

58-F-3050

Hwy. # 11

By Pass, EARLTON

B9730

TROW, SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

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Project: C108/J163

April 21, 1958.

58-F-305 C

Mr. A. M. Toye,
Bridge Engineer,
Dept. of Highways of Ontario,
280 Davenport Rd.,
Toronto, Ont.

Attention: Mr. J. McAllister

Foundation Investigation
Hwy. #17 By pass, Earleton

Dear Sirs:

Enclosed herewith is our report on the foundation conditions underlying the proposed Highway #17 By pass of Earleton, Ont.

In this report we have recommended the use of a multispan bridge with spill-through piers and abutments and embankment approach slopes of $3\frac{1}{2}:1$ for the railway crossing area. A single span structure appeared unfavourable because the varved clay underlying the site will not support a more concentrated embankment loading unless stage construction and a comprehensive drainage system is installed. Even if drainage is accelerated the accompanying rapid settlement of the railway will involve considerable maintainance. A settlement of almost eight feet has been predicted for a scheme involving single span structure and fill up to grade level of 31 feet adjacent to the tracks. Consolidation will be at an extremely slow rate if subsurface drainage is not used.

We have indicated that construction of embankments can proceed to final grade level on $1\frac{1}{2}:1$ slopes beyond Stations 246 and 255, but that 2:1 slopes should be used between these locations. In addition, a ten foot stabilizing berm, tapering from 44 feet wide at the railway boundary to zero at Stations 255 and 247, also is required.

The bridge structure must be founded upon end-bearing piles driven to hard limestone bedrock located between 42 and 45 feet below existing ground surface. No corrosion or deterioration problems

resulting from contact with the varved clay, are anticipated.

We hope that the contents of this report assist you in selecting a structure for this railway crossing. If your review of the contents of this report requires additional clarification, please do not hesitate to contact us.

We sincerely regret any inconvenience to you as a result of our delay in submitting this report.

Yours very truly,

W. Trow

William A. Trow (P. Eng.)

WAT/lt
Encls.

DEPARTMENT OF HIGHWAYS OF ONTARIO
280 DAVENPORT ROAD
TORONTO, ONTARIO.

FOUNDATION INVESTIGATION
HIGHWAY #17 BY PASS, EARLTON, ONT.

G108 J163

Trow Soderman and Associates

April 22 1958

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FOUNDATION INVESTIGATION
ONTARIO NORTHLAND RAILWAY OVERPASS AND EMBANKMENTS
HIGHWAY #17, EARLTON, ONT

This report presents the results of the foundation investigation performed at the site of the proposed railway overpass which forms part of the Highway #17 By-pass of Earlton, Ontario. Because the design of the bridge structure will be determined by the requirements for stability of the approach embankments, a considerable portion of the report has been devoted to this latter problem.

Description of the Site and Embankment Failure Records

The proposed relocation of Hwy. #17, across the north boundary of Earlton, Ont., crosses a flat plain which comprises a portion of the great clay belt, a lacustrine deposit of glacial Lake Timiskaming. The land is used for general mixed farming and drainage for crops is accomplished by means of ditches dug to a depth of approximately 2 feet below the general ground level. At the time of the investigation, ice and water lay in some of the ditches, indicating a high water table condition. Little variation in this level would be anticipated however, even during the dry summer season.

Although the embankments of the proposed highway relocation passed over heavy clay, limestone bedrock is reported to outcrop in wooded land about one-half mile to the east of the site. This outcrop is evident, also, in McNamara's quarry 2 miles to the north. The strippings of this quarry represented the source of embankment fill when the construction of the by-pass was initiated just prior to World War II. Limestone bedrock was reported also at the bottom of a 71 foot well about one block west of the railway and about 200 feet north of the present highway.

Although eye-witness reports of events immediately prior to the embankment failure, which lead to the cessation of construction, have suffered slightly from natural failings of memory and imagination, general agreement exists on the following three significant matters: (a) that the embankment was constructed to full height of approximately 26 feet right up to the railway right of way, (b) that failure of the easterly embankment occurred suddenly, by the mechanism of spreading, shortly after this height was reached, and (c) that, as a consequence, considerable fill was removed and spread beside the embankment in order to reduce the danger of ground movements under the railway. Mr. Greig, retired McNamara Co. employee, advises that about one-third of the highway fill was removed and redistributed.

The present profile of the highway fill represents the level following this pre-war remedial work. Possible zones of failure have been obscured by farm cultivation or by the post-slide relocation of fill. Significant evidence for appraising present stability still exists, however, the most important being the fact that the maximum fill height is now approximately 20 feet and that this embankment state exists at approximately $1\frac{1}{2}:1$ slopes. There is little doubt therefore that the heavy clay of this area is able, at least, to support this embankment loading condition.

Although the land was partly snow covered during the investigation, numerous boulders and coarse gravel were noted over the fill surface. According to Mr. Greig and local witnesses, the fill was obtained from the overburden of McNamara's quarry two miles to the north, and as such, contained a variety of particles, sizes ranging from boulders to lean clay. An inspection of the quarry area indicated that a large percentage of the stripped ground must have consisted of coarser sands and gravels. No borings were made through the embankment because of the high concentration of boulders.

Field Investigation

The borings of this investigation were performed during the period from Nov. 25th to Dec. 2nd 1957, using conventional drilling methods. This involved the repetitive procedure of driving 3-inch I.D. casing to the required sampling depth, washing out the disturbed soil confined within the casing, recovering an undisturbed sample of the clay below casing level using 2-inch I.D. Shelby tubes, performing field vane strength measurements in the undisturbed soil below sampling depth and re-driving the casing to the next sampling interval.

Because of the marked uniformity of the soil with depth, samples were taken at 10 foot intervals, starting 5 feet below ground level. The shear strength of the soil within this 10 foot interval was determined by laboratory measurements on the undisturbed samples and by field vane tests made at approximately 3 foot increments of depth. Prior to the performance of the field vane tests, the soil was completely remoulded above a depth of 18 inches from the vane tips. Allowance was made for the resistance offered by this remoulded soil in the computation of in situ shear strengths.

Because the soil was obviously too soft to support concentrated bridges abutment loads, efforts were made to determine the levels of the underlying refusal stratum. This was accomplished by driving a 2-inch diameter cone, under 350 ft.lbs. of energy per blow, to the depth where the drive hammer bounced. This refusal stratum was confirmed, by 6 feet of rock drilling in hole 1, to be a hard limestone with some thin seams about 1 foot below bedrock level.

Water, for the drilling operations, was hauled from McNamara's quarry, 2 miles to the north. This water had to be heated to avoid freezing of the hose lines and other equipment during the sub-zero weather of the field investigation period.

The locations of all borings are shown on drawing 1 and are referenced to the railway signal located approximately at the intersection of the projected south embankment slope and the west side of the main railway line. The elevations of each boring were referenced to the top of the west rail of this line directly opposite the aforementioned signal post. Several views of the railway crossing and boring locations are shown at the end of the report.

All samples were carefully sealed with low-melting point wax upon recovery and were immediately placed on the seat of a heated car in order to avoid freezing during the sub-zero weather. All sampling and testing equipment was, also, warmed before insertion into the borehole as an additional precaution against freezing. The field sampling operations were supervised at all times by the author.

Description of Soil Types

On the basis of the borings and penetration tests of the investigation, quite uniform soil and bedrock conditions appear to exist at all locations tested. The subsoil profiles for each boring are presented in drawings 3 to 10 and the average profile, representative for the site, is shown in drawing 2.

Reference to this latter drawing shows that the predominant soil type is a thick deposit of varved clay consisting of alternate layers of soft, very plastic clay and non-cohesive silt or rock flour. The thickness of each phase of a varve ranges up to a maximum of about 0.6 inches, with the clay somewhat thicker than the silt. Views of partially dried samples illustrating the varving are shown at the end of the report. The upper levels of the clay have been dessicated due to surface drying as evidenced by the sharp increase in shear strength and corresponding reduction in moisture content, by the decrease in thickness of the varves and by the predominant brown colour toward the ground surface. The consistency of the clay phase is defined by a liquid and plastic limit of approximately 74% and 27% respectively, and a moisture content below the dessication zone, of about 90%. The silt portion of each varve had little or no plasticity. Chemical analyses indicated a slight basic condition and a free sulphate content well within tolerable limits for concrete. The test results for a depth of 6 feet in hole 3 are: pH = 8.23, free sulphate = 140 ppm.

At a depth of approximately 34 feet, the varved clay gradually changes or merges into a mixture of silty sand with gravel which becomes predominant just above the bedrock surface.

Although confirmed at one location only, there is little doubt that the refusal depths noted are indicative of bedrock. The depth to bedrock ranged from about 42 to 45 feet below ground surface corresponding to Elev. 771 and 767 feet respectively. Refusal at corresponding depths in a penetration test 400 feet to the north indicate that the varved clay and underlying materials are similar over a broad area.

Discussion of Strength Properties

As stated in the introductory remarks, the requirements for the stability of the approach embankments will determine in large measure what type of bridge crossing can be used to overpass the railway at this site. The fact that the concentrated loads of the bridge itself require support on end bearing piles to bedrock does not alter this argument. Since the final decisions regarding embankment stability must be based upon certain assumptions of the strength of the clay above bedrock, some discussion on this particular matter appears warranted. This discussion

will be concerned with answering four specific questions:

- (a) What is the present in situ strength and, hence, what shear strength should be used in analysing the stability of the embankment, assuming the clay does not consolidate to greater strengths during construction?
- (b) What has been the consolidating effect of the embankment fill since placement about 18 years ago, i.e., what is the increase in undrained strength of the soil with effective pressure?
- (c) How quickly will the clay consolidate when subjected to embankment loads?
- (d) What are the basic or effective strength characteristics of the clay in the event that stage construction or drainage is feasible?

The first of these questions have been resolved by two independent methods of shear strength measurement, i.e., in situ vane tests and undrained triaxial tests run at overburden pressure conditions. The summarized results of these tests are recorded in table 1, as well as in drawings 3 to 8 for each boring. The stress strain curves for each undrained triaxial test are shown in drawing 13.

Because of the unusual nature of this varved clay, i.e., layers of very plastic, soft, compressible clay interbedded with layers of tough, relatively incompressible silt, one might speculate on what test, if any, truly represents the capacity of this material at failure. In the undrained triaxial test failure took place by a squeezing out of the clay horizontally between the layers of silt. The percent strain at failure was approximately 5 to 8, although a lower failure strain might be anticipated under field conditions. The vane test involved measurements of strength of interbedded materials having quite different characteristics, the silt phase of which is not applicable for this method of test. However, since good agreement between these two independent methods of test was obtained, as demonstrated by the summarized results shown to the right of the soil profile in drawing 2, one can only assume that the silt varves were reduced to a state of zero resistance due to excess pore pressure as the torque of the vane was applied and therefore that the measurements are for the clay phase only. In passing, it should be noted that the sensitivity of the clay ranged from about 6 to 15, a fact to consider if the clay is to be subjected to strain much in excess of failure.

Therefore, in answer to the first query, it is assumed that the in situ shear strength for relatively rapid rates of loading, is as determined by the field vane and undrained triaxial tests. The conservative average values noted to the right of drawing 2, have been used in subsequent analyses of immediate embankment stability. The use of these strength values in the analysis of the original embankment failure has resulted in factors of safety close to unity for the maximum fill heights obtained.

With regard to the second question, the results from borings 3 and 5 indicate no increase in strength due to the consolidating effect of the adjacent embankment even though the lateral consolidating pressures reach

values of approximately 500 p.s.f. Some indication of the ratio between undrained cohesive strength and effective pressure can be obtained from the slope of the strength versus depth line of drawing 2, and from the results of three consolidated undrained triaxial tests shown in drawing 14. In the former instance, the C/P ratio has a value of approximately 0.31 and the slope of this average line projects above present ground level to intersect the depth ordinate at an equivalent consolidating pressure of approximately 1000 p.s.f. This possible state of preconsolidation is confirmed, to some extent by the consolidation test results for Hole 3 - 16 feet, but not for Hole 5 - 16 feet. The increase in strength with effective pressure from the three consolidated undrained tests, shown in drawing 14 and in table 1, can be shown to have an average value of $C/P = 0.22$ with a range from 0.209 to 0.23. This computation, however, makes no allowance for the possibility that the clay has been preconsolidated, which, if true, would result in higher values for C/P. The assumption of this report is that the increase in shear strength with 100% consolidation under any given pressure is equal to $C/P = 0.27$. This is in agreement with values noted by Skempton for a clay of similar plasticity.*

Assuming this relationship between shear strength and effective pressure, it can be shown that a consolidating embankment pressure of about 1000 p.s.f., the equivalent of $8\frac{1}{2}$ feet of embankment, would be required to achieve the shear strengths presently existing below the dessicated zones of the clay at this site. Since these strengths are already available in areas unaffected by the embankment load, it would appear that the application of weights in excess of $8\frac{1}{2}$ feet of embankment are necessary to achieve an increase in shear strength, and therefore that the consolidating effect of a large part of the existing embankment has been negligible.

