[redacted]

W.P. 104-60

Materials and Research Division

November 7, 1962.

Professor D. Townsend,
Civil Engineering Department,
Ellis Hall,
Queen's University,
Kingston, Ontario.

Re: O.J.H.R.P. Project No. 24,
New Liskeard Embankment
W.J. 62-F-99.

Dear Dave:-

W.P. 104-60

Enclosed, please find a photostatic copy of the instrument readings of our co-operative research project at New Liskeard during the placing of the protective cover of fill four feet above the top of the culvert.

The results appear to be consistent so far.

My personal regards.

Sincerely,

KYLo

Kwan Y. Lo.

KYL/MdeF
Attach.

62-F-99

ONTARIO

DEPARTMENT OF HIGHWAYS

New Liskeard, Ontario,
September 17, 1962.

Mr. A. Scermac,
Foundation Section,
Department of Highways,
Parliament Buildings,
Toronto, Ontario.

Re: Installation at O.N.R. Overpass - Tri-Town By-Pass

Dear Tony:

The following is a list of the materials supplied by Canadian Longyear and left at the site.

1. 394 ft. of Std. 3/4" black pipe.
2. 372 ft. of Std. 1½" black pipe.
3. 11 ft. of Std. 4" black pipe.
4. 2 caps to fit 4" pipe.
5. 9 - 2" ship augers.
6. 9 - adapters for augers.
7. 11 - 2 ft. sections of E - rods.
8. 2 - 5 gal. pails of bentonite (4 pails were shipped, 2 were returned).
9. Approx. 5 lbs. of lumite cement.

Also enclosed are the two packing slips for the valves and connectors which I believe should be forwarded to Ken Selby.

Sincerely,

David E. Hilts

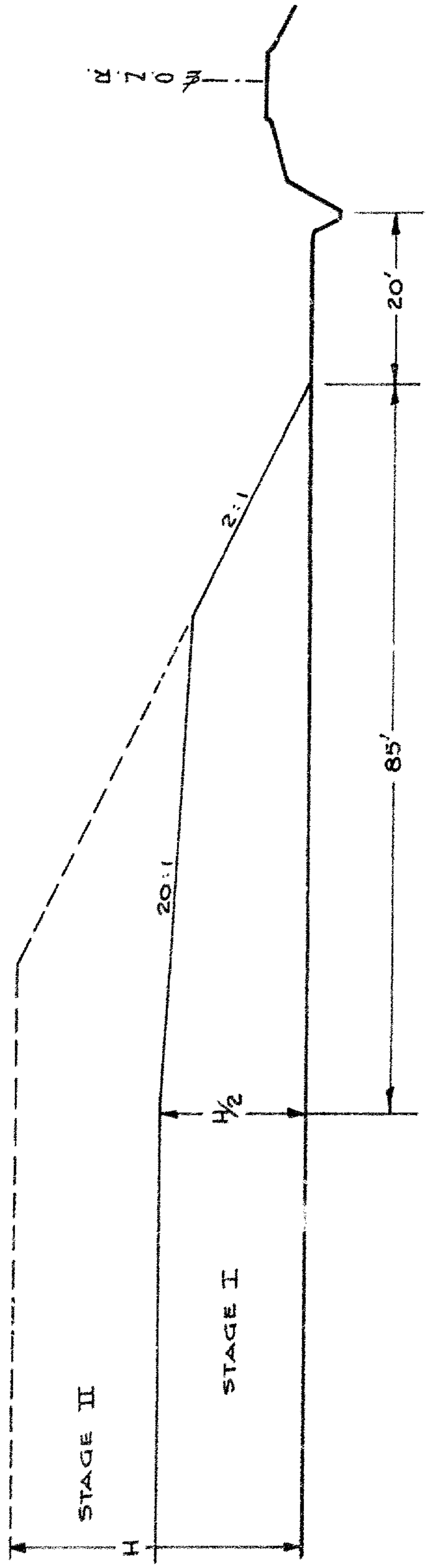
David E. Hilts.

DEH/ld

Attached.

SEP 18 2 PM 1962

SUGGESTED APPROACH EMBANKMENT CONSTRUCTION



NOTE: The time of Stage II construction will be determined by the Foundation Section

ORIGINATED M. Devata	DEPARTMENT OF HIGHWAYS - ONTARIO	SCALE 1 inch = 20 feet
DRAWN H.D. Reed	MATERIALS & RESEARCH SECTION	W. P. NO. 104-60
CHECKED M.D.	O.N.R. CROSSING	JOB NO. 62-F-99
APPROVED [signature]	HIGHWAY Nº 11	
DATE 4 Oct. 1961	TRI-TOWN BY-PASS, DIST. 14	DWG. NO. ~~X~~-29

File 62-F-99

Longyear
CANADIAN LONGYEAR, LIMITED
General Office and Manufacturing Plant
North Bay, Ontario, Canada TELEPHONE 474-2800
P.O. BOX 330 — CABLE LONGYEAR TELEX 027694

October 8, 1964

SPECIAL DELIVERY

Mr. Devada
Senior Foundation Engineer,
Material and Testing Division
Ontario Department of Highways,
Downsview, Ontario

Dear Mr. Devada:

As per your telephone request of today's date, we are attaching one print each of SKB.12373, Driving Sleeve and SKD.12374 - Adapter to 2" Auger. The Augers used were 2" O.D. x 18" long, having "A" Rod Box threads on the top end, and are called "Ship Augers"

We trust the above information is what you require.
Kindly return the prints when they have served their purpose.

Yours very truly,

Canadian Longyear Limited

C.W. Steele

Chief Draftsman

C.W. Steele:RMK
Encs.

WP-104-60-2
Re: Bridge
ONR Overpass at
New Liskeard.

Mr. W. Wigle,
Program Engineer.

Mr. A. Rutka.

December 4, 1967.

Hwy. 11, W.P. 104-60-020, ONR Overpass at New Liskeard

I have checked with the Foundation Section and have been advised that the settlement measurements of the loaded pier are continuing, the latest readings being taken this Fall.

The total settlement is approximately 3" so far, and it seems to be levelling off. We are confident that the bridge can be constructed as planned. However, we can take further measurements if the bridge is not programmed for construction next year.

I understand that the Bridge Office is now considering a subway as an alternate, so it would be wise to get further information from them before this structure is programmed for construction.

AR

A. Rutka,
Materials & Testing Engineer.

AR:pa

cc: Foundation Office (Rm. 110)

W.P. 104-60-20

Re: O.N.R. Overpass at New Liskeard.

Mr. D. R. Davis,
Bridge Engineer,
Bridge Division.

Attention: Mr. G. S. Grabski

Foundations Office,
Materials & Testing Div.,
Room 107, Lab. Bldg.

December 1, 1966

Re: O.N.R. Overpass at New Liskeard
— District #14 —

WP 104-60-020

This is to summarise the discussion held in your office on November 23, 1966, regarding the results of the instrumentation of the test footing at the above mentioned crossing.

After half of the embankment was removed, the test footing was built at station 802+20 and loaded with what is to be the dead load of the structure. The dimensions of the test footing correspond to those of the proposed structure.

Observations of the footing settlements were carried out for a period of eleven months and are still being continued. For this period the footing has settled about 2-3/4 inches, of which approximately 1-1/4 inches was immediate settlement.

During this period of time, the pore pressures at the culvert (some 30 ft. east), have decreased in the order of 2.5 to 3.0 p.s.i.

It is also estimated that there is still some 5.0 p.s.i. of excess pore pressures remaining in the ground, and these are in the process of dissipation.

Simplifying the problem somewhat, it can be assumed that during the dissipation of the remaining 5.0 p.s.i. of excess pore-water pressures, another 3.0 inches of settlements will probably take place. Consequently, the total settlement will amount to close to 6 inches.

The rate of pore pressure dissipation and settlement will decrease as time goes on and the hydraulic gradient becomes smaller. It is estimated that it will take more than ten years for the remaining settlements to take place.

It is our suggestion that the bridge be built as planned - i.e., on piers with spread footings resting 5 ft. below the surface of the remaining 9 ft. of fill (Drawing D-5882-P3). The piers supporting the span over the O.N. Railway should, however, be supported by end-bearing piles driven to refusal on the underlying rock.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

cont'd. /2 ...

Mr. B. E. Davis,
Bridge Engineer.

Attn: Mr. C. S. Grebski

December 1, 1966

Since it can be assumed with a reasonable degree of accuracy that settlements of the piers, when built, will be in the order of 2.5 to 3.0 inches within the first year, and it is predicted that they will further settle approximately the same amount before equilibrium is reached, it is suggested that provisions for lowering the span over the railway be incorporated in the design. This, of course, will call for the building of the entire bridge six inches higher than actually required.

Because of the possibility of differential settlements between individual piers, it is recommended that provisions be made at each pier for jacking and shimming of bridge bearings.

We wish to advise you also, that the above was discussed with Dr. H. Q. Golder on November 29, 1966, and he is in agreement with our interpretation and recommendations.

AGS/MdeF

A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. J. Curtis
E. R. Saint
Foundations Office
Gen. Files

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

files WP 104-60-2

C.N.R. Bypass, Tri-Town Bypass.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Testing Div.

Mr. K. Y. Lo,
Supervising Foundation Engr.,
Foundation Section.

November 24, 1965

W.P. 104-60-020.

Re: O.N.R. Bypass Tri-Town Bypass
Dist. 14 (New Liskeard) W.P. 104-60

This memo is a summary of the design and field work carried out for the above project between the period of August to date.

In a memo dated [illegible], to the Bridge Office, it was pointed out that, based on the results of field observations of pore pressure and settlement, stage construction was not feasible. Various schemes were then considered. A meeting was held on August 24, 1965, at which Dr. H. Q. Golder, Messrs. A. Rutka and A. G. Stermac were present.

In a letter to Mr. A. Rutka, dated October 5, 1965, Dr. Golder summarized the discussions and conclusions arrived at during this and subsequent meetings (August 26 and September 2, 1965).

The scheme suggested by Dr. Golder and discussed in these meetings, was to remove 9 ft. of the existing embankment and build the bridge on spread footings resting on the remaining 9 ft. of fill.

The factor of safety of the clay against shear failure due to the superimposed load of the footing was checked and found to be satisfactory if the footing is not placed more than six feet deep from the top of the remaining fill.

The remaining question, then, is that of settlement, which depends on the residual pore pressures after load removal as well as the superimposed load from the footing. It is also necessary to explain the unusual behaviour of the pore pressures and settlements observed at station 802+50, so as to lend confidence to further design work.

An hypothesis was put forward by the writer, based on some previous experimental work on the stress-strain-pore pressure relationship of sensitive, normally-consolidated clays (Lo, 1961). Beneath the embankment, the soil is loaded to a factor of safety very close to unity. In fact, a failure occurred between stations 804+50 and 808+00 in June 1963. Within this region, therefore, plastic deformation occurs and the shear strain progressively builds

cont'd. /2 .....

up excess hydrostatic pressure. This process proceeds simultaneously with consolidation of the subsoil. The pore pressure, settlement and change in undrained strength will, therefore, depend on the net result of these two independent phenomena. This explains the high pore pressure and almost constant rate of settlement after 2-1/2 years of embankment construction.

In order to define more accurately the strength profile along the centre line on the west side of the railway and to check the above hypothesis, four additional boreholes were put down. Seven piezometers and three settlement augers were installed at station 796+00. It was found that the vane strengths at this location were over 500 p.s.f. as compared to a value of 320 p.s.f. from a previous analysis of the 1963 failure. The pore pressures at the centre line observed, were much lower than those at station 802+50 as shown in the table below.

Depth below original ground surface (ft.)	Excess Pore Pressure (p.s.i.)	
	Station 802+50 Oct. 1965	Station 796+00 Oct. 1965
15	10	2.3
30	10	5.2
45	11	2.9
60	10	1.5

It is obvious that the pore pressures at station 796+00 are much lower than those at corresponding depths at station 802+50, although the embankment is 2 or 3 ft. higher at the latter location. This fact, therefore, is consistent with the hypothesis. Calculations of residual pore pressures after removal of 9 ft. of fill and settlement subsequent to the application of footing load, were carried out.

(a) Calculation of Residual Pore Pressure at Station 802+50:

The residual pore pressure at various depths at the centre of the embankment may be computed by -

cont'd. /3 ......

1. Using the usual assumption that the change in pore pressure is equal to the change in vertical stress, i.e., neglecting the effect of shear stresses.

2. Using the pore pressure-strain theory as suggested, but simplifying the calculation by neglecting the elastic rebound for regions where the shear stress exceeded the shear strength of the soil, say within 40-ft. depth. Below this depth, the pore pressure changes will be assumed to be equal to the change in mean principal stresses.

The results of calculation are shown in Fig. 1, in which the existing excess pore pressures are also plotted. The actual residual pore pressure will most likely lie between curves 1 and 2, but closer to the curve based on method 2.

(b) <u>Calculation of Settlement after Removal of 9-ft. Fill and application of Footing Load:</u>

Calculation was performed for footing size of 36 ft. by 6 ft.[1] at a bearing pressure of 1-1/2 $T/ft.^2$.

The total settlement arises from two sources: (a) the dissipation of the excess residual pore pressure; and (b) dissipation of excess pore pressure resulting from the footing pressure.

Curve 2 was used to establish the starting point on e-log p curves of the appropriate consolidation tests. Calculations show that the total final settlement is of the order of 1-1/2 ft., of which 6 in. is due to the footing load, after correcting for the thickness of the silt layers.[2]

Note 1 -- Full-scale footing was changed to 36 x 7.5 ft.

Note 2 -- A factor of .75 was used, as the ratio of thickness of silt to clay layers is at least 1:4.

cont'd. /4 ...

An alternative approach of estimating the settlement is to assume that 8 ft. of fill was applied initially and the footing then constructed. The consolidation part of the settlement occurred to date, should then be subtracted from the result to arrive at an estimated settlement which will occur.

The total settlement under 8 ft. of fill and footing load was computed to be 43 in.

Using the method of isochrones and taking $C_v = 26.7$ ft.^{2}/yr. and further assuming that the present settlement is mostly confined to the top 45 ft. of the clay deposit, the consolidation settlement which has occurred is 10 in., the degree of consolidation being approximately 30%.

The total settlement which will occur is, therefore, 25 in. after correcting for the thickness of the silt layers.

It will be seen that both methods yield results of the similar order of magnitude. The calculated total settlement is about 1-1/2 to 2 feet as compared to a total settlement of 6 feet if the 19-ft. embankment were left in place.

At the present time, 9 feet of fill is being removed and measurement of residual pore pressure taking place. A more reliable estimate of settlement may be carried out when these results and those of the full-scale loading test are available.

It has also to be noted that the calculation has been carried out for station 802+50 only. At places where the shear strengths of the soil are higher, the residual pore pressures will be much smaller and, therefore, the expected settlement will be much less.

It is suggested that the following items of further work should be carried out:

1. Calculate the residual pore pressures at other points of the subsoil so that the whole pattern may be compared with field measurements.

2. Revise the settlement computation using measured residual pore pressure at station 802+50, and compare with results of full-scale loading test.

3. Study the field pore pressure and settlement records at stations 802+50 and 796+00 to arrive at an appreciation of the variation of settlement with respect to change in loading and subsoil conditions.

cont'd. /5 ......

4. Estimation of settlement at each pier and abutment location for design work.

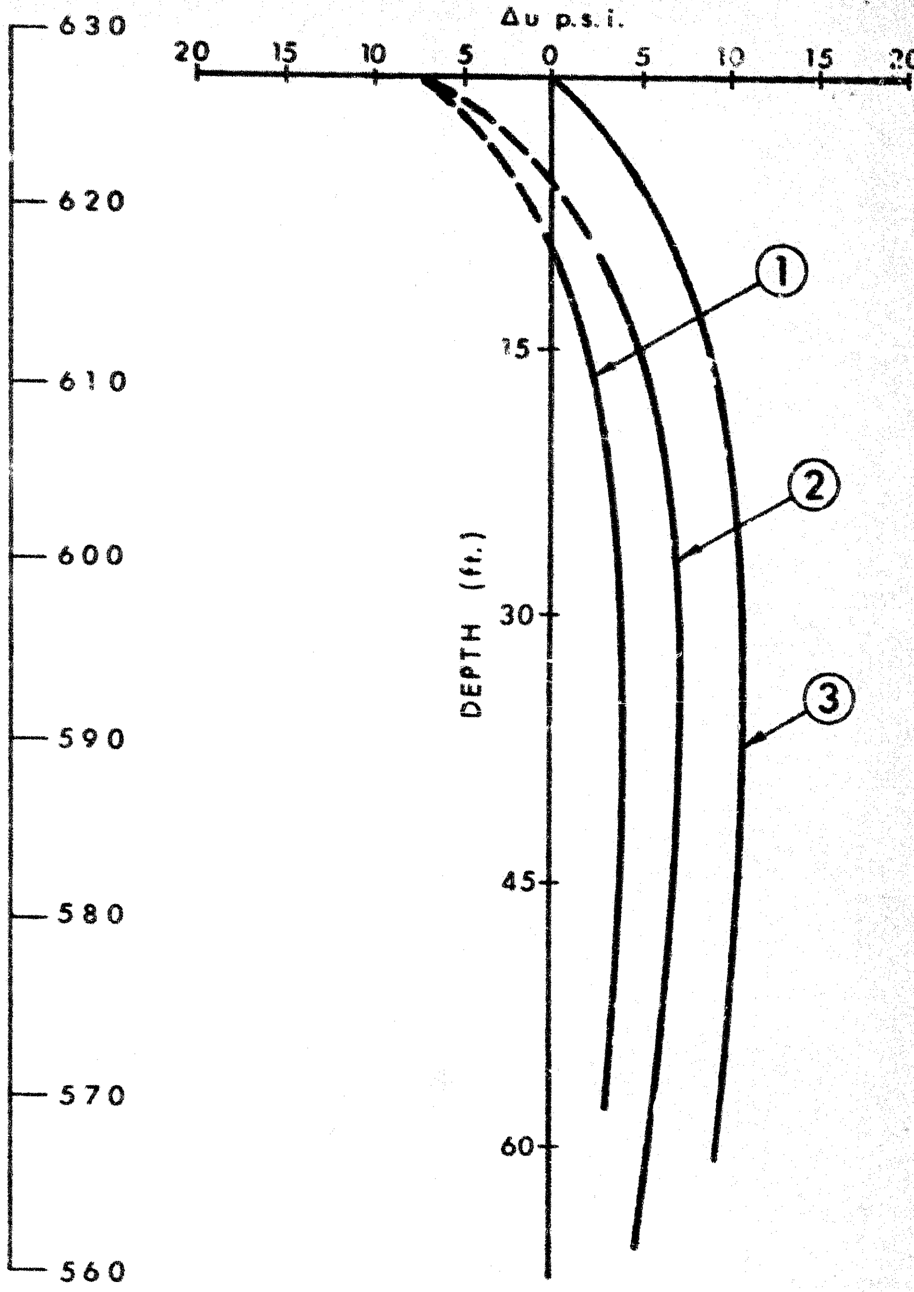
KYL/MdeF
Attach.

K. Y. Lo.

K. Y. Lo,
SUPERVISING FOUNDATION ENGINEER

cc: Dr. H. Q. Golder
Mr. A. Rutka
Mr. S. McCombie
Mr. C. Grebski

Foundations Office
Gen. Files



(1) – Residual p.w.p. assuming $\Delta u = \Delta\sigma_1$

(2) – Residual p.w.p., method 2

(3) – Existing excess p.w.p. Oct. 1965

CC: Gen. Files

Mr. G. A. Wrong,

Principal Soils Engineer.

Materials & Research Section,

(Foundations Office).

October 4, 1961.

Re: C.N.R. Crossing - Hwy. #11,
Tri-Town By-Pass,
District #14.

Attached to this memo, you will find Drawing No. X-29 showing the recommended construction sequence of the approach embankments. Stage No. 1 can be started whenever desirable or convenient, while the time for the start of Stage No. 2 will be determined by the Foundation Section.

For this purpose, settlement plates and piezometers will be installed and readings carried out to determine the amount of settlements and the percentage of consolidation.

Stage construction has been decided upon, basically because of two main reasons:-

1) Large settlements that would have been detrimental for the structure, will already have taken place by the time the structure will be built.

2) The full height of embankment will be built without berms.

AGS/MdeF

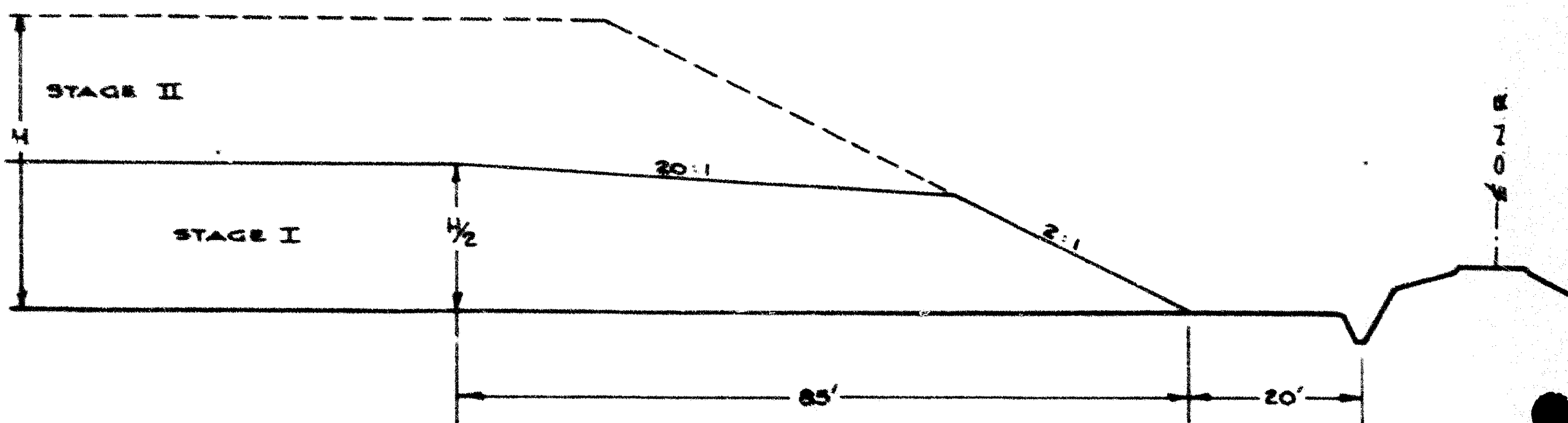
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office
Gen. Files.

Copy also sent to Chester Grebski, Bridge Office

GRADING CONTRACT No 61-247
GRADING W.P. 102-60

SUGGESTED APPROACH EMBANKMENT CONSTRUCTION



NOTE: The time of Stage II construction will be determined by the Foundation Section

ORIGINATED M. Devata
DRAWN H.D. Reed
CHECKED M.D.
APPROVED
DATE 4 Oct. 1961

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION
O.N.R. CROSSING
HIGHWAY No 11
TBI TOWN TWP.-FALCO, DIST. 14

SCALE 1 inch = 20 feet
W.P. NO. 104-60
JOB NO.
DWG. NO. X-29

cc: Gen. Files.

Mr. R. S. Chapman,
District Engineer,
New Liskeard, Ontario.

Attention: Mr. B. Aitken,
Construction Engr.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Materials & Research Division.

September 7, 1962.

Re: Commencement of Filling Operations at
C.N.R. Overhead - W.P. 104-60, District #14.

You are kindly requested to advise this Section at least one week in advance of commencement of fill construction at the C.N.R. overhead structure. As you know, a great deal of effort and quite a lot of money was spent for the instrumentation at this site, and it is therefore most important that all the necessary preparations and readings be carried out in time.

