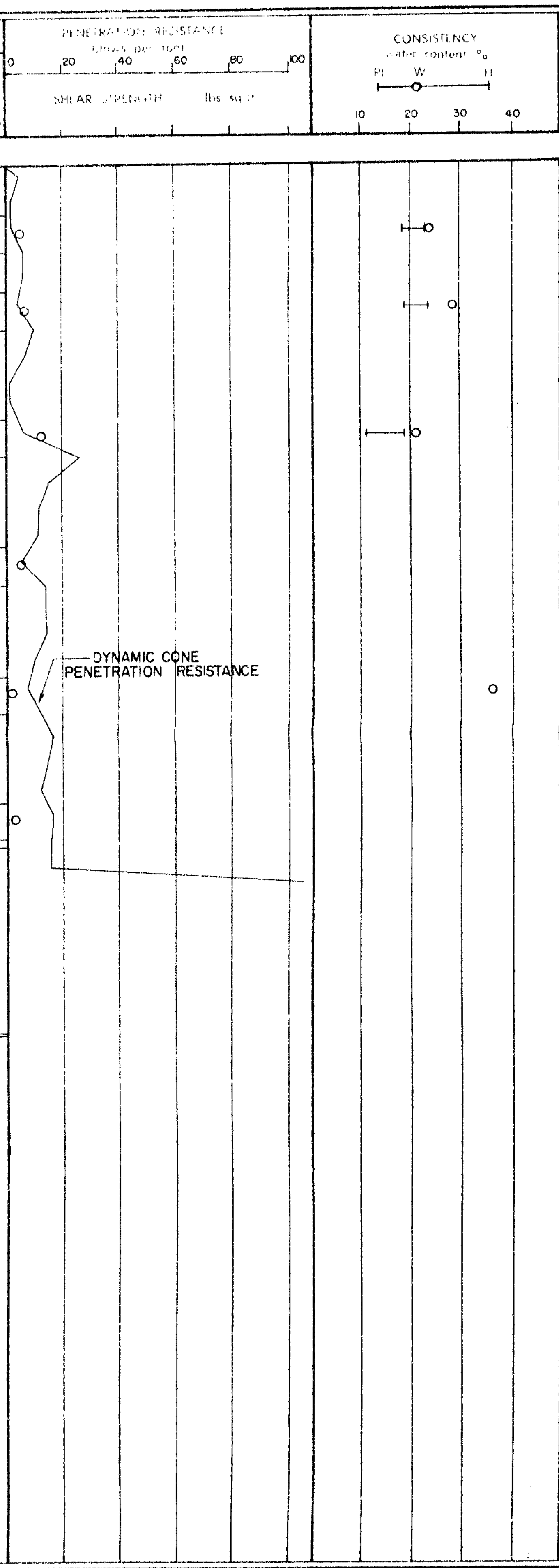
OUR REFERENCE NO. 2-4-10

CLIENT ONTARIO DEPARTMENT OF HIGHWAYS
PROJECT BRIDGE OVER BEVERLEY CREEK
LOCATION REVISED HIGHWAY No. 42. DELTA, ONT.
DATUM ELEVATION 307.5

METHOD OF BORING WASHBORING
DIAMETER OF BOREHOLE $2\frac{7}{8}$"
DATE APRIL 17. 1962.

ENCLOSURE NO. 2

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES N (or % recovery of sample)
307.5	0	TOPSOIL			
305.0		traces of organics; brownish / greyish; FIRM SLIGHTLY PLASTIC SILT WITH SOFT, SILTY CLAY LAYERS	1	SS	6
	5		2	SS	7
300.0					
	10		3	SS	13
295.0		COMPACT GREY FINE SAND			
	15		4	SS	6
290.0					
	20		5	SS	?
285.0		GREY, SOFT SILTY CLAY			
	25		6	SS	3
280.0		WHITE CRYSTALLINE LIMESTONE WITH BLACK SPECKLES	7	AXT CORE 75 % REC.	STEADY PRESSURE NO LOSS OF WATER
	30		8	AXT CORE 100% REC.	
275.0					
	35				



REMARKS

NOTATIONS:

- SS denotes disturbed sample taken with a split spoon sampler
- o denotes standard penetration resistance
- ▽ denotes water level
- denotes the depth where the borehole caved in upon withdrawal of casing
- +ST denotes shear strength and sensitivity as measured by field vane test
- ▲ denotes shear strength as obtained in undrained quick triaxial compression test
- ● denotes shear strength as obtained from unconfined compression test

VERTICAL SCALE: 1 IN. TO 5 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: V. H. CH'D: L.S.