In answering the query about the rate of consolidation of the clay, reference is made to the results of the two consolidation tests of drawing 11 and of the consolidated undrained tests of drawing 14. The coefficient of consolidation, C_v , expressing this characteristic is a function of the drainage path and modulus of compressibility and is measured in square feet per day unit. There was good agreement between the above two methods of test, although the results for the consolidation tests apply to vertical drainage through the clay and the consolidated triaxial tests permit horizontal drainage through each silt varve. A C_v value of about 0.002 sq.ft./dy. was obtained for the range of pressures anticipated and is indicative of a very slow rate of consolidation. Because the clay smear across the extremities of each varve of the samples would tend to prohibit drainage in the triaxial tests, extra precautions were taken, in hole 1 - 15 feet, to remove this smear from each silt layer prior to test. This operation did not alter the results. However, since it seems reasonable that each silt layer should be more permeable than the adjacent clay and therefore act as a potential drainage layer during the application of load, an additional test was done on a manufactured sample constructed entirely of the silt phases of a varve. The results of this test are also presented in drawing 14 and indicate a rate of drainage ten times greater than the other three measurements. Although this test will be influenced somewhat by disturbance, it does show that the silt layers of the varved clay can be counted upon to hasten consolidation provided a proper field drain system is installed. This matter

* Geotechnique 1948 No.1

is considered in greater detail later in the report.

The basic strength characteristics used for analysis of long term stability or for estimating the rate of embankment construction are indicated by the results of the slow drained triaxial test recorded in drawing 12. This drained strength parameter for the clay is expressed by $C' = 0$, and $\phi' = 24^\circ$. Although the tested samples consisted of alternate layers of clay and silt, these strength characteristics are believed to be representative of the weaker clay in which actual failure took place. Experience with other soils indicates that the silt or rock flour should have strengths corresponding to $C' = 0$ and $\phi' = 30^\circ$.

Analysis of Embankment Stability

Before attempting the stability analyses required for the selection of safe embankment sections, a check will be made on the reliability of the methods and assumptions used in investigating the failure mechanism which halted construction about 18 years ago. According to the reports indicated earlier, the embankment was built with about 1½:1 slopes and reached a height of approximately 26 feet when failure occurred.

These embankment conditions have been analysed by the sliding block method in design sheet 1. The sliding block assumes failure along a horizontal surface, i.e., along a clay layer or a silt layer at some depth below the ground. Failure occurs when the horizontal thrust from the embankment exceeds the shearing resistance of the soil along the plane of failure and the passive resistance generated at the toe of slope. The horizontal thrust, in this analysis, assumes an earth pressure at rest condition acting at the instant of failure. Although this assumption is somewhat severe, it is not unreasonable considering that the three consolidated undrained tests of drawing 14 failed at a very low percent strain. The sliding block method has been used because subsequent analyses have shown it to indicate the lowest factors of safety.

Reference to design sheet number 1 shows that the most probable path of failure was along a silt layer at a depth of about 8 feet. This is the approximate lower limit of the zone of dessication and the level where the shearing resistance is least. When allowance is given for the distribution and dissipation of embankment pressures with depth, the clay layers are found to be somewhat more stable for this height of 26 feet.

The shearing or frictional resistance generated in the silt when original embankment construction was begun can be expressed by the equation:

$$S = N \tan \phi$$

where N is equal to the effective vertical pressure
and ϕ is the angle of internal friction of the silt, taken here as 30° .

Because of the extremely compressible nature of the clay, all embankment loads would generate equivalent excess pore pressures in the clay and silt

and therefore the only value of N , effective in developer friction, is the weight of overburden existing prior to construction. At 8 feet this value of N is approximately 404 p.s.f., and therefore the corresponding frictional resistance in the silt at failure was 233 p.s.f. Although a lesser value of N is available at shallower depths, conditions of prestress due to dessication exist at these levels which hold the silt in a denser state. The lower moisture content measurements in the silt at a depth of 5 feet in all holes confirm this view.

The results of design sheet 1 can be used also for determining the maximum safe heights for the embankment if construction is renewed. At the present time the critical conditions in the silt no longer apply because the embankment weight has been in place long enough to squeeze out all excess pore pressures. Therefore the value of N on any silt seam under the embankment can be increased by the considerable weight of the embankment fill. The strength of the clay phase of any varve now becomes the determining factor with regard to stability and the conclusion of the earlier discussion is that the increase in clay strength due to consolidation for most of the embankment area is negligible.

Reference to design sheet No.1 and to the Summary of Factors of Safety in Table 3, shows that an embankment height of 26 feet on $1\frac{1}{2}:1$ slopes is just safe although the factor of safety is somewhat too low. According to the embankment grades indicated, a height of 26 feet is exceeded only between Stations 247 and 255. The present fill heights, effective for about 18 years, at these Stations, ranges from 9 to 20 feet respectively. At Station 246, 12 feet of fill presently exists. It therefore, will be quite safe to continue construction on $1\frac{1}{2}:1$ slopes to final grade west of Station 255 and east of Station 246. The increment of additional fill is too great for the use of $1\frac{1}{2}:1$ slopes between Station 246 and 247, however, and a value of $2:1$ is recommended.

In order to provide a minimum factor of safety of 1.3 immediately after construction, the embankment height of 31 feet adjacent to the railway will require slopes of $3\frac{1}{2}:1$. The factor of safety for failure in the silt is slightly less than this for conditions of instantaneous loading, but some horizontal dissipation of excess pore pressure can be anticipated as the embankment is built to final height. Berms 10 feet high and 44 feet wide placed out from the sides of $2:1$ embankment slopes would provide equivalent protection to $3\frac{1}{2}:1$ slopes. These berms could be reduced in width uniformly to zero at Stations 247 and 255 respectively. Due to redistribution of fill load after the original embankment failure, some of this berm material is already in place.

The overpass requirements adjacent to the Ontario Northland railway require special treatment if the stability of the railway is to be maintained. If subsurface drainage or stage construction is not to be utilized, the embankment must be provided with $3\frac{1}{2}:1$ slopes and the bridge structure must be multi-span with spill-through piers and abutments. As stated in earlier paragraphs, the weight of the bridge must be carried on end-bearing piles to limestone bedrock, located between 42 and 45 feet below present ground surface. No batter piles will be necessary.

As an additional precaution against failure, it is recommended that a trench drain be installed to a depth of approximately 25 feet at the upper quarter point of the embankment slope, as shown in drawing 15. This drain will be at right angles to the road centre line and it should be carried laterally beyond the embankment. All trenches should be back-filled with coarse sand and gravel as the excavation work progresses.

If on the other hand, a single span bridge is required with fill placed to full height against the abutments, means must be provided to consolidate the clay to a more stable state, and this, from a contractor's view, must be done in a relatively short time. It can be shown that single span bridge abutments at this site can support a maximum safe differential fill height above railway level of only 12 feet, if danger of railway heaving is to be avoided. This means that the underlying soil must be permitted to drain sufficiently quickly that a state less than 62% consolidation is never obtained. This is the equivalent of the 19 additional feet of embankment fill. Sand-filled trenches at 10 foot intervals placed parallel to the road centre line, dug to a depth of about 30 feet and extending back from the abutments for a distance of about 30 feet, should accomplish this desired consolidation in about one year's time. Lateral drains would be required to lead this drained water off to the sides of the embankment and piezometric control would be desirable to check on the actual rate of pore water dissipation.

If subsurface drainage and associated stage construction is given consideration, it also may be desirable to use 2:1 slopes for the entire critical zone between Stations 247 and 255. If this is the case, the aforementioned drains should be continued at the 10 foot spacing between these Station limits. Piezometric control again would be required, the critical excess pore pressure for a factor of safety of 1.3 being about 7 p.s.i. The computations for this estimate are indicated in design sheet 2. The cost of this drain installation will be considerable.

Another factor favouring the use of a multispan bridge concerns the compressibility of the underlying varved clay. The two consolidation tests for the clay phases of the varved clay, shown in drawing 11, indicated a compression index ranging from 1.5 to 2.5, values comparable to the very compressible Leda clay of the Ottawa valley. They are not unreasonable considering that the liquid limit of the clay is about 71% and the natural moisture content reaches 90%. No consolidation test was performed on the silt, but experience indicates that it should be greater than about 0.2.

The compression index, C_c , is a dimensionless term defining the compressibility of a soil beyond its existing preconsolidation limit. It can be used to indicate total settlement, S , when applied to the equation:

$$S = H \frac{C_c}{1+e} \log_{10} \frac{P_o + \Delta p}{P_o}$$

where H is the thickness of compressible soil in feet,
 e is the in situ void ratio

P_o is the max. state of prestress presently existing in the soil
 and Δp is the additional increment of load added, in this case,
 31 feet of embankment fill.

Neglecting the upper 4 feet of dessicated surface clay and assuming the underlying 30 feet of soil consists of 60% clay and 40% silt, it can be shown that the overall long term settlement for 31 feet of fill next to the railway abutments will be about 8 feet. If drainage is provided much of this movement could be expected to take place during the construction period. Constant readjustment of track level, therefore, would be required. If no drainage is provided the application of this fill next to the abutment would result in failure.

However, this value of settlement still applies for the junction point of fill and abutment, even if a multi-span structure and $3\frac{1}{2}:1$ approach slopes are used. If no drainage is installed, this amount of consolidation will theoretically take four centuries for completion. One-half of the settlement, with no drainage, would take about 85 years. Therefore, if side berms and a multispan bridge are utilized, the magnitude of settlement should be of academic interest only. The use of this arrangement would appear to involve less uncertainty than a scheme involving steeper slopes, a single bridge span and the drainage system required for it. The vibrating effect of train traffic also would be of less concern. The amplitude of vibration was found to be quite noticeable during the investigation period.

Conclusions

The comments and observations of the preceding sections of this report can be summarized by the following statements.

- (1) The route of the proposed By-pass of Highway 17, north of the town of Earlton, crosses a plain of extremely compressible varved clay approximately 35 feet thick. Its strength and other physical properties are discussed in the report.
- (2) Stability analyses have shown that the original embankment failure adjacent to the Ontario Northland Crossing occurred at a fill height of 26 feet. When construction is resumed, the placement of fill can be carried to final grade level on $1\frac{1}{2}:1$ slopes east of Station 246 and west of Station 255. Between these Stations, the embankment should have 2:1 slopes and, in addition, should be provided with berms 10 feet high, increasing from zero width at Stations 247 and 255 to 44 feet wide at the railway boundaries. Some of this berm material is already in place as a result of fill redistribution following the original pre-war slide.
- (3) Although stage construction and vertical drains would permit the use of a single span structure and avoid the need for stabilizing berms, considerable control and some construction delay would accompany such a scheme. In addition, the estimated 8 foot settlement for the maximum fill height of 31 feet would require continued readjustment of the railway line. If no drains are installed, the duration of settlement is expected to last for four centuries.
- (4) The recommended structure for the railway overpass is a multispan bridge incorporating spill-through piers and abutments and fill slopes of $3\frac{1}{2}:1$. The piers and abutments can be supported on end-bearing piles driven to bedrock at depths ranging from 42 to 45 feet below present ground sur-

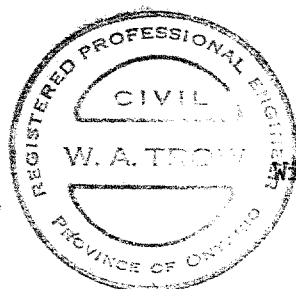
face. The capacity of the piles will be determined by their structural capacity considered as short columns. As an additional precaution against embankment failure, a trench drain should be dug under the front face of each embankment at right angles to the road centre line at or close to the upper quarter point of the spilled-through fill. This trench should lead out beyond the sides of the embankment in order to permit the free drainage of pore water. The trench should be back-filled with coarse sand and gravel and this operation should be carried on as the excavation work proceeds. Sketches of the suggested embankment approaches are shown in drawing 15.

(5) The use of this drainage system may hasten the original stages of consolidation of adjacent fill but should have little effect at the abutment points. With $3\frac{1}{2}:1$ approach slopes little or no settlement of the railway is anticipated.

(6) No deterioration of concrete or steel members is anticipated due to prolonged contact with the varved clay.

(7) The above remarks concern a railway crossing at the position attempted about 18 years ago. The alternative to this is to cross the railway at a point two miles to the north where bedrock comes closer to the ground surface.

WAT/lt
April 22, 1958.
C108/J163



W. A. Trow

William A. Trow (P.Eng.)

TABLE NO. 1
SUMMARY OF STRENGTH MEASUREMENTS IN P.S.F.

