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SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

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JAN 12 1960

Project: J 608

January 6, 1961.

De Leuw Cather & Co. of Canada Ltd.,
1491 Yonge St.,
Toronto, Ont.

Attention: Mr. H. Van Bodegom

WP

Foundation Investigation
Proposed Overhead - Michigan Central Rly.,
County Road No. 11

Dear Sirs:

The enclosed report contains the results of a foundation investigation completed in December 1960 at the above-noted railway crossing.

As indicated during recent conversations on this subject, the site is underlain by a very deep deposit of lean clay which continues for a depth of 112 feet, the estimated level of bedrock. As a result of surface drying, the top approximately 40 feet of this clay exists in an overconsolidated state. The degree of overconsolidation is very great within the first approximately 15 feet below the surface, but the effects of surface drying become less marked with increasing depth. Below 40 feet, the soil is in equilibrium with present overburden pressures only.

Two thick strata of coarse silt are interbedded with this clay deposit. The thicknesses of these layers vary in a north-south direction, particularly on the west side of the railway. Because of this fact and because of the oblique angle of the crossing, there is a tendency for differential settlement across the width of the bridge structure.

Although the safe bearing value of the soil is in the order of 9000 p.s.f. at and below Elev. 602 feet, considerable settlement of this bridge structure must be anticipated. This movement will be due almost entirely to the weight of the approach fill.

Settlement calculations indicate that about 40 percent of the foundation compression, including all elastic movements, will be complete by the time the embankment fill has been raised to full height. At this stage of construction the abutments will have settled considerably more than the piers. In addition, because of the greater weight of fill, the obtuse corners of the bridge will be lower than the acute ends.

In order to anticipate this large abutment settlement, it is recommended that they be built about 3 inches too high. By doing this, they will be closer to the desired level when construction is to be started on the superstructure. Some additional 'building up' of the bearing surfaces of the abutments may be required at this stage, but the adjustments should not be as great as the settlement calculations of this report indicate.

The computed maximum differential settlement occurring between the abutments and piers during and following the placement of girders and decking is about 5 inches. Again, a lesser amount of actual movement is anticipated. Provision for this movement could be made by installing simple screw jacks under the bridge girders. In this same final period, the estimated maximum settlement along the length of the bridge piers is computed to be 2 inches. The settlement of the bridge and approach fill should be essentially complete within a period of 4 years. A reduction in overhead clearance of the order of $3\frac{1}{2}$ inches has been estimated as a result of the settlement of the bridge piers.

The computed maximum settlement of the highest portions of the embankment is in the order of 23 inches. In the estimation of actual fill requirements however, it is recommended that an allowance for a total settlement of 15 inches, only, should be made. A linear decrease in settlement with lower fill heights should be assumed.

There will be a definite differential compression of the soil under the embankment along the path of the 6 inch gas main which passes between the two lines of hydro towers a few feet west of the proposed bridge structure. The probable differential movement is in the order of one foot over a length of 60 feet. If this differential settlement cannot be tolerated, the gas line should be re-routed.

If all settlement of the bridge is to be avoided, the structure must be founded on H piles driven to rock. The disadvantage of this costly program is that severe differential settlement will be experienced between the bridge and its approaches. Friction piles will not perform satisfactorily because a large proportion of the settlement occurs in the more compressible soil found below the 40 foot depth.

A source of Class B gravel has been located about 4 miles away on McKenny Road, south-east of Cook's Mills. This appears to be the closest pit to this project.

We hope that the comments of this report are of assistance to you in the preparation of foundation designs for this structure. We shall be pleased to review any particular aspect of this investigation, or the observations and conclusions arising out of this study, which you may wish to discuss.

It has been our pleasure to serve you in this regard.

Yours very truly,

W. Trow

William A. Trow (P. Eng.)

Encls.
WAT/lt

DE LEUW CATHER & CO. OF CANADA LIMITED
1491 YONGE ST., TORONTO, ONT.

FOUNDATION INVESTIGATION
PROPOSED OVERHEAD - MICHIGAN CENTRAL RLY.,
COUNTY ROAD NO. 11

Project: J 608 January 6, 1961

William A. Trow & Associates Ltd.

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FOUNDATION INVESTIGATION
PROPOSED GRADE SEPARATION
COUNTY RD. NO. 11 AND M.C.R.R.
WELLAND, ONTARIO

Project

The Michigan Central Railroad crosses County Road No. 11, about 2 miles east of the city of Welland. It is proposed to eliminate the existing level crossing at this site by the construction of an overhead bridge. This overhead will consist of a three span structure with embankment approaches.

This report describes the soil conditions encountered during the field investigation at this site. Recommendations are made concerning the most suitable type and depth of foundation, the safe bearing capacity of the soil and the probable settlements to be anticipated under the bridge foundations and embankment approaches. The availability of suitable embankment fill in the locality is also discussed.

Site Description

This site is located at the skew crossing of the Michigan Central Railroad and County Road No. 11. The double track railroad has been constructed about 4½ to 6 feet above the surrounding flat countryside. Small embankment approaches bring the existing road up to this height at the crossing.

The site lies in part of an extensive, essentially level clay plain formed by the glaciolacustrine deposits of Lake Warren. This lake was one of a succession of glacial lakes which, at one time, covered south western Ontario. The site is relatively well drained, and is surrounded by farmlands with a few houses situated a short distance to the west.