GEOTECHNICAL DATA SHEET FOR BOREHOLE . 2 . . .

OUR REFERENCE NO 2-4-10

CLIENT: ONTARIO DEPARTMENT OF HIGHWAYS
PROJECT BRIDGE OVER BEVERLEY CREEK
LOCATION REVISED HIGHWAY No. 42. DELTA, ONT.
DATUM ELEVATION 303.9

METHOD OF BORING WASHBORING
DIAMETER OF BOREHOLE $2\ {}^{7}/_{8}$"
DATE APRIL 17. 1962.

ENCLOSURE NO. 3

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES N or Advancement of Sampler	PENETRATION RESISTANCE blows per foot (0 20 40 60 80 100) / SHEAR STRENGTH lbs/sq ft	CONSISTENCY water content % (PL W LL)	REMARKS
303.9	0	TOPSOIL						FOR NOTATIONS SEE ENCL. 2.
300.0	5	DARK BROWN SILTY GRAVELLY SAND WITH ORGANIC REMAINS	1	SS	28			
			2	SS	2			
295.0	10	GREY, SOFT SLIGHTLY PLASTIC SILT						
290.0	15	COMPACT GREY FINE SAND	3	SS	9			FOR GRAIN SIZE DISTRIBUTION OF SA 3 SEE ENCLOSURE 7.
285.0			4	SS	7			
280.0	20 / 25	BUFF, HARD CALCAREOUS SANDSTONE	5	AXT CORE 97% REC.	STEADY PRESS. NO LOSS OF WATER			

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: V.H. CH'D: L.S.

GEOTECHNICAL DATA SHEET FOR BOREHOLE 4 & CONE 6

OUR REFERENCE NO. 2-4-10

CLIENT: ONTARIO DEPARTMENT OF HIGHWAYS
PROJECT: BRIDGE OVER BEVERLEY CREEK
LOCATION: REVISED HIGHWAY No. 42. DELTA, ONT.
DATUM ELEVATION: CONE AND BOREHOLE: 304.4

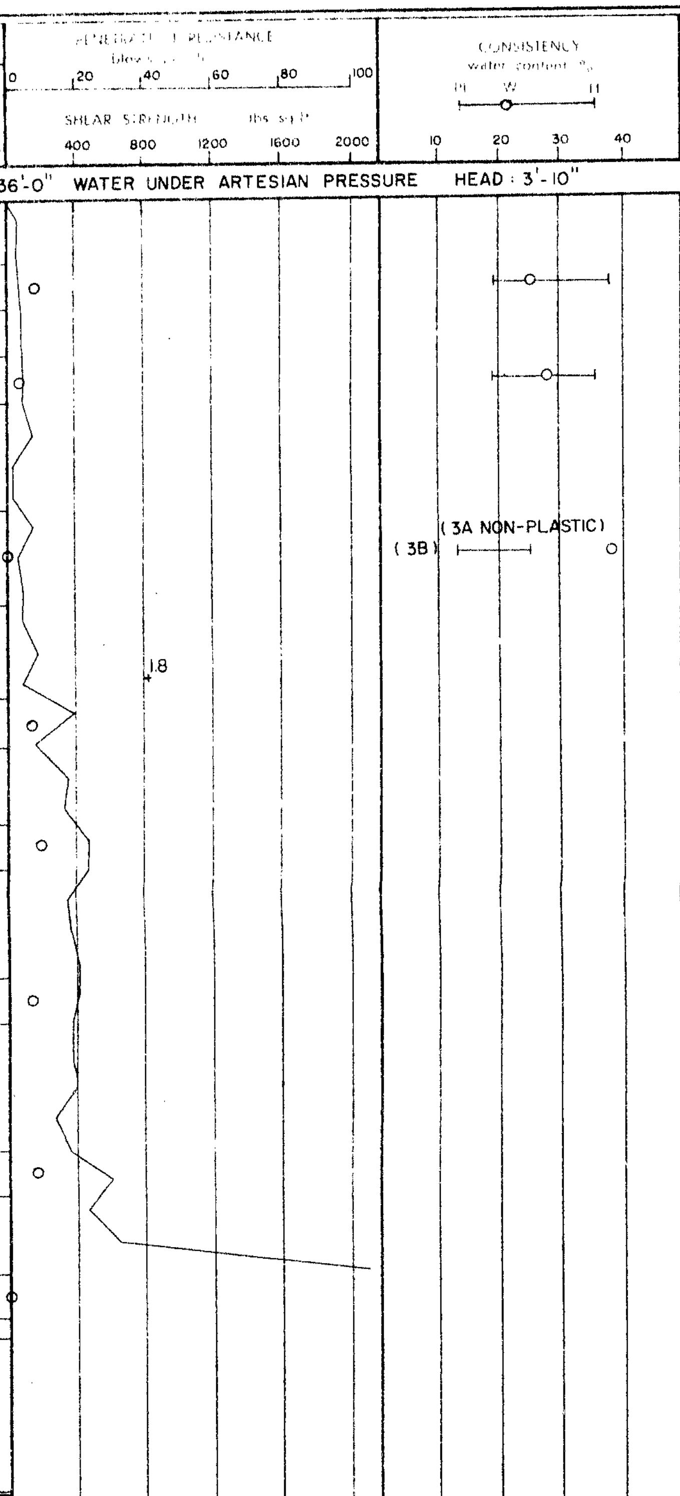
METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: APRIL 18 & 19. 1962.