Depth Feet	Hole 1		Hole 2		Hole 3		Hole 4		Hole 5		Hole 6	
	Qu	V	Qu	V	Qu	V	Qu	V	Qu	V	Qu	V
2												
3				1075 s=4.8			1584 s=3.2		2200 s=16.7			
4		1892 s=5.4			> 2200						2178 s=11	
5					1500 s=5	887 s=6.0						
6	1200		NO		1060				820			
7											poor test ↓	
8		748 s=5.7			352 s=4.6		473 s=12.3		953 s=13		395	605 s=13.7
9				506 s=9.2								
10			TESTS				484		429 s=9.7			
11											583 s=8.8	
12		518 s=7.8		462 s=9.3	412 s=7.5		506 s=5.1		563 s=17		519 s=15	
14			DONE		473 s=8.6				419 s=13			
15					460(good) cq=870		717 s=6.1 774 s=5.9					
16	500 cq=932								425 s=7.4 cq=865			
17		550 s=7.2										
18				517 s=5.5	484 s=11				475 s=10			
							660 s=6					
20		561 s=7.3		594 s=6.4	450 s=5						583 s=13.2	
							682 s=15.5		484 s=8.8			

TABLE NO.1 - (CONT.)

Depth Fest	Hole 1		Hole 2		Hole 3		Hole 4		Hole 5		Hole 6	
	Qu	V	Qu	V	Qu	V	Qu	V	Qu	V	Qu	V
22			671 s=6.8		500 s=5		616 s=8				603 s=8	
24	605 s=7.9				580 (patched)		462 s=5.4		605 s=7.9			
26												
28	693 s=7		704 s=5.1		539 s=4.9		748 s=8.5					
30	847 s=8.6		704 s=9.1		1023 s=5		792 s=12		792 s=7.2		761 s=9.6	
32			880 s=2.2		869 s=2.6		946 s=14.3					
	605 s=6.1								858 s=6.5			
34			792 s=1.6								825 s=8.4	

LEGEND: Qu - Undrained Triaxial Test
V - Vane
s - Sensitivity
cq - consolidated undrained test.

TABLE NO. 2

SUMMARY OF OTHER LABORATORY TEST MEASUREMENTS

AND ASSOCIATES

Depth	Consistency % Dry Weight						Natural Unit Weight p.c.f.						
	Natural		Moisture		Content		L.L.	P.L.	Natural		Unit Weight		p.c.f.
	Hole								Hole				
	1	2	3	4	5	6			1	3	4	5	6
2													
3													
4													
5		c=83		c=76.1 s=20.1 comp=45.4	c=63.6 s=30.7		71	29.5	H3			110.2	
6	c=65.1 s=30.5		c=63.3 s=28.9				74	29	H1	110.2		106.5	
7				comp=73.9									111
8													
9													
10													
14													
15				comp=57.4 " 53.7									
16	Qu c=90.7 s=33.7	c=87.1 s=35.2	c=88.0 s=35	comp=46.1	c=89 s=30.5		76.4	26.2	H1-s	103.5	103.8	102.5	109.5
17	Cq c=66.7 s=25.2		Cq c=79.5 s=27.7		Cq c=66.2 s=24.2		c 74.7 s 30.9	27 22.1	H3 H3				
26	c=73.1 s=34.3	c=76.7 s=57.3	c=80.2 s=33.5	comp=75.8			79.0	26	H3		106.2	102.5	
35	c=57												
36			c=42.7	c=62.4									

NOTE: Hole 3 - 5 ft.: pH = 8.23
free Sulph.= 140 ppm

LEGEND: L.L.-Liquid Limit P.L.-Plastic Limit
Comp. -Composite sample s -Moist.Cont.Silt c -Moist.Cont.Clay
Cq-Mois.Cont.at end of consolidated undr.test. Sd-Mois.Cont.at end of slow drained test

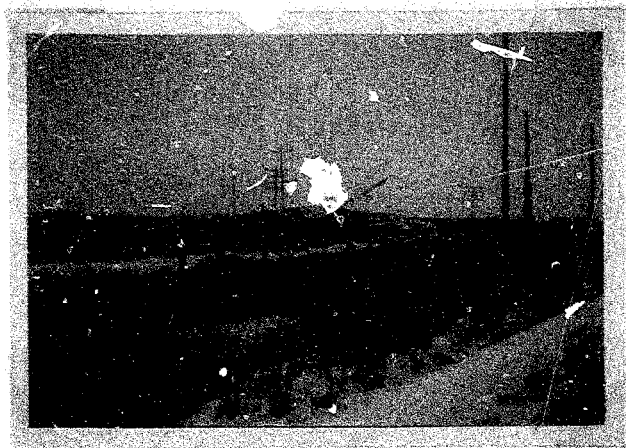
LEGEND: L.L.-Liquid Limit Cq-Mois.Cont.at end of
P.L.-Plastic Limit consolidated undr.test.
Comp. -Composite sample Sd-Mois.Cont.at end of
s -Moist.Cont.Silt slow drained test
c -Moist.Cont.Clay

NOTE: Hole 3 - 5 ft.: pH = 8.23
free Sulph.= 140 ppm

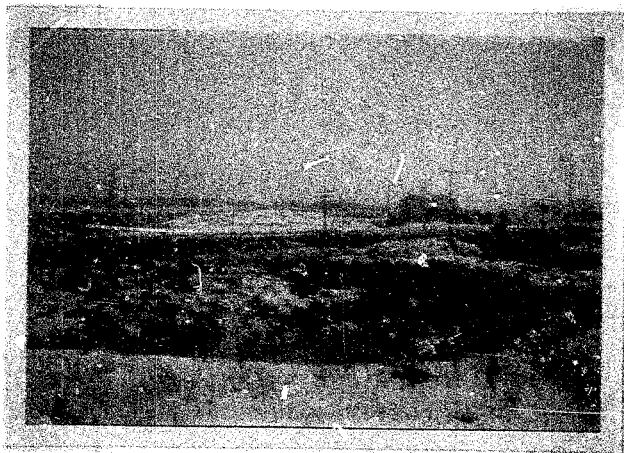
TABLE NO. 3

SUMMARY OF STABILITY ANALYSIS

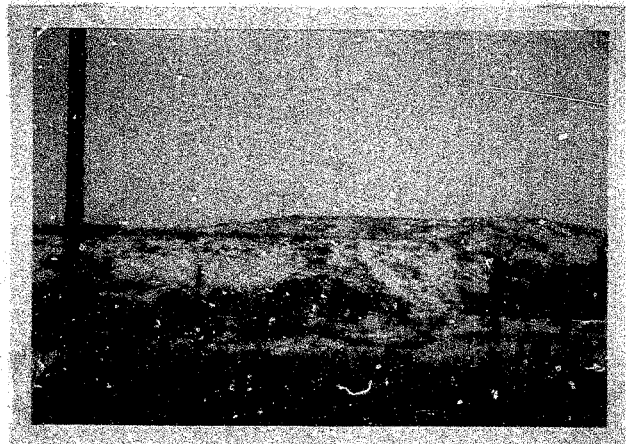
Embankment Height Feet	Side Slopes	Factors of Safety	
		Sliding Block	Circular Arc (Result for various trial circles)
25	1½:1		1.335
	2:1		1.235; 1.274; 1.47; 1.49
26	1½:1	1.34(8'); 1.128(20'); 1.18(35')	
28	2½:1	1.53(10'); 1.275(20'); 1.292(30')	
30	2:1		0.98
31	1½:1	Clay 1.044(10'); 0.952(20'); 1.02(30'); Max. safe pore pressures (F.S.=1.3) Silt=9.2 psi; Clay = 6.5 psi	1.055; 1.05; 1.225
	3:1	Clay 1.405(10'); 1.205(20'); 1.216(30') Silt 1.17(10'); 1.175(20'); 1.275(30')	
	3½:1	Clay 1.31 (20') Silt 1.265 (10')	



View looking North-west
(railway signal reference indicated by
arrow)



Looking East from Station No. 254



Looking South-west from railway toward
west embankment



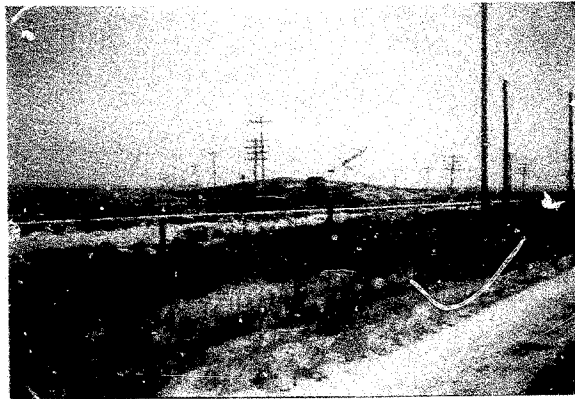
Looking east - drill on hole No. 3



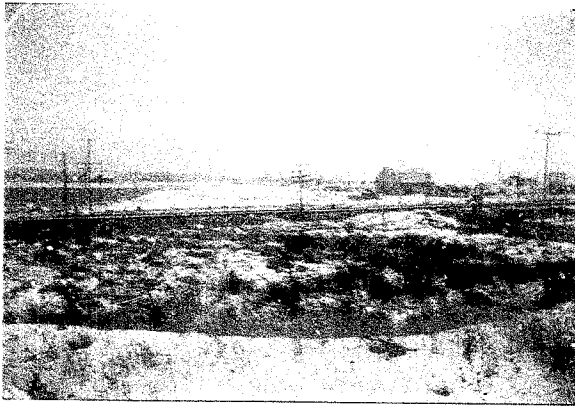
Looking west from Station 248

VARIOUS VIEWS OF HWY. #17 BY PASS CROSSING OF ONTARIO NORTHLAND
RAILWAY, EARLTON, ONTARIO.

SUPERIMPOSED DOCUMENT MAY
APPEAR AS MULTI-FEED ON FILM.



View looking North-west
(railway signal reference indicated by
arrow)



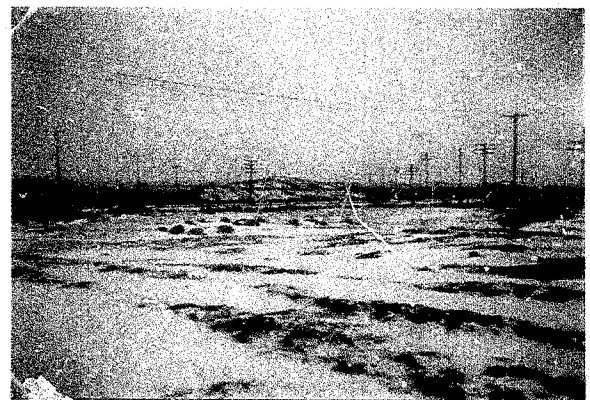
Looking East from Station No. 254



Looking South-west from railway toward
west embankment



Looking east - drill on hole No. 3



Looking west from Station 248

VARIOUS VIEWS OF HWY. #17 BY PASS CROSSING OF ONTARIO NORTHLAND
RAILWAY, EARLTON, ONTARIO.

SUPERIMPOSED DOCUMENT MAY
APPEAR AS MULTI-FIELD ON FILM.

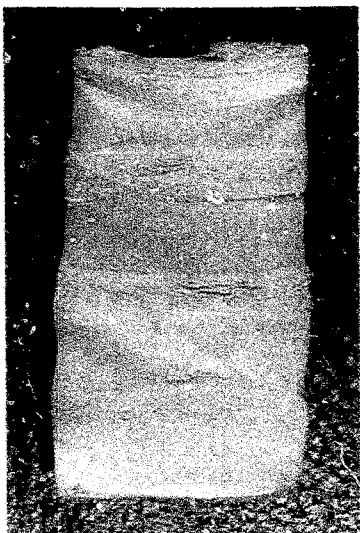


Before drying.



After drying one day.