Antifreeze is going to be put into the piezometer leads in order to prevent any freeze-up in the leads. However, if the placement of the fill is scheduled for a time when very low temperatures are already experienced, it would be most desirable to have the entire length of the culvert covered with an earth cover at least four feet thick. Would you please endeavour to have this precautionary measure carried out.

AGStermac
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF

cc: Foundations Office
Gen. Files.

cc: [redacted]. Files

WP - 104-60-2

Re: C.N.R. Overpass - Tri-Town Bypass.

Mr. B. R. Davis,
Bridge Engineer,
Bridge Division.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

Attention: Mr. S. McCombie

June 29, 1965

WP. File - 104-60-020

C.N.R. Overpass -- Tri-Town Bypass,
Dist. #14 (New Liskeard) W.P. ~~104-64~~

A meeting was held on June 24, 1965, to discuss the alternative schemes for the above project. The purpose of this memo is to recapitulate briefly, the history of the investigation and design of this project and to record the various schemes we propose for this crossing, based on the results of field observation for the past two years.

The original foundation investigation for the crossing and approach fills was undertaken by Geocon, Ltd. in 1960. The subsoil was found by the Consultant to be a thick deposit of laminated and varved clay, underlain by banded limestone bedrock at a depth of 150 - 160 ft. The consistency of the varved clay was found to be soft to firm, having values of shear strength as low as 300 p.s.f. at some locations.

The Consultant recommended an overhead crossing with approach embankments of 33 ft. high, necessitating 120 ft. long berms. The final settlement was expected to be of the order of 100 inches. In view of these shortcomings, Geocon suggested to investigate alternative routes for the crossing and mentioned the possibility of a crossing by means of a subway.

The alternate routes were investigated by H. Q. Golder and Associates in 1961. It was found that soil conditions did not change significantly along the C.N.R. tracks within one mile north and south of the proposed line. Therefore, the present alignment was adopted.

A decision was then reached to go ahead with the construction of the overhead, utilizing stage construction for the embankments. The first stage called for a height of fill of 20 ft., followed by the full height, after sufficient strength increase in the subsoil is obtained by consolidation under the load of the first stage. In order to observe the pore pressures and settlements caused by the load, piezometers and settlement gauges were installed by this Section near the crossing under the proposed embankment.

cont'd. /2 ...

During construction of the first stage, in June 1963, a failure took place some 500 ft. east of the crossing. The height of the fill at the failure was approximately 18 ft. A short berm was designed by this Section to increase stability, and the fill was brought back to 18 ft. in height. Further placement of the fill was called to a halt.

Results of observations for the past two years, together with additional field and laboratory tests carried out on samples from a borehole put down in October 1964, show that:

(a) The excess pore pressure has dissipated by only 15% or less in the last two years.

(b) No increase in strength of the subsoil was noticeable. In fact, there is a slight decrease of the undrained shear strength.

(c) The settlement is still progressing at a rate of approximately 5 in./year, and there is no trend for the rate to decrease.

From the above conclusions, it is obvious that the stage construction method as originally contemplated, is not feasible if construction of the overpass is to be carried out in the near future. In order that construction can proceed, one of the following alternatives should be adopted:

A) <u>Overhead Structure Supported on End-Bearing Piles Driven to Bedrock with Berms for Approach Fill</u>:

This solution will necessitate approx. 31-ft. high embankments, which, in turn, require berms of about 100 - 120 ft. length for stability. The height of the berms are designed to be half of the heights of the embankments. Because of the required 120-ft. length of the end berms, the overhead will have a length of about 460 ft., the abutments being at around Stations 797+80 and 802+40. Spill through type of abutments are recommended in order to maintain 2:1 side slopes for the approach fills.

Bedrock was observed around El. 475 - 467 ft., some 155 - 163 ft. below ground level Piles should be driven to practical refusal, at around above elevations. The safe bearing pressure on the piles will be governed by the structural strength of the particular type of piles used.

Pile driving through the present fill will not be feasible due to the large amount of boulders (1 - 4 ft. diam.) in the embankments. Excavations of the existing fill down to the original ground will be necessary at the locations of pile driving. 2 horizontal to 1 vertical side slopes should be maintained for these excavations.

cont'd. /3 ...

A) (cont'd.) ...

The recommended sequence of pile driving and fill construction is as follows:

1.) Excavation of fill in pile driving locations.

2.) Pile driving and forming the pile caps.

3.) Refill of the excavations and construction of the berms to the designed length and height.

4.) Construction of the approach embankments to the final grade.

By adopting this method of crossing, a new detour will be necessary farther from the centre line of the highway; also, additional land should be expropriated to provide room for the long berms.

B) <u>A Long Overhead Structure</u>:

This solution calls for an overhead of approximately 1,250 ft. length in order to eliminate approach embankments higher than 19 ft. The locations of the two abutments will be at around Stations 794+30 and 806+80, respectively. The height of the approach fills at the abutments will be about 19 ft. Embankments of such heights are stable with very short berms or no berms at all, provided they are constructed with side slopes 2 horizontal to 1 vertical.

The present fill between Stations 794+30 and 806+80 will not be required any longer, so the material of this section could be removed and used for other purposes. Removal of the fill should precede pile driving. The bridge should also be supported on end-bearing piles or caissons, driven to bedrock as discussed under Section A).

Spill-through type of abutments are recommended.

C) <u>Crossing the O.N.R. Tracks by means of a Subway</u>:

This solution will require an approx. 160-ft. long bridge with spill-through abutments, supported on end-bearing piles on bedrock. Length of piles will be some 20 - 30 ft. shorter than those required for the overhead.

The approach cuts for the subway should be constructed with side slopes 3 horizontal to 1 vertical. It is estimated that the deepest cut at the crossing will be about 20 ft. A 1 - 2 ft. thick granular blanket or sand cushion is suggested to be used to protect the cuts.

cont'd. /4 ...

C) (cont'd.) ...

The ground water level should be lowered to below the bottom of the proposed cuts by provision of a deep drainage system to the Wabi River, some 2,000 ft. to the east.

In case of adopting the crossing by subway, the present fill must be completely removed.

It was agreed that the Bridge Office would study these schemes in further detail.

The above recommendations for the various schemes are preliminary and for estimating purposes only. Further detailed work is required on a chosen scheme for final design.

KYL/MdeF

K. Y. Lo,
SUPERVISING FOUNDATION ENGINEER

cc: Mr. G. M. Sinclair

Foundations Office (2)
Gen. Files

cc: Ge[illegible] Files.

Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Section.

February 10, 1961.

<u>Attention: Mr. C. S. Grebski</u>.

Re: O.N.R. Overhead,
Hwy. #11, Dist. #14,
<u>W.P. 104-60.</u>

Attached to this memo, you will find a sketch showing the required distance between the existing and the detoured railway tracks. The distance was determined on the basis of a factor of safety of 1.3. Ten more feet were added to provide some additional safety necessary because of vibrations caused by the passing train.

L. G. Soderman,
PRINCIPAL FOUNDATION ENGR.

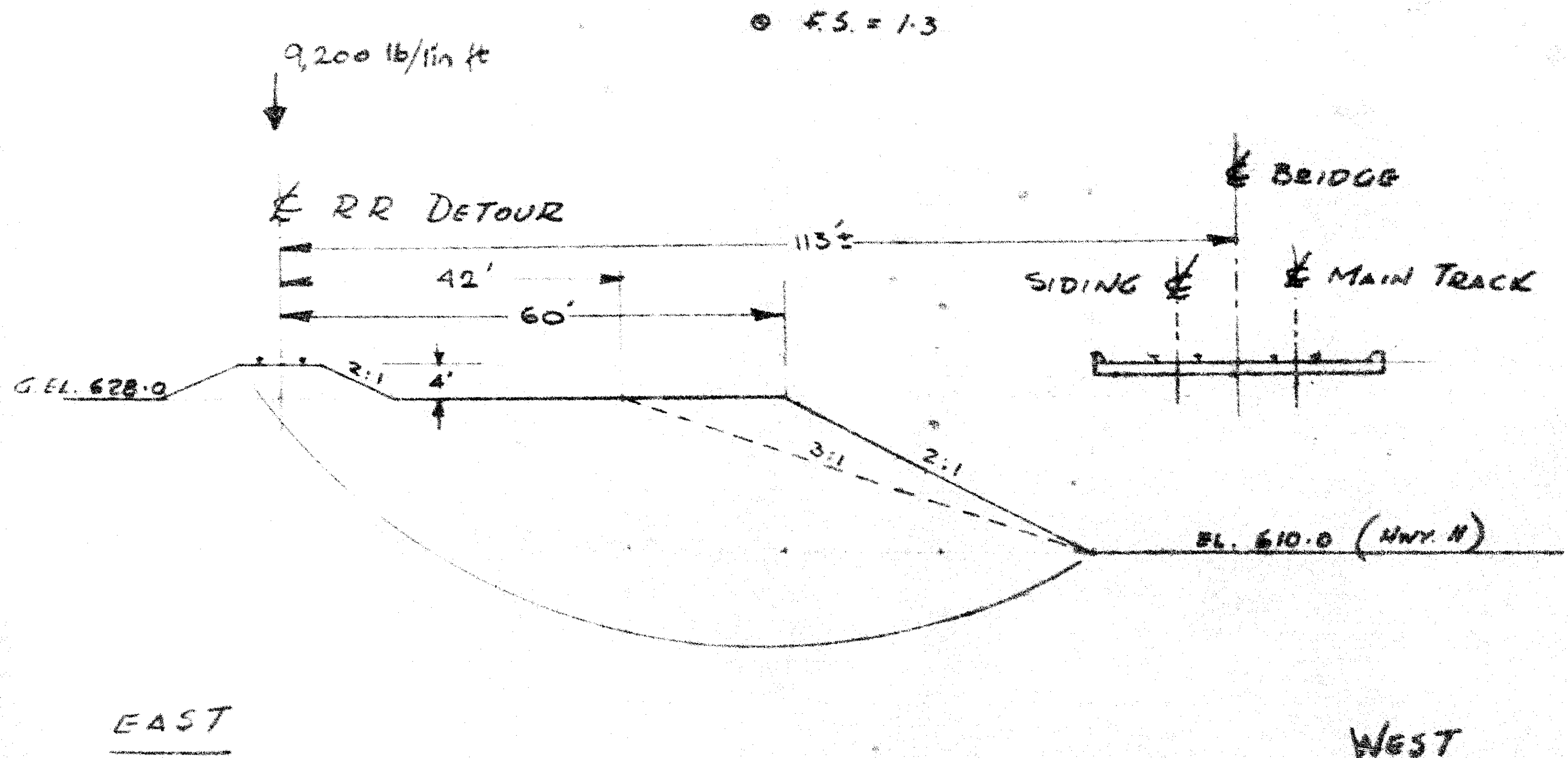
Per:

AGStermac

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

AGS/MdeF
Attach.

cc: Foundations Office
Gen. Files



O.N.R. SUBWAY

DETOUR DISTANCE

SCALE - 1" = 20'

cc: Gen. Files

Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Section.

February 1, 1961.

REVIEW OF PRELIMINARY PLAN by Foundations Office.

Attention: Mr. C. S. Grebski.

Re: O.N.R. Overhead,
Hwy. #11, Dist. #14,
W.P. 104-60.

We have reviewed the preliminary Drawing No. D 4807-B of the three span alternative structure, and have found the embankment slopes stable. A number of stability analyses have been carried out in order to check this stability, making different assumptions concerning the live load and shear strength of the underlying clay. Even when it was assumed that no shear strength is left in the clay through which the piles are driven, and that the piles have no beneficial resisting influence, the computed factor of safety was slightly in excess of one. It is therefore our opinion that the shown preliminary disposition is safe and can be finalized as such.

We believe that this information will prove adequate for your future design work. However, should there be any other questions you would like to discuss, please feel free to call on our Office.

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

(V. Korlu,
PROJECT FOUNDATION ENGR.)

al

VK/MdeF

cc: Foundations Office
Gen. Files

cc: Mr. N. D. Smith

December 16, 1959.

Geocon, Limited,
14 Haas Road,
Rexdale, Ontario.

Attention: Mr. V. Milligan.

Re: Site Investigations,
Tri-Town Bypass - District #14,
-- Three Structures --
1. Hwy. 65 Interchange W.P. 124-60.
2. Wabi River Crossing W.P. 103-60.
3. O.N.R. Overhead W.P. 104-60.

Dear Sir:-

Please consider this your authority to carry out foundation investigations at the above noted sites.

A tentative boring and sampling program has been discussed with your Mr. N. McCannon and he has been provided with the necessary site plans. We anticipate that soil conditions will consist of a deposit of varved clay overlying limestone bedrock. An evaluation of the strength (both laboratory and in-situ vane measurements) and compressibility characteristics of the subsoil will be required - both at the structure locations and in the areas underlying the approach fill embankments.

A summary of the number and location of borings, as discussed with Mr. McCannon, is as follows:-

Hwy. #65 Interchange - W.P. 124-60.

Two detailed sampled borings with vane tests between sample intervals, required at diagonally opposite corners of abutment footings. If pile foundations appear necessary, 10 feet of bedrock core should be proved in each of these borings.

cont'd. /2 ...

Hwy. #65 Interchange - W.P. 124-60 ... (cont'd.) ...

Two wash borings with no samples recovered, should be carried out at the corners of the other diagonal. If piles appear necessary, these wash borings should be carried to refusal, or to prove at least 10 feet of a competent underlying stratum.

In addition to the borings at the structure, three sampled borings are requested at the following centre line chainages: 782+00; 784+00; and 778+00. Information obtained in these borings should be sufficient to design the embankment sections.

Wabi River Structure - W.P. 103-60.

A three span structure is anticipated at this site. One detailed sampled boring including in-situ vane tests and one wash boring to prove a competent bearing stratum for piles, is requested at each abutment location.

If end-bearing piles appear necessary, at least 10 feet of bedrock core should be recovered in each sampled boring. Holes should be located 25 ft. left and right of road centre line. The tentative centre pier location is chainage 819+00, and one detailed sampled boring on centre line is requested at this chainage.

C.N.R. Overhead - W.P. 104-60.

At each abutment location - i.e., chainages 800+00 and 800+60, one sampled boring and one wash boring, as at Wabi River, is requested. In addition, centre line sampled borings are requested at chainages 798+50 and 802+00.

It is understood that boring rigs will be mobilized from Kirkland Lake. Our Project Engineer, Mr. A. Loh, has been assigned as Liaison Engineer for these projects and he is to be advised of the boring results immediately each hole is completed. The extent of laboratory work required and analyses necessary, will be authorized upon our mutual study of field information.

If the tentative boring and sampling program appears inadequate or too elaborate, based upon a study of field data, modifications of this program will be made.

cont'd. /3 ...

It is important that we are informed of results as they become available from the field and we request that you contact Mr. Loh directly. Authorization for the use of a foil sampler will be given if this device appears economically justified.

The District personnel have been requested to assist you with surveying work necessary.

Would you please submit to us your proposal for carrying out the work, as detailed in this tentative program. Please address same to: Mr. A. Rutka, Acting Materials & Research Engineer, Materials and Research Section, Ontario Department of Highways, Parliament Buildings, Toronto 2, Ontario.

Ten copies of the completed reports for these three structures should be submitted for distribution, to the Foundation Office, at your earliest convenience.

LGS/McF

Yours very truly,

A. Rutka,
A/MATERIALS & RESEARCH ENGINEER

cc: Messrs. C. McCombie
G. K. Hunter
R. S. Chapman
E. R. Sand
N. D. Smith

Foundation Section
Gen. Files (2)

THE PERFORMANCE OF AN EMBANKMENT ON A DEEP DEPOSIT OF VARVED CLAY

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K. Y. LO *Professeur Auxiliare, Département du Génie Civil, Université Laval, Québec (Formerly Supervising Foundation Engineer, Department of Highways, Ontario)*
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It is generally accepted that a clay deposit containing near-horizontal layers or bands of coarser grained material exhibits greater permeability in the horizontal than in the vertical direction. It has also been pointed out (Milligan, Soderman, and Rutka, 1962) that, owing to the faster rate of consolidation of such deposits under external load, they are ideally suited for stage construction.

In 1960 the Ontario Department of Highways proposed to construct a bypass of Highway 11 in the Tri-Town region in Northern Ontario. The northerly portion of the bypass at the town of New Liskeard lies on a deep deposit of varved clay of glacial origin (Figure 1). Among other structures along the proposed road, the overpass over the Ontario Northland Railway necessitated the construction of long approach embankments of some 30–33 ft. in height (Figure 2). Stability calculations showed that on account of the low strength of the underlying varved clays an embankment of such height could be built only with long berms or by utilizing stage construction.

The latter scheme was adopted, with the construction of a 20 ft. high fill as the first stage. It was believed that the consolidation of the varved clay would take place much faster than it would in a comparable deep deposit of homogeneous clay and that the larger part of the induced pore pressure would dissipate within a few years. It was also assumed that the corresponding strength increase would improve the soil properties to such a degree that the full height of the fill could then be safely built. In order to observe the pore pressures and settlements caused by the embankment load, piezometers and settlement augers were installed under the proposed fill near the crossing.

During construction of the first stage, in June 1963, a failure occurred some 500 ft. east of the crossing, as reported by Lo and Stermac (1964). The height of the fill at the time of failure was approximately 18 ft. The failed section was redesigned with a short berm and further placement of the fill was brought to a halt. By the summer of 1965 the observations of the piezometers and settlement

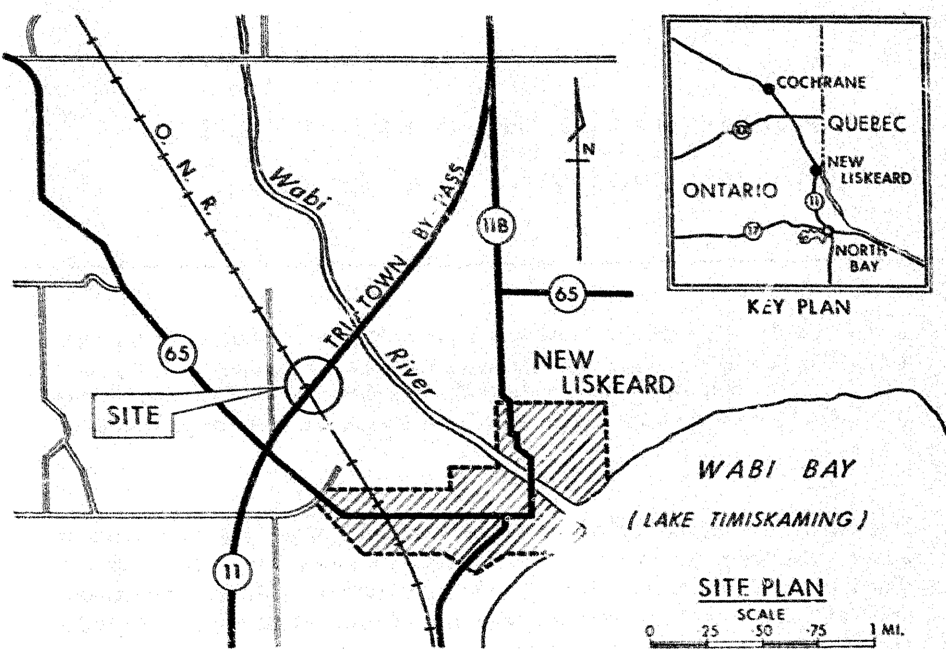


FIGURE 1. Site plan

augers, as well as additional field and laboratory tests on samples of boreholes put down later, showed that: (*a*) The excess pore pressure had dissipated by only approximately 25 per cent. (*b*) No increase of strength of the subsoil was noticeable. Indeed, there was a slight decrease of the undrained shear strength. (*c*) The settlement was still progressing at a rate of about 5 in./year and there was no trend to show that the rate might decrease. From the above findings it became obvious that the stage construction as originally contemplated would have to be abandoned.

Several alternative schemes were therefore considered for the crossing, among them a longer structure to limit approach fills to 19 ft. For the foundations, end bearing piles driven to bedrock and friction piles are being studied. Crossing by means of a subway is also being contemplated. Because of a number of still unresolved engineering problems and also because of economic considerations, it was decided, however, to give priority to the investigation of the overhead scheme in a modified form. This modified design called for the removal of 9 ft. of fill and the use of spread footings for the bridge piers, placed in the remaining 9 ft. of fill. The feasibility of the scheme had to be confirmed by a full-scale loading test.

In this paper, a description of the soil properties and instrumentation for the measurements of pore pressure and settlement is given. The results reported include: (*a*) pore pressures and settlements at various depths under the full

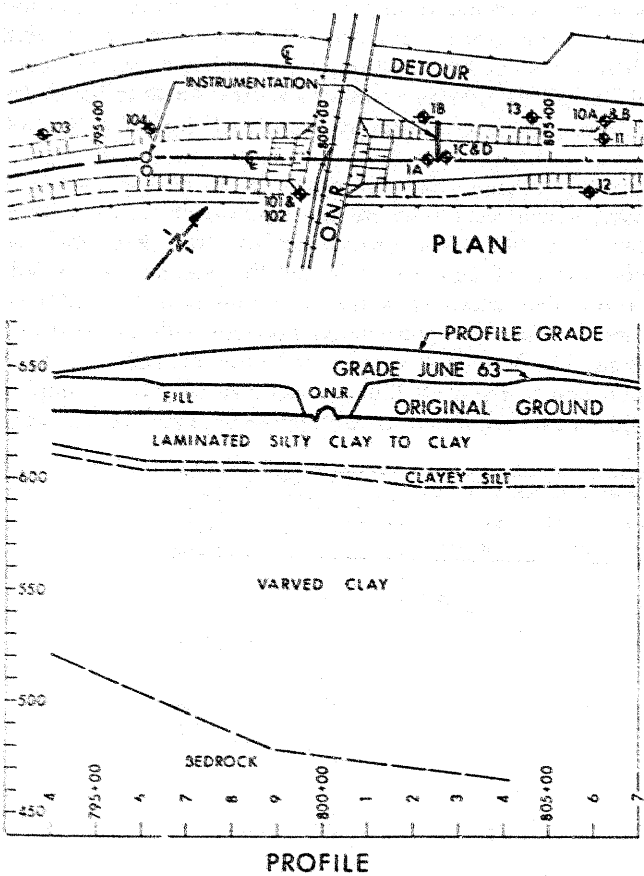


FIGURE 2. Plan and profile

load of the [illegible]8 ft. embankment; (*b*) pore pressures and settlements during and subsequent to partial removal of the embankment load; and (*c*) settlement of the full-scale test footing. The results of field observations are discussed in relation to their effects on the design problem. Detailed analyses of these results are outside the scope of this paper.

SOIL CONDITIONS

The site of the crossing is part of the region known as the "Little Clay Belt." The subsoil consists essentially of varved and laminated deposits of silty clays or clays (dark layers), and clayey silts (light layers). The geotechnical properties of the varved clay in the New Liskeard area have been studied by various authors (Milligan, Soderman, and Rutka, 1962; Eden and Bozozuk, 1962; Raymond and Hilts, 1964; Lo and Stermac, 1965), and the physical-chemical properties have been investigated by Soderman and Quigley (1965).

The area under consideration included the proposed overhead crossing at the ONR tracks and approximately 600 ft. east and 800 ft. west of the crossing along the bypass, where the stage construction of the embankment was contemplated (Sta. 794+00–808+00). Within this area several series of boreholes were put down at different stages of design and construction during the last four years.

These field investigations showed that the upper 20 ft. of the subsoil is composed of silty clays, having very faint minute laminae of clayey silts. A layer of approximately 8 ft. below the ground surface was desiccated, as evidenced by the higher values of shear strengths and the presence of mottled, oxidized pockets of clays. The thickness of the silt seams is in the order of $\frac{1}{32}$–$\frac{1}{8}$ in. Figures 3 and 4 show two representative borelogs with the results of laboratory and field tests. The boreholes were located near Sta. 802+50.