ENCLOSURE NO. 4

DEPTH OF HOLE 36'-0" WATER UNDER ARTESIAN PRESSURE HEAD: 3'-10"

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	SAMPLE NUMBER	TYPE	No. of blows	REMARKS
304.4	0	TOPSOIL	AFTER THE HOLE CAVED IN			FOR NOTATIONS SEE ENCL. 2.
		BROWN FIRM LEAN CLAY WITH SAND AND SILT LAYERS	1	SS	8	
300.0	5		2	SS	3	
295.0	10	GREY SILT WITH LAYERS OF LEAN CLAY AND FINE SAND	3	SS	0	SA No. 3. WAS DIVIDED INTO TWO PARTS: SILT 3A AND LEAN CLAY 3B.
290.0	15					FOR GRAIN SIZE DISTRIBUTION OF SAS 1, 2, 3A, 3B AND 4 SEE ENCLOSURE 7 & 8
		COMPACT GREY FINE SAND	4	SS	7	
285.0	20		5	SS	9	
280.0	25		6	SS	7	
275.0	30	layer of silt	7	SS	8	
270.0	35	increasing silt content	8	SS	0	
		SOFT GREY & RED CLAY				
265.0	40	CORE RECOVERED: HARD GREY SLATE AND WHITE CRYSTALLINE LIMESTONE	9	AXT CORE 25% REC.	STEADY PRESSURE ALL WATER RETURNS	

THE BALANCE OF THE CORE FELL BACK INTO THE BOREHOLE



VERTICAL SCALE: 1 IN TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: V.H. CH'D: L.S.

GEOTECHNICAL DATA SHEET FOR BOREHOLE 5 & CONE 3

OUR REFERENCE NO. 2-4-10

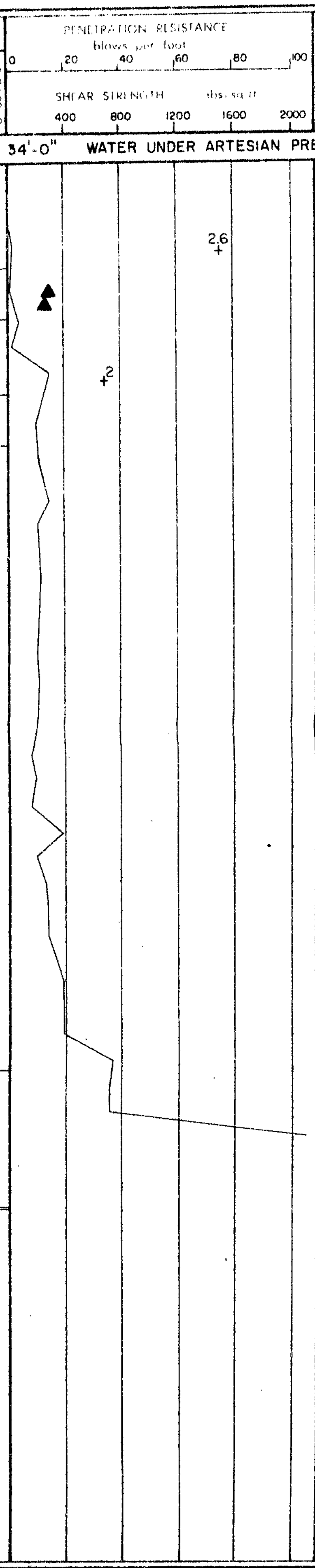
CLIENT: ONTARIO DEPARTMENT OF HIGHWAYS
PROJECT: BRIDGE OVER BEVERLEY CREEK
LOCATION: REVISED HIGHWAY No. 42, DELTA, ONT.
DATUM ELEVATION: BH. 5 : 304.5 CONE 3 : 301.9