Two sections of typical sample of undessicated
varved clay



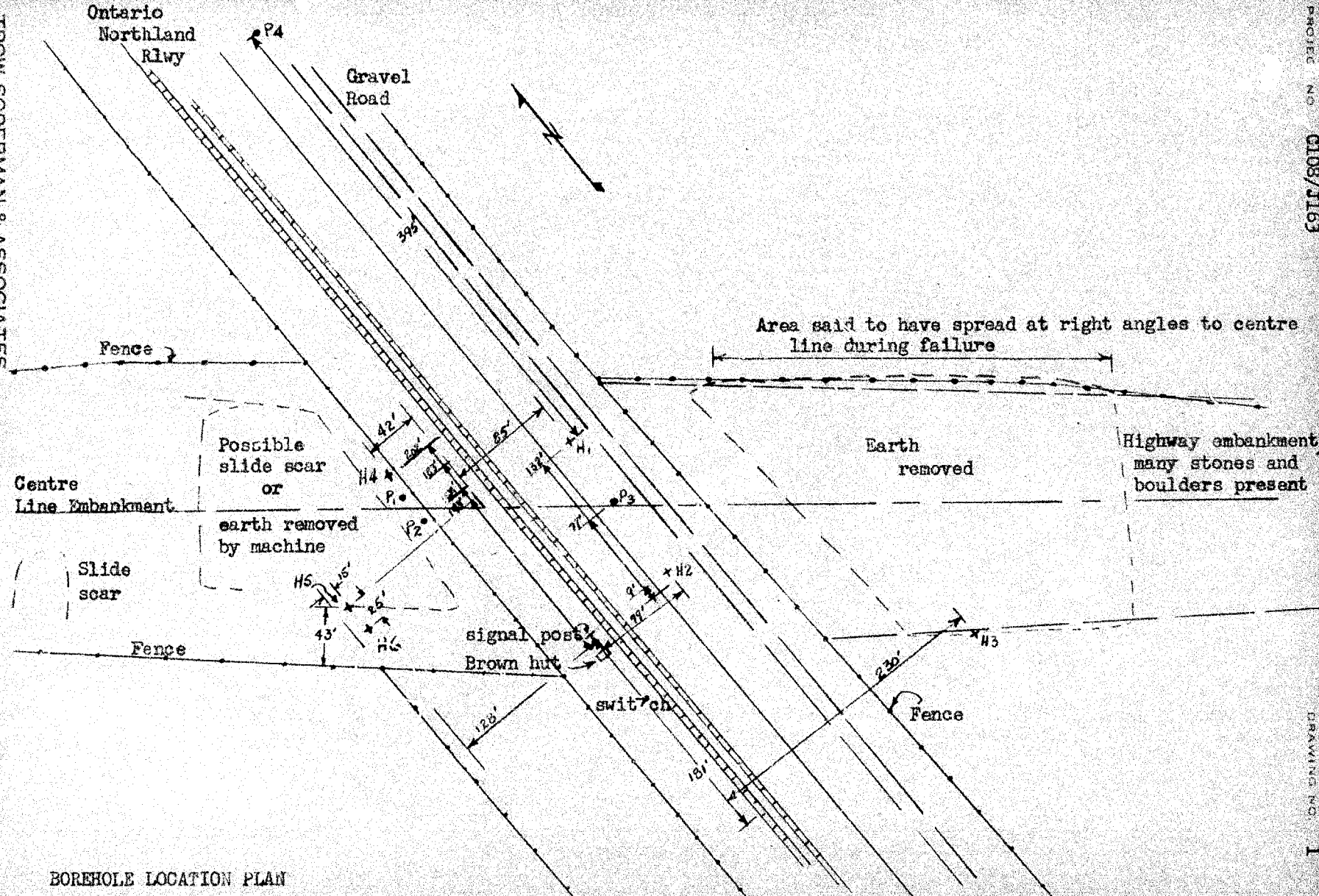
Before drying.



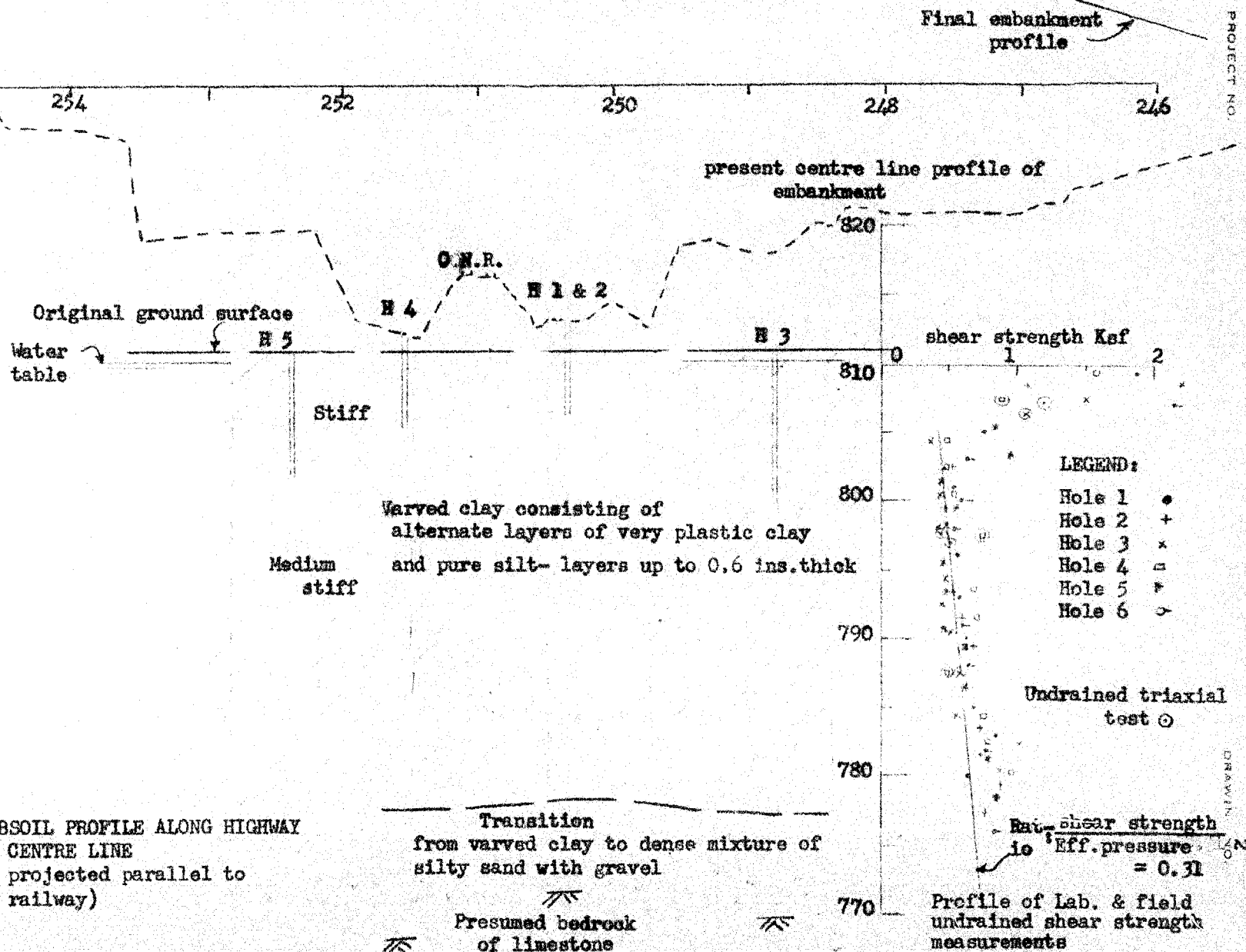
After drying one day.

Two sections of typical sample of undessicated
varved clay

TROW SODERMAN & ASSOCIATES



BOREHOLE LOCATION PLAN



TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT: Ontario Northland Overpass

LOCATION: Earlton, Ont.

HOLE LOCATION: S-1 Awg. 1

HOLE ELEVATION AND DATUM: 813.2 assumed top west
rail by signal post = 816.5

BOREHOLE NO. 1

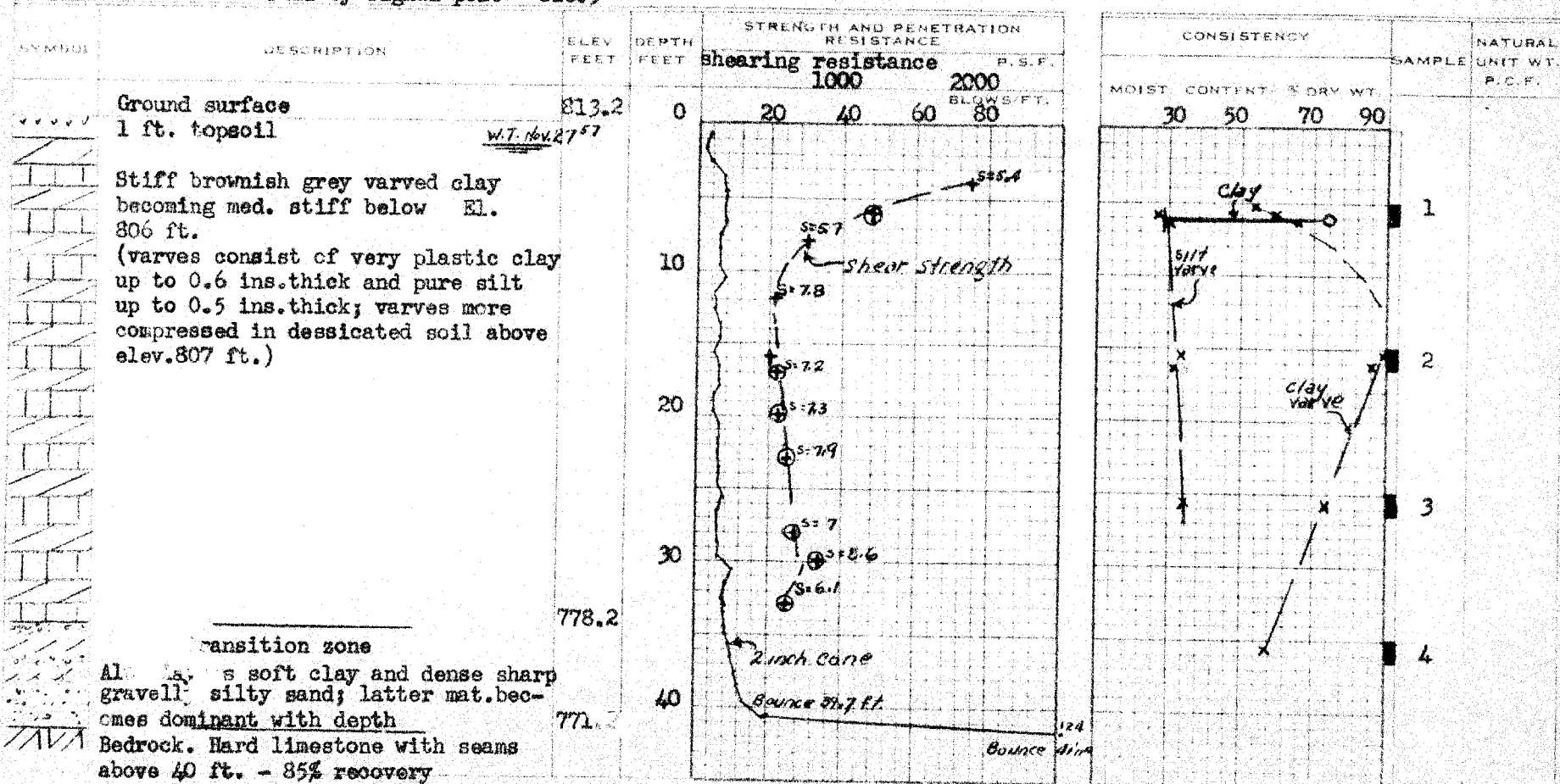
FIELD SUPERVISOR: WT

DRILLER: MG

PREP: WT

LEGEND

- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (QU)
- VANE TEST (C) AND SENSITIVITY (S)
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT



TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Ontario Northland Overpass
LOCATION Earleton, Ont.