In the upper portion due to the very thin laminations, only bulk samples were tested. The liquid limits were found to vary between 43 and 76 per cent,

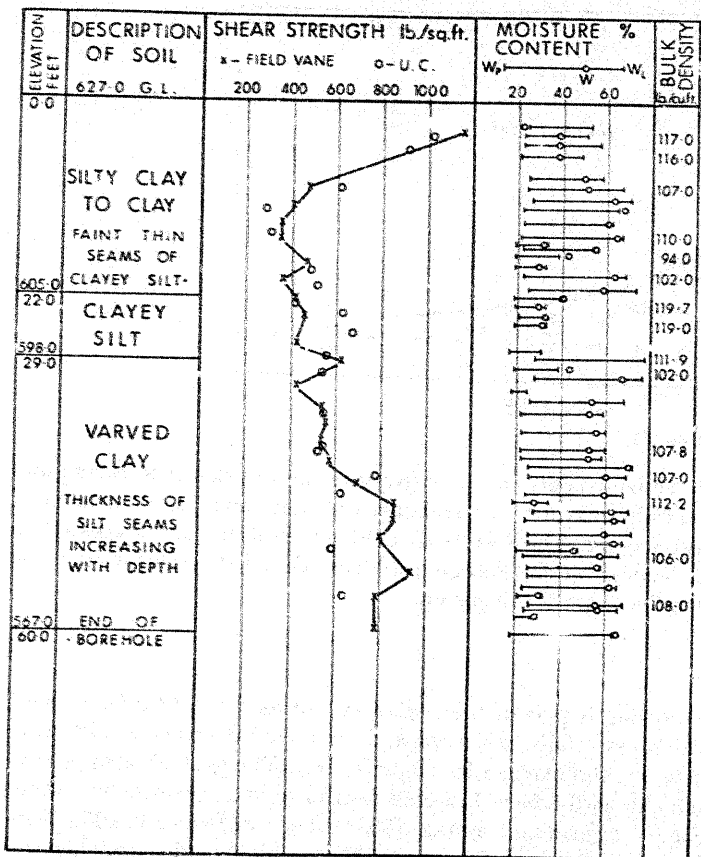


FIGURE 3. Results of field and laboratory tests: bore hole 1A and B

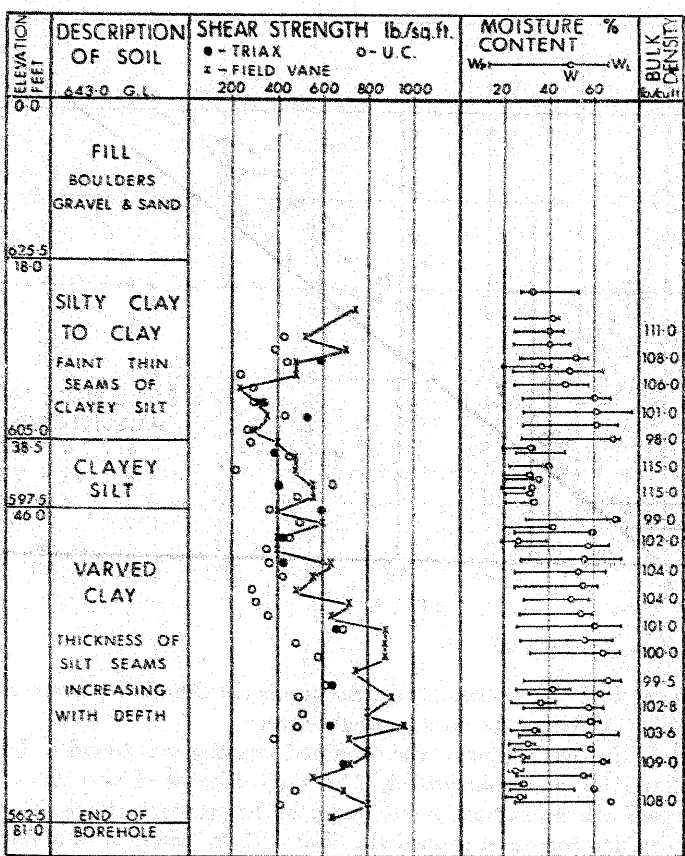


FIGURE 4. Results of field and laboratory tests: bore hole IC and D

the plastic limits from 23 to 28 per cent. Typical values of Atterberg limits are plotted on Figure 5. The natural moisture contents of the thin laminated layer zone lie both below and above the liquid limits, yielding values of the liquidity indices of 0.64–1.40.

Underlying the laminated stratum a 3–7 ft. deep silt layer was observed, with hardly any visible stratification, under which the varved clay of a more regular nature follows. The separate varves are composed of dark grey clay or silty clay and light grey clayey silt; the thickness of the individual varves varies and there appears to be a tendency of increasing thickness of silt seams with depth. The lamination is near-horizontal, but occasional inclined varves were also observed. The thickness of the individual seams is between 1/32 and 1½ in.

The average values of liquid limits for the dark clay and the light silt seams were estimated to be 72 and 27 per cent respectively, while those of plastic limits were 31 and 20. The computed highest and lowest values of the liquidity

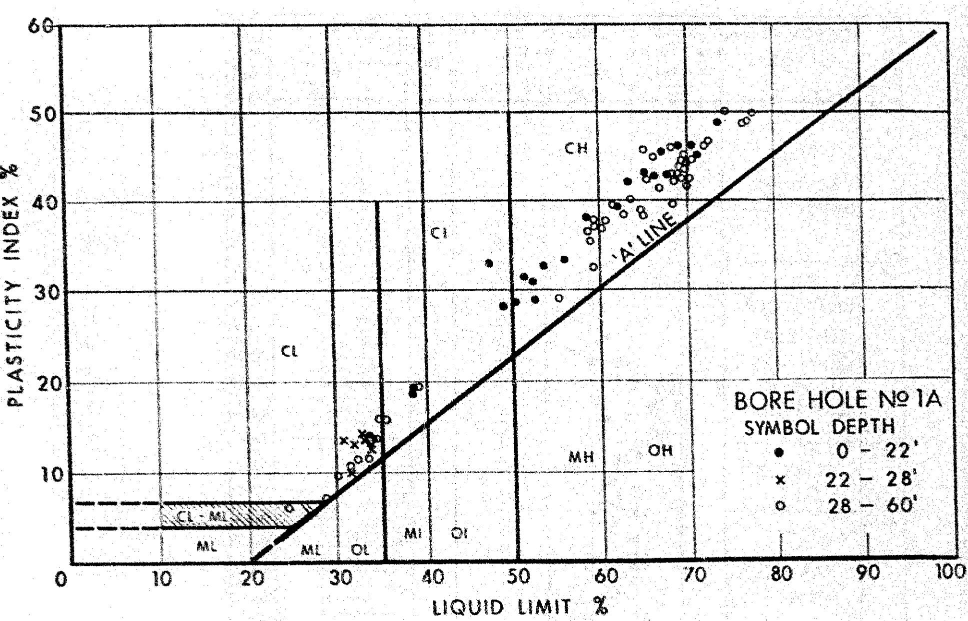


FIGURE 5. Plasticity chart, New Liskeard

indices were 1.30 and 0.41. There seems to be no significant difference between the values of liquidity indices of the dark and light layers.

The total depth of the overburden at the proposed crossing was found to be about 150 ft. followed by an approximately 2 ft. thick deposit of very dense silty sand and gravel, which, in turn, is underlain by limestone bedrock. The depth of the overburden increases toward the Wabi River, where it is about 170 ft. deep and decreases toward the crossing of Hwy. 65, where the bedrock is exposed at the surface. The undrained shear strength of the overburden was determined by field vane tests, unconfined compression, and unconsolidated undrained triaxial tests. Since there appears to be no reason to place more reliability on any one of the three types of tests for reasons discussed elsewhere (Lo and Stermac, 1965, also closure to discussions 1966), the weighted average values were computed.

A careful study of the strength determined at various locations along the embankment showed that the strength is minimum at regions around the failure area but increases in both directions towards Wabi River and Hwy. 65. The average undrained strength at three locations is plotted in Figure 6, which illustrates this horizontal variation of strength. Although the weighted averages of all the test results have been shown in Figure 6, it is important to note that this trend of variation is present also when each type of test is considered separately.

Typical void ratio/pressure curves of oedometer tests are presented on Figure 7.

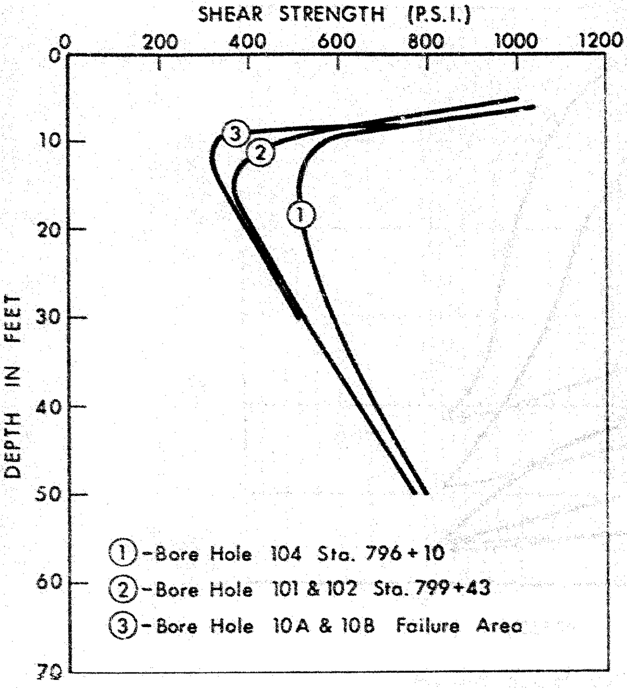
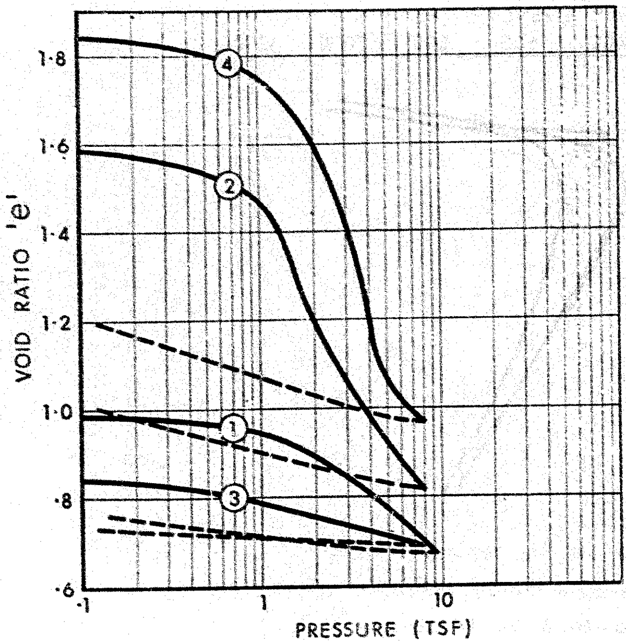


FIGURE 6. Variations of average shear strength at different locations

INSTRUMENTATION

In order to observe the pore water pressures and settlements in various locations beneath the fill, piezometers and settlement points were installed. On Figure 8 the layout of the instrumentation at Sta. 802+50 is shown. The instrumentation was carried out jointly by Queen's University, Kingston, and the Department of Highways, Ontario, in 1962, and formed part of an Ontario Joint Highway Research Project. At later dates, additional piezometers and settlement augers were put down near the original ones ("L" group) and also at the west side of the crossing at Sta. 796+00 ("B" group).

The first set of 30 piezometers was installed in four locations below the fill at Sta. 802+50, at various depths from 15 ft. to 120 ft. below the original ground. The location of the first group was beneath the centre-line of the fill, the second below the shoulder of the 18 ft. high embankment, the third at the toe, and the fourth at 40 ft. outside the fill. Within the upper 60–75 ft. depth, "Peaker"-type piezometers (modified Casagrande piezometers) were installed, with two polyethylene leads to facilitate flushing the instruments. The piezometers below 75 ft. were of the porous brass type (Geonor piezometers). They were provided with one tube only, so flushing and cleaning were not possible because of the great depth involved. The installation of the latter piezometers



BORE HOLE № 1A

CURVE	SAMPLE	DEPTH	C_c	%		
				W_P	W_L	W
1	3	6' 2"	0·351	22	50	38
2	9	21'	0·938	25	68	60
3	10 (SILT)	24'	0·100	19	33	31
4	22	51'4"	1·290	25	67	59

FIGURE 7.

were not satisfactory; in fact none of the instruments registered reliable pressures from 1963 on. Possible factors in causing this malfunction might have been (*a*) the difficulty of de-airing and flushing of the piezometers at great depth or (*b*) the fact that, since the polyethylene tubes were not cased in by drill rods or casings, they could have been bent, twisted, and broken owing to the large settlements. Tubes of the piezometers installed later were protected by standard E and A rods, and so far, have been found to perform satisfactorily.

The settlement augers consisted of 2 in. diameter ship augers attached to ¾ in. pipes, the upper 4 ft. of which were assembled in 2 ft. sections. The latter could then be removed as settlements increased. The settlement rods were encased by 1.5 in. diameter casings. Surface settlement plates were installed 6 in. below ground.

The leads of the piezometers together with the settlement rods (below the fill) were housed in a 48 in. CIP, so that readings of the instruments could be

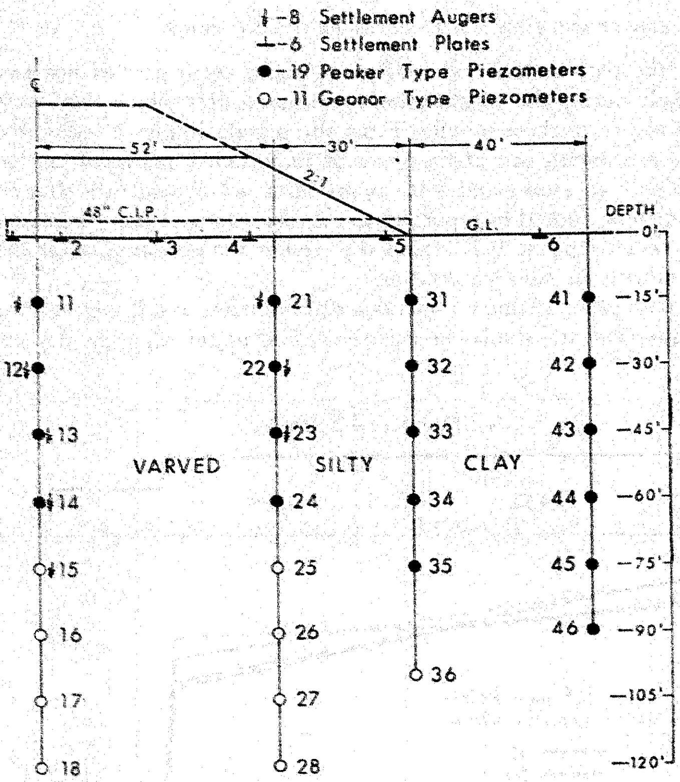


FIGURE 8. Instrumentation, Sta. 802 + 50

taken during and after construction. The piezometers were assembled with Bourdon pressure gauges by means of a header. The gauges were provided with mercury null indicators similar to those used in laboratory triaxial tests, so that a whole group of piezometers could be read by means of one pressure gauge.

In the early fall of 1965, additional (Geonor) piezometers were installed at Sta. 796+00, where the shear strength of the subsoil was observed to be higher than at Sta. 802+50. Four piezometers were put down at the centre, 15, 30, 45, and 60 ft. below the original ground; also three piezometers at the shoulder at depths 30, 45, and 60 ft. Three settlement augers were installed as well, just below the original ground, two of them being at the centre and one below the shoulder of the fill. The purpose of this latter instrumentation was to compare the pore pressures and settlements within stronger and weaker soil strata that were induced by a superimposed load of approximately the same magnitude.

In order to check the accuracy of the original piezometers at Sta. 802+50, five piezometers (Geonor) were placed at the centre of the fill near the original ones in the fall of 1965 ("L" group on Figure 13).

RESULTS OF SETTLEMENT AND PORE WATER PRESSURE OBSERVATIONS

Observations of the piezometers and settlement points were carried out two or three times daily during the construction operations, after which they were read at intervals of two or three months. From the practical point of view, the centre group of instruments are of the greatest importance, inasmuch as the highest induced pore pressures and settlements occurred beneath the centre. This paper is therefore limited to reporting the settlements and pore pressures registered by the centre group. It is intended to report the performance of the rest of the instruments in later publications.

The results of pore pressure measurements for piezometers at different depths below the embankment are shown in Figure 9. The upper diagram depicts

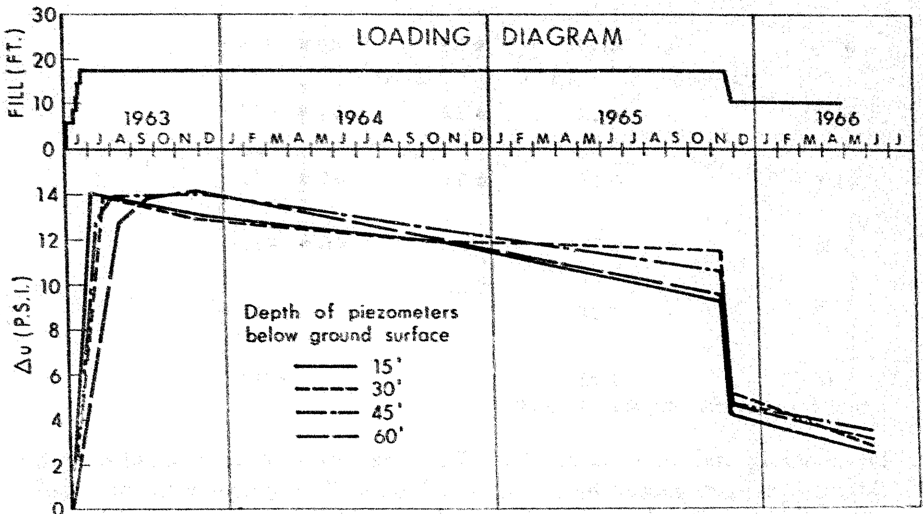


FIGURE 9. Variation of excess pore pressure, Sta. 802 + 50

the history of construction. The full height of 18 ft. of embankment was completed in approximately three weeks within 100 ft. on either side of Stat. 802+50. The load was put on in 2 to 3 ft. lifts running from toe to toe. The fill material was granular with pebbles and boulders. It will be seen from Figure 9 that there was very little dissipation of pore pressure during the first few months after the full load was put on. In fact, the average degree of dissipation after 2½ years was only 25 per cent for this group. The maximum pore pressure registered was 14 lb./sq. in.

Figure 10 depicts the surface settlements versus time at Sta. 802+50. The settlement forms a regular pattern with its magnitude decreasing from the centre to the toe of the embankment. The largest settlement below the centre of the fill during the period from June 1963 to November 1965 was registered as 25 in. (settlement plate no. S-1). The "immediate" settlement, which

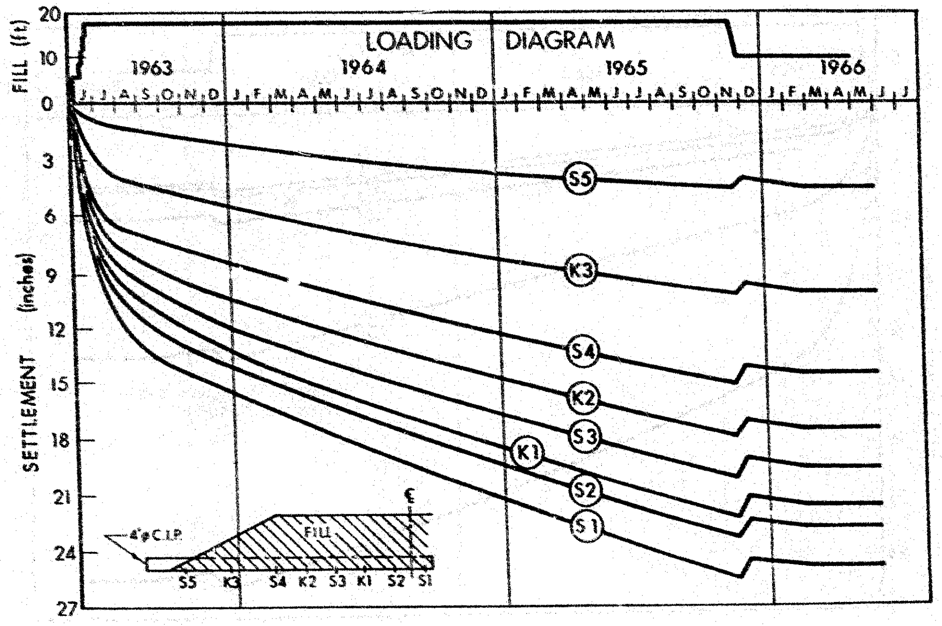


FIGURE 10. Variations of surface settlements versus time

occurred during the construction period, was approximately 8 in. As can be seen, the settlements proceeded at about the same rate, without any noticeable slowing down until half of the fill was removed (in November 1965).

On Figure 11, settlements at various depths beneath the centre of the embankment are presented as registered by the settlement augers.

Results of pore pressure measurements at Sta. 796+00 are shown in Figure 12. The embankment at this location was 16 ft. high and was constructed at approximately the same period as that portion at Sta. 802+50. However, when the pore pressures at these two stations are compared, it is obvious that the pore pressures at Sta. 796+00 are much lower. At October 1965, the highest value of the excess pore pressure was 5 lb./sq.in. registered by the piezometer at 30 ft. depth. The excess pore pressure at other depths are lower, being from 2 to 3 lb./sq.in. At the same period, the excess pore pressure at Sta. 802+50 was about 10 lb./sq.in.

It was observed that settlement points at Sta. 796+00 during the period from September 1965 to March 1966 registered surface movement of less than 0.4 in. Within the same period of time the surface settlements at Sta. 802+50 would have been in the order of 2.5 in. if no fill had been removed.