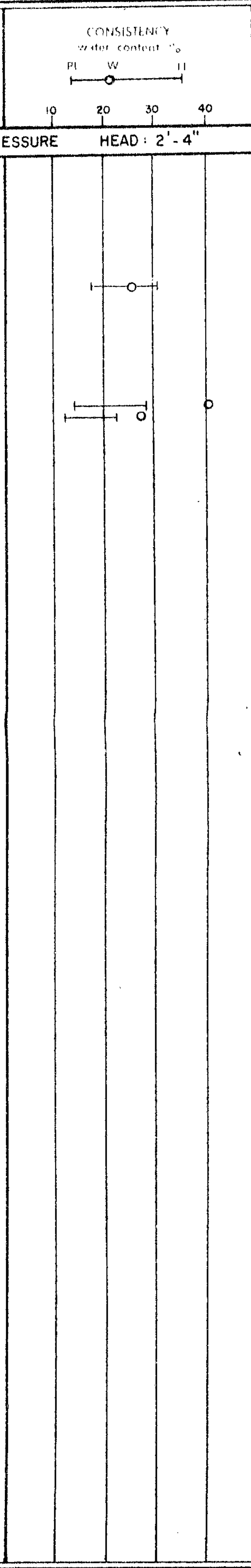
METHOD OF BORING: WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: APRIL 18 & 19, 1962.

ENCLOSURE NO. 5

DEPTH OF HOLE: 34'-0" WATER UNDER ARTESIAN PRESSURE HEAD: 2'-4"

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLES NUMBER	TYPE	-N- or Advancement of Sampler	REMARKS
304.5	0	TOPSOIL				FOR NOTATIONS SEE ENCL. 2 & 4.
		BROWN FIRM LEAN CLAY WITH SAND AND SILT LAYERS	DEPTH OF HOLE 11'-0"			TW denotes undisturbed sample
300.0	5		1	TW	PUSH	
295.0	10	GREY SILT WITH LAYERS OF LEAN CLAY AND FINE SAND	2	TW	PUSH	
290.0	15					
285.0	20					NO SAMPLING WAS DONE BETWEEN 11'-0" AND 35'-4" DEPTH. CHANGES NOTED BY OBSERVING WASHWATER
280.0	25	COMPACT GREY FINE SAND				
275.0	30					
270.0	35	GREYISH LIMESTONE				
		ROSY LIMESTONE				
		WHITE LIMESTONE				
		ROSY LIMESTONE	3	AXT CORE 97% REC.	STEADY PRESSURE ALL WATER RETURNS	
265.0	40	YELLOW LIMESTONE				





VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: V.H. CH'D: L.S.

OUR REFERENCE NO. 2-4-10

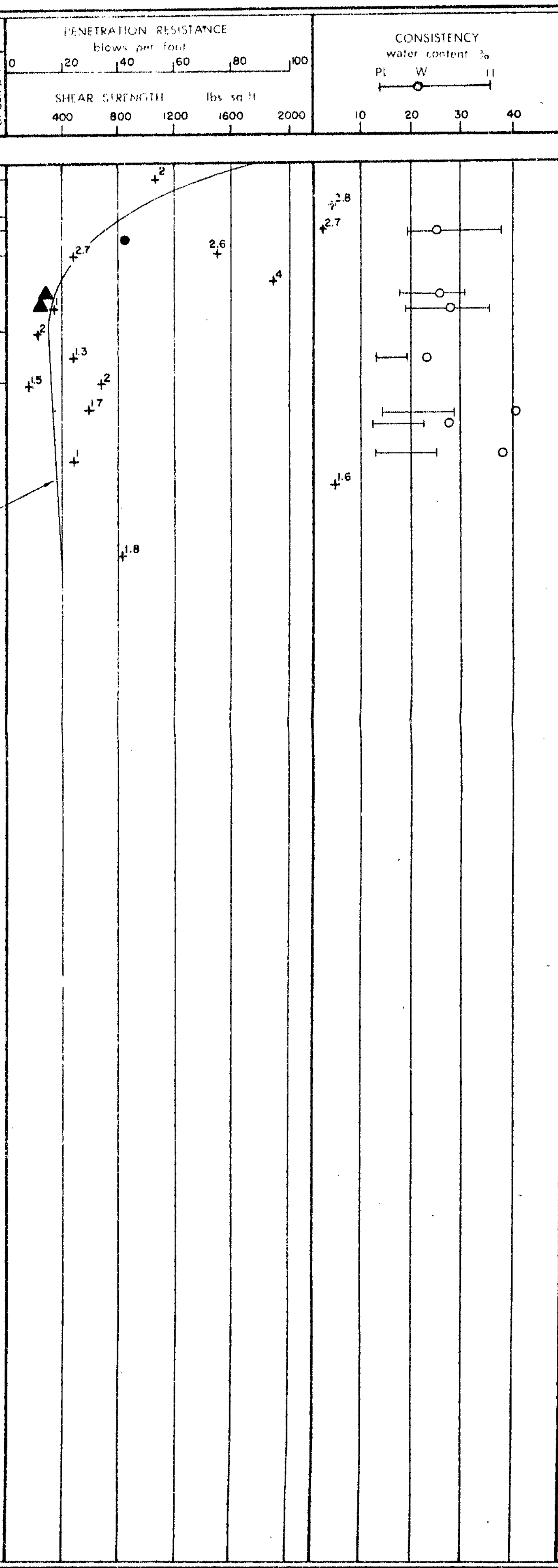
GEOTECHNICAL DATA SHEET FOR BOREHOLE 7 & SUMMARY
(NORTH SHORE ONLY)