The proposed bridge will be a three span skew structure, with open type abutments which allow the approach fill to spill through. The approaches will be constructed to a maximum height of 32 feet at the abutments with 2:1 side slopes.

Numerous underground services are located in and around this site. In particular a 6 inch provincial gas main runs 140 feet west of and parallel to the railroad track. In its present position this line will be overlaid by the maximum height of approach fill.

Due to the extensive nature of this lacustrine clay plain, very few areas of suitable granular fill material are found in the locality. Such deposits that do exist are the remains of the old lake beach. The largest deposits of such material lie just west of Fonthill, about 11 miles from the site, where two quarries are currently being worked. An old pit on McKenny Road, located about 4 miles south east of the site was also appraised during the field investigation.

Subsoil Description

Initially four borings were made at the site at the locations indicated on the accompanying drawing No. 1. A fifth borehole, E, was then put down to further define the level and thickness of the various strata.

The soil types encountered in each hole, together with a record of some of their physical properties are presented in drawings 2 to 6. The relevant information from these borehole logs has been summarized and presented in drawing No. 1 in the form of estimated subsoil stratigraphical profiles.

Reference to this drawing shows 6 to 12 inches of topsoil overlying a stratum of stiff to hard red brown silty clay with gravel sizes. The upper 10 to 15 feet of this soil exists in a dessicated hard condition, with moisture contents almost at the plastic limit. Measured undrained shear strengths, in this upper zone were in excess of 4750 psf. The lower horizon of this soil is considerably less stiff with shear strengths in the order of 1000 psf. The stratum is terminated in hole A at a depth of $24\frac{1}{2}$ feet or El 582 feet and rises slightly in a westerly direction to El 586 feet in hole E. On the north side of the road, however, this stratum extends to a depth of about 34 feet or El 575 feet in hole B and again rises about 2 feet in a westerly direction to El 577 feet in hole D.

This stiff clay deposit rests on a stratum of medium dense cohesionless medium to coarse red brown silt. The thickness of this deposit varies considerably from $3\frac{1}{2}$ feet in hole D to an average maximum of 20 to 25 feet in the other holes.

Underlying this cohesionless soil a deposit of medium stiff red brown silty clay was found and this material extends to the full depth of boring or 100 feet in holes A and D. Laboratory tests indicate that this clay is in a "normally loaded" state which term implies that it has become adjusted only to the weight of the present overburden. A thin stratum of cohesionless silt, similar to that mentioned above, was encountered below 76 feet or El 530 feet in hole A and in hole D it was found to extend between El 530 and El 520 feet.

Bedrock was not proven in these holes but a cone was driven to refusal at 112 feet in hole D. Drawing 1a indicates the bedrock levels from well drilling records in the vicinity. They are in agreement with this refusal depth.

Water level observations in the boreholes show that the water table is located 2 to 3 feet below ground surface or at El 604.5 feet. The higher level recorded in hole C is believed to be due to the entry of surface runoff during an overnight storm.

Overhead Bridge Foundations

In the previous section it was mentioned that approximately 10 to 15 feet of hard desiccated silty clay was found to exist at the surface of this site. This clay is entirely competent to support both the overhead structure and the approach embankments. Spread footings for the bridge should be located 4 feet below the ground surface for frost protection and to avoid soil temporarily softened by frost action. From the levels obtained during the field investigation this depth corresponds approximately to El 602 feet.

The safe allowable net bearing pressure to use for these footings may be computed from the expression:

$$q = \frac{N}{F} C$$

where: F is the factor of safety required to keep settlement within tolerable limits; a value of F = 3 is usually applied

C is the undrained shear strength of the soil under the footing, assumed from triaxial tests to have an average minimum value of 4,750 psf;

and N is a bearing capacity factor which depends upon the shape and the depth to breadth ratio of the footing. For a continuous strip type footing near the ground surface N = 5.70.

Substituting these values in the above expression a safe pressure in excess of 9000 psf is obtained. According to current practice this is close to the maximum pressure usually permitted for a soil of this type. This limitation provides a margin of safety in case of undetected local pockets of softer material. When using this pressure, footings should be designed to carry the dead weight of the structure and overlying approach fill plus the maximum possible live load. Practical design considerations may, however, require larger footings than will be determined from the above bearing value.

Excavation for the footings in this material should present no problems. The sides of the excavation can be cut almost vertical. Because of the impermeable nature of the clay, ground water flow into the excavation will be negligible.

Heavily desiccated clays of this type, however, soften slightly on prolonged contact with free water. Therefore, the final 9 inches of soil in the footing area should not be excavated until immediately prior to the pouring of concrete.

As mentioned previously the proposed approach embankment will remain quite stable. Vegetation and other compressible surface material should first be removed before constructing the approaches.

Settlement of Bridge Structure and Approach Fill

As mentioned in a previous section, the soil underlying the site consists of approximately 110 feet of silty clay intersected by two distinct strata of coarse silt.

The various field and laboratory test results indicate that the effects of surface drying and consequent over consolidation of the silty clay extend down to a depth of about 40 feet below the ground surface. At greater depth the clay appears to be "normally loaded" which term signifies that it is in equilibrium under present overburden weights only. These observations are supported by the strength-depth relationship shown on drawing 7 and by the estimated preconsolidation pressures of samples from various depths shown on drawings 8 to 11.