In order to determine the effect of consolidation on the shear strength of the subsoil, two boreholes were put down (B.H. 1C and 1D) near the instrumentation at Sta. 802+50 in the fall of 1964. These boreholes were some 10 ft. from B.H. 1A and 1B, which were carried out prior to the construction of the fill

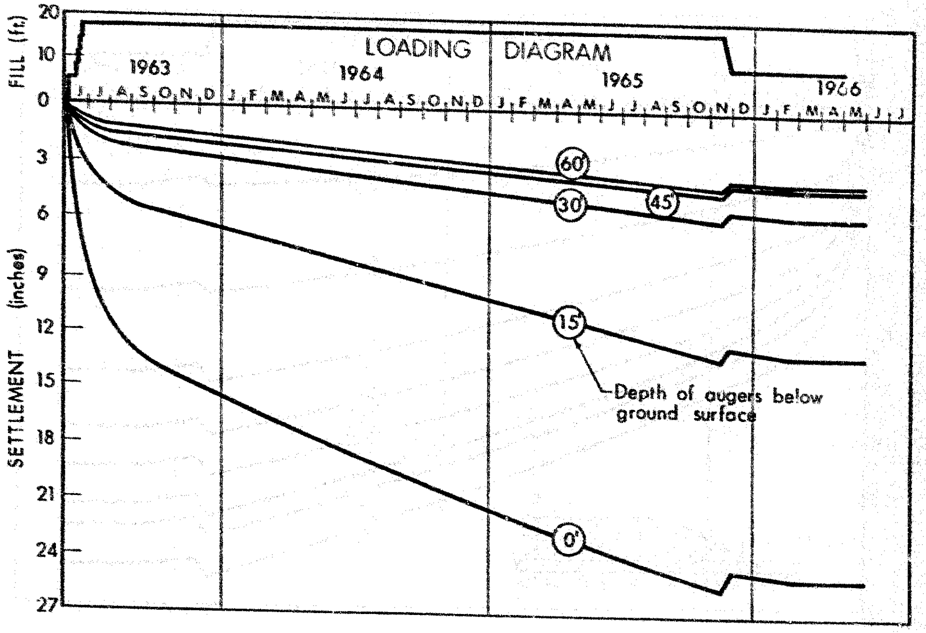


FIGURE 11. Settlements below the centre of fill at various depths

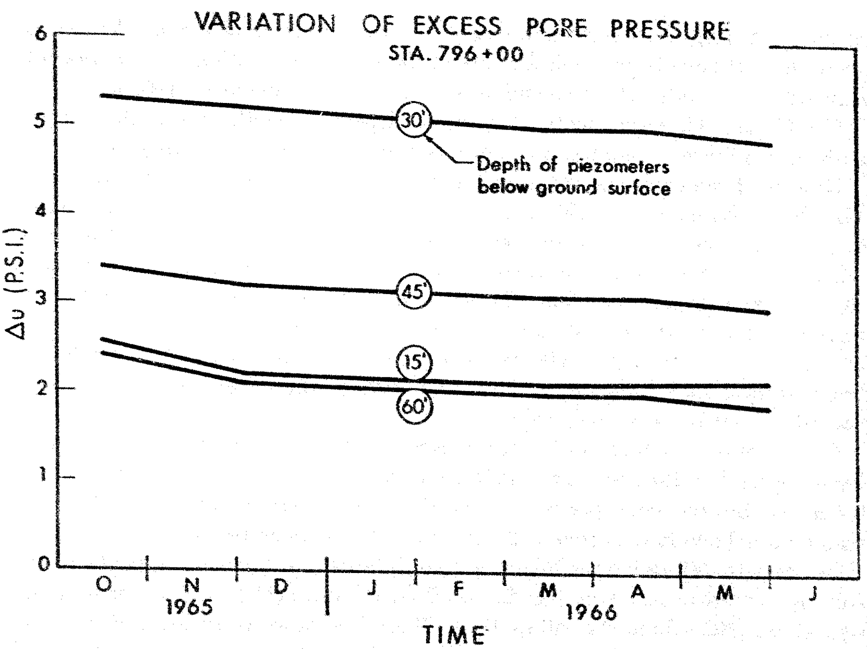


FIGURE 12. Variation of excess pore pressure, Sta. 796 + 00

(Figs. 3 and 4). No increase of shear strength was observed whatever but, rather, a slight decrease at depths of 16–22 ft.

In summarizing the observations up to the early fall of 1965, it can be seen that at Sta. 802+50 where the undrained shear strength of the soil was generally low (see Figure 6) the high values of pore pressure still persisted, the rate of settlements showed no slowing down, and no increase of the shear strength was noticeable. On the other hand, at Sta. 796+00, where the shear strength was generally higher, the magnitude of the induced excess pore pressure was less than half the pressure registered at Sta. 802+50 by October 1965. Hardly any settlement was measured within the six months of observation.

It should be mentioned that the original design of the stage construction was based on undrained shear strength values of 500 lb./sq.in.; the weaker zones were unfortunately discovered only later. Had the assumed value been the true shear strength of the subsoil along the entire length of the fill, the consolidation would probably have occurred similarly as it did at Sta. 796+00.

MODIFICATION OF THE ORIGINAL DESIGN

By the early fall of 1965, it became clear from the observations discussed in the previous section that the originally proposed stage construction would not be feasible in the near future. A modified scheme of the crossing was therefore adopted. The new crossing scheme called for the removal of half the height of the fill within the length of the 1308-ft. long bridge, and spread footings for the piers supported within the remaining 9 ft. of the embankment. It was assumed that, because of the preloading of the soil by the 18-ft. fill, only small further settlements would occur under the 9-ft. fill.

To assure the maintenance of the desired clearance over the railway tracks, the end piers at the crossing were to be founded on H-piles driven to bedrock. It was also decided that a test footing be built near Sta. 802+50 and loaded with the design dead load of the pier and the weight of one span of the bridge (220 tons) in order to study the settlements for a period of half to one year.

The removal of the upper half of the fill within a 500 ft. length of the test area, together with the construction and loading of the test footing, was carried out during the latter part of November 1965. The layout of the loading box and the piezometers beneath the middle of the fill are shown on Figure 13. The effect of the partial removal of the embankment load was observed by the settlement points as well as by the piezometers to be almost instantaneous. The magnitude of the rebound of the subsoil at the original ground beneath the centre of the fill was 1.1. in. and roughly 0.5 in. at a depth of 60 ft. (see Figs. 10 and 11)

The drop of the excess pore pressure (Δu) registered by the centre group of piezometers was in the order of 5 lb./sq.in. (Fig. 9). Figure 14*b*, *c*, and *d* illustrate the drop of Δu during the time of load removal at the other three locations. It can be observed that beneath the centre, shoulder and toe of the fill the drop of pressure was 50–60 per cent of the pressure registered just

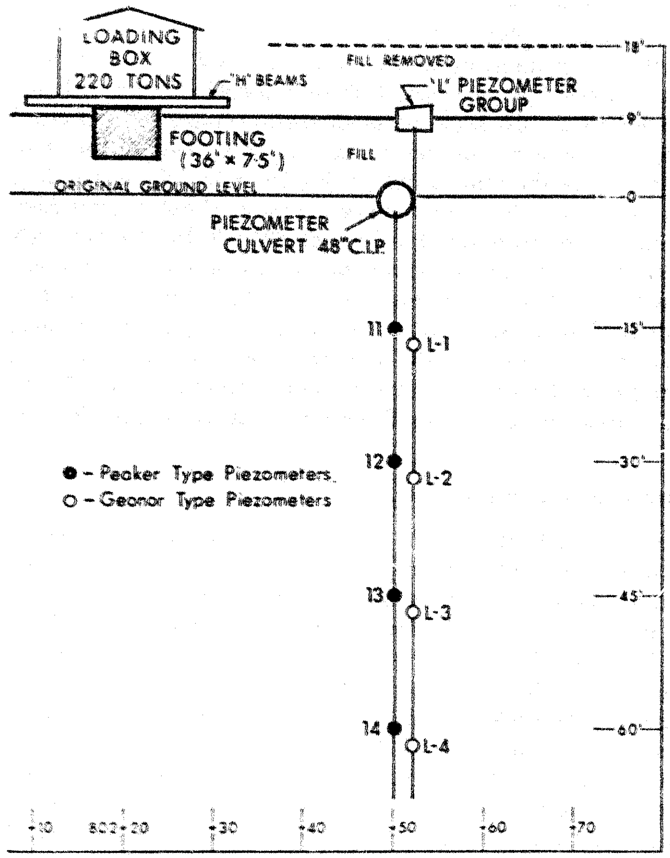


FIGURE 18. Settlement below test footing

before the stress release. However, at the location outside the fill the pressure decreased less than 20 per cent. It is quite obvious that the rate of dissipation during the period from November 1965 to June 1966 was somewhat faster than before the removal of the embankment load.

Observations of settlements of the test footing were carried out after the test footing was fully loaded. The loading box contained 220 tons of sand, yielding a unit load of 0.82 tons/sq.ft. The results of observation are plotted in Figure 15. A very small amount of settlement occurred below the embankment from November 1965 (load removal) to February 1966, and no further settlement was observed up to May 1966 by the points inside the culvert.

Settlements below the footing, however, appear to be quite substantial. Settlement within the six-month period was about 2 in., of which approximately 1 in. may be taken as immediate, the rest being consolidation settlement. The projected consolidation settlement for the first year may be taken to be 1.5–1.8

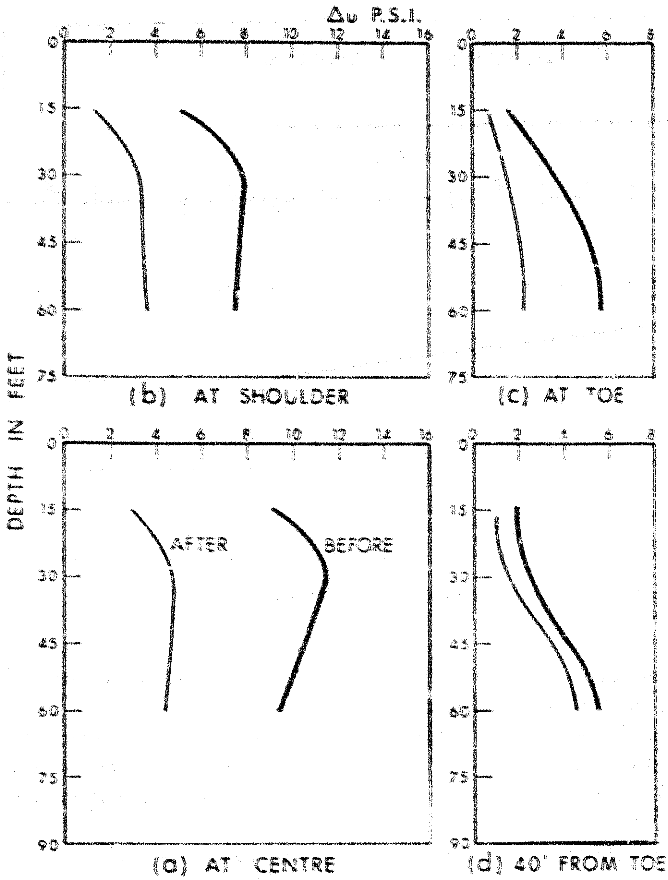


FIGURE 14. Drop of excess pore pressure due to the removal of 9 ft. of fill

in., but it is felt that the total settlement due to the footing will not exceed 4 in. and the rate of settlement will be relatively fast. It should be remembered that a large portion of the stresses is concentrated in the 4 ft. layer of granular fill below the footing, thus transferring diminished stresses to the weaker soil strata.

CONCLUDING REMARKS

The soils investigation, instrumentation, and performance of an embankment founded on a thick deposit of varved clay have been described. Accounts have also been given of the pore pressure and settlement behaviour subsequent to partial removal of the embankment load, as well as the performance of a full-scale test footing. While the results of all these field observations await

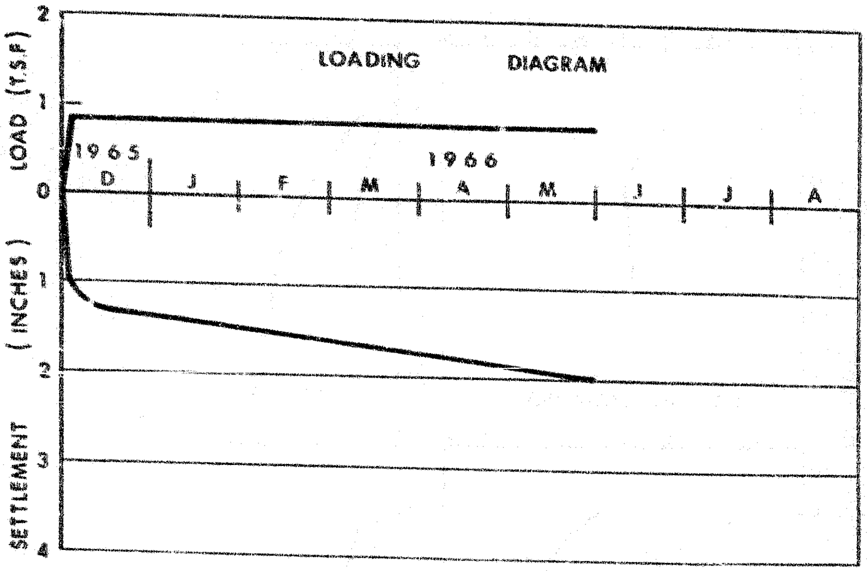


FIGURE 15. Layout of test footing and piezometers (at centre of fill)

further detailed analysis, the present study serves to emphasize several important points from the practical point of view.

1. In the same and apparently uniform soil deposit, important horizontal variation of strength may occur. The variation may not be large in the absolute value of shear strength, but it is sufficient to alter the design approach completely in a soft clay.

2. When an embankment is designed with a low factor of safety, pore pressures set up by shearing strains must be accounted for. These pore pressures are generated under sustained load and are strain-dependent. A hypothesis based on the pore pressure-strain concept (Lo, 1961) appears to be able to interpret the observed phenomena consistently.

ACKNOWLEDGMENTS

Dr. G. P. Raymond, Queen's University, participated in the layout of the instrumentation, the major portion of which was carried out by Mr. D. Hilts as part of his M.Sc. thesis. Mr. M. Devata, Senior Foundation Engineer, Department of Highways, Ontario, provided guidance during this field work. Dr. H. Q. Golder of H. Q. Golder and Associates Ltd., Toronto, participated in the revision of the original approach and contributed to the revised design. The encouragement and guidance by Mr. A. Rutka, Materials and Testing Engineer, Department of Highways, Ontario, is greatly appreciated.

The authors are indebted to Mr. H. W. Adcock, Assistant Deputy Minister, Engineering, Department of Highways, Ontario, for permission to publish this paper.

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DISCUSSION

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All field records are of immense importance to a better understanding of soil behaviour. We are grateful to the authors for recording their experiences in connection with the performance of a deep deposit of varved clay subjected to an embankment loading.

It is perhaps worth noting that the geotechnical properties of the varved clay described by Raymond and Hilts (1964) and referred to by the authors along with results reported by Townsend, Raymond, and Cruickshanks (1964) were obtained from a borehole some 37 ft. from the original instrumentation described by the authors. A reanalysis of the laboratory tests along with additional tests on the same material has been presented by Raymond and Chan (1966). The above references are given so that anyone who wishes to perform his own analysis of this case record may do so, although a more detailed analysis is at the moment underway.

One of the most important aspects of this paper is that it describes a planned stage construction of an embankment which had to be abandoned for some alternative scheme. Very rarely do we see in print case records which did not work out as planned.

Some of the difficulties which arise when attempting to predict the performance of this case record can be summarized as follows. First, one wonders whether the use of larger factors of safety as suggested by the authors would have caused faster rates of dissipation of the pore-water pressures. The writer would tend to agree with the authors that it would, for reasons that have been stated earlier by Raymond (1965), (1966a).

Second, it may be noted that the modulus of deformation of this soil deposit as reported by Raymond and Hilts is about 10 to 15 times higher at the bottom of the deposit than at the top. This difference is indirectly confirmed by the vane tests reported, if it is agreed that the undrained shear strength is proportional to the undrained modulus. It is quite possible that such a variation in modulus will affect the initial distribution of pore-water pressures.

Third, shown on Figure 1 of this discussion is a plot of field void ratio against compression index for this varved clay. It is quite apparent that there is much variation in the soil properties. At the moment it would appear that the only way of analysing this material is to try to obtain an average field void ratio to give an average compression index or an average value of the consolidation characteristics (the same type of plot being used to determine the other consolidation characteristics).

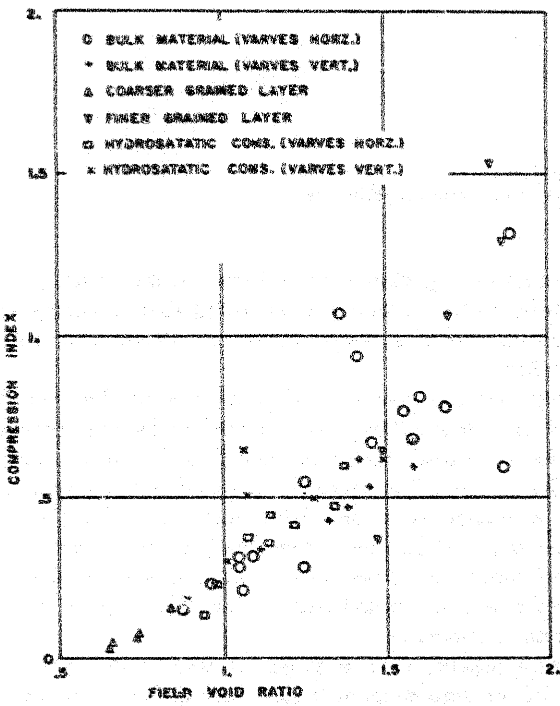

FIGURE 1. Correlation betwen compression index and field void ratio of New Liskeard varved soil

Fourth, in such a deep deposit it is felt that some effect of the variation of the soil's characteristics with respect to depth should be taken into account. Some work along these lines has been presented by Schiffman and Gibson

(1964), Davis and Raymond (1965), and Raymond (1966b). Further work will, it is hoped be presented by the writer and will acount, also, for the variation in consolidation coefficients during the consolidation process.

Fifth, some consideration of lateral drainage must be given.

While it is not our intention to elaborate on any of these points at this time, it is perhaps worth recording that the Ontario Department of Highways and Queen's University have been co-operating through the Ontario Joint Highway Research Programme to obtain a better understanding of the effects of these points.

It should be needless to add that considerable savings could have been realized had it been possible to predict the performance of this embankment. This underlines the necessity for continued research on the consolidation and settlement behaviour of soils.

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The importance of field measurements has been well demonstrated by the authors of this paper. In the opinion of the writers it would not be possible, using present theories, to predict the settlement rates and pore pressure response which has been observed in this case.

From the soil profile and the consolidation tests it may be assumed that most of the consolidation settlement occurred in the 10-ft. layer between depths of 12 and 22 ft. The stress applied to this layer has just equalled or exceeded the measured preconsolidation pressure and the settlement of the layer is estimated to be about 5 in. during the first month, 10 in. after 6 months, and 15 in. after 24 months. At the end of two years, therefore, the layer has compressed about 12 per cent without any gain in strength. This is also the layer of minimum initial strength through which most of the failure surface passed and significantly it is the most sensitive layer with a liquidity index of 1.5.

It is known that in a sensitive soil under constant stress the pore pressure will increase with time due to structural breakdown and presumably this can lead to undrained failure if these pressures cannot dissipate quickly enough. It has been observed in the laboratory that a substantial consolidation is required to compensate for the loss in strength which occurs when the preconsolidation pressure is slightly exceeded (Crawford, 1963). A similar observation was made in sensitive clay under an earth fill at Kars, Ontario, just south of Ottawa (Eden, 1960).

This fill was placed in 1959 over a layer of extremely sensitive Leda clay. The vertical pressure of the fill, about 3000 lb./sq.ft., exceeded the preconsolidation pressure of the clay. In 1963, some 3½ years after the fill had been placed, the clay layer had settled nearly 15 in. and the excess pore water pressure caused by the fill had dissipated to about 5 ft. excess head. In July 1963, field vane tests were conducted through the fill to determine whether any gain in strength due to the consolidation could be observed. The results showed the same average vane strength as the tests conducted before any fill had been placed. Hence a substantial volume decrease in the clay layer had not resulted in any strength gain and, therefore, the loss in strength due to the structural breakdown in the clay was just as significant as the gain in strength from the consolidation process.

In the New Liskeard fill it is apparent that from the time of construction until the time of unloading (2½ years) the pore pressures were increasing at a rate almost equal to the maximum rate of dissipation. As soon as the load was reduced (in November 1965) the pore pressures immediately decreased to less than half their previous values. Despite this substantial drop in the hydraulic gradient the observed rate of dissipation increased. This is a most significant observation because it suggests that the relief of overburden has stopped the strain-mechanism that was creating the pore pressures on a continuing basis. Following this change in stress level, therefore, the dissipation should follow a more normal rate consistent with the consolidation theory. It would be useful to have an extension of Figure 9 in the authors' closure.

Further questions of interest are: 1. What were the ranges of computed amount and rate of settlement under the fill? 2. What was the method and rate of loading of the consolidation tests? 3. Was there any evidence of lateral movement outside the failure area? 4. What pore pressures were observed below the 60 ft. depth?

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The paper presents an interesting and valuable example of a case history of fill construction over soft ground. It points out clearly that stage construction may present many problems which may be difficult to investigate beforehand. The spacial variation in strength experienced at New Liskeard occurs often at many sites and for this reason a conservative approach to safe initial fill heights may be advisable.

The following points are presented for discussion.

1. What were the C_v values and how did they vary?
2. How were these values used in the calculations for design?
3. In a similar situation how would the results of this investigation be interpreted to get a satisfactory design? In other words had a failure not occurred, would stage construction have been feasible?

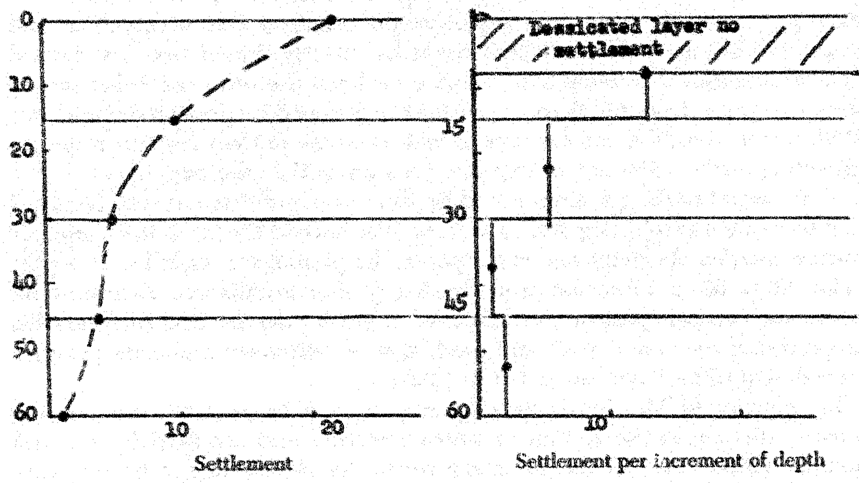

FIGURE 16

4. Plotting the settlements (Fig. 16) between the various points of measurement it appears that about 50 per cent of the settlement occurs in the top 15 ft. If the top 8 to 10 ft. is desiccated it appears likely that there would be hardly any settlement in this section, leaving only about 7 ft. of materials in which 50 per cent of the settlement has occurred. Would the authors care to comment on the reasons for this and do they consider that this layer may have been extensively remoulded?

If this were the case, the remoulding may have formed a more impermeable layer preventing drainage of the underlying layer and thus delaying the process of consolidation. The fact that decreases in strength occurred tends to support this hypothesis; a remoulded layer may well lose strength faster than it can be regained by consolidation. This was found to be the case under fill on the Burnaby Freeway.

5. Sensitivity values are not quoted. What were they? Did they give any indication that remoulding due to shearing might have caused an increase in the compressibility and a decrease in the permeability?

6. In order to speed up the dissipation of pore pressures, was the use of toe relief drains in the form of drilled in sand drains considered? These may have been effective in providing a shorter drainage path.

Reply by A. G. STERMAC, K. Y. LO, and A. K. BARSVARY

The authors are grateful to the discussers for their interesting contribution. The main purpose of the paper was to present the results of field observations and therefore only the conclusions that could be directly drawn from the factual data were outlined. Attempts to compare in detail the observed behaviour of the varved clay deposit with some existing theories will be presented elsewhere. With this in view, the authors restrict their response to the questions raised to providing further information that has been omitted in the paper.

With regard to the questions raised by Crawford and Eden, it may be noted that there are no visible signs of movement at or beyond the toe at the section of instrumentation. As mentioned in the paper, the piezometers installed at depths below 60 ft. did not function properly shortly after installation. Therefore, no information on pore pressure was obtained at greater depths. The consolidation properties of the clay deposit and prediction of settlement and pore pressure were described by Raymond and Hilts (1964).

In response to Mr. Readshaw's queries, it may be pointed out that the failure adjacent to the section of instrumentation and reported by Lo and Stermac (1965) was not the governing reason for abandoning stage construction. The main concern was the persistently high pore pressures and the slow rate of dissipation, together with the lack of increase in shear strength of the clay 2½ years after the embankment was constructed.