HOLE LOCATION See Dwg. 1

HOLE ELEVATION AND DATUM 811.5 assume top west
rail by signal post = 816.5

BOREHOLE NO. 2

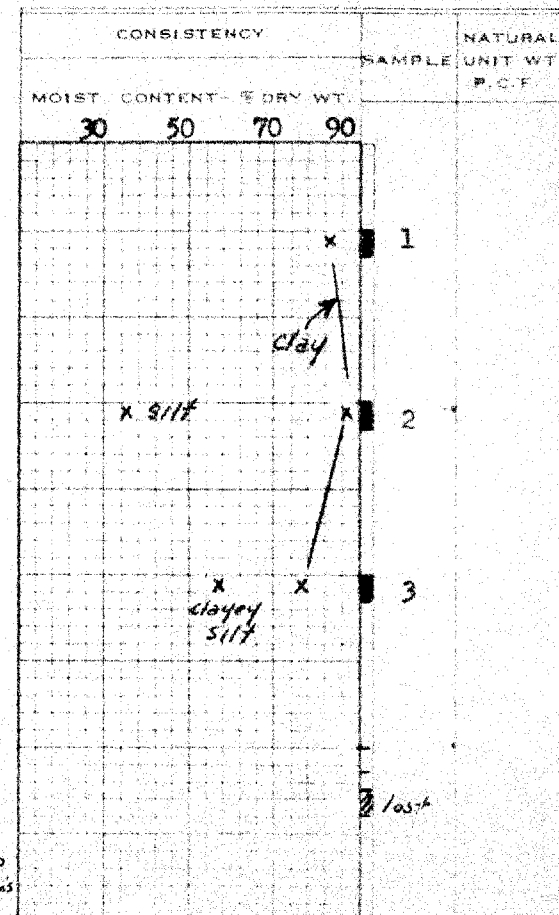
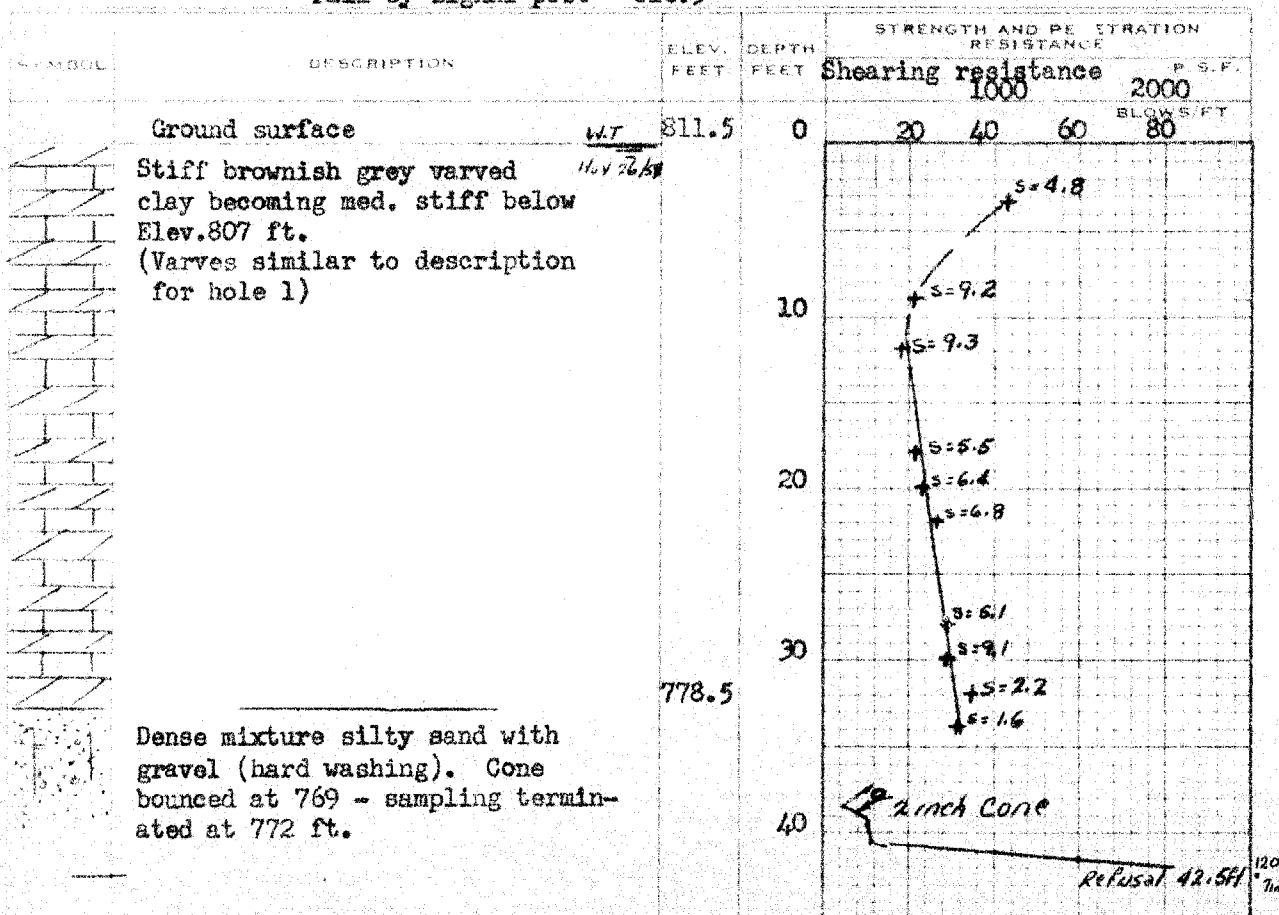
FIELD SUPERVISOR WT

DRILLER MG

PREP. WT

LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION (QU)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND
LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT



TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

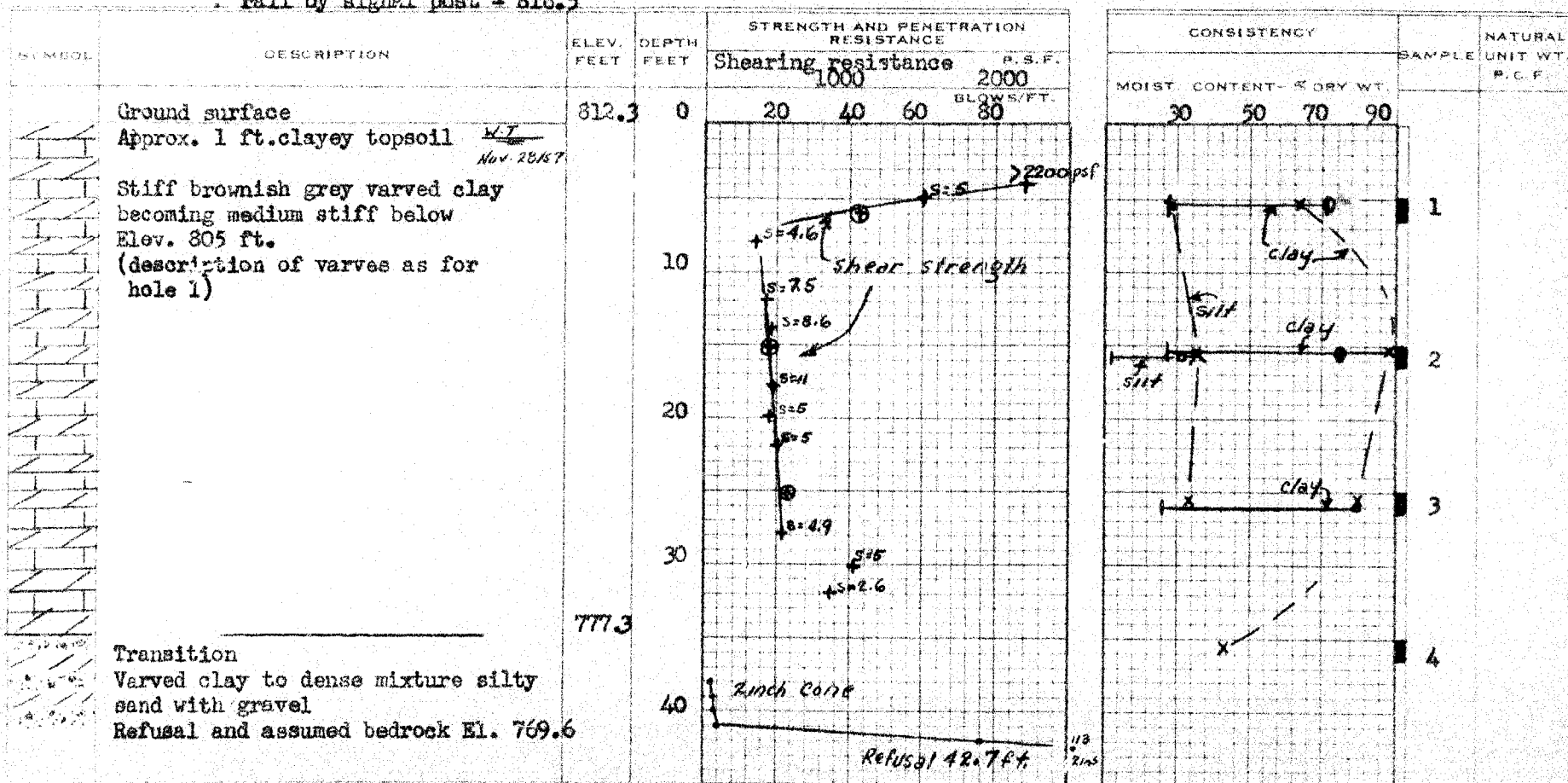
PROJECT Ontario Northland overpass
LOCATION Earlton, Ont.

HOLE LOCATION See Dwg. 1

HOLE ELEVATION AND DATUM 812.3 assumed top west
rail by signal post = 816.5BOREHOLE NO. 3
FIELD SUPERVISOR WT
DRILLER MG
PREP. WT

LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION (QU)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND
LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT



TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Ontario Northland Overpass

LOCATION Earlton, Ont.

HOLE LOCATION See Dwg. 1

HOLE ELEVATION AND DATUM 812.3 Assumed top west
rail by signal post = 816.5

BOREHOLE NO. 4

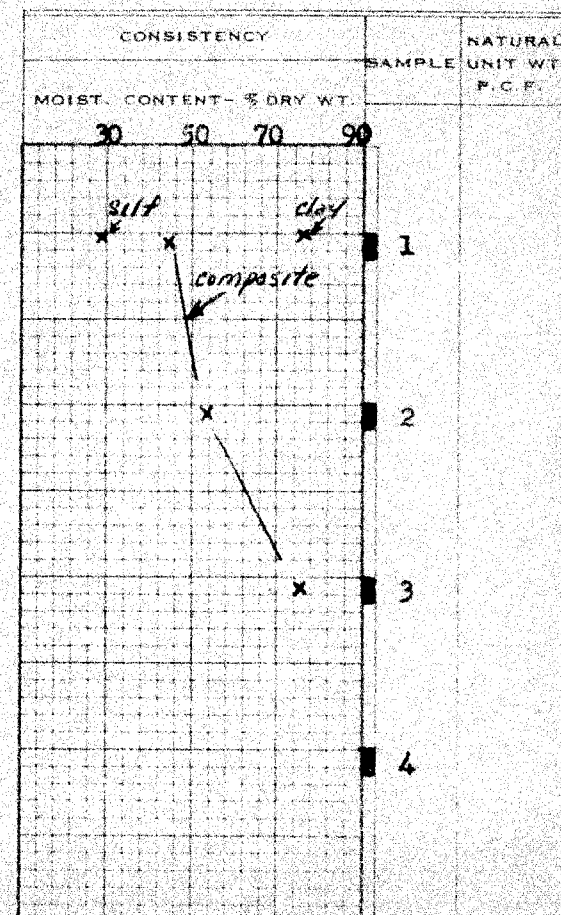
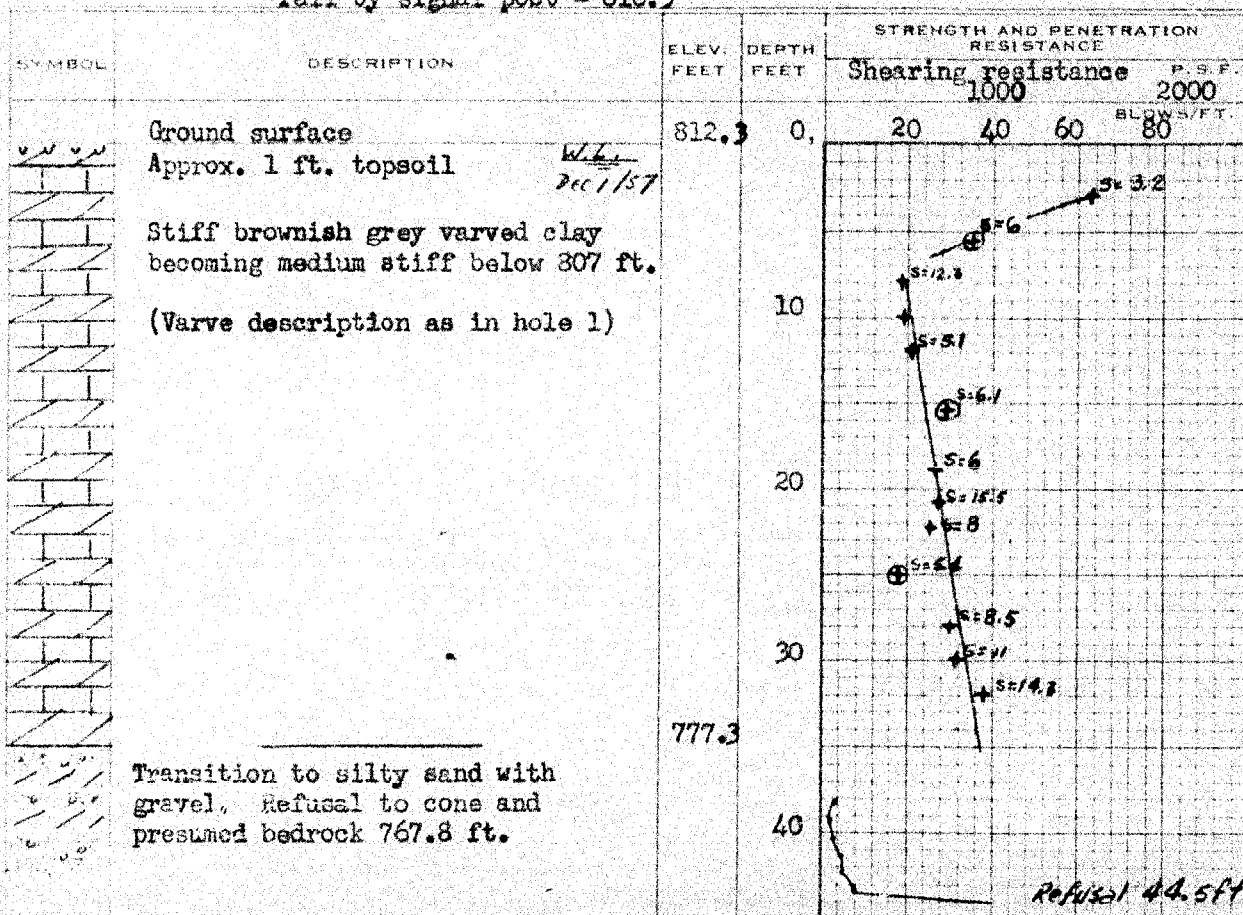
FIELD SUPERVISOR WT

DRILLER MG

PREP. WT

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION [Qu]
 VANE TEST [C] AND SENSITIVITY [S]
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Ontario Northland Overpass

LOCATION: Earlton, Ont.