The distribution of pressure below El 602 feet from the footings of the bridge structure will be confined essentially to the very stiff desiccated clay near the surface. Settlements under the footings due to sustained bridge loads, therefore, will be small.

However the influence of the weight of the approach fill extends to the full depth of the clay deposit. Appreciable settlements will therefore occur due to the normally loaded nature of the softer clay ground below about 40 feet.

The variations in thickness of the coarse silt strata have been mentioned previously. Consolidation due to expulsion of pore water will be almost negligible in this soil. Therefore where a large difference in the thickness of this stratum occurs over a short distance, such as between holes D and E, appreciable differential settlements will occur. In addition the unsymmetrical loading from the approaches due to the large skew will induce additional differential movement.

Consolidation tests were performed on three samples from hole D and on one sample from hole A. The results of these tests are presented in drawings 8 to 11. Corrections have been made, according to accepted procedures, in order to account for the effects of sample disturbance and to simulate, as closely as possible, the field consolidation line. At a depth of 24 feet the soil appears to be over consolidated by a pressure of about 3500 psf in excess of the overburden weight. Since the pressures from the embankment will not exceed this loading, settlements in this zone will be small.

Samples from depths of 44 feet and 71 feet, although somewhat disturbed because of the presence of gravel sizes, indicate the previously mentioned normally consolidated condition existing at this depth.

Assigning appropriate values of the coefficient of compressibility, m_v, as determined from the foregoing tests, settlements under each end of the piers and abutments were computed. These computations are outlined in Appendix B. Account has been taken of the relative stiffness of the upper desiccated clay compared to the lower deposits of more compressible material, and of the modification of the stress distribution into the soil resulting from this lack of uniformity.

According to the computations in Appendix B, a maximum settlement of about 10 to 12 inches will occur at the abutment locations, and the differential movement between abutments and piers will be in the order of 8 inches. Because of variations in the intensity of embankment loading and differences in thickness of compressible clay a differential movement in the order of 3 to $3\frac{1}{2}$ inches has been computed along the length of the bridge piers.

The accuracy of these settlement estimates can be verified to some extent by recent settlement observations made by the Department of Highways for embankment fill placed at another location in the general vicinity of this site. The thicknesses of compressible soil are somewhat less in this instrumented example. The total movement at the end of construction, or after a period of $1\frac{1}{2}$ months, was observed to be in the order of 4 inches. At this time the consolidation of the subsoil appeared to be well advanced. At the present time, after a period of about 6 months the movement is in the order of 5 inches. Some additional long term settlement is anticipated.

On the basis of this information it would appear that the estimates of settlement made in this report, are too large even though allowances have been made for sample disturbance, surface crust effects and other factors. In addition it should be appreciated that settlement of the structure will be well advanced by the time the approach embankments have been constructed to full height. Therefore, if the superstructure of the bridge is built after this time, the disturbance to it should not be nearly as great as these estimates would suggest.

According to theoretical computations illustrated in Dwg. 16, about 40 percent of the settlement will be complete by the time the embankments are up to full height. If work on the superstructure is delayed to this time, then only about 60 percent of the above movements should be experienced. In addition, if the pouring of the bridge deck is delayed for another 3 months, and if bridge girders are raised level at this time, only about 45 to 50 percent of the computed settlement should be expected. In view of the great lengths of the piers and abutments the resultant differential movements should be acceptable.

The presence of a gas main under the proposed west approach embankment has been mentioned in the previous section of the report. If the line is maintained in its present position, it will settle with the surrounding ground under the embankment. It is believed that the line should be flexible enough to accommodate these settlements without damage to the pipe. The estimated differential movements under the embankment in the vicinity of the gas main are indicated on Figure 3 of Appendix B. A differential movement in the order of 15 inches over a length of 60 feet is indicated. This estimate is believed to be too high. However it may be advisable to relocate the line to avoid future maintenance problems under the approaches.

Settlement of the abutments and piers can be eliminated by founding these elements on end bearing steel H - piles driven to bedrock. Relative movement of 12 inches between approaches and abutment will still occur. It is not considered, however, that the elimination of bridge settlement justifies the extra expense of such piles. Friction piles cannot be used satisfactorily at this site because the main body of compressible soil lies at great depth below 40 feet.

In view of the anticipated foundation movements it is understood that consideration is being given to an overhead structure which incorporates simply supported spans. Although the differential settlements within this bridge should not be too great if work on the superstructure is delayed as indicated above, it may be desirable to provide a system of jacks under the bridge girders. Adjustments then can be made as required.

Granular Fill

As mentioned in a previous section of this report, the nearest commercial quarries are located at Fonthill, about 11 miles from this site. Two suitable quarries at that Village are Moyer's Ltd. and Fonthill Sand and Gravel Ltd. It has been confirmed that these quarries can supply material conforming to the Dept. of Highways specifications for Class A granular base course, although the gravel has not always met all the requirements of this standard. Because of this fact, it is recommended that tests be performed at frequent intervals to ensure compliance with the specifications, if these sources of supply are to be utilized.