CLIENT: ONTARIO DEPARTMENT OF HIGHWAYS
PROJECT: BRIDGE OVER BEVERLEY CREEK
LOCATION: REVISED HIGHWAY No. 42. DELTA, ONT.
DATUM ELEVATION: 304.4

METHOD OF BORING: AUGERING
DIAMETER OF BOREHOLE: 2 1/2"
DATE: MAY 9. 1962.

ENCLOSURE NO. 6

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	REMARKS
304.4	0	TOPSOIL			FOR NOTATIONS SEE ENCL. 2.
		BROWN FIRM LEAN CLAY WITH SAND AND SILT LAYERS density : 123 pcf.	1	TW	
			2	TW	TW denotes undisturbed sample
300.0	5	GREY SILT WITH LAYERS OF LEAN CLAY AND FINE SAND density : 112 pcf.	3	TW	
295.0	10				NO UNDISTURBED SAMPLE WAS OBTAINABLE BELOW 8'-6" DEPTH
290.0	15				
285.0	20	COMPACT GREY FINE SAND $N_{av.} = 8$ water under artesian pressure in this stratum head: el. 308.4			
280.0	25				
275.0	30				
270.0	35	SOFT CLAY			
		BEDROCK			
265.0	40				



VERTICAL SCALE: 1 IN. TO 5 FT

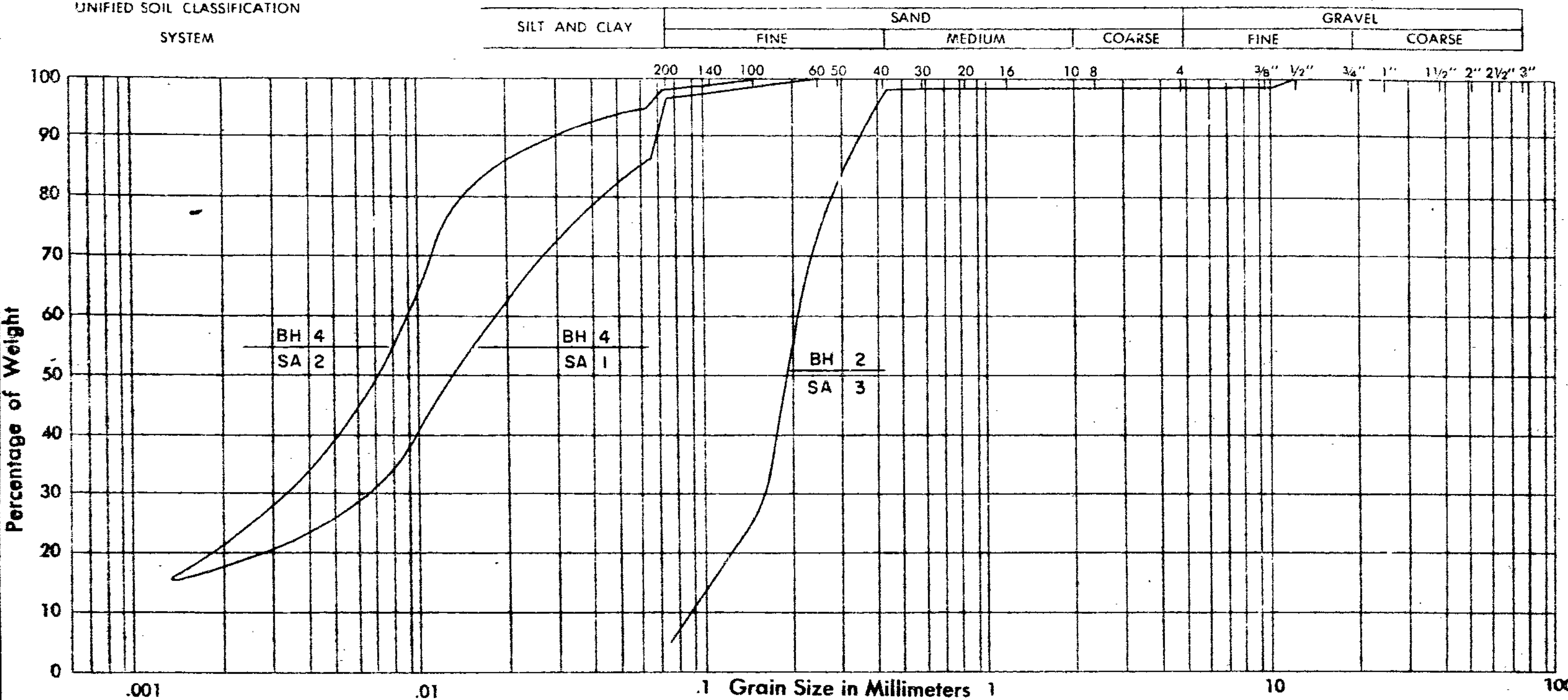
DOMINION SOIL INVESTIGATION LIMITED

MADE: V.H. CH'D: L.S.

DOMINION SOIL INVESTIGATION LIMITED

GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 2-4-10



PROJECT: REV. HWY. 42. & BEVERLEY CR.

LOCATION: DELTA, ONT.

BOREHOLE NO.:	2	4	4
SAMPLE NO.:	3	1	2
DEPTH OF SAMPLE:	11'	3'	6'
ELEVATION OF SAMPLE:	292.9	301.4	298.4

COEFFICIENT OF UNIFORMITY

COEFFICIENT OF CURVATURE

Classification of Sample and Group Symbol:

2/3 :	FINE SAND	SP
4/1 :	SILTY CLAY	CI
4/2 :	SILTY CLAY	CI

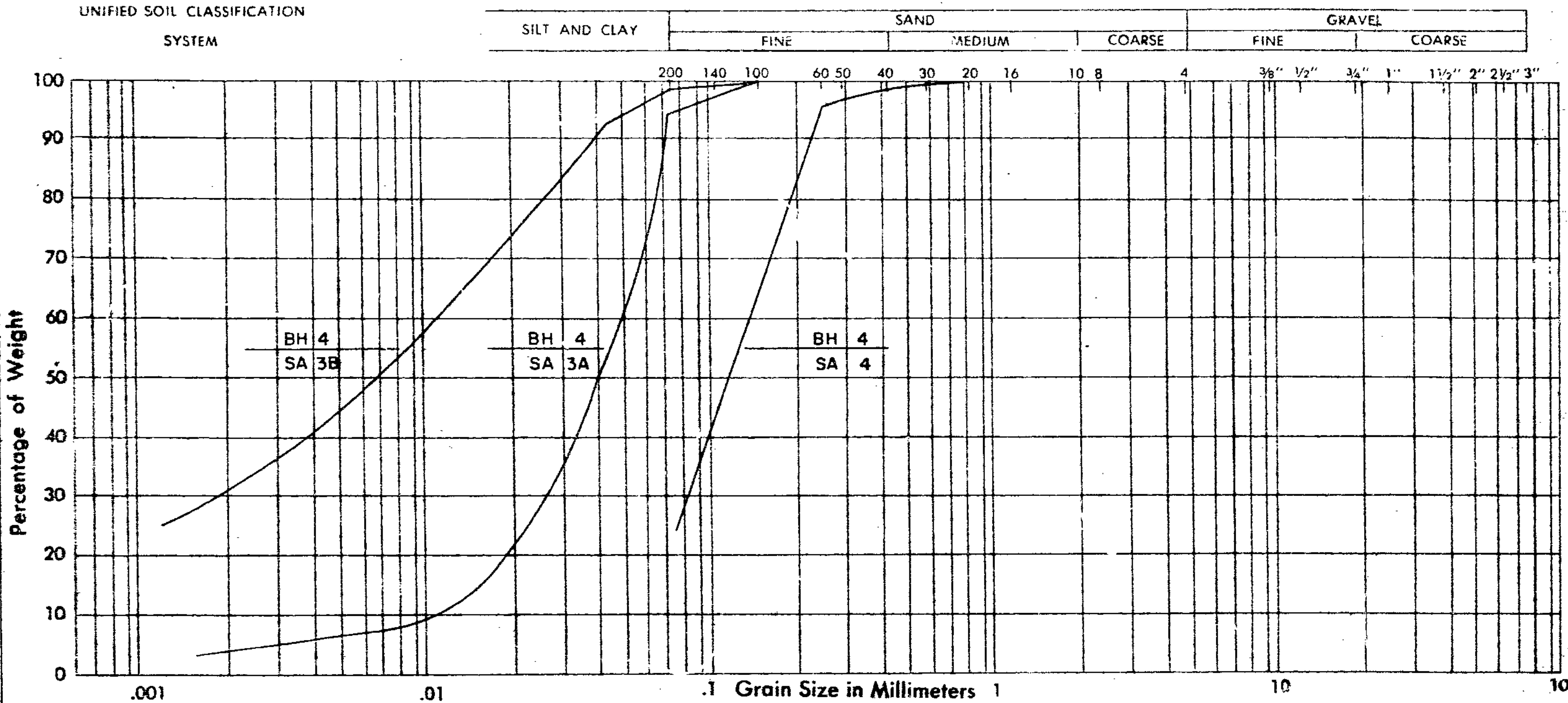
PLASTIC PROPERTIES:		4/1	4/2
LIQUID LIMIT	% =	38.5	36.3
PLASTIC LIMIT	% =	19.4	19.1
PLASTICITY INDEX	% =	19.1	17.2
MOISTURE CONTENT	% =	25.8	28.8
ACTIVITY =		1.06	0.77