HOLE LOCATION See Dwg. 1

WIDE ELEVATION AND DATUM 811.5 Assumed top west

rail by signal post = 816.5

BOREHOLE NO. 3

FIELD SUPERVISOR **WT**

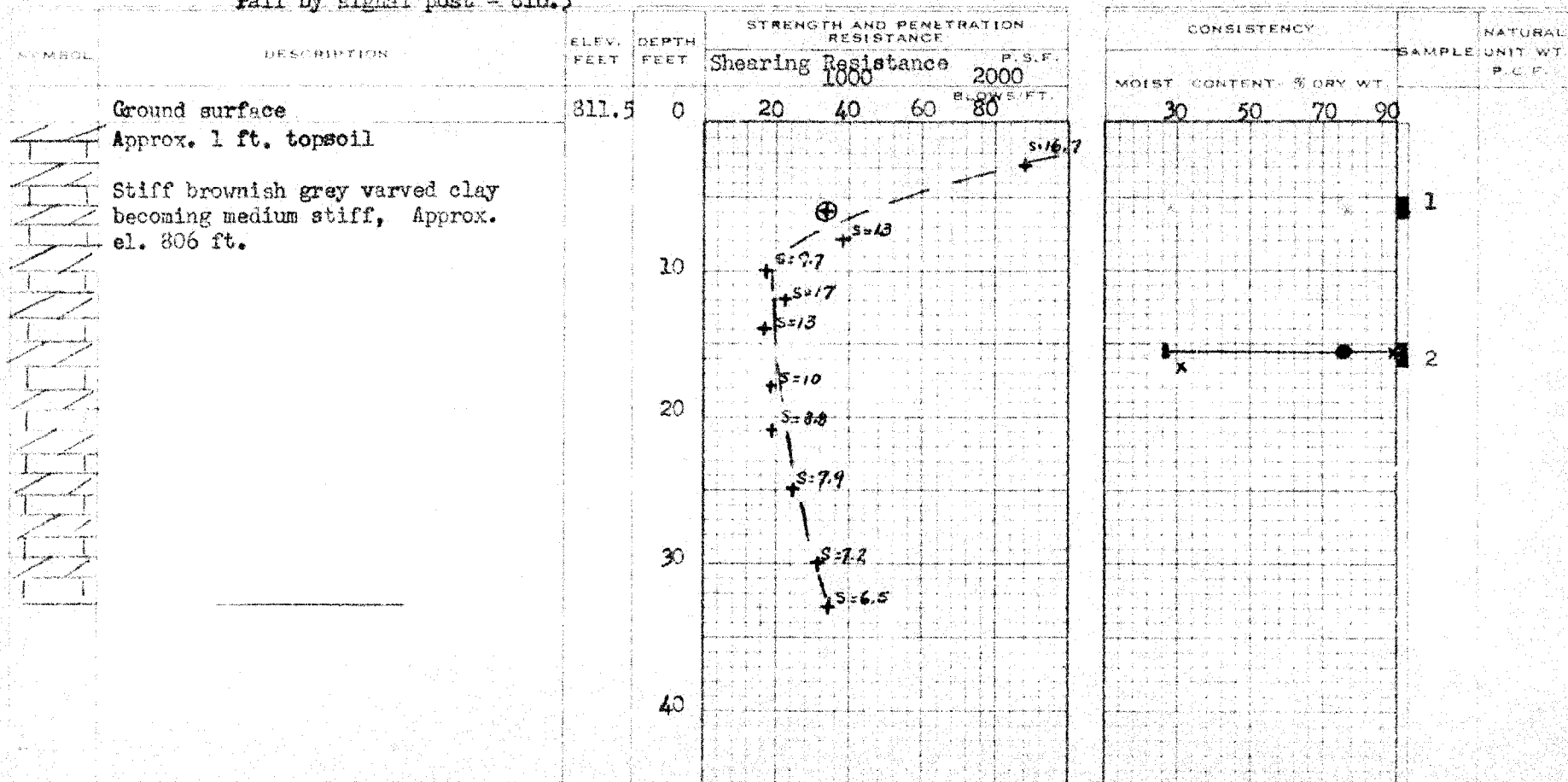
DRILLER MC

FREE. WT

DRAWING NO. 7

LEGEND

- 2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND
LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT



PROJECT NO. C108J163

DRAWING NO. 8

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Ontario Northland Overpass

LOCATION Earlton, Ont.

HOLE LOCATION See Dwg. 1

HOLE ELEVATION AND DATUM 311 - Assumed top west
rail by signal post = 816.5

BOREHOLE NO. 6

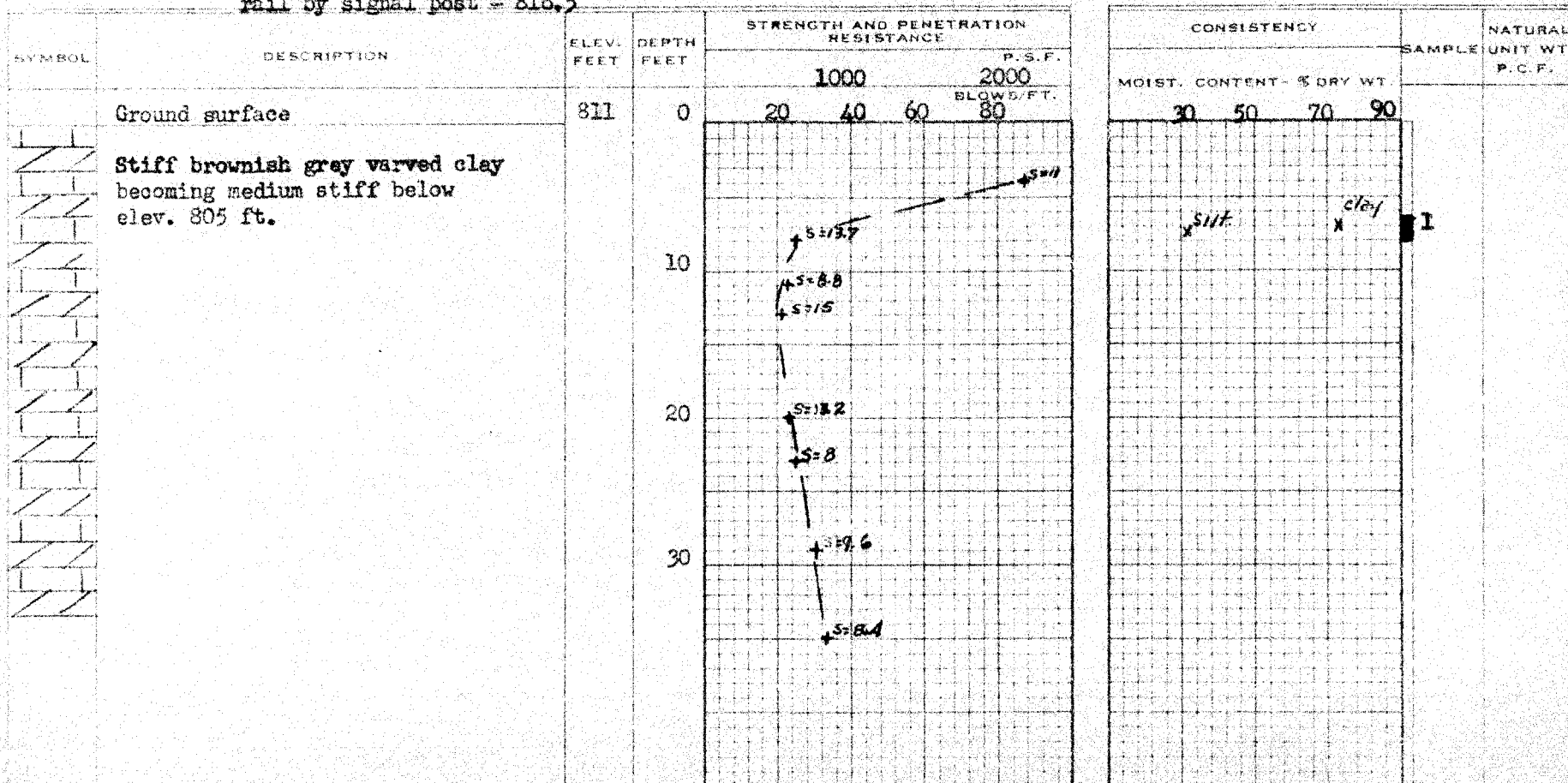
FIELD SUPERVISOR **WT**

DRILLER

PREP. NT

LEGEND

- 2" DIA. 5" LIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION [Qu]
VANE TEST [C] AND SENSITIVITY [S]
NATURAL MOISTURE AND
LIQUIDITY INDEX.
LIQUID LIMIT
PLASTIC LIMIT



TPOW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Ontario Northland Overpass

LOCATION Earleton, Ont.

HOLE LOCATION See Dwg. 1

HOLE ELEVATION AND DATUM 812.4

BOREHOLE NO. **P1 & P2**

FIELD SUPERVISOR

DRILLER

PREP

LEGEND

- 2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION (Qu)
PLANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND
LIQUIDITY INDEX.
LIQUID LIMIT
PLASTIC LIMIT

[illegible]

PROJECT NO C108 J163

DRAWING NO. 10

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT	Ontario Northland Overpass
LOCATION	Earlton, Ont.
HOLE LOCATION	See Dwg. 1
HOLE ELEVATION AND DATUM	811.5

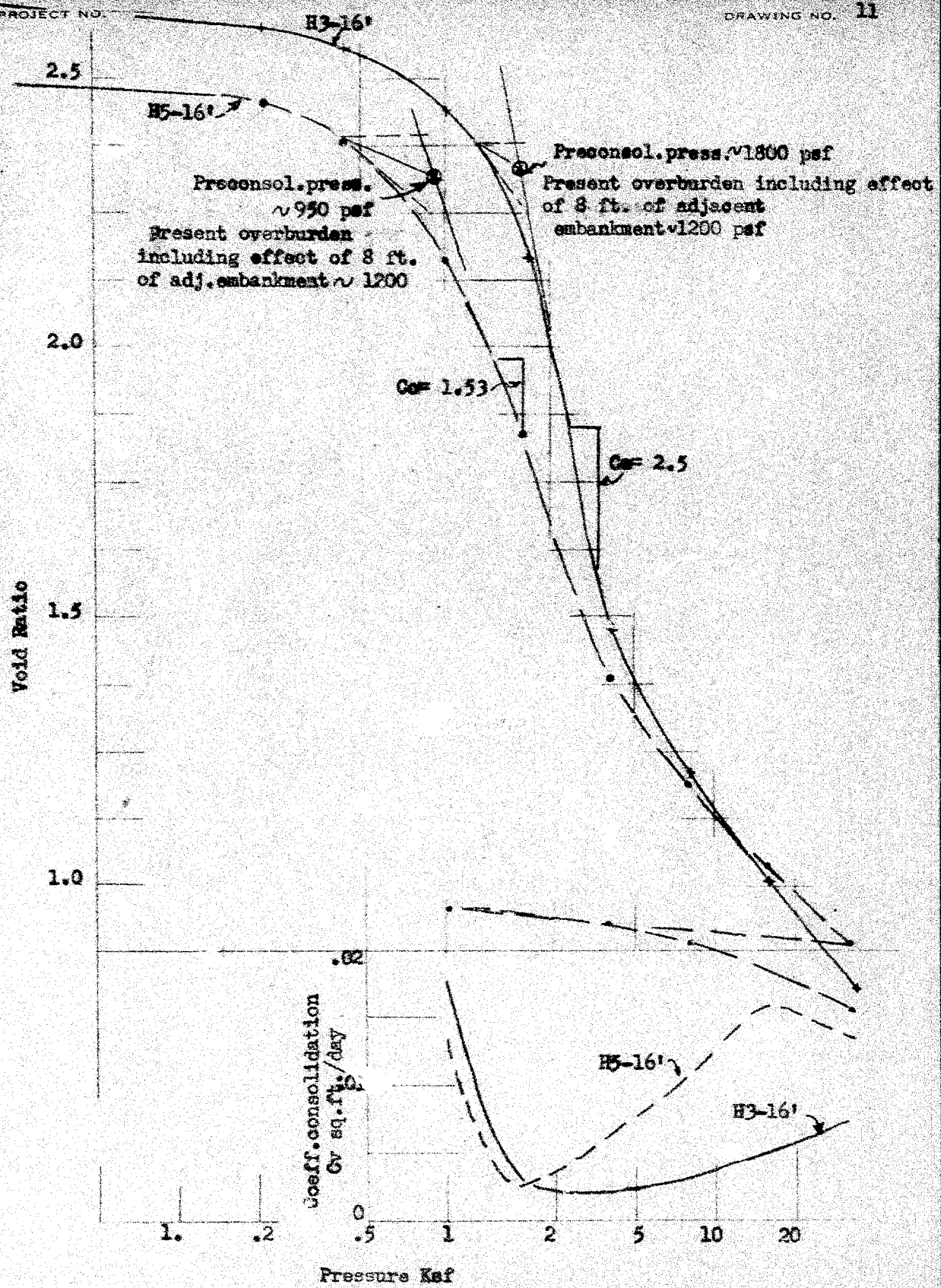
BOREHOLE NO. **P3 & P4**
FIELD SUPERVISOR _____
DRILLER _____
PREP. _____