An old pit beside McKenny Road about 4 miles south-east of the site was appraised during the field investigation. Several samples of the exposed soil, believed to be representative of sections of this quarry, were obtained. The results of grading tests performed on these samples are presented in Dwg. 8 and Dwg. 9.

These results show that the gravel in the unworked area in the north and north-east parts of the pit is satisfactory for Class B granular base course. It is estimated that this area contains at least 15,000 cubic yards of this material.

Two samples of sandy soil were also obtained from previously worked areas to the south of the quarry pond. According to the grain size distribution curves shown on Dwg. 13, these soils are poorly graded and are not suitable as Class B material.

It is recommended that frequent grading tests be carried out during excavation of the gravel from this site to ensure that it complies with the Class B grading. It is possible that a minor amount of screening may be required, although the grading curves do not indicate the need for this.

If acceptable as a source from an economic and aesthetic point of view, the top very stiff to hard clay existing in the first approximately 10 feet below the ground in this general railway crossing area should be suitable to supplement this granular fill for the lower courses of the embankment. Its moisture content lies at or below the plastic limit, which condition coincides with that required for optimum density.

Conclusions

The conclusions and recommendations presented in this report are summarized as follows:

1) The soil at this site consists essentially of a medium stiff to stiff silty clay, with a hard dessicated surface crust 10 to 15 feet thick. Interbedded with this soil are two coarse silt strata which vary in thickness over the site. Although not proven, bedrock is assumed to lie about 112 feet below the surface. According to several well drilling records, this is the level where bedrock was intersected.

2) The overhead bridge can be founded on spread footings located 4 feet below the present ground surface or at approximate Elev. 602 ft. A safe bearing pressure of 9000 p.s.f. is recommended.

3) The maximum settlement at the abutment locations has been computed to be about 12 inches. According to theoretical computations and laboratory tests, the differential movement between the abutments and piers and also along the length of these members will be considerable as well. On the basis of field measurements that are available for somewhat similar installations, however, these estimates of foundation movement are believed to err

on the high side. In addition, the settlement following the installation of the bridge superstructure will be of a much smaller order since about 40 to 50 percent of the overall foundation movement will have occurred prior to this time. In making this statement, it is assumed that the embankment construction has been completed. The greatest differential settlement of the superstructure after this time has been estimated to be in the order of 2 inches. This movement will occur along the length of the east pier.

4) The theoretical settlement of the interior piers has been computed to vary between 2 and 5 inches. For purposes of providing clearance over the railway, a movement of about $3\frac{1}{2}$ inches should be anticipated.

5) As stated above, the maximum settlements of the abutments have a computed range between 10 and 12 inches. About 4 to 5 inches of this movement should be complete at the end of embankment construction. These estimates probably err on the high side. However, a large percentage of this differential settlement must take place during construction, as evidenced by the D.H.O. measurements referred to in the report. Therefore, it should be safe to anticipate this movement by building the abutments about 3 inches too high. By the time the superstructure is to be installed, the abutments should be at the required level or possibly somewhat lower. The balance of the movement can be provided for by shimming or other means before the bridge girders are placed. If settlement plates are installed, a more reliable prediction of abutment and pier movements can be made.

6) Theoretical computations indicate that the maximum settlement of the embankment, well back from the abutments, will be about 23 inches. This estimate, again, is probably somewhat severe. For fill estimating purposes, it is suggested that provision be made for a total of 15 inches of movement. A linear decrease in this settlement should be assumed for lower embankment fills both laterally and longitudinally along the centre line of the fill.

7) Theoretically, a differential settlement in the order of 15 inches can be anticipated between the crest and toe of the highest parts of the embankments. This movement acts over a length of 60 feet. This differential settlement should not be detrimental to a flexible gas pipe line. All settlement should be essentially complete after a period of about 4 years.

8) In view of the settlements to be expected at this site, it is recommended that the design of the bridge incorporate simply supported girders. It is understood that this design scheme is being considered.

9) Granular fill conforming to D.H.O. specifications for Class A base course is obtainable from either of two pits at Fonthill.

10) A suitable deposit of gravel, conforming to D.H.O. Class B specifications for granular base course, is located in a quarry on McKenny Road. There appears to be sufficient material at this location to supply all of the Class B needs for this project.

P.G.M.Imrie

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Jan.6/61

Peter G. Imrie (P. Eng.)

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APPENDIX A

Field Investigation and Laboratory Procedures

Initially all holes were put down with continuous flight auger equipment. The holes were 5 inches in diameter and were uncased. Hobes B , C, D and E were advanced to their full depths in this manner. Similarly hole A was bored to a depth of 50 feet, where further progress was precluded due to the collapse of the surrounding wet silt. Hole A1, put down nearby, was cased to 68 feet below which depth the unsupported hole remained open. This hole was advanced by washing ahead.

Undisturbed samples of the clay were obtained by levering or driving 2 inch or 3 inch inside diameter shelby tube piston samplers into the soil ahead of the boring. Near the surface where the clay was very stiff, open drive shelby tube samplers were used. Disturbed samples of clay and cohesionless material were obtained with a standard 2 inch outside diameter split spoon sampler. The penetration resistance of the soil so obtained was determined by the number of 350 ft. lb. hammer blows required to extend the penetration of the spoon from 6 inches to 18 inches.