It must be noted that the so-called desiccated crust of the deposit is not incompressible. The average shear strength of the first 8 ft. is approximately 850 lb./sq.ft. and the highest strength measured is 1340 lb./sq.ft. with liquidity index generally greater than 0.5. The large settlement that occurred within the first 15 ft. of the deposit, however, no doubt induced considerable straining of the clay with simultaneous reduction in strength due to structural breakdown. The same phenomenon would occur at greater depth, though to a smaller degree. For this reason, the use of sand drains, although they may have accelerated the dissipation of pore pressure, does not necessarily produce an over-all gain in shear strength sufficiently large for the second stage construction.

The sensitivity of the clay determined from the same field tests ranged from 4 to 20, averaging 10, as reported previously (Lo and Stermac, 1965).

Professor Raymond suggests several factors which should be considered in the analysis of the observed behaviour of the varved clay deposit. The authors are pleased to see that the case record is serving the purpose of arousing interest in further study of the behaviour of varved clay deposits, which have caused several major problems in the design and construction of embankments in these areas.

DEPARTMENT OF HIGHWAYS, ONTARIO

FAILURE OF AN EMBANKMENT FOUNDED ON VARVED CLAY

by

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Report No. 54

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SUMMARY

During the construction of a 19-ft. high roadway embankment at New Liskeard, Northern Ontario on varved clay, a failure involving half the width of the embankment occurred, extending for a length of 350 ft. A comprehensive investigation was carried out, including continuous sampling and in-situ vane tests at 2-ft. intervals to locate the failure zone.

Stability analyses were carried out both for the failed and the stable side of the embankment, assuming different values of fill strength and depth of tension cracks.

Prior to construction, 30 piezometers and 14 settlement gauges were installed at a section close to the failure region. Extrapolation of the pore-pressures measured at this location to the failure section enabled the stability study to be performed in terms of effective stresses. Analyses were also carried out for the instrumented section both in terms of total and effective stresses.

The results of this study show that:

(a) The discrepancy between in-situ vane and laboratory tests carried out in the varved clay stratum is of the same order of magnitude as those experienced in homogeneous deposits.

(b) The $\phi = 0$ analysis gives approximately correct factors of safety in all the cases analyzed when both fill strength and effect of tension cracks are taken into account.

(c) The factors of safety given by effective stress analysis appear to be too high.

The remedial measures subsequently carried out for the failure region are described.

Introduction

It is not uncommon for geotechnical problems to arise in the construction of highways in Northern Ontario where a varved clay belt stretches between Cochrane and New Liskeard. An embankment failure at Earlton was reported by Milligan, Soderman and Rutka,(1), and the foundation failure of a silo, near New Liskeard, was investigated by Eden and Bozozuk (2). While these papers, among others, have thrown some light on the validity of the conventional methods of dealing with varved clays, many problems, both fundamental and practical, remain unsolved. Several studies of varved clays, with special emphasis on field behaviour, have therefore been carried out in connection with the design and construction of several important bridges and high embankments. This program included observation of pore-pressures due to pile driving described elsewhere by Lo and Stermac (3), settlement and pore-pressure changes under embankment loading and increase of bearing capacity of friction piles with time. This paper describes another study in this series and reports the investigation and analyses of an embankment failure during construction.

Description of Failure

The site is located on Highway 11 approximately one mile north-west of New Liskeard, Ontario, where an embankment 4,000 ft. in length was being constructed between Highway 65 and the Wabi River, as part of the Tri-Town Bypass. A plan of the site is shown in Fig. 1.

Construction of the embankment between station 800 + 00 and 811 + 00, near the Ontario Northland Railway Crossing, began in the early part of June, 1963, (Fig. 2). At that time, a detour on the north side of the embankment was completed. The fill material was granular, with a high percentage of pebbles and boulders placed in layers 2 to 3 ft. thick, running from toe to toe. Compaction was achieved by an 8-ton pneumatic roller.

A new lift of fill material was being placed on June 27th, starting from station 808 + 00 towards decreasing chainages. When the lift reached station 804 + 50 at 4:00 p.m., the northern half of the embankment between these two stations suddenly failed. The slide took place without any warning and no cracks were observed prior to failure. Movement was completed in a matter of one minute.

The length of the fill involved was approximately 350 ft. and the height of the fill within the region of failure was 18.5 ft., decreasing to 13 ft. at the eastern extremity of the slide. The back of the slide ran almost parallel to and at a short distance from the centreline of the embankment, as shown in the plan in Fig. 2. The fill subsided vertically for a distance of 12 to 14 ft. (Figs. 3 and 4). The soil was pushed up at the toe and was badly cracked (Fig. 5). Some of these cleavages were as deep as several feet, as seen in Fig. 6. A culvert newly installed, was also forced up by the slide. The geometry of the slide approaches a classical case of plane strain condition.

Investigation of the Slide

The investigation of the slide was started four days after the failure. Four boreholes were put down at the positions shown in Fig. 2. In borehole 10-A, within the failure area, continuous 2-in. Shelby samples were taken. In borehole 10-B, at a distance of 5 ft. away, vane tests were performed at 2-ft. intervals in order to compare results of laboratory and field tests at the same elevation. Borehole 12 was put down on the stable side of the embankment and borehole 13 was drilled at the west extremity outside the slide. Previous to this investigation, continuous 3-in. piston samples were obtained at the location of the instrumentation for pore-pressure and settlement measurements (station 802 + 50). These boreholes outside the failure zone furnish information on the horizontal variation of the properties of the varved clay deposit.

Soil Conditions at Failure Site

The subsoil at the site consists of a layer of laminated silty clay approximately 8 ft. thick lying on an extensive varved clay stratum extending to 155 ft. below the ground surface. Below this depth there is a 1-ft. layer of very dense sand and gravel overlying limestone bedrock.

Samples recovered from the boreholes were carefully studied in the laboratory. Wherever possible, Atterberg limits and moisture contents were determined on individual lamina. The undrained strengths were determined both by unconfined compression and unconsolidated undrained triaxial tests, with special reference to the mechanism of failure, the percentage of silt and clay layers in the specimen and the failure zone as well as the location of the failure plane with respect to the orientation of the varves.

The silty clay stratum is desiccated, with brown pockets showing the effects of oxidation. At the natural state, the samples exhibit no lamination. On drying, however, a faintly laminated structure is evident. The thickness of the laminae is of the order of 1/32 to 1/16 in. The liquid limit varies from 43 per cent to 53 per cent, with an average of 47 per cent, while the plastic limit ranges from 20 per cent to 23 per cent, averaging 21 per cent. The moisture contents lie between 21 per cent and 39 per cent.

Underlying the desiccated laminated clay stratum is an extensive deposit of varved clay whose upper boundary varies generally between El. 618 and 620 ft. The dark layers are composed of grey clay of high plasticity or occasionally silty clay, and the light layers are clayey silt. The thickness of the individual layers varies from 1/32 to 2 in. but there appears to be no definite trend towards increase in layer thickness with depth.

The orientation of the varves is mainly horizontal in boreholes 12 and 13. In borehole 10, however, varves with an angle of inclination as high as 45 degrees to the horizontal were observed at depths of 10 to 20 ft. Some typical specimens of the varved clay are shown in Figs. 7 to 11. Detailed descriptions of each sample and the results of field and laboratory tests are contained in Tables 1 to 3.

The liquid limits of the dark layers vary between 60 and 73 per cent, while the plastic limits range from 21 to 34 per cent. In the light layers, the ranges of the liquid and plastic limits are 28 to 47 per cent and 19 to 26 per cent respectively. With a few exceptions, the moisture contents of the varves are close to the liquid limit, exhibiting a trend of decreasing with depth. The values of the liquidity index of each layer at the same depth do not seem to be significantly different. The activity of the dark layers and light layers are 0.5 and 0.3 respectively. The sensitivity of the bulk samples was found to range from 4 to 20, averaging 10 from field vane tests.

A mineralogical study of the deposit was reported by Soderman and Quigley (4). It is significant to note that both montmorillonite and vermiculite are present in the dark layers. Consolidation tests performed on the clay layers showed that the deposit is slightly overconsolidated. A profile of the consolidation history of the deposit is shown in Fig. 12.

Detailed results of the laboratory unconfined and unconsolidated undrained triaxial tests are shown in Tables 1 to 3. A study of the experimental data indicates that:

(a) The mode of failure is not determined by the type of test,

(b) The mechanism of failure, whether by bulging or shearing on a plane depends on both the thickness of the individual layers and the orientation of the varves.

If the layers are arbitrarily designated as "thick" for thickness exceeding 1/4 in., and as "thin" if they are less than 1/4 in., the samples may be divided into the following categories according to the manner of failing:

1. For samples composed of horizontal thin layers, shear planes develop across the varves (14 specimens).

2. For samples composed of thick clay laminae and thin silt laminae, shear planes develop across the varves, independent of orientation (12 specimens).

3. For samples composed of horizontal thick laminae, plastic failure occurs (4 specimens).

4. For samples composed of inclined thick laminae, shear planes develop along the clay laminae (6 specimens).

It is emphasized that the above statements are not meant to be conclusions but rather, observations for this particular deposit of varved clay. It is important to note that the majority of the samples failed in shear. For the four samples that failed by bulging, no squeezing or spreading of individual layers was noted. This is consistent with the result that the liquidity indices for the individual layers are of the same order of magnitude. The fact that the type of failure in this varved clay deposit does not differ from that of homogeneous clay strata suggests that the conventional methods of strength determination and stability analysis discussed by Tschebotarioff and Bayliss (5) may be applicable.

A detailed comparison between the undrained strengths measured by in-situ vane tests and laboratory tests, at the same elevation in Borehole 10, shows that the strength determined in the laboratory is on the average, 20 per cent lower. The general strength profiles of the different types of tests, however, are in much better agreement in Boreholes 12 and 13, except at depths greater than 40 ft. These observations do not differ significantly from those generally experienced in homogeneous clay deposits.

For convenience of discussion on the strength properties, the clay deposit is arbitrarily divided into three layers as shown in Table 4. It is clear from Table 4 that the average strengths in the three layers of boreholes 10 and 13 agree closely, while the strengths are definitely, though only slightly, higher on the south side of the embankment (borehole 12). The same conclusion may be reached by studying Figs. 13 to 15.

For the purpose of stability calculations, therefore, undrained strengths of 800, 320 and 460 p.s.f. for layers 1, 2 and 3 will be used for the failure region. For the south side, the strengths used will be 1,000, 360 and 570 p.s.f.

Soil Conditions at Station 802 + 50

Details of soil properties at the location of the instrumentation will not be dealt with in detail, but the results of field and laboratory tests of a typical borehole are shown in Fig. 16. It may be seen that the soil conditions are similar to those at the site of the failure. The average undrained strengths for the soil strata arbitrarily divided as above will be 800, 340 and 500 p.s.f. for layer 1 (10 ft.), layer 2 (8 ft.) and layer 3, respectively.

Observations on the Failure Zone

In an attempt to detect the failure zone, careful examination was made on each sample from borehole 10. At a depth of 11 ft. 3 in. to 12 ft. 3 in., a zone of distorted and disturbed material was observed, (Fig. 9). An unconfined compression test performed on a specimen taken within this zone indicated a shear strength of 150 p.s.f., the lowest value observed in all tests carried out, and a value less than half the average strength of layer 2. It is very probable, therefore, that this sample forms part of the failure surface of the slide.

Triaxial Tests with Pore-Pressure Measurements

Three series of consolidated undrained triaxial tests with pore-pressure measurements were performed on samples taken from boreholes 10-A, 12 and 13. The results of the tests are summarized in Table 5.

The tests were carried out at a strain rate of 1 per cent per hour at which speed the pore pressures in the constituent layers of the specimen are probably equalized or differ only slightly. The results, however, show some scatter. The factors which control the effective stress parameters are, among others, the orientation of the varves, the thickness or relative thickness of the laminae and the mechanism of failure. For example, the varves of the samples in Series 1 are inclined, in Series 2 are contorted and in Series 3 are horizontal. A detailed discussion of these factors is outside the scope of this paper. The values of effective cohesion and angle of shearing resistance which are considered to be reasonably representative for the stratum in which failure occurred are $C' = 100$ p.s.f., $\phi' = 25^{o}$. But, as will be seen later, stability condition is also investigated for the case assuming $C' = 0$.

Stability Analysis

Conventional stability analyses were carried out both in terms of total ($\phi = 0$) and effective stresses. In order to investigate the validity of these methods more fully, both the failure and the stable sections of the embankment were analyzed.

For the total stress analysis, calculations were performed for the following sections:

(a) Failure section at station 806 + 00 which is located approximately at the centreline of the slide (Fig. 17).

(b) The stable side (south) of the embankment at station 806 + 00 (Fig. 18).

(c) The instrumented section at station 802 + 50 (Fig. 19).

Effective stress analyses were carried out for sections (a) and (c) only.

The strength properties applicable to each location have been discussed in the previous section. For the fill material, it is assumed that $C' = 0$ and $\phi' = 35^{o}$.

In all stability analyses, the assumptions of full mobilization and virtually no fill strength, as well as depth of tension cracks (d) were alternated so as to obtain the minimum factors of safety for the following conditions:

(1)	$d = 0$	, with no fill strength	(F_{oo})
(2)	$d = 0$	, with full fill strength	(F_{of})
(3)	$d = 12$ ft.,	with no fill strength	(F_{do})
(4)	$d = 12$ ft.,	with full fill strength	(F_{df})

In addition, calculations were carried out for the probable slip surface at station 806 + 00.

From the external geometry of the failure and the probable failure zone found from a sample at borehole 10-A, it has been possible to construct the slip surface with a reasonable degree of accuracy.

Total Stress Analysis

The results of the total stress analyses for different sections using different assumptions of fill strength and depth of tension cracks as mentioned previously are shown in Table 6.

It will be seen from Table 6 that the effect of assuming no tension crack is to reduce the factor of safety by 4 to 8 per cent, while the fill strength accounts for 6 to 10 per cent of the value of the factor of safety. All the factors of safety obtained except F_{oo} which involves the severe assumptions of zero tension crack and no fill strength are consistent with the observed facts with regard to the stability conditions of the embankment at each location.

The positions of the critical and the actual failure arc at station 806 + 00 are shown in Fig. 17. The two circles do not coincide and the factor of safety calculated from the failure arc is higher than the minimum factor of safety by approximately 10 per cent. This inconsistency is inherent in $\phi = 0$ analysis, Skempton (6).

The results of the analyses show that using the average strength of all field and laboratory tests, and taking tension crack and fill strength into account, the empirical use of $\phi = 0$ analysis leads to an accuracy within 15 per cent on the safe side of the factor of safety. If the average undrained strength determined by in-situ vane test in borehole 10 is used, the factor of safety similarly calculated is close to unity. Similar results have been obtained by Eden and Bozozuk (2) and Milligan et al (1). However, there appears to be no compelling theoretical reason as to why in-situ vane strength should be employed (Table 4).

Effective Stress Analysis

The factors which affect the computed factor of safety are, as in the case of total stress analysis, the depth of tension crack and mobilization of fill strength. In addition, the pore-water pressure distribution and the shear strength parameters in terms of effective stress, have to be defined. The pore pressures used in the analysis are those measured at station 802 + 50 where the height of the embankment is the same as at the slide and the soil conditions are very similar. Details of the measurement and analysis of pore-pressures and settlements will be reported elsewhere. It is believed that these pore pressures should be very close to those existing prior to failure. The shear strength parameters have been discussed in a previous section and the values used in the analyses are $C' = 100$ p.s.f. $\phi' = 25^{o}$.

Calculations show that the factor of safety is increased by approximately 5 per cent if tension crack of 12 ft. deep is assumed while full fill strength accounts for about 20 per cent of the factor of safety. By coincidence the difference in the factor of safety between $C' = 0$ and $C' = 100$ is also approximately 20 per cent. Therefore, factors of safety obtained by taking $C' = 100$ and no fill strength are approximately the same as assuming $C' = 0$ and full fill strength.

If tension cracks are taken into account and fill strength neglected, then the factor of safety obtained for the failure is 1.25. Using the probable failure arc, the calculated factor of safety is 1.33. If $C' = 0$, the figures drop to 1.03 and 1.11, respectively.

Analyses carried out at station 802 + 50 yield a factor of safety for $C' = 100$ greater than 1.4 and for $C' = 0$ greater than 1.3.

It is seen, therefore, that factors of safety given by the effective stress analysis appear to be on the unsafe side by more than 20 per cent unless the stringent assumptions of $C' = 0$ and no fill strength are used.

Remedial Measures

The embankment was reconstructed after failure to its former grade by adding a berm of half the height of the embankment. In design of the berm, a factor of safety of 1.3 was used. The rebuilt embankment has so far been quite stable.

Conclusions

As a result of this study, the following conclusions may be drawn with regard to this particular case:

(a) The amount of discrepancy between in-situ vane and laboratory undrained tests is of the same order of magnitude as those experienced in homogeneous clay deposits.

(b) The conventional $\phi = 0$ analysis gives approximately correct factors of safety when both fill strength and tension cracks are taken into account.

(c) The effective stress analysis yields factors of safety on the unsafe side by more than 20 per cent unless the stringent assumptions of no fill strength and zero cohesion intercept are made.

It is emphasized that the treatment expounded in the paper is entirely of an empirical nature and extrapolation of the conclusions reached herein should be made only with extreme care. Lo has shown elsewhere that even for homogeneous but anisotropic clays, the shear strength determined by the vane and undrained tests would depend on the degree of anisotropy (7). There is also a lack of both theoretical or experimental methods of studying the stress-strain pore pressure properties of the individual layers. A more logical approach to the problems of a layered soil cannot be adopted until such studies are carried out.

Acknowledgement

The field work reported in this paper was carried out by Mr. A. Barsvary, Project Foundation Engineer.

This paper is presented by permission of Mr. H.W. Adcock, Assistant Deputy Minister (Engineering), Department of Highways, Ontario.

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TABLE 1 - DETAILED RESULTS OF LABORATORY AND FIELD TESTS - BOREHOLE 10A

Sample No.	Depth (ft.)	Description	w_L (%)	w_p (%)	w (%)	I_L	U.C. (PSF)	ε_f (%)	Q_u (PSF)	ε_f (%)	Mode of Failure	Vane Strength (PSF)
1	2.0 - 3.5	Silty Clay, oxidized pockets. Mottled.	48.0	23.4	30.2	0.28	1,093	20	826	8	SH. A.	
2	4.0 - 5.5	H. Cl.= 1/32 - 1/16 in. Si.= 1/32 - 1/16 in.	46.2	21.5	36.4	0.60	720	7	591	10	SH. A.	
3	5.8 - 7.5	H. Cl.= 1/16 - 1/8 in. Si.= 1/32 - 1/16 in.	48.0	22.4	36.6	0.91	350	20	308	13	U.C. = P. Q_u = SH. A.	520
4	7.8 - 9.5	H. Cl.= 1/8 - 1/4 in. Si.= 1/32 - 1/16	72.9	25.8	69.0 45.6	0.92	161 250	20 5	30[illegible]	11	U.C. = Undefined Q_u = SH. A.	360
5	9.8 - 11.5	I. Cl.= 1/2 in. Si.= 1/32 - 1/8 in.	59.8	22.8	65.8 73.8 77.6 78.2	-	315 344 282	2.5 6 4.5			SH. A.	380
6	11.8 - 13.5	I. Cl.= 1/2 in. Si.= 1/4 in.			65.6 68.8 32.7 29.2		164 (Failure Zone)	20	415	10	U.C. = P. Q_u = Undefined	360
7	13.8 - 15.5	I. Cl.= 1.0 - 1.5 in. Si.= 1/4 - 1/2 in.	71.0	23.6	54.5 81.7	0.65	241	20	278	7	U.C. = SH. - Undefined Q_u = SH. Cl.	400
8	15.8 - 17.5	I. Cl.= 1.0 in. Si.= 1/2 - 1 in.	64.3 29.6	25.5 20.6	77.8 28.1	1.34 0.83	207	10	374	10	SH. Cl.	360
9	17.8 - 19.5	I. Cl.= 1.0 - 1.5 in. Si.= 1/2 in.	65.6	25.9	86.9 28.0	1.53	312	10	399	15	SH. Cl.	400
10	19.8 - 21.5	H.& Ir. - Cl.= 3/8 - 1/2 in. Si.= 1/4 - 3/8 in.	71.7 29.4	28.5 21.6	74.2 30.6	1.06 1.23	392	20	510	19	P.	620
11	21.8 - 23.5	H. Cl.= 1.5 - 2.0 in. Si.= 1.5 - 2.0 in.	29.3	21.3	75.7 32.2 31.0	1.21	362 315	20	514	17	P.	400
12	23.8 - 25.5	Ir. No details.	75.4 30.2	30.4 22.7	77.4 [illegible] 68.7	1.04	282	20	550	15	U.C. = SH. former slip Q_u = -	360
13	25.8 - 27.5	" " "	42.6	26.3			247 349	20 7	479	7	-	520
14	27.8 - 29.5	" " "	30.8	21.1			459 669	20 20	761	20	P.	480

LEGEND

Description

H. = Horizontal varves

I. = Inclined varves

Ir. = Irregular or contorted

Cl. = Clay or silty clay

Si. = Clayey Silt

Mode of Failure

SH. = Failure along a shear plane

P. = Bulging failure

A. = Failure plane across the varves

Cl. = Failure plane in clay layer

NOTE: U.C. = Undrained Strength from Unconfined Compression Tests.

Q_u = Undrained Strength from Unconsolidated Undrained Triaxial Tests.