LEGEND

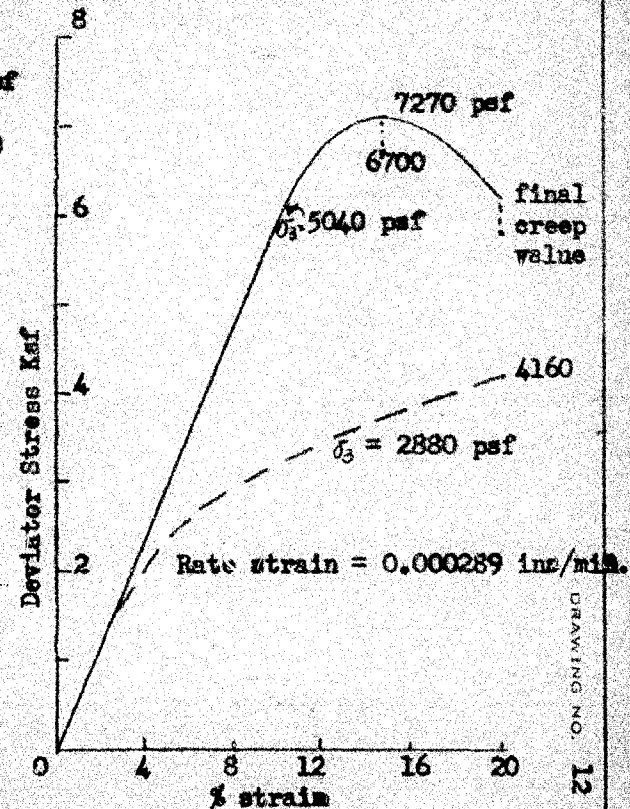
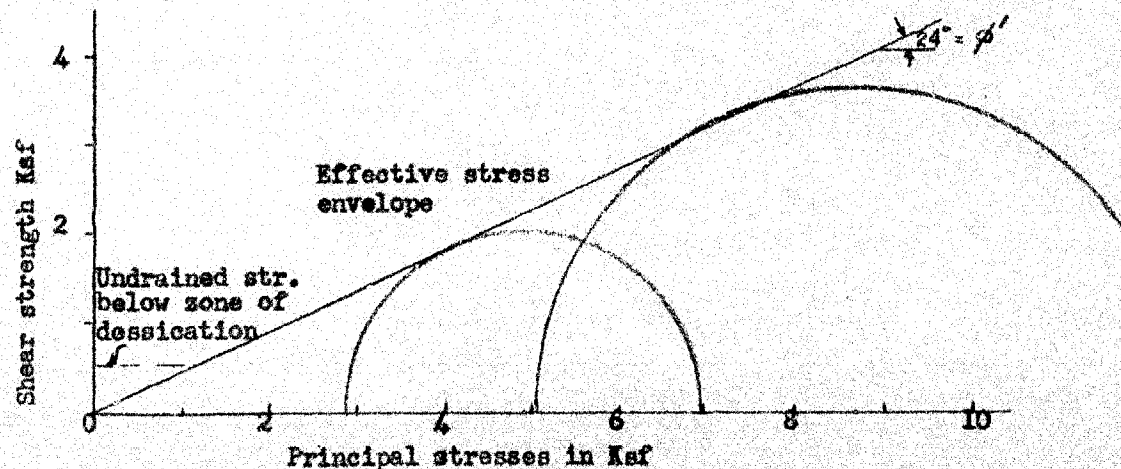
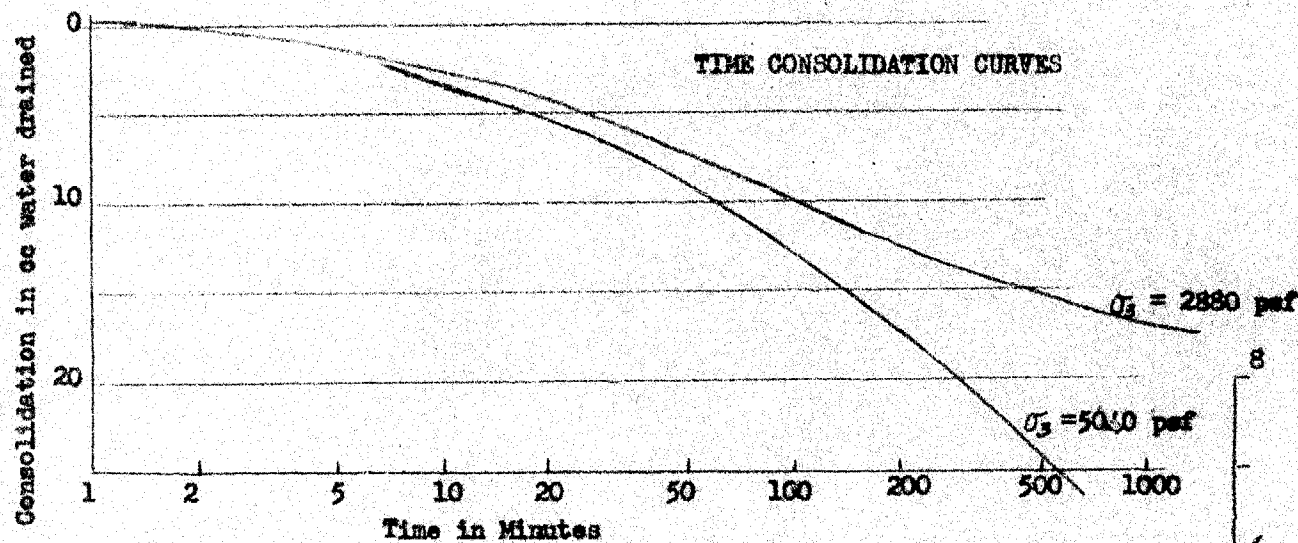
- 2 ¹¹ DIA. SPLIT TUBE
2 ¹¹ SHELBY TUBE
2 ¹¹ SPLIT TUBE
2 ¹¹ DIA. CONE
CASING
2 ¹¹ SHELBY
1/2 UNCONFINED COMPRESSION [Qu]
VANE TEST [C] AND SENSITIVITY [S]
NATURAL MOISTURE AND
LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

[illegible]

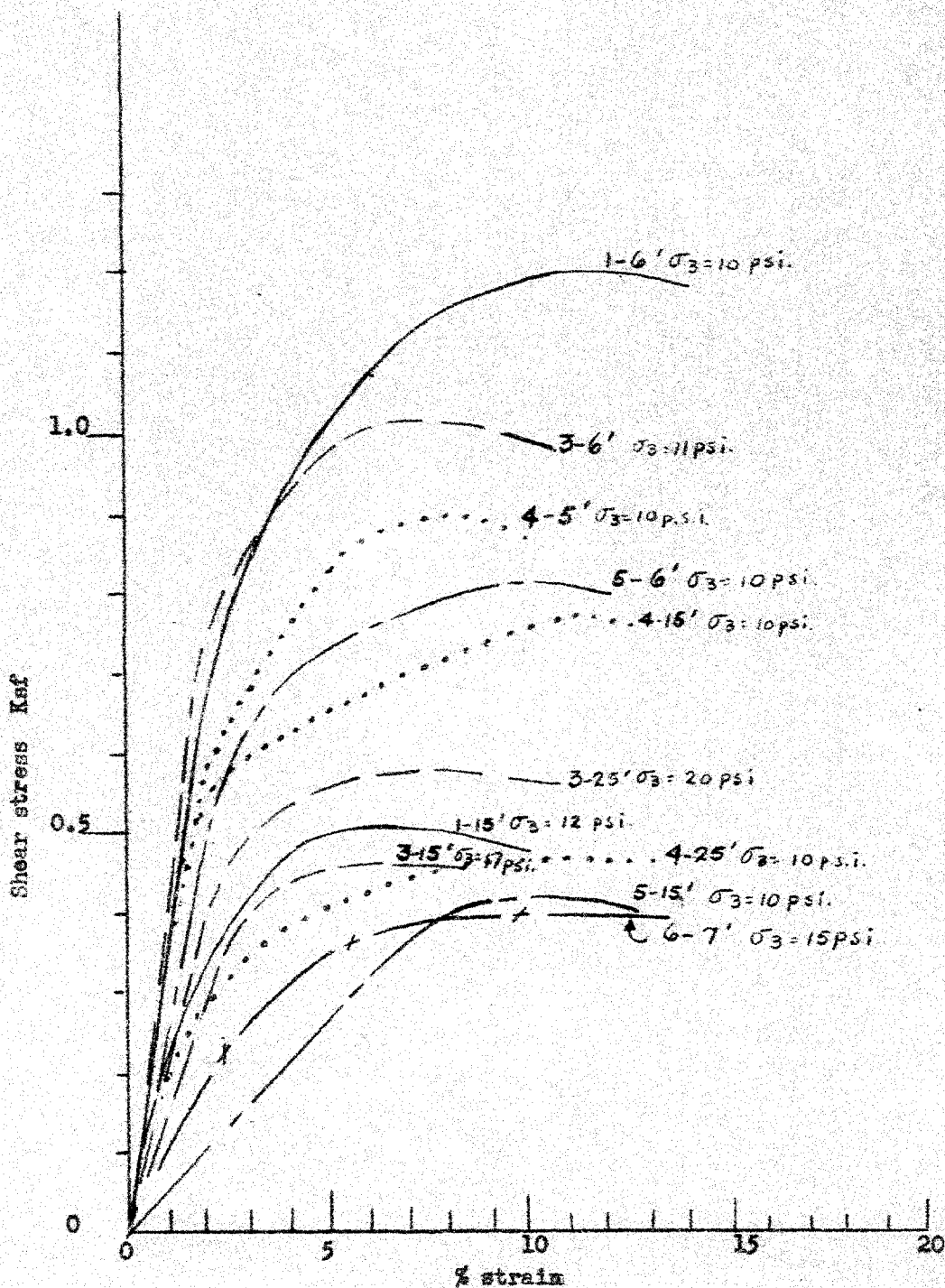
CONSISTENCY		SAMPLE	NATURAL
MOIST. CONTENT - % DRY WT.			UNIT WT. P.C.F.



CONSOLIDATION CHARACTERISTICS FOR CLAY PORTION OF A VARVE



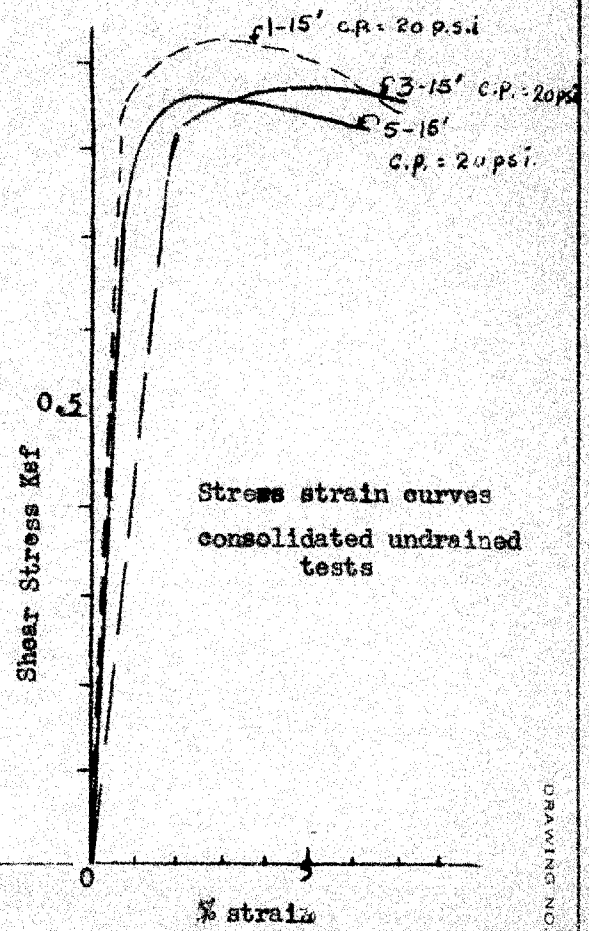
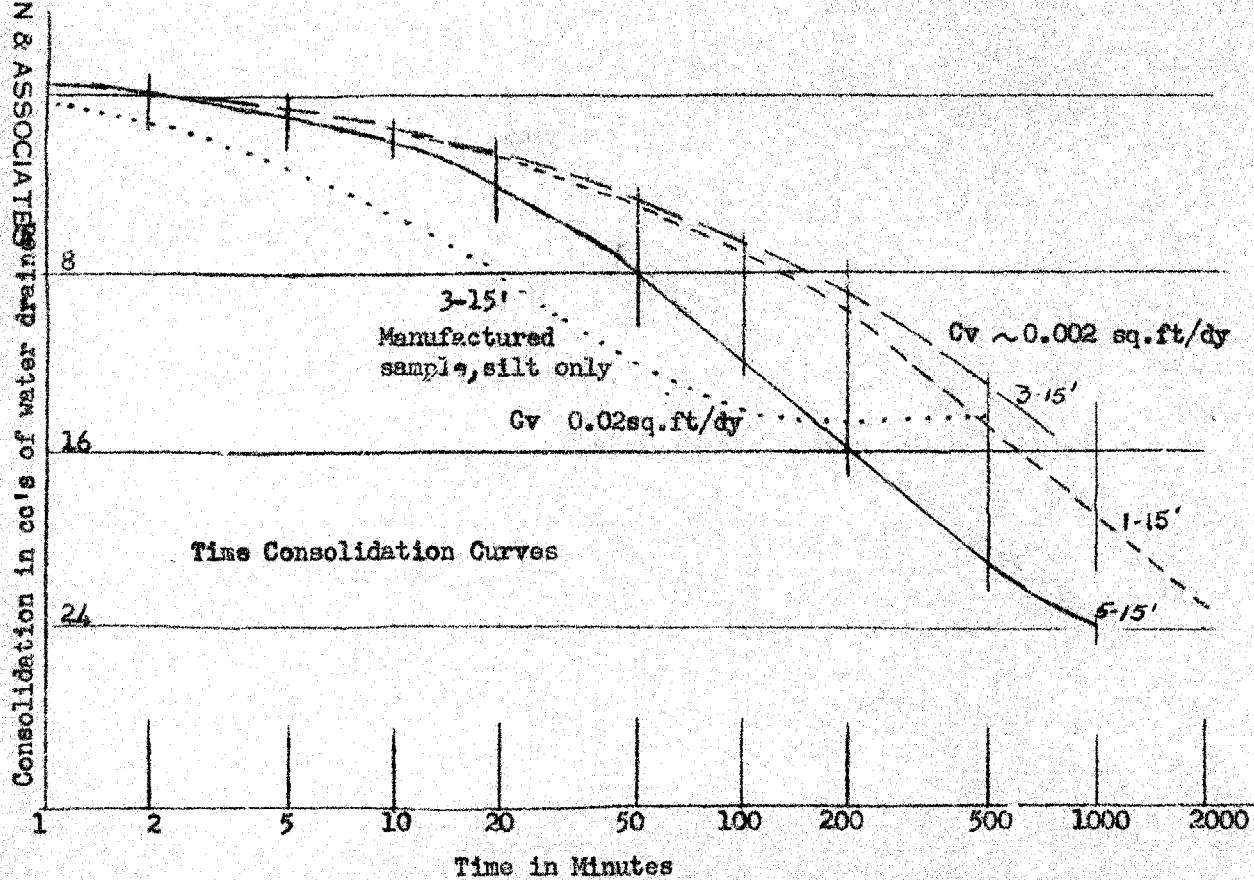
RESULTS OF SLOW DRAINED TRIAXIAL TESTS ON SAMPLES FROM DEPTH OF 8 FEET IN HOLE 6