In-situ shear strength measurements of the undisturbed clay soils were carried out using vanes 2-5/8 inches and 2-1/8 inches in diameter depending upon the clear diameter of the hole. After test the soil was completely remoulded and the test was repeated. The ratio of the undisturbed to remoulded strength is defined as the sensitivity of the soil. The very stiff soil near the ground surface was found to be beyond the capacity of the vane.

The shelby tube samples were sealed at the site and the disturbed samples were wrapped in tinfoil and placed in moisture proof plastic bags. In the laboratory, natural water content determinations were carried out on all cohesive samples and Atterberg limit tests performed on selected specimens. Representative shelby tube samples were subjected to triaxial compression tests with cell pressure at least equal to overburden pressure. The undrained shear strength of the soil is computed as one half of the deviator stress at failure. Natural unit weights of the soils was calculated from the weight of the cylindrical specimens. Consolidation tests were performed on representative shelby tube samples to provide information for a settlement analysis.

Ground water level in the holes was observed during the field testing period until no further rise occurred.

Representative samples of granular material were obtained from the previously mentioned quarries. In the laboratory, grading tests were performed on these samples.

The records of these field and laboratory measurements are recorded on the borehole logs as drawings 2 to 6 of this report. Stress-strain curves of the triaxial tests are shown on drawings 14 to 15 and the consolidation test results on drawings 8 to 11. The grading curves of the McKenny Road pit samples are presented in drawings 12 and 13.

The levels of the ground beside all holes were referred to the top of rail as shown on drawing 1.

APPENDIX B

Settlement Computations

A. General

The approaches are assumed to have 2:1 side slopes, a top width of 66 feet and to possess a unit weight of 130 pcf. The assumed load distribution is indicated in Fig. 1. The abutments and intermediate piers are assumed to have 14 feet and 7 feet wide continuous footings, 95 feet long, respectively. The abutment footings carry 1600 kips and the piers 1800 kips due to sustained bridge loads.

The pressure increments at any depth due to the above loads are determined by using a Newmark chart.

The following thicknesses of clay soil are assumed in which consolidation settlement occurs.

1) West Abutment and Pier

Depth (ft.)	North End (ft.)	South End (ft.)
0–20	20	20
20–40	15	5
40–60	20	10
60–80	20	20
80–100	10	10

2) East Abutment and Pier

Depth (ft.)	North End (ft.)	South End (ft.)
0–20	20	20
20–40	15	5
40–60	10	15
60–80	15	15
80–100	20	20

In both 1) and 2) relatively incompressible material is assumed below 100 feet.

Computations both of the immediate or elastic settlement occuring as weight is applied and of longer term consolidation settlement have been made.

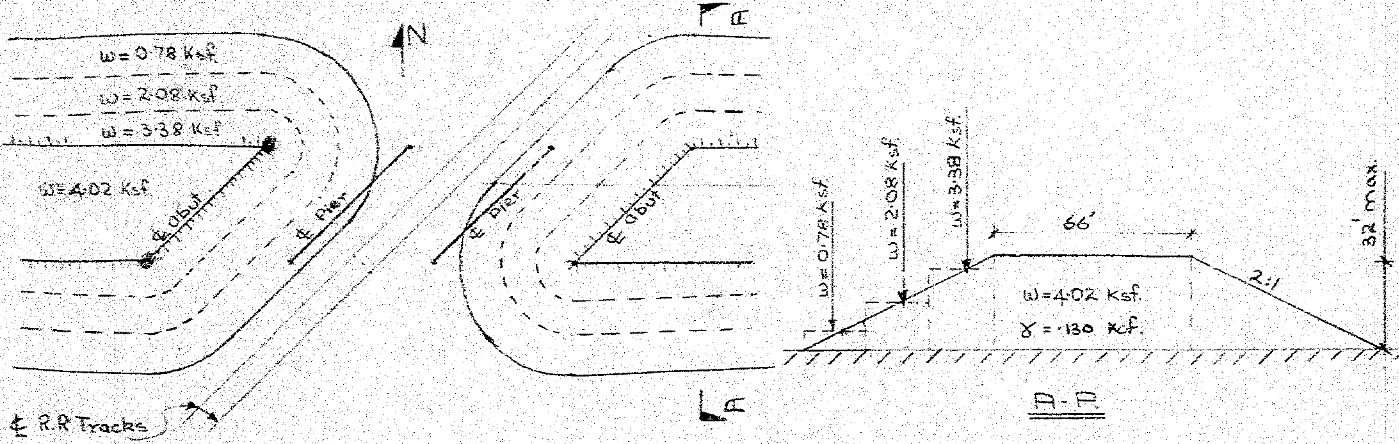

Fig. 1

Sketch showing assumed distribution of embankment load.

B. Consolidation Settlement

The expression used for computing this movement is:

$$\Delta H_c = (H . \Delta p . m_v)\mu$$

where: H is the depth of the stratum considered,

Δp is the average increase in stress within the stratum

m_v is the appropriate coefficient of compressibility as determined from the consolidation tests and

μ is a factor relating estimated field consolidation to laboratory test results. *

The average values of m_v and μ used in these calculations are:

0-40 feet	m_v = 0.0026 ft.^{2}/kip	μ = 0.6
40-100 feet	m_v = 0.010 ft.^{2}/kip	μ = 0.8

For settlements due to abutments and pier loads μ = 0.5.