Enclosure No. 7.

DOMINION SOIL INVESTIGATION LIMITED

GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 2-4-10



Enclosure No. 9.

PROJECT: REV. HWY. 42 & BEVERLEY CR.

LOCATION: DELTA, ONT.

BOREHOLE NO.:	4	4	4
SAMPLE NO.:	4	3A	3B
DEPTH OF SAMPLE:	17'	12'	12'
ELEVATION OF SAMPLE:	287.4	292.4	292.4

COEFFICIENT OF UNIFORMITY

COEFFICIENT OF CURVATURE

Classification of Sample and Group Symbol:

4/4	SILTY FINE SAND	SP-ML
4/3A	SILT	M
4/3B	LEAN CLAY	CL

PLASTIC PROPERTIES: 4/3B

LIQUID LIMIT	% =	25.6
PLASTIC LIMIT	% =	13.9
PLASTICITY INDEX	% =	11.7
MOISTURE CONTENT	% =	38.1
ACTIVITY =		0.377

DOMINION SOIL INVESTIGATION LIMITED

Our Ref. No. 2-4-10

Prep. By V.H.

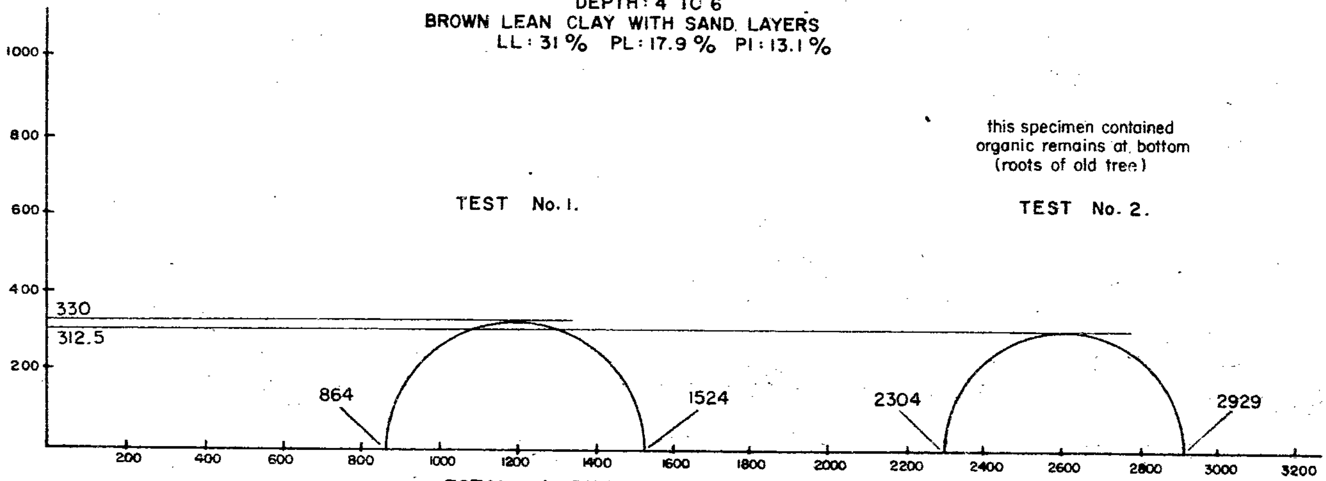
BOREHOLE No. 5. SAMPLE No. 1.