TABLE 2 - DETAILED RESULTS OF LABORATORY AND FIELD TESTS - BOREHOLE 12

Sample No.	Depth (ft.)	Description	W_L (%)	W_p (%)	W (%)	I_L	U.C. (PSF)	ε_f (%)	C_u (PSF)	ε_f (%)	Mode of Failure	Vane Strength (PSF)
1	2.0 - 4.0	Silty Clay. Oxidized pockets. Mottled.	53.4	20.3	33.0	0.38			1338	20	SH. A.	
2	6.7 - 7.9	H. thin varves. Size not measured.	46.6 47.2	20.1 21.6	38.7 39.6	0.70 0.70	731	12	738	2.5	SH. A.	
	9.5											220
3	10.3 - 11.3	Cl. Ir. Silt pockets and seams.	71.8 64.7	30.0 28.7	57.7 62.1 29.5	0.66 0.91	418	3	445	4	SH. A.	
	13.5											320
4	14.1 - 15.9	Cl.	60.2 60.2 56.0	23.3 23.8 22.2	68.0 63.2 56.0 56.8	1.21 1.08 1.07			326	2	SH. Cl.	
	17.5											360
5	18.1 - 19.9	I.& Ir. Cl.= [illegible] in. Sl.= [illegible] in.	65.2 68.7 37.4	24.1 26.1 20.5	69.5 67.0 33.2	1.10 0.96 0.75	373	14	462	16	U.C. = SH. Cl. C_u = -	
	21.5											440
6	22.4 - 23.7	H. & Ir.	31.1	20.9	26.8 66.8 30.9	0.98	594	14	641	20	U.C. = SH. A. C_u = P.	
	25.5											460
7	26.5 - 27.6	H. Cl. = 1/32 - 1/8 in. Sl. = 1/4 - 4 in.	30.4 29.4	20.7 20.4	30.3 31.0	0.99 1.18	716	19	623	20	U.C. = SH. A. C_u = P.	
	29.5											640

LEGEND - (Refer to Table 1)

TABLE 3 - DETAILED RESULTS OF LABORATORY AND FIELD TEST - BOREHOLE 13

Sample No.	Depth (ft.)	Description	w_L (%)	w_p (%)	w (%)	I_L	U. C. (PSF)	ε_f (%)	q_u (PSF)	ε_f (%)	Mode of Failure	Vane (PSF)
1	2.0 - 4.0	Silty Clay. Oxidized pockets. Mottled.	48.2	20.5	30.6	0.37	1,330	17	1,220	10	SH. A.	
	6.0											600
2	6.5 - 8.5	H. Cl.= 1/16 - 1/2 in. Si.= 1/32 - 1/16in.	43.8	20.0	38.8	0.81	342	15	543	7	SH. A.	
	10.0											400
3	10.5 - 12.5	H. Cl.= 1/4 - 3/4 in. Si.= 1/32- 1/8 in.	66.1 42.4	23.1 20.2	61.5 40.0	0.89 0.89	240 247	13 6			SH. A.	
	14.0											340
4	14.5 - 16.5	H.& Ir. - Cl.= 1/4 - 1/2 in. Si.= 1/8 - 3/8 in.			77.8 28.4		210 365	15 13			SH. A.	
	18.0											240
5	18.5 - 20.5	H. Cl.= 1/8 - 1/2 in. Si.= 1/8 - 3/8 in.	69.4 28.0	24.3 22.4	77.8 28.2	1.19 1.03			420	13	Undefined	
	22.0											400
6	22.5 - 24.5	I. Cl.= 3/4 - 2 in. Si.= 1/4 in.	44.3 31.9	28.9 19.8	42.4 75.0 58.2 75.8	0.66 0.69	421 326 611	9 14 20	529	16	SH. Cl.	
	26.0											480
7	26.5 - 28.5	Clayey Silt. Few H. Cl. 1/32 in.	29.2	19.7	30.7	1.16	940	20	458	20	P. No varves	
	30.0											560
8	31.5 - 33.5	Upper 8 in. Clayey Silt. Below that: H. Cl.= 1/2 - 1/8 in. Si.= 1/8 in.	31.0	19.2	32.0 60.5 57.4	1.09	350	3.3			SH. A.	
	35.0											560
9	36.0 - 38.0	H. Cl.= No information Si.= " "	60.4	21.6	62.8 56.2	1.06	537	8	539	2	SH. A.	
	40.5											500
10	42.0 - 44.0	H. Cl.= 1/8 - 3/16 in. Si.= 1/16 in.	68.8 66.5	25.5 24.2	62.0 61.5 66.0	0.85 0.89	560 580	5 5	515	4.3	SH. A.	
	45.5											860
11	46.5 - 48.0	H. Cl.= 1/4 in. Si.= 1/16 in.	72.2	34.6	61.7 64.8 64.5	0.81	571	8	779	3	SH. A.	
	49.5											840
12	50.0 - 52.0	H. Cl.= 7/16 - 11/16in. Si.= 1/16 - 5/16 in.	73.0 47.0	24.7 21.6	68.0 63.0 44.0 42.0	0.90 0.96	600 800	4 2	705	2.6	SH. A.	
	53.5											860
13	54.0 - 56.0	H. Cl.= 3/4 - 1.5 in. Si.= 1/8 - 1/4	66.4	25.7	52.4 51.3 51.8	0.66	334 565	6 6	762 560	10 5	SH. A.	
	57.5											860
14	58.0 - 60.0	H. Cl.= 1/2 - 3/4 in. Si.= 1/2 - 3/4 in.	64.5 28.0	24.6 22.2	63.0 62.0 25.0 25.0	0.94 0.19	700	-	572	11	P.	

LEGEND (Refer to Table 1)

TABLE 4 - SUMMARY OF UNDRAINED SHEAR STRENGTHS

Borehole No.	Layer	Depth Ft.	Vane	Average Undrained Strength p.s.f. Unconfined Compression	Unconsolidated Undrained	Weighted Average
10	(1)	0 - 8	910 (3)	720 (3)	600 (3)	740
	(2)	8 - 20	380 (6)	270 (8)	350 (5)	320
	(3)	20 - 30	490 (6)	390 (8)	370 (5)	470
12	(1)	0 - 8	1120 (1)	1050 (2)	730 (1)	990
	(2)	8 - 20	300 (3)	390 (2)	410 (3)	360
	(3)	20 - 30	510 (3)	650 (2)	580 (2)	570
13	(1)	0 - 8	600 (1)	840 (2)	880 (1)	800
	(2)	8 - 20	330 (3)	270 (4)	420 (1)	310
	(3)	20 - 30	480 (3)	380 (2)	490 (2)	450

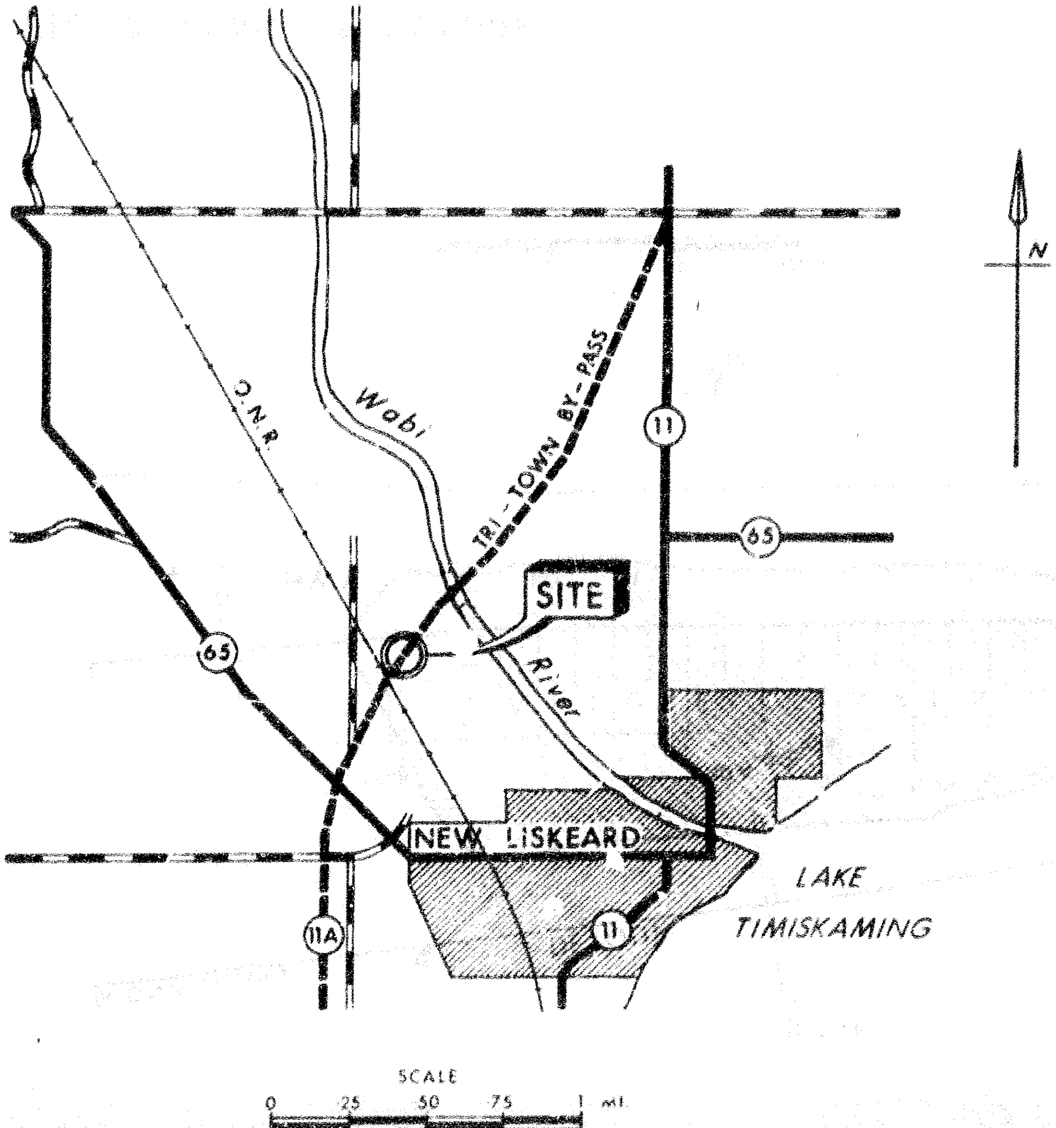
NOTE: ( ) Indicates number of tests.

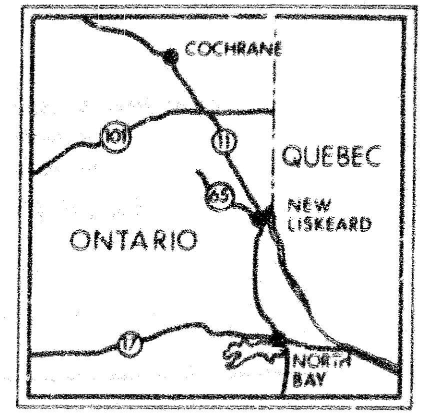
TABLE 5 - RESULTS OF TRIAXIAL TESTS

Series	Borehole No.	Depth Ft.	C' P.S.F.	Ø' Degrees
1	10A	12 - 16	80	27
2	12	14 - 16	240	22
3	13	14 - 20	230	24

TABLE 6 - RESULTS OF TOTAL STRESS ANALYSIS

Cross Section	Factor of Safety				C PSF
	F_{oo}	F_{of}	F_{do}	F_{df}	
806 + 00 Failure Arc	0.74 0.81	0.83 0.93	0.80 0.88	0.87 0.97	C_1 = 800 C_2 = 320
806 + 00 Stable Side	1.07	1.13	1.12	1.19	C_1 = 1,000 C_2 = 360
802 + 50 (Stable)	0.97	1.07	1.05	1.13	C_1 = 800 C_2 = 340