* " A Contribution To The Settlement Analysis of Foundations on Clay" A.W. Skempton and L. Bjerrum. Geotechnique, Vol. 7, No. 4

1) West Abutment

a) Due to Approach Fill

Depth (ft.)	μ	m_v ft.^{2}/kip	North End Δp (ksf)	H (ft.)	ΔH_c (ins.)	South End Δp (ksf)	H (ft.)	ΔH_c (ins.)
0-20	0.6	.0026	3.42	20	1.29	3.64	20	1.37
20-40	0.6	.0026	2.84	15	0.79	3.25	5	0.30
40-60	0.8	.010	2.22	20	4.26	2.80	10	2.66
60-80	0.8	.010	1.80	20	3.45	2.45	20	4.70
80-100	0.8	.010	1.45	10	1.39	2.04	10	1.95
					11.18			10.98

b) Due to Sustained Bridge Loads

Settlement due to these loads is negligible below 20 feet. Assuming 30 degree load spread, the average pressure increase, Δp, between 4 - 20 feet, is 0.66 ksf.

$$\Delta H_c = (m_v . \Delta p . H) u$$

$$= (.0026 \times .66 \times 192)\ 0.5$$

$$= 0.16 \text{ ins.}$$

2) West Intermediate Pier

b) Due to Approach Fill

Depth (ft.)	μ	m_v ft.^{2}/kip	North End Δp (ksf)	H (ft.)	ΔH_c (ins.)	South End Δp (ksf)	H (ft.)	ΔH_c (ins.)
0-20	0.6	.0026	0.07	20	0.03	0.86	20	0.32
20-40	0.6	.0026	0.17	15	0.05	1.06	5	0.10
40-60	0.8	.010	0.26	20	0.50	1.13	10	1.09
60-80	0.8	.010	0.32	20	0.62	1.06	20	2.03
80-100	0.8	.010	0.37	10	0.36	1.00	10	0.96
					1.56			4.50

b) Due to Sustained Bridge Loads

Settlement due to these loads is negligible below 20 feet. Average pressure increase, Δp, at either end between 4 – 20 feet is 1.06 ksf.

$$\Delta H = (m_v . \Delta p . H) \mu$$

$$= (0.0026 \times 1.06 \times 192)\ 0.5$$

$$= 0.265 \text{ ins.}$$

3) East Abutment

a) Due to Approach Fill

			North End			South End		
Depth (ft.)	μ	m_v (ft²/kip)	Δp (ksf)	H (ft.)	ΔH (ins.)	Δp (ksf)	H (ft.)	ΔH (ins.)
0–20	0.6	0.0026	3.64	20	1.37	3.42	20	1.28
20–40	0.6	0.0026	3.25	15	0.92	2.84	5	0.26
40–60	0.8	0.010	2.80	10	2.69	2.22	15	3.20
60–80	0.8	0.010	2.45	15	3.54	1.80	15	2.59
80–100	0.8	0.010	2.04	20	3.92	1.45	20	2.79
					12.44			10.12

b) Due to Sustained Bridge Loads

As in 1 b) above ΔH_c = 0.16 ins.

4) East Pier

a) Due to Approach Fill

			North End			South End		
Depth (ft.)	μ	m_v (ft²/kip)	Δp (ksf)	H (ft.)	ΔH (ins.)	Δp (ksf)	H (ft.)	ΔH (ins.)
0–20	0.6	0.0026	.86	20	0.32	0.07	20	0.02
20–40	0.6	0.0026	1.06	15	0.30	0.17	5	0.01
40–60	0.8	0.010	1.13	10	1.09	0.26	15	0.38
60–80	0.8	0.010	1.06	15	1.52	0.32	15	0.47
80–100	0.8	0.010	1.00	20	1.92	0.37	20	0.71
					5.15			1.59

b) Due to Sustained Bridge Loads
As in 2 b) above ΔH_c = 0.26 ins.

C) Immediate Settlement

The immediate or elastic settlement of the soil can be computed from the expression:

$$\Delta H_i = \frac{0.5\,\Delta pH}{E} \quad *$$

where Δp is the average increase in pressure due to the weight of the applied loads.
H is the depth of the stratum
E is the "elastic" modulus of the soil.

The appropriate values of E for the clay soil are estimated from a consideration of the stress-strain curves and the undrained shear strength of this soil with allowances made for sample disturbance. The corresponding value for the coarse silt soils is obtained from a relationship between this modulus and the penetration resistance.**

These values are:

Clay	0-40 feet	E = 800 kips/ft.^{2}
Clay	40-100 feet	E = 200 kips/ft.^{2}
Silt		E = 400 kips/ft.^{2}

1) West Abutment

a) Due to Approach Fill

Depth	E	North End			South End		
		Δp	H	ΔH_i	Δp	H	ΔH_i
(ft.)	(ksf)	(ksf)	(ft.)	(ins.)	(ksf)	(ft.)	(ins.)
0 - 20	800	3.42	20	.51	3.64	20	.54
20 - 40	800	2.84	15	.32	3.25	5	.12
	400	2.84	5	.21	3.25	15	.73
40 - 60	200	2.22	20	1.33	2.80	10	.84
	400	2.22			2.80	10	.42
60 80 - 80	200	1.80	20	1.08	2.45	20	1.47
80 -100	200	1.45	10	.43	2.04	10	.61
	400	1.45	10	.22	2.04	10	.30
				4.10			5.03