DEPTH: 4' TO 6'

BROWN LEAN CLAY WITH SAND LAYERS

LL: 31 % PL: 17.9 % PI: 13.1 %

SHEAR STRESS PSF



TOTAL NORMAL STRESS — PSF

TEST SPECIMEN DATA:		Test No. 1	Test No. 2
	unit weight:	124 pcf.	110 pcf.
	water content:	25.1 %	31.4 %
	spec. gravity (ass'd):	2.70 gr/ccm	2.40 gr/ccm
	degree of saturation:	96 %	95 %
	void ratio:	0.7	0.79
	strain at failure:	20 %	20 %

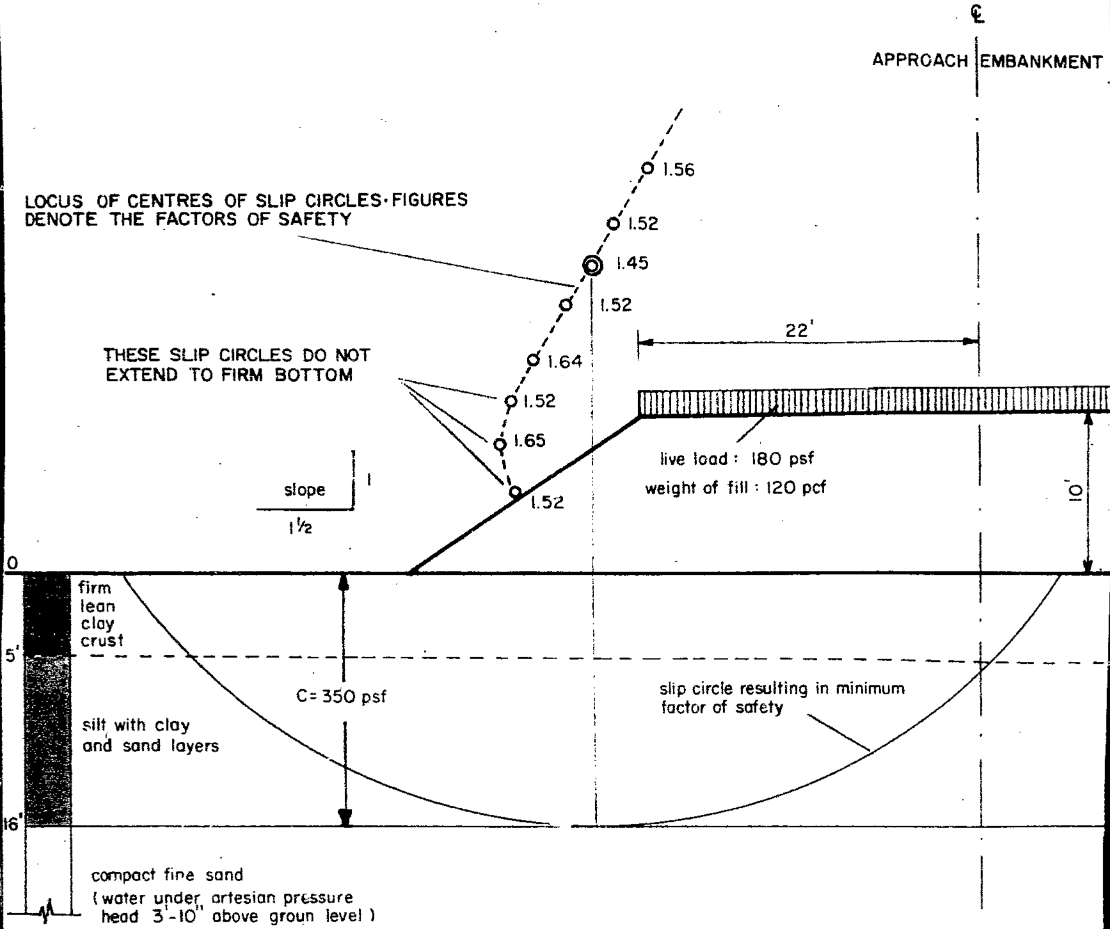
UNDRAINED QUICK TRIAXIAL COMPRESSION TEST RESULTS

Enclosure No. 9.

Our Ref. No. 2-4-10

Enclosure No. 10.

Prep. By V.H.



THE STABILITY OF APPROACH EMBANKMENT

SCALE : 1" TO 10'

Dominion Soil Investigation Limited