KEY PLAN

FIG. I: PLAN OF SITE

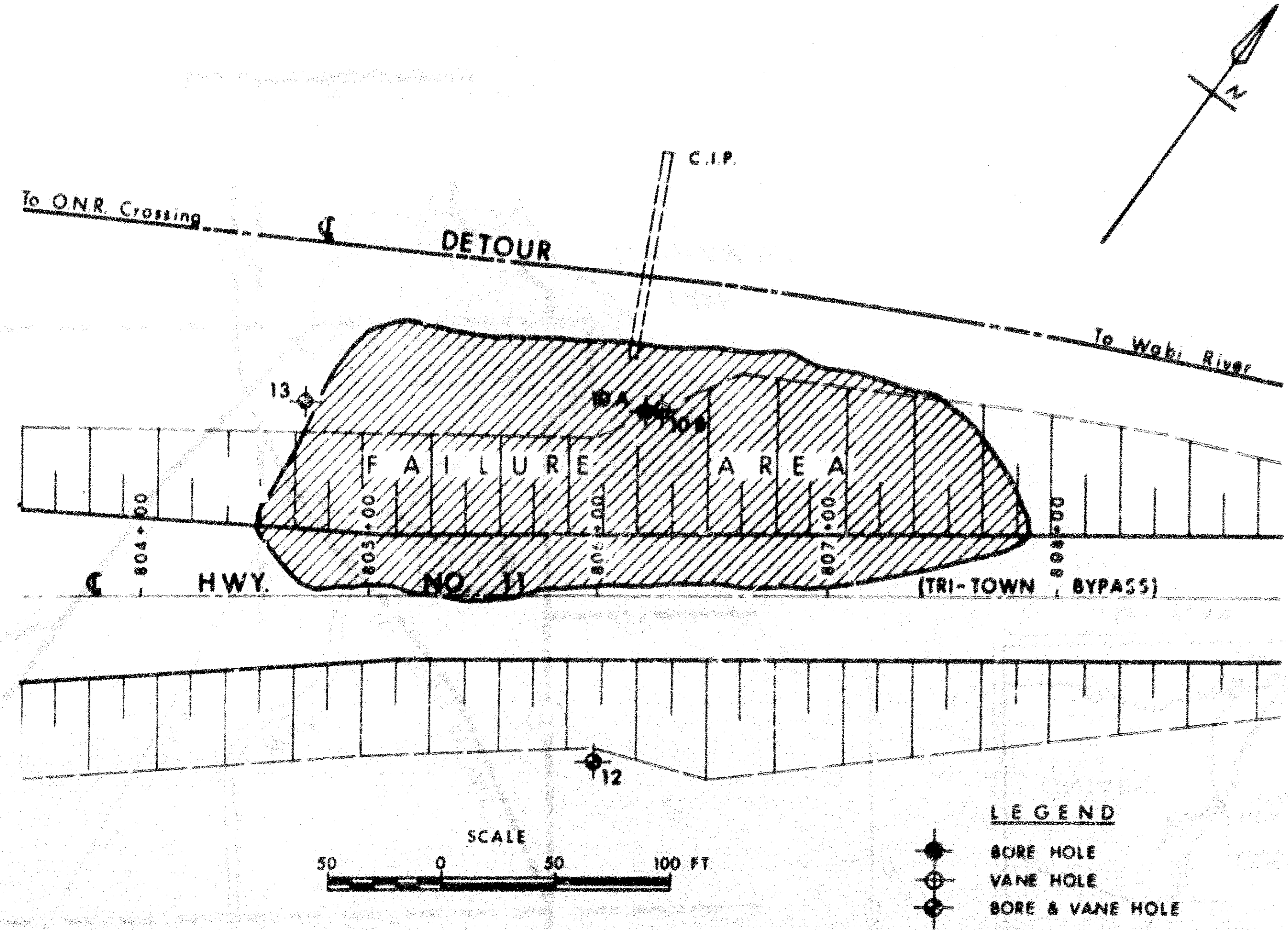


FIG. 2: PLAN OF FAILURE

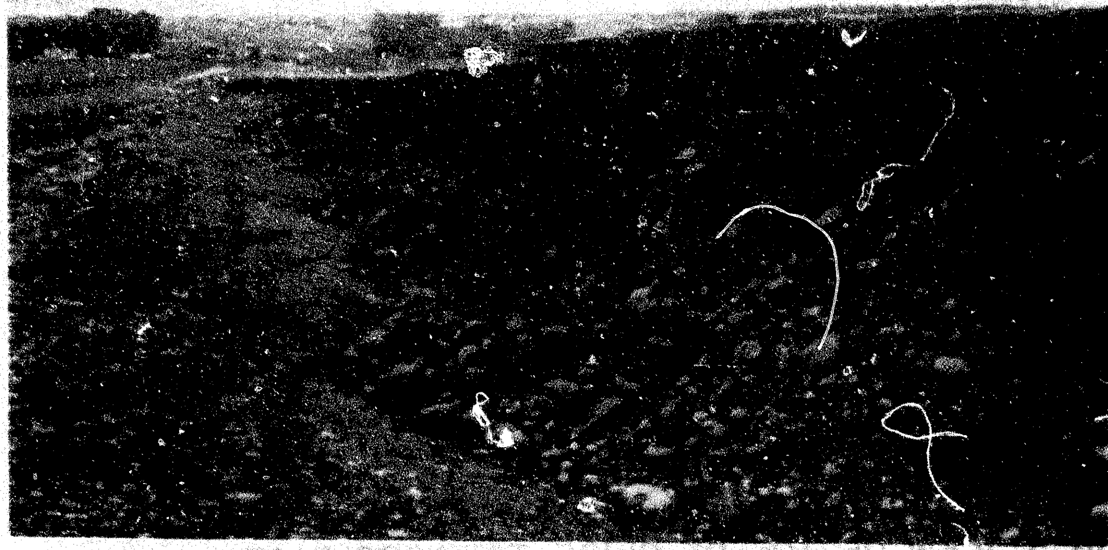


FIG. 3: VIEW OF FAILURE TOWARDS EAST

FIG. 4: VIEW OF EXPOSED PORTION OF EMBANKMENT AFTER FAILURE

FIG. 3: VIEW OF FAILURE TOWARDS EAST

FIG. 4: VIEW OF EXPOSED PORTION OF EMBANKMENT AFTER FAILURE

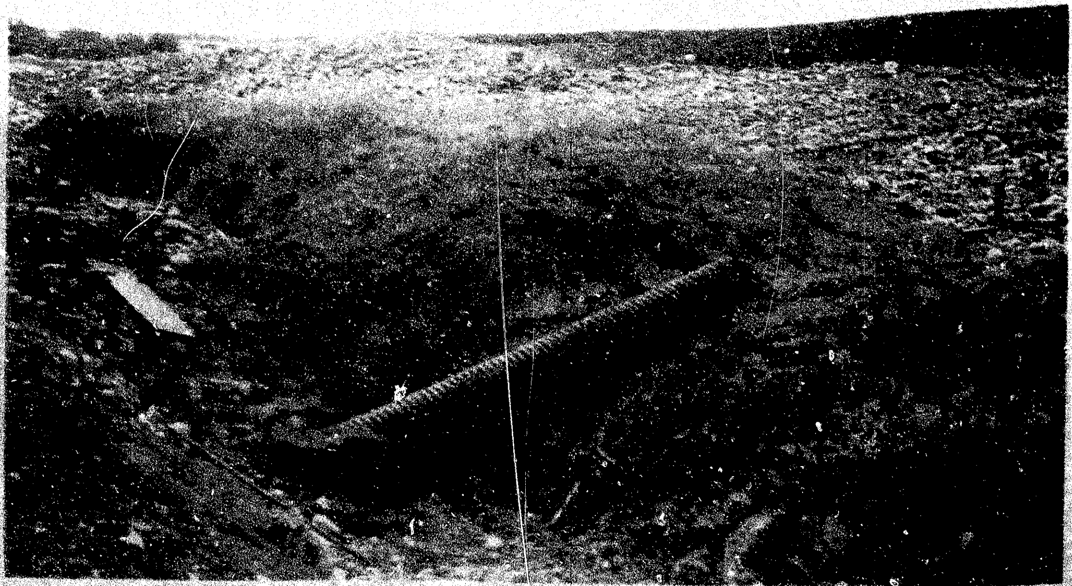


FIG. 5: VIEW OF TOE OF SLIDE TOWARDS EAST

FIG. 6: VIEW OF TOE OF SLIDE TOWARDS WEST

FIG. 5: VIEW OF TOE OF SLIDE TOWARDS EAST

FIG. 6: VIEW OF TOE OF SLIDE TOWARDS WEST

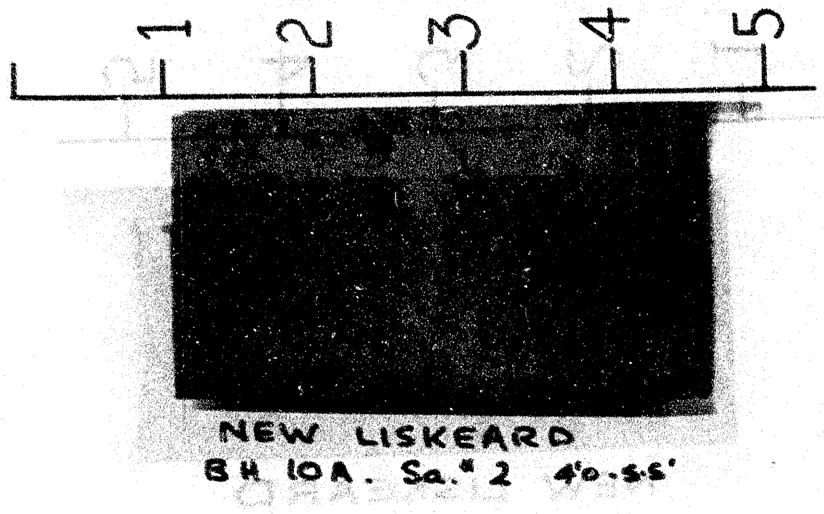


FIG. 7: A TYPICAL SAMPLE OF LAMINATED SILTY CLAY

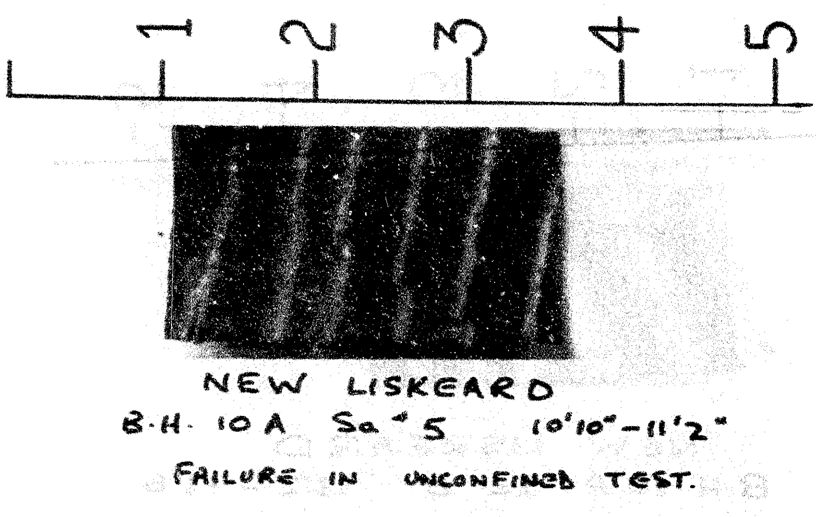


FIG. 8: A TYPICAL SAMPLE OF THE VARVED CLAY DEPOSIT

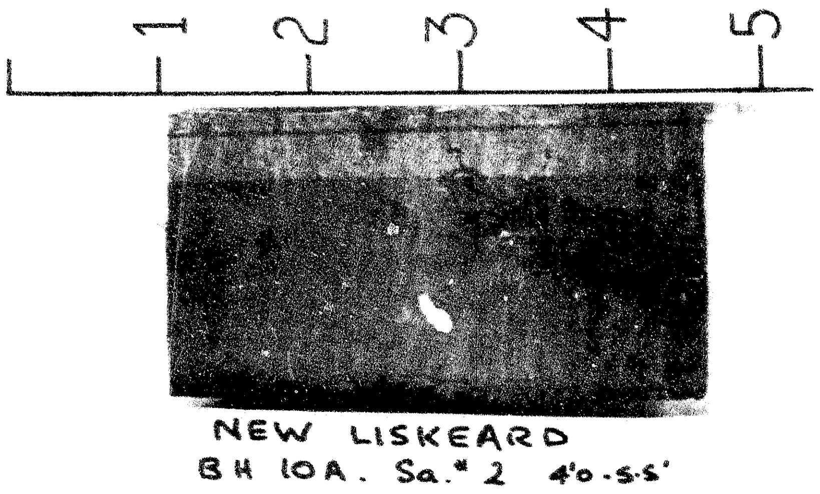


FIG. 7: A TYPICAL SAMPLE OF LAMINATED SILTY CLAY

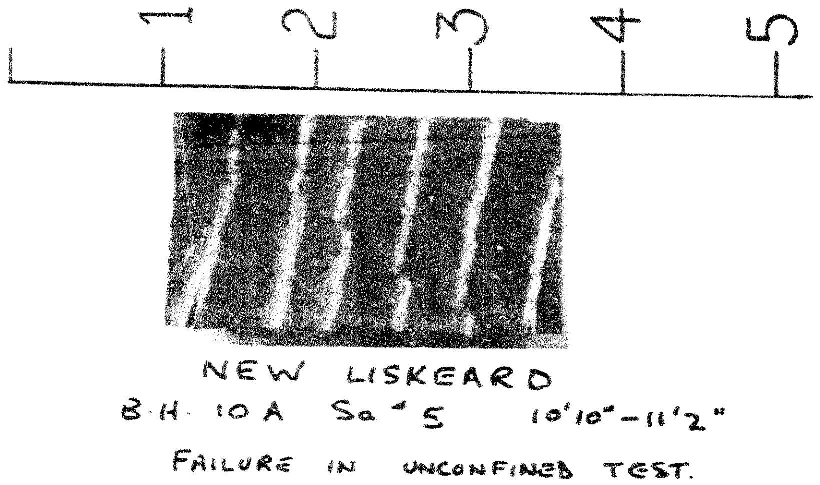


FIG. 8: A TYPICAL SAMPLE OF THE VARVED CLAY DEPOSIT

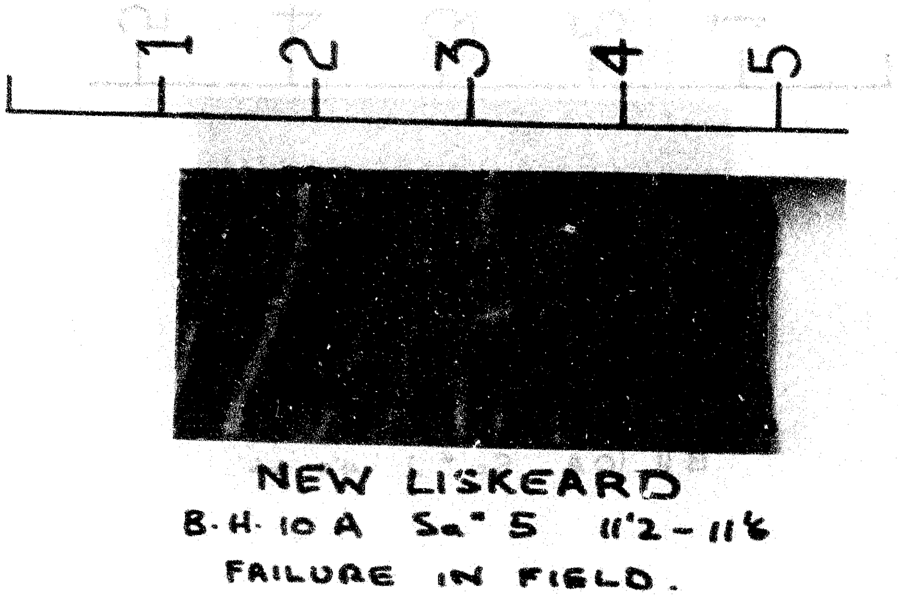


FIG. 9: PROBABLE FAILURE ZONE RECOVERED FROM BOREHOLE 10

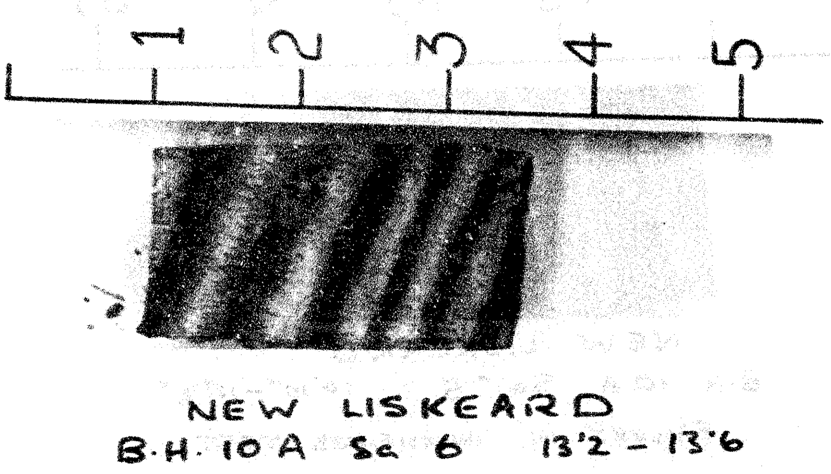


FIG. 10: A TYPICAL SAMPLE OF THE VARVED CLAY DEPOSIT

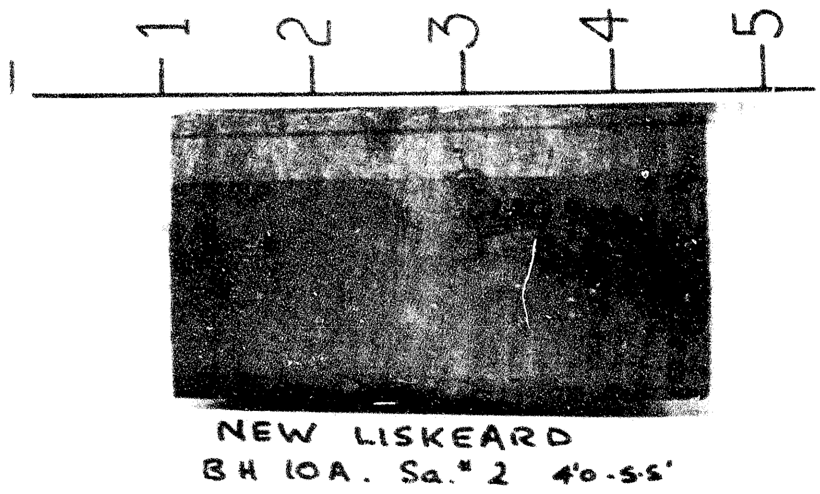


FIG. 7: A TYPICAL SAMPLE OF LAMINATED SILTY CLAY

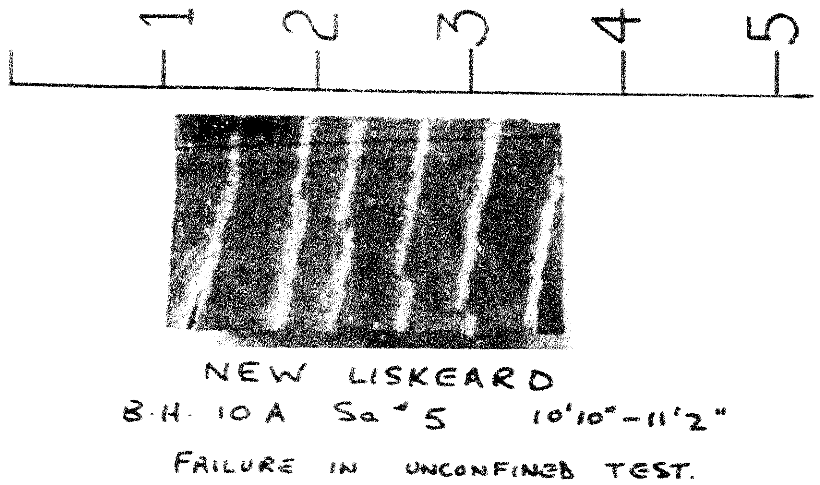


FIG. 8: A TYPICAL SAMPLE OF THE VARVED CLAY DEPOSIT

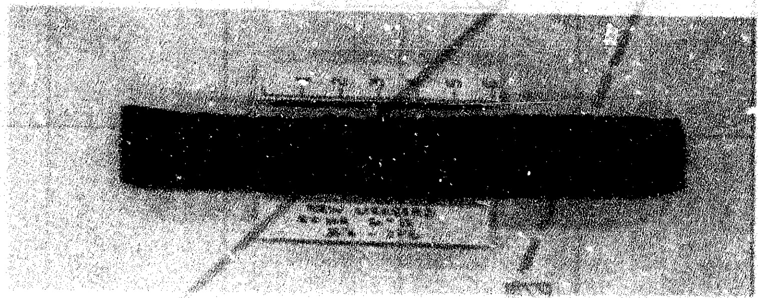


FIG. 11: A TYPICAL SAMPLE OF THE VARVED CLAY DEPOSIT

FIG. II: A TYPICAL SAMPLE OF THE VARVED CLAY DEPOSIT

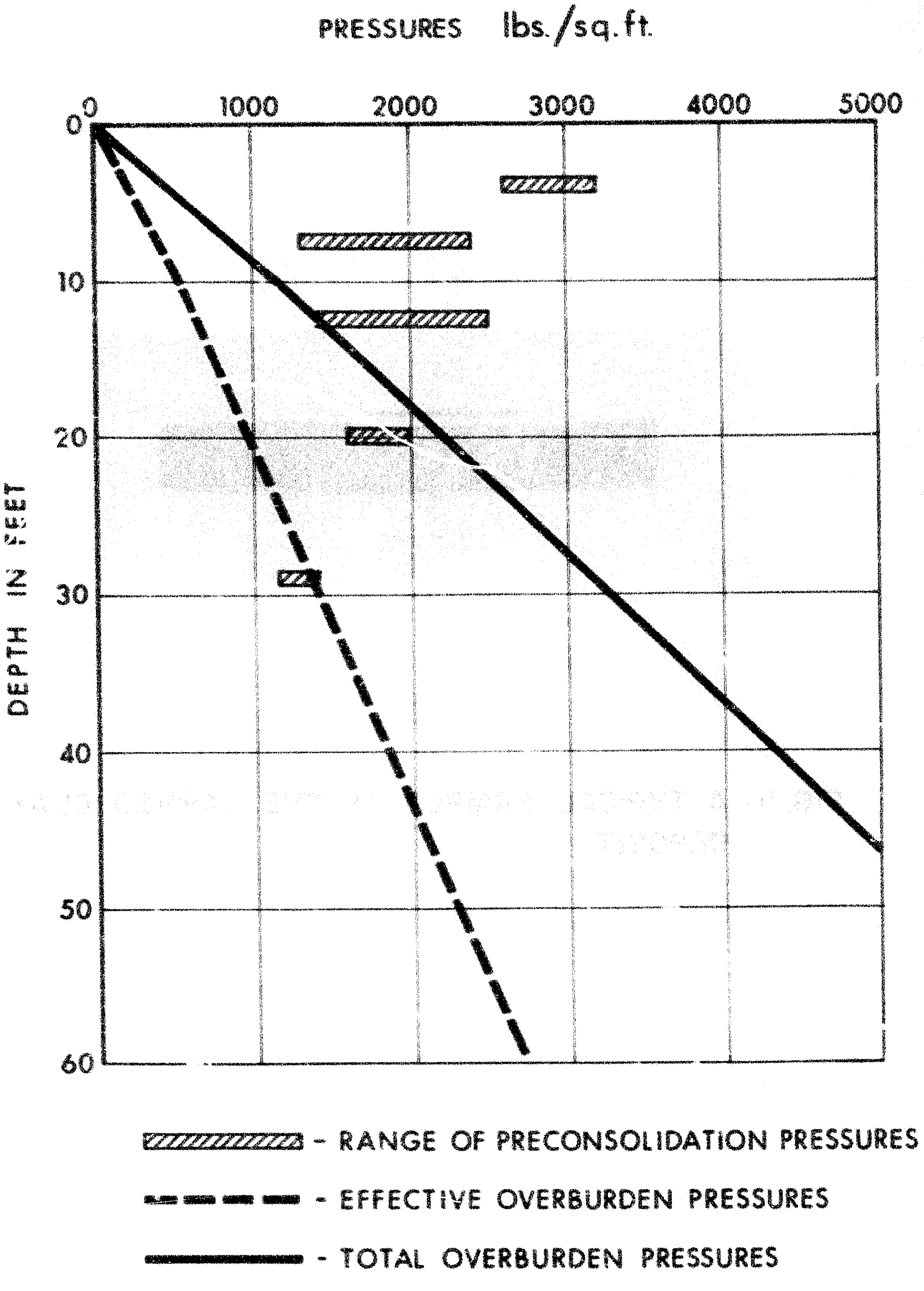


FIG. 12: PROFILE OF CONSOLIDATION HISTORY

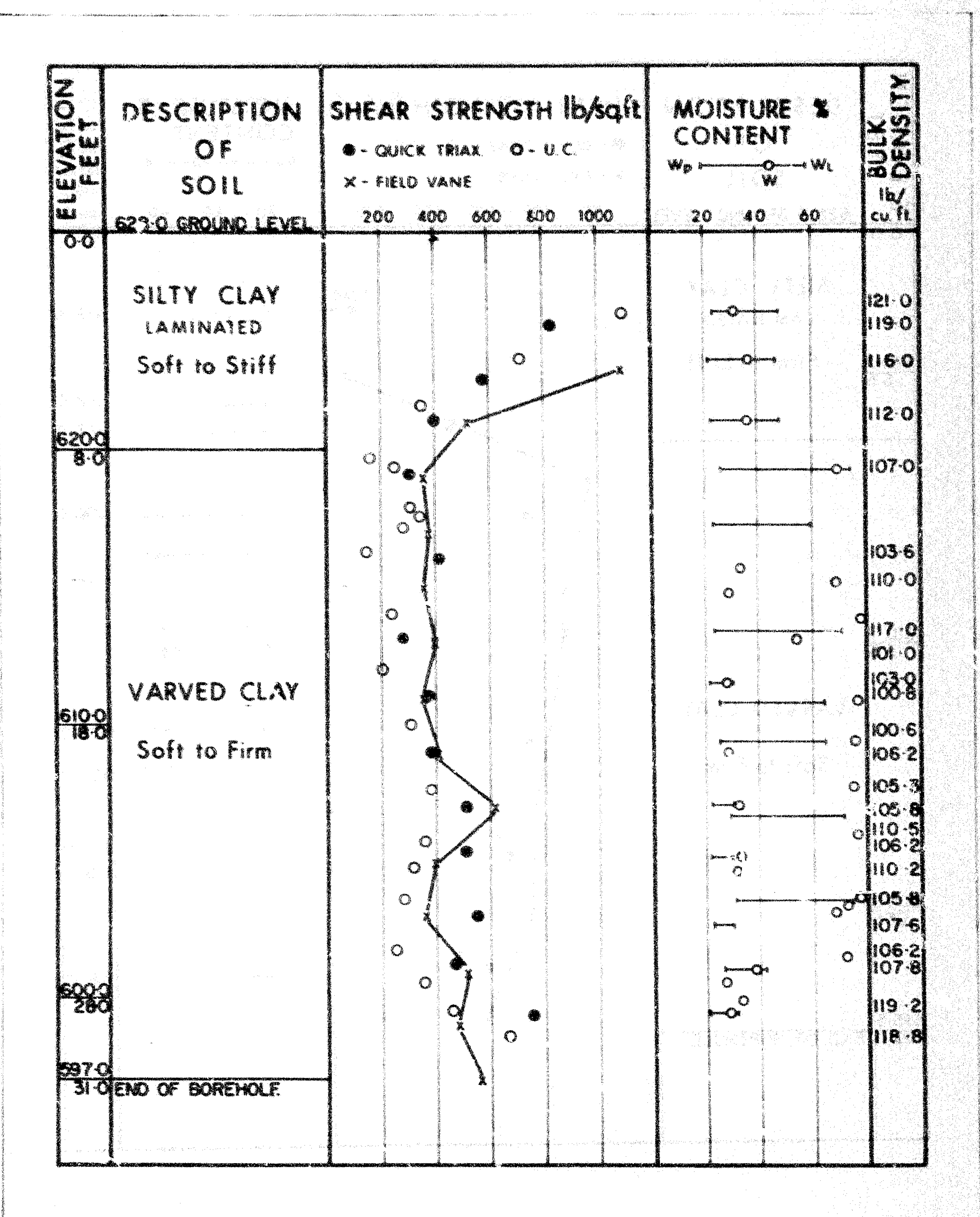


FIG. 13: RESULTS OF FIELD AND LABORATORY TESTS, BOREHOLES 10A AND 10B

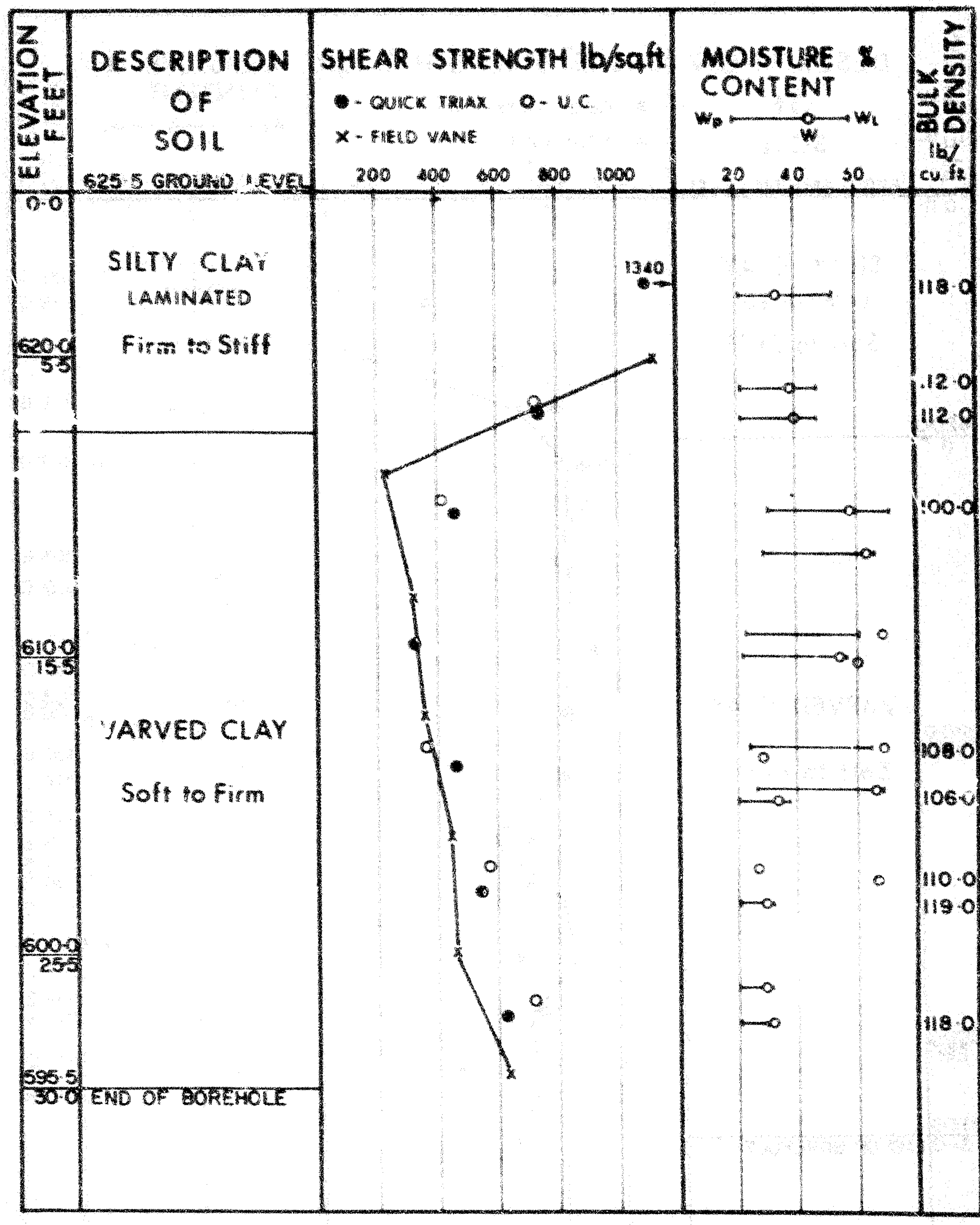


FIG. 14: RESULTS OF FIELD AND LABORATORY TESTS, BOREHOLE 12

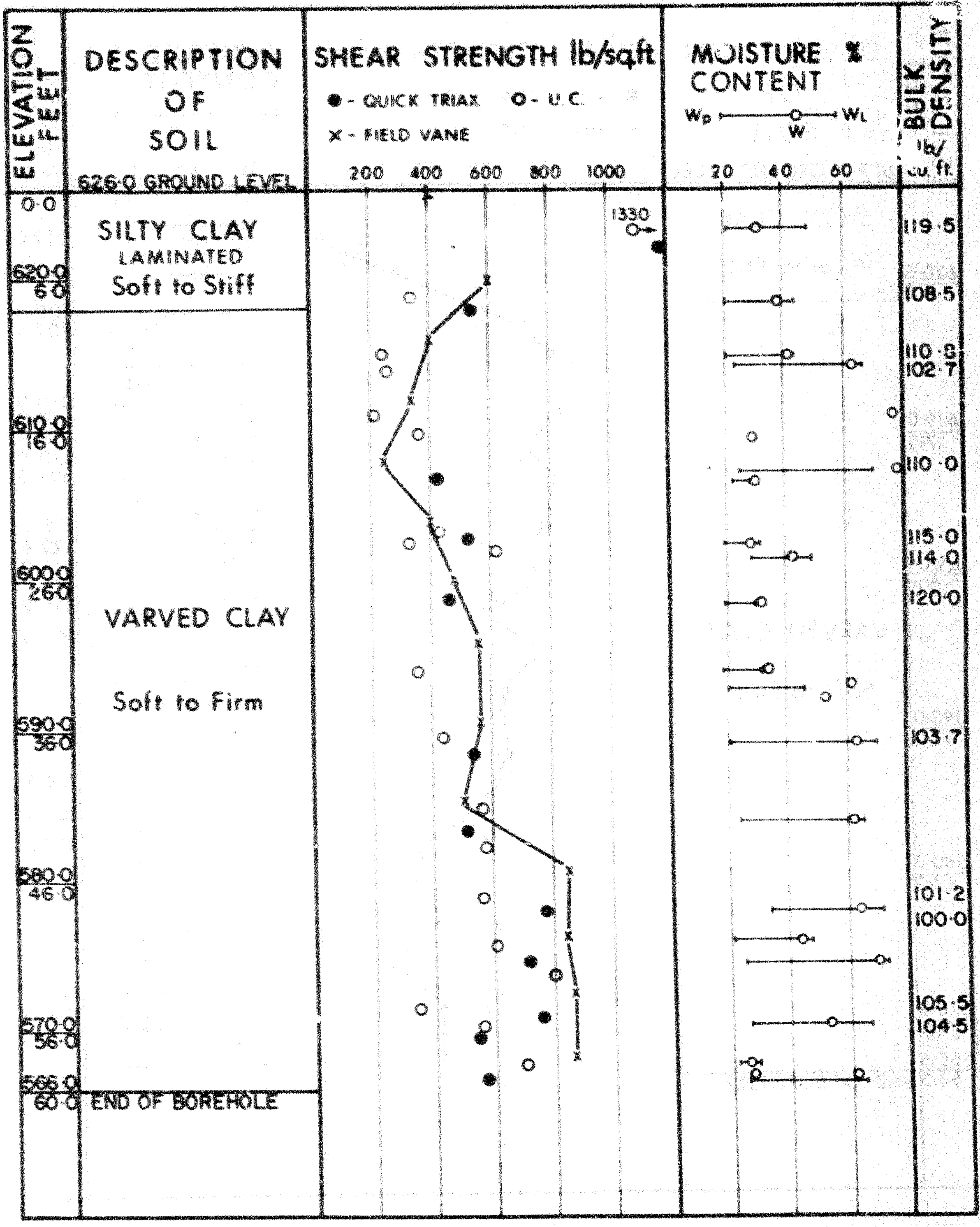


FIG. 15: RESULTS OF FIELD AND LABORATORY TESTS, BOREHOLE 13

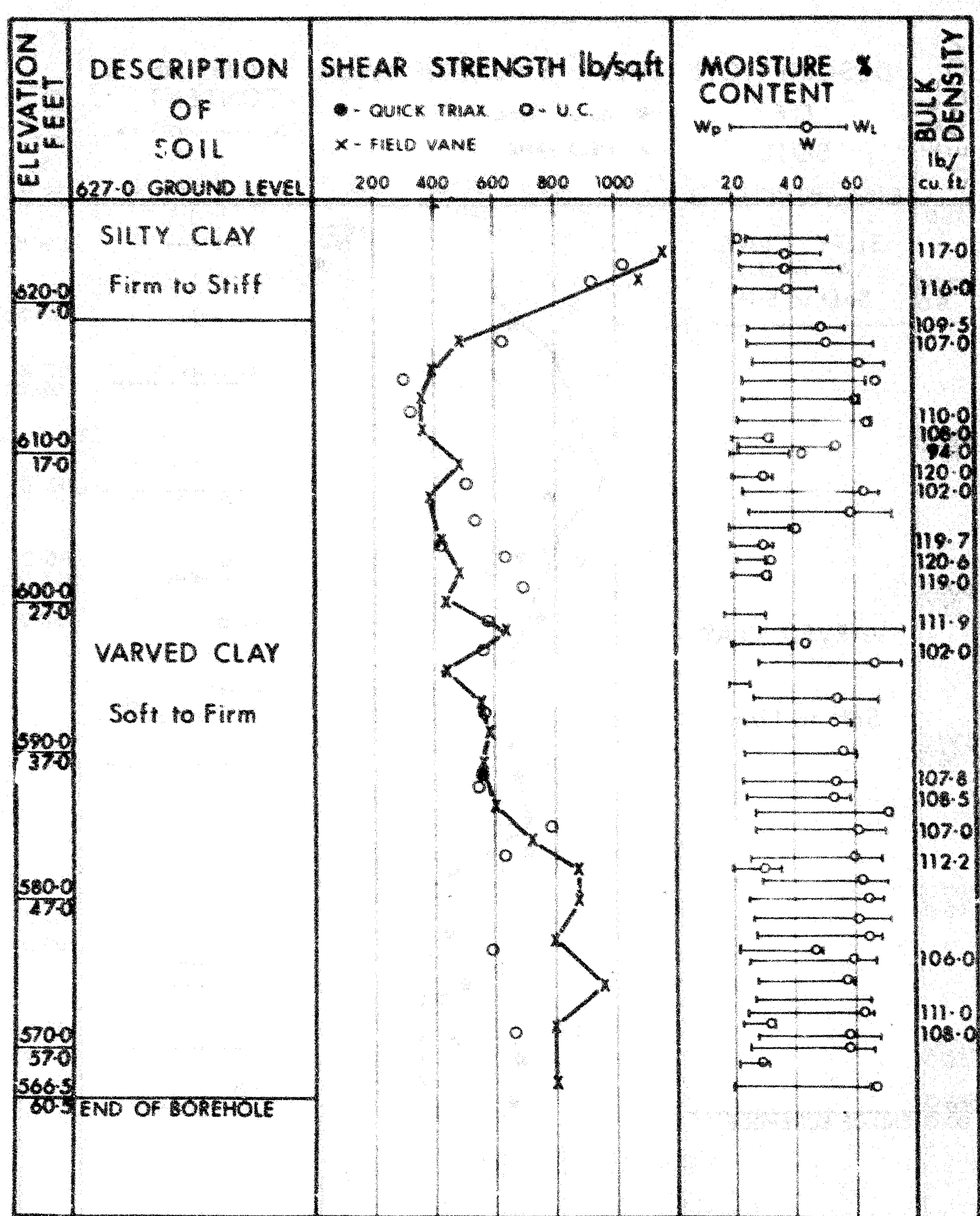
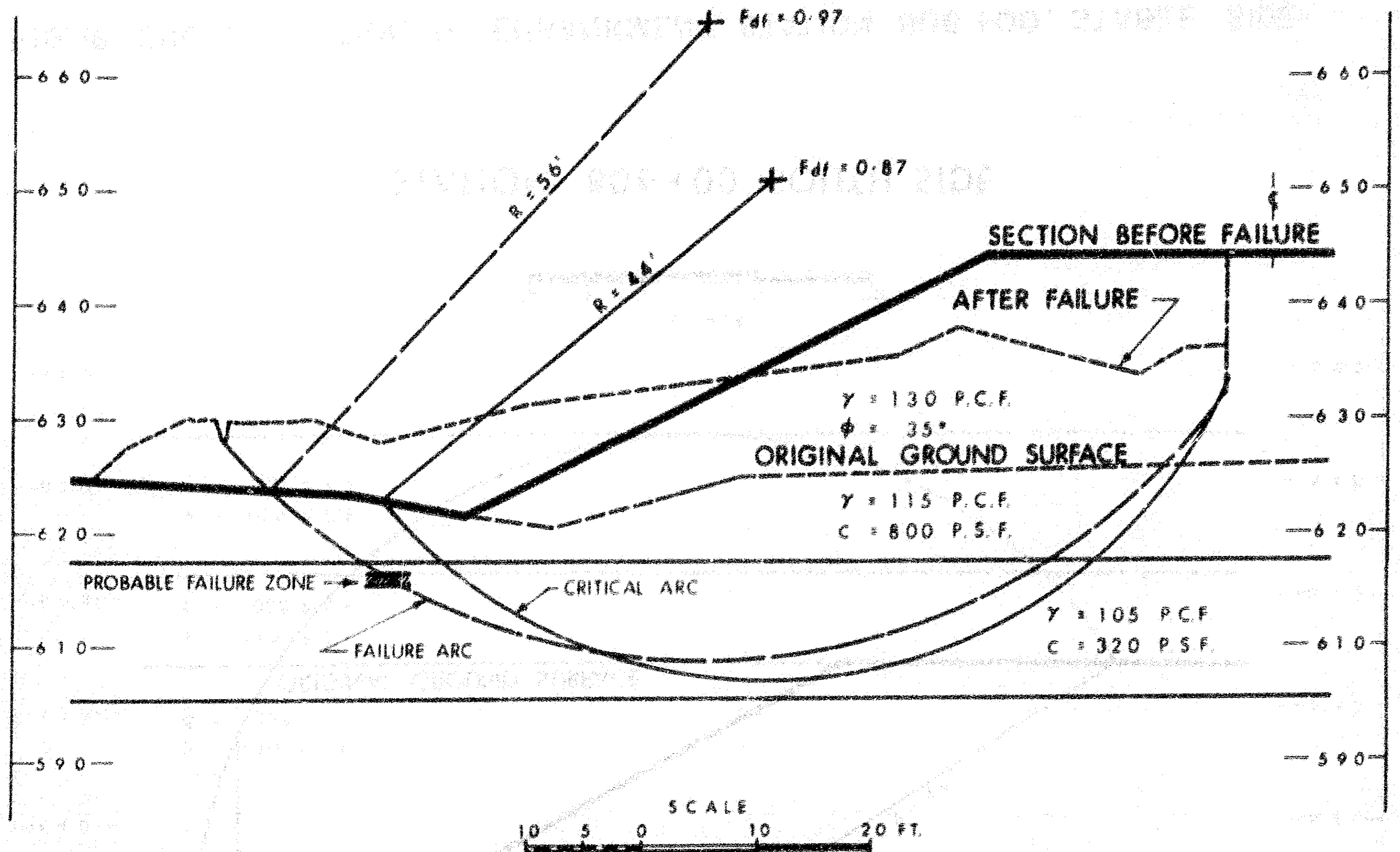
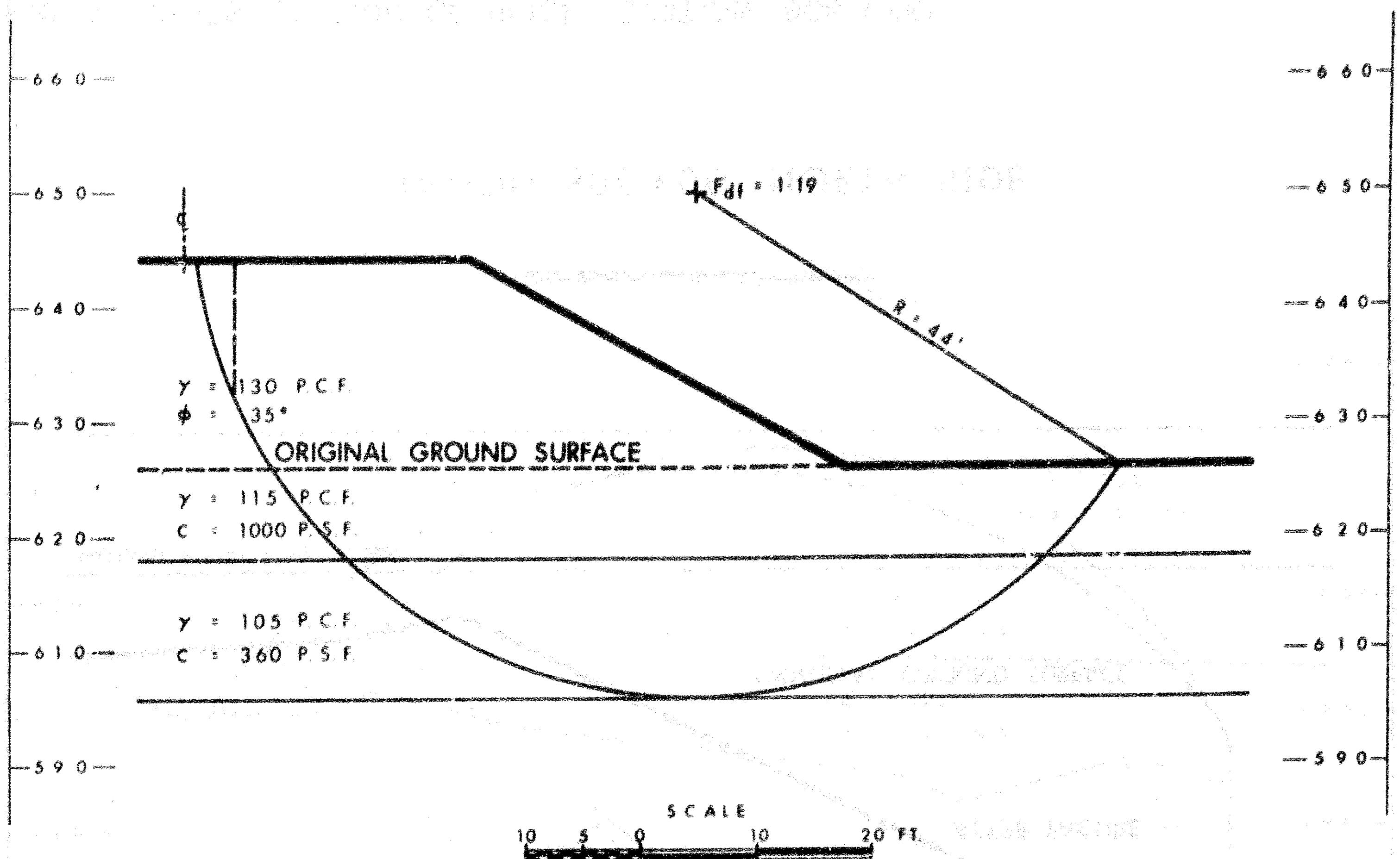


FIG. 16: RESULTS OF FIELD AND LABORATORY TESTS, BOREHOLES 1A AND 1B



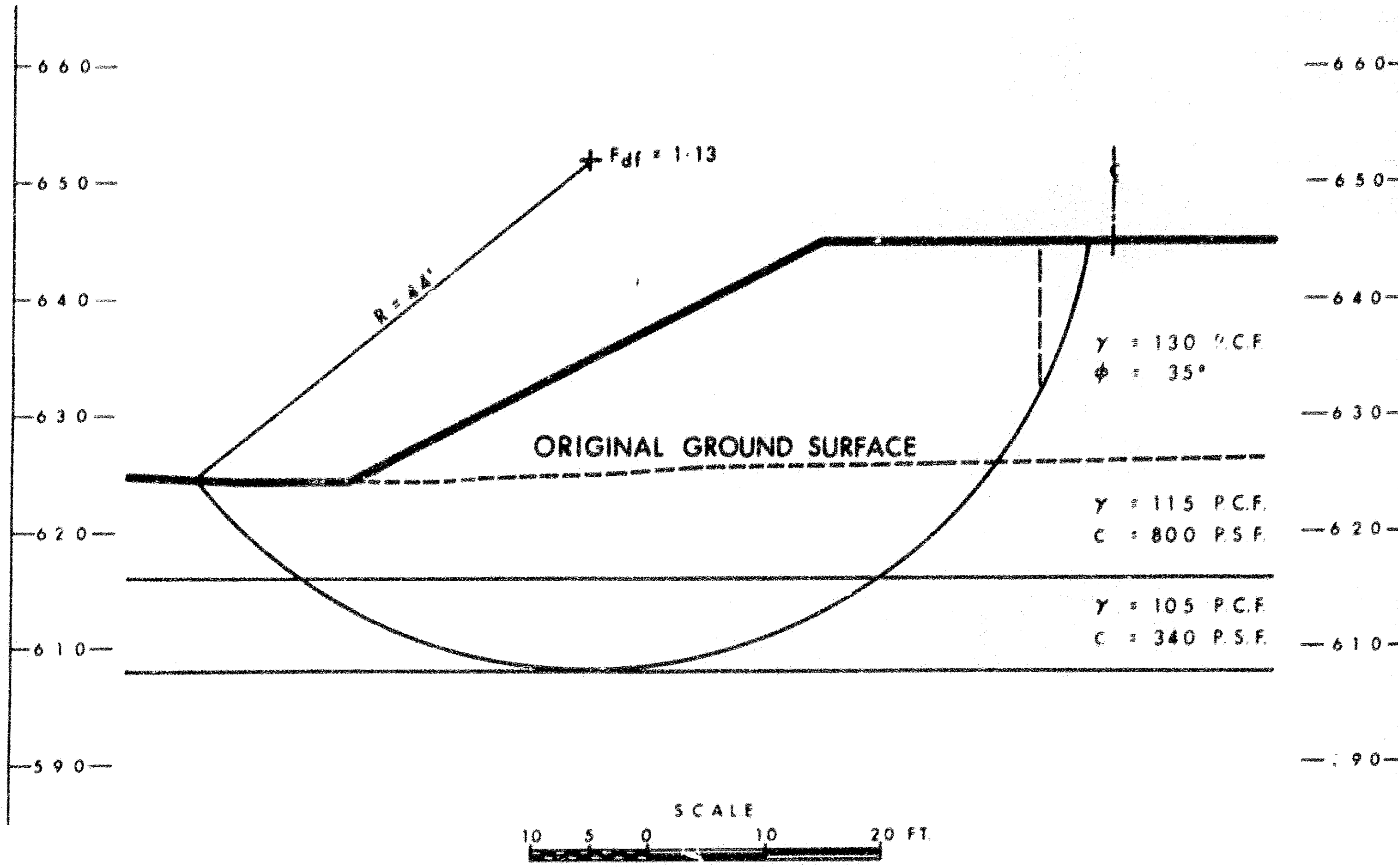
STATION 806+00 NORTH SIDE