* $\Delta H_i = \int_0^z \left(\frac{\Delta\sigma_z}{E} - \frac{\mu\Delta\sigma_x}{E} - \frac{\mu\Delta\sigma_Y}{E}\right) = \int_0^z 0.5\frac{\Delta\sigma_z}{E}$ for $\Delta\sigma_x = \Delta\sigma_Y = \frac{1}{2}\Delta\sigma_z$ & $\mu = 0.5$

** 'Imperial College Lectures in Soil Mechanics' - 1959-60

b) Due to Sustained Bridge Loads

The computation is similar to that for consolidation settlement, but 0.5/E is substituted for m_v

$$\Delta H_i = 0.5/E.\ \Delta p.H.$$

$$= (\frac{0.5}{800} \times 0.66 \times 192)$$

$$= \underline{0.08 \text{ inches}}$$

2) West Intermediate Pier

a) Due to Approach Fill

Depth (ft.)	E (ksf)	North End Δp (ksf)	North End H (ft.)	North End ΔH_i (ins.)	South End Δp (ksf)	South End H (ft.)	South End ΔH_i (ins.)
0–20	800	0.07	20	.01	0.86	20	0.13
20–40	800	0.17	15	.02	1.06	5	0.04
	400	0.17	5	.02	1.06	15	0.23
40–60	200	0.26	20	.15	1.13	10	0.34
	400	0.26			1.13	10	0.17
60–80	200	0.32	20	.19	1.06	20	0.63
80–100	200	0.37	10	.11	1.00	10	0.30
	400	0.37	10	.05	1.00	10	0.15
				0.55			1.99

b) Due to Sustained Bridge Loads

The computation is similar to that for consolidation settlement, but 0.5/E is substituted for m_v

$$\Delta H_i = \frac{0.5}{E} \Delta pH$$

$$= (\frac{0.5}{800} \times 1.06 \times 192)$$

$$= 0.12 \text{ ins.}$$

3) East Abutment

a) Due to Approach Fill

Depth (feet)	E (ksf)	North End Δp (ksf)	H (ft.)	ΔH_i (ins.)	South End Δp (ksf)	H (ft)	ΔH_i (ins.)
0-20	800	3.64	30	0.54	3.42	20	0.51
20-40	800	3.25	15	0.36	2.84	5	0.10
	400	3.25	5	0.25	2.84	15	0.64
40-60	200	2.80	10	0.84	2.22	15	1.00
	400	2.80	10	0.42	2.22	5	0.16
60-80	200	2.45	15	1.10	1.80	15	0.81
	400	2.45	5	0.18	1.80	5	0.13
80-100	200	2.04	20	1.23	1.45	20	0.87
				4.92			4.22

b) Due to Sustained Bridge Loads

As in 1(b) above ΔH_i = 0.08 inches

4) East Pier

a) Due to Approach Fill

Depth (ft.)	E (ksf)	North End Δp (ksf)	H (ft.)	ΔH_i (ins.)	South End Δp (ksf)	H (ft.)	ΔH_i (ins.
0-20	800	0.86	20	0.13	0.07	20	0.01
20-40	800	1.06	15	0.12	0.17	5	0.01
	400	1.06	5	0.08	0.17	15	0.04
40-6-	200	1.13	10	0.34	0.26	15	0.12
	400	1.13	10	0.17	0.26	5	0.02
60-80	200	1.06	15	0.47	0.32	15	0.14
	400	1.06	5	0.08	0.32	5	0.02
80-100	200	1.00	20	0.60	0.37	20	0.22
				1.99			0.58

b) Due to Sustained Bridge Loads

As in 2(b) above ΔH_i = 0.12 inches

D. Two Layer System Modification

The foregoing immediate and consolidation settlements have been calculated assuming a Boussinesq stress distribution. The presence of the more rigid upper layer has the effect of reducing the stresses, and hence the settlements, throughout the underlying soil. Tabulated results are available only for the stress distribution under a uniform circular load. * Using these results, the expected modification of the settlements is computed below.

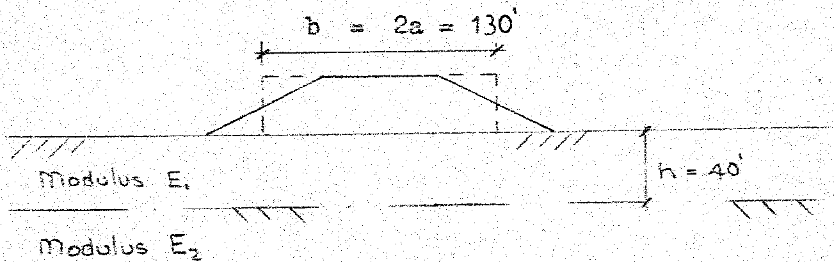

Fig. 2.
Sketch showing layout of two layer system.

The ratio E_1/E_2 for this problem can be determined in two ways.

a) $E_1/E_2 = m_{v2}/m_{v1} = 0.010/0.0026 \approx 4$

b) by the computation of E from the stress-strain curves, drawings 14 and 15. Values of E_1/E_2 so computed range from 21 to 3.