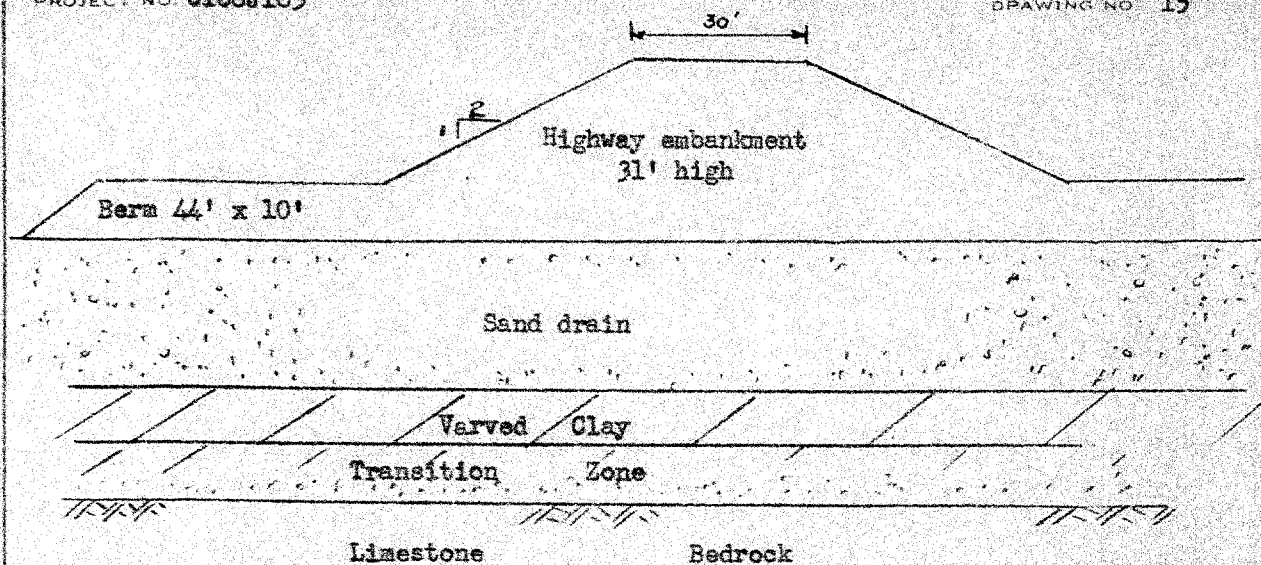
UNDRAINED TRIAXIAL STRESS STRAIN CURVES

TROW SODERMAN & ASSOCIATES

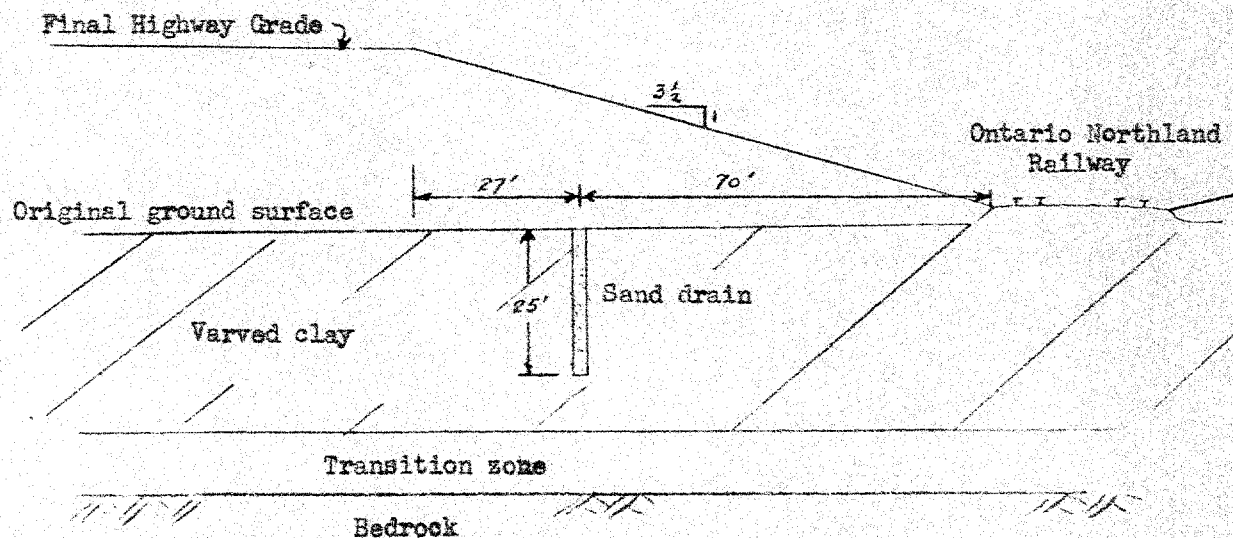
TROW SODERMAN & ASSOCIATES



RESULTS OF CONSOLIDATED UNDRAINED TESTS ON FOUR SAMPLES
CONSOLIDATED AT CELL PRESSURE (c.p.) OF 20 psi



Section of the embankment approaches at height 31 feet



LONGITUDINAL PROFILE SHOWING RECOMMENDED EMBANKMENT AND DRAIN REQUIREMENTS ADJACENT TO THE RAILWAY

Sliding Block Analyses of Existing Embankment Failure

Conditions: Failure height = 26 feet. side slopes $1\frac{1}{2}:1$

Assumptions: Water table - depth of 1 foot.

Soil Prop. - as shown

Distribution of Embankment pressure, N, from Carothers tables for $\frac{1}{2}$ triangular loading plus loading for edge of continuous footing.

Earth pressure - at rest condition in embankment, i.e., $= k(\frac{1}{2}\gamma h^2)$ where $k = 0.5$.

Total Active Pressure

8 ft. = 28,976 plf

20 " = 60,608 plf

35 " = 96,203 plf

Total Passive Resistance

8 ft. = 23,348 plf

20 " = 46,838 plf.

35 " = 85,313 plf

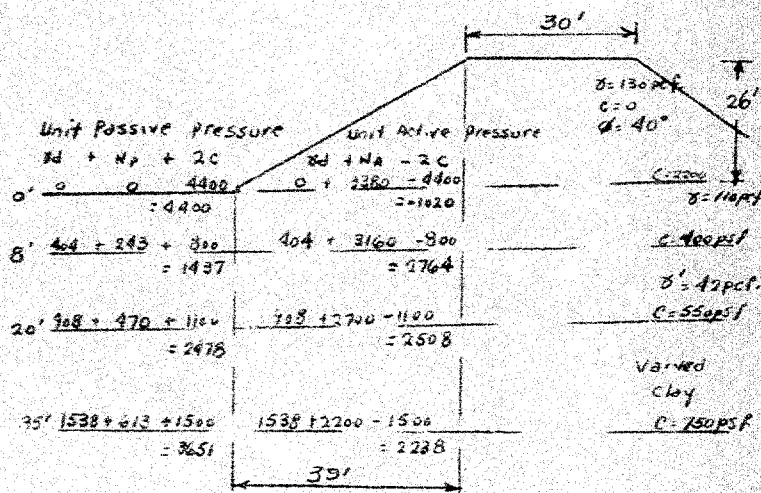
Sliding Resistance

Clay

8 ft. = $400 \times 39 = 15,600$

20 " = $550 \times 39 = 21,450$

35 " = $750 \times 39 = 29,250$

Factor of Safety for slide in Clay

$$8 \text{ ft.} = \frac{23348 + 15600}{28976} = 1.34$$

$$20 \text{ ft.} = \frac{46838 + 21450}{60608} = 1.3$$

$$35 \text{ ft.} = \frac{85313 + 29250}{96203} = 1.18$$

Sliding Resistance in silt at 8 feet at time of failure
 $= 39 \times N \tan \phi = 39 \times 404 \times .58 = 9100 \text{ plf.}$

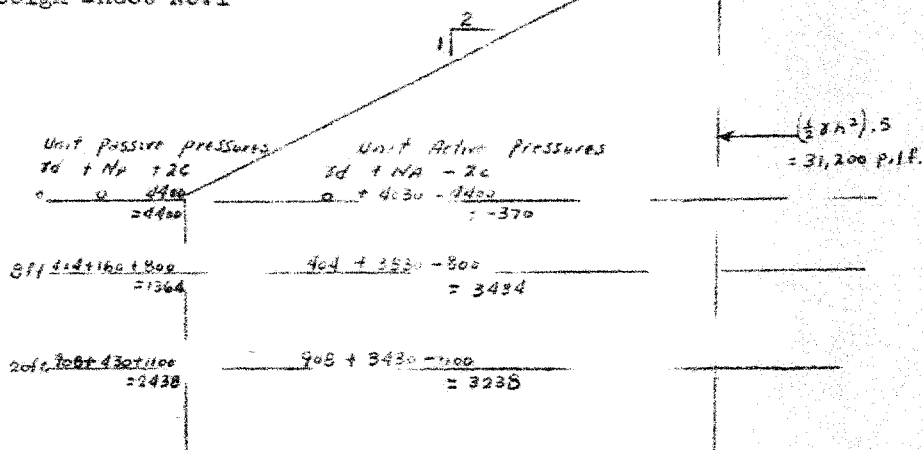
Factor of safety against sliding just over unity.

PROJECT NO. C108J163

DRAWING NO.

COMPUTATIONS OF PERMISSIBLE SAFE PORE PRESSURES FOR 2:1 EMBANKMENT SLOPE AND H = 31 FEET

Soil Values as in Design Sheet No. 1

 P_A

$$8 \text{ ft.} = 31,200 + \frac{-370 + 3434}{2} \times 8 = 43,456$$

$$20 \text{ ft.} = 43,456 + \frac{3434 + 3238}{2} \times 12 = 83,488$$

 P_p

$$8 \text{ ft.} = \frac{4400 + 1364}{2} \times 8 = 23,056$$

$$20 \text{ ft.} = 23,056 + \frac{1364 + 2438}{2} \times 12 = 45,868$$

Average normal pressure on plane of failure

$$8 \text{ ft.} = 2414 \text{ psf}$$

$$20 \text{ ft.} = 2918 \text{ psf}$$

$$(P_A - P_p) 8 \text{ ft.} = \frac{20400}{62} = 330 \text{ psf}$$

$$(P_A - P_p) 20 \text{ ft.} = \frac{37620}{62} = 607 \text{ psf}$$