FIG. 17: CROSS-SECTION OF SLIDE, STATION 806 + 00



STATION 806+00 SOUTH SIDE

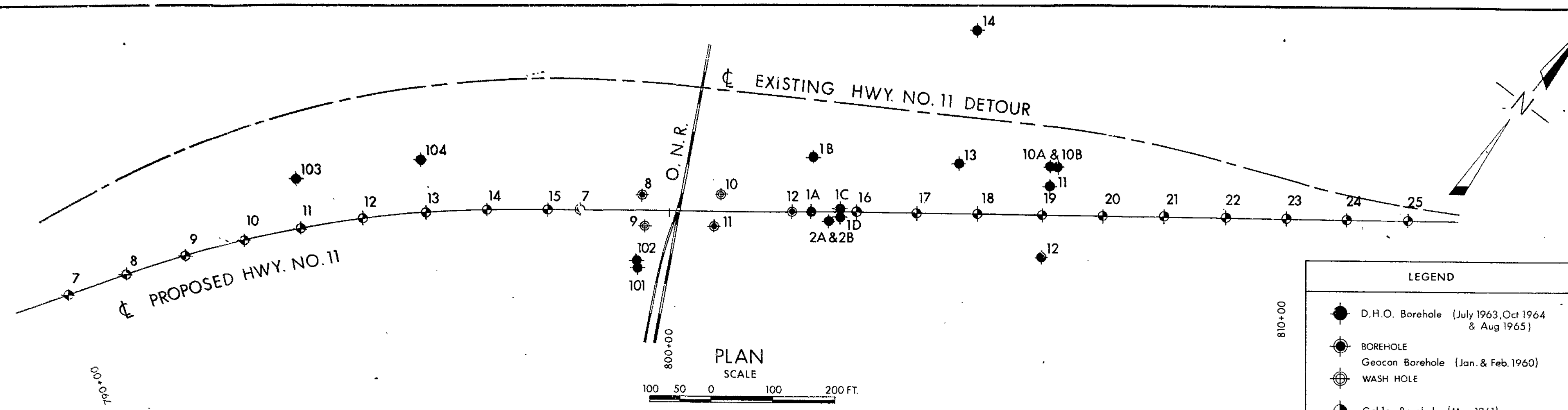
FIG. 18: CROSS—SECTION OF ENBANKMENT, STATION 806+00, STABLE SIDE

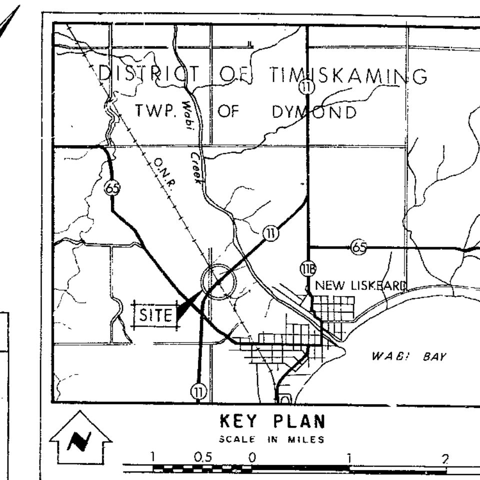


STATION 802+50 NORTH SIDE

FIG. 19: CROSS-SECTION OF ENBANKMENT, STATION 802+50

#62-F-99
WP.#104-60
O.J.H.R.P.
PROJ. #24
NEW. LISKEARD
EMBANKMENT





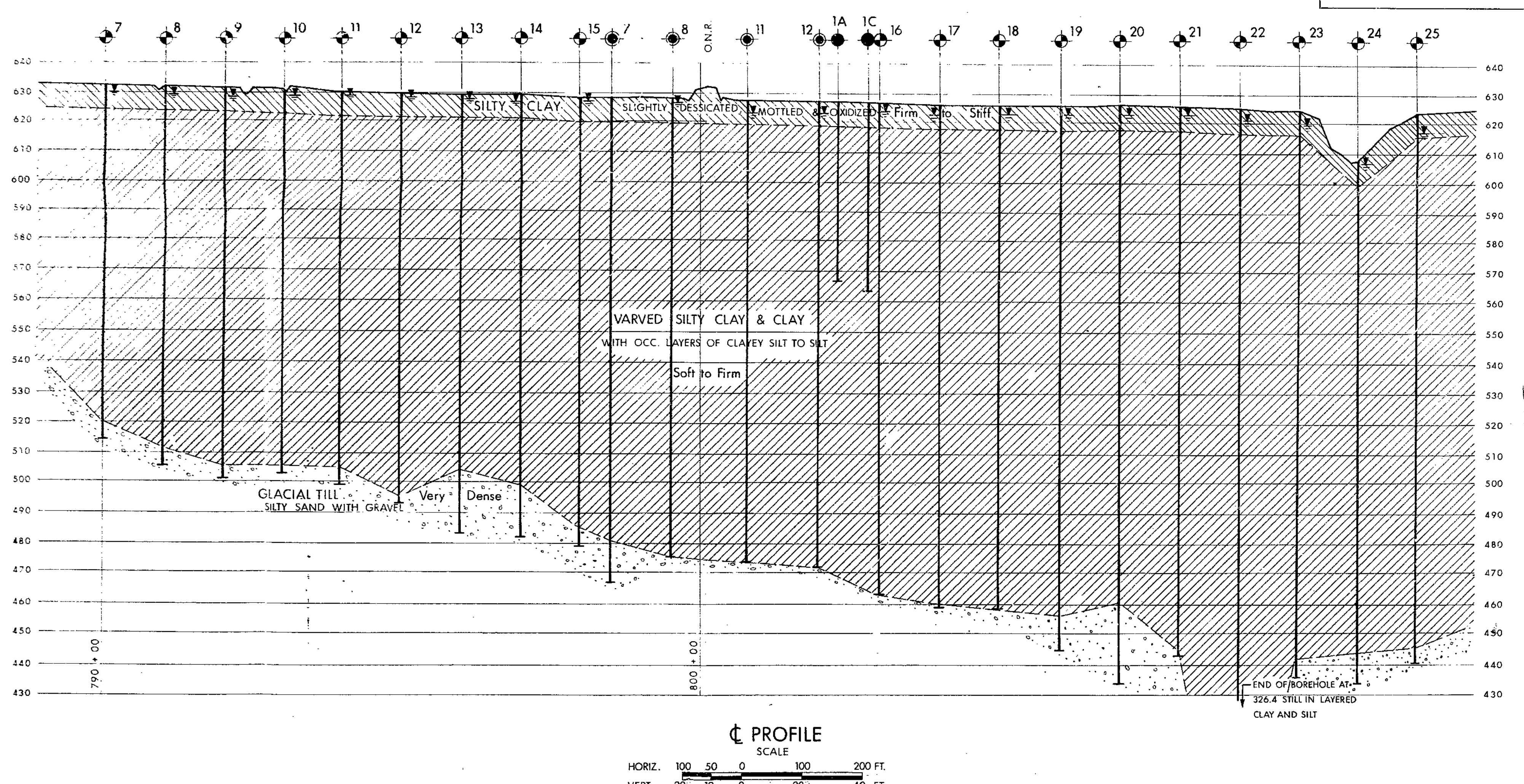
LEGEND

- D.H.O. Borehole (July 1963, Oct 1964 & Aug 1965)
- BOREHOLE Geocon Borehole (Jan. & Feb. 1960)
- WASH HOLE
- Golder Borehole (May 1961)
- Water Levels Established at Time of Field Investigation

	NO.	ELEVATION	STATION	OFFSET
D.H.O.	1A	627.0	802+18	℄
	1B	627.0	802+26	87' LT
	1C	643.5	802+70	3' LT.
	1D	643.5	802+70	3' RT.
	2A	635.5	802+53	14' RT.
	2B	635.5	802+50	14' RT.
	101	629.5	799+43	96.5' RT.
	102	629.5	799+43	85' RT.
	103	630.6	794+03	79' LT
	104	628.5	796+10	85' LT.
	10A	628.0	806+14	78' LT.
	10B	628.0	806+20	78' LT.
	11	635.0	806+14	47' LT.
	12	625.5	806+00	70' RT.
	13	626.3	804+66	87' LT.
	14	625.0	804+92	195' LT.
Geocon	7	628.7	798+50	℄
	8	628.9	799+65	27' LT.
	9	628.9	799+55	25' RT
	10	627.9	800+78	25' LT.
	11	628.0	800+70	25' RT.
	12	627.2	802+00	℄
Golder	7	632.1	790+00	"
	8	631.8	791+00	"
	9	631.2	792+00	"
	10	631.4	793+00	"
	11	630.0	794+00	"
	12	629.6	795+00	"
	13	629.2	796+00	"
	14	629.1	797+00	"
	15	628.8	798+00	"
	16	626.9	803+00	"
	17	625.8	804+00	"
	18	626.2	805+00	"
	19	626.1	806+00	"
	20	625.8	807+00	"
	21	625.5	808+00	"
	22	624.6	809+00	"
	23	623.4	810+00	"
	24	607.2	811+00	"
	25	623.5	812+00	"

— NOTE —

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.



PRINT RECORD

NO.	FOR	DATE

REVISIONS

DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTAR

MATERIALS & TESTING DIVISION - FOUNDATION

ONTARIO NORTHLAND RAIL... AND APPROACHES

KING'S HIGHWAY NO. 11 DIST. NO. 14

DIST. TIMISKAMING TOWN OF NEW LISKEARD

TWP. DYMOND LOT 7 CON. II

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D A.B.	CHECKED	W.P. NO. 104-60	M.B.T. DRAWING NO. 62-F-99A
DRAWN B.S.	CHECKED	JOB NO. 62-F-99	
DATE 19 FEB. 1968		SITE NO	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	