The tabulated results are given only for a/h = 1 and $E_1/E_2 = 10$, but as an approximation these can be applied to this problem.

Depth (ft.)	Boussinesq Stress / Applied Stress	Modified Stress / Applied Stress	Modified Stress / Boussinesq Stress
0–20	0.98	0.90	0.92
20–40	0.78	0.44	0.56
40–60	0.55	0.26	0.47
60–80	0.36	0.20	0.56
80–100	0.24	0.16	0.67

* "Soil Mechanics for Road Engineers" – Road Research Laboratory, D.S.I.R., P. 435

The foregoing reduction factors are believed to be not entirely representative, since the crust thickness and ratio of E_1/E_2 may not be as great as assumed here. In addition, the embankment loading condition is somewhat more severe than the circular load used in this reference. For purposes of these computations, therefore, a compromise reduction of 0.7 will be used.

The corrected settlements in inches, including those due to footing loads, therefore, are as follows:

		West Abutment	West Pier	Differ-ential	East Abutment	East Pier	Differ-ential
North End	H_i	2.95	0.51		3.60	1.52	
	H_c	7.99	1.36		8.87	3.76	
	Total	10.94	1.87	9.07	12.47	5.28	7.19
South End	H_i	3.52	1.52		3.04	0.53	
	H_c	7.82	3.42		7.24	1.38	
	Total	11.34	4.94	6.40	10.28	1.91	8.37
Differential		0.40	3.07		2.19	3.37	

About thirty percent of the settlement noted above is the result of elastic compression of the soil. Another 15 percent approximately of consolidation settlement should have taken place by the time the embankment is completed and the abutments and piers are in place. At this stage of construction, adjustments can be made to allow for the differential settlement occurring up to this time. The bridge superstructure will be affected only by the movements which occur following this period. Thus the total and differential movements will be only about 55 percent of the values noted above. If the installation of the deck slab is delayed for about 3 months after the embankments are in place, and if the superstructure girders are raised to a level position at this time, the subsequent movements should be only about 45 to 50 percent of the values noted above.

E. Settlement of Approach Fill

Maximum settlement under the embankment centre line occurs approximately 100 feet behind the abutment.

Immediate and consolidation settlements at this point are computed below. The respective thicknesses of clay and silt are assumed similar to those under the northwest corner of the bridge. Similar values of m_v and E to those assumed in the previous sections of this appendix are also used.

			Immediate		Consolidation		
Depth (ft.)	H (ft.)	Δp (ksf)	E (ksf)	ΔH_i (ins.)	u	m_v (ft.^{2}/kip)	ΔH_c (ins.)
0-20	20	4.02	800	.60	0.6	.0026	1.51
20-40	15	3.76	800	..43	0.6	.0026	1.06
	5	3.76	400	.28			
40-60	20	3.34	200	2.01	0.8	.010	6.41
60-80	20	2.75	200	1.65	0.8	.010	5.29
80-100	10	2.44	200	0.73	0.8	.010	2.34
	10	2.44	400	0.37			
				6.07			16.61

In the previous sections of this appendix, a correction factor of 0.70 was applied to the computed settlements to allow for the presence of the more rigid upper crust. This figure was obtained by modifying tabulated results for a uniform circular load to allow, among other factors, for the applied surface load distribution in the abutment and pier area.

The approach embankment load distribution at the point considered above is similar to an infinitely continuous strip footing. In view of this condition it is believed that the correction factor will be nearly unity.

The total resultant settlement will therefore be:

$$\Delta H = \Delta H_i + \Delta H_c$$
$$= 6.07 + 16.61$$
$$= \underline{22.67 \text{ inches}}$$

In a similar manner the settlement at two points due north of this location were computed. These results are presented in Fig. 3 below.

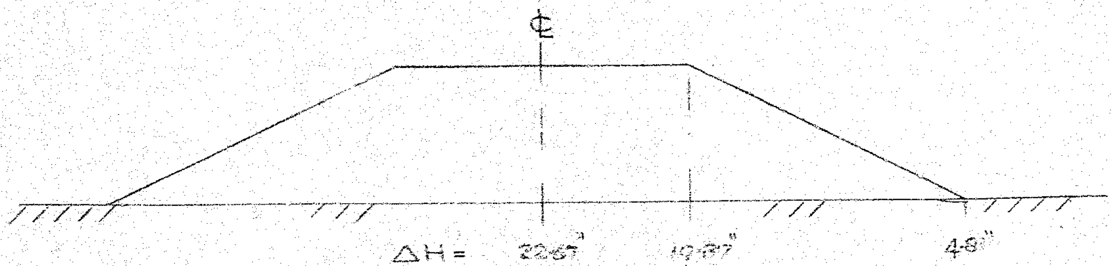

Fig. 3.

Settlements under embankment, 100 feet west of abutment.

F. Time for Settlement

The time settlement curve for the north east corner of the east abutment is presented in Dwg. 16. About 80 percent of the consolidation settlement should be complete at this location after a period of $1\frac{1}{2}$ years. All movement should be essentially complete at all locations after about 4 years. A more accurate prediction of the final foundation movements can be obtained by installing settlement plates in the abutment and pier areas at the initiation of embankment construction.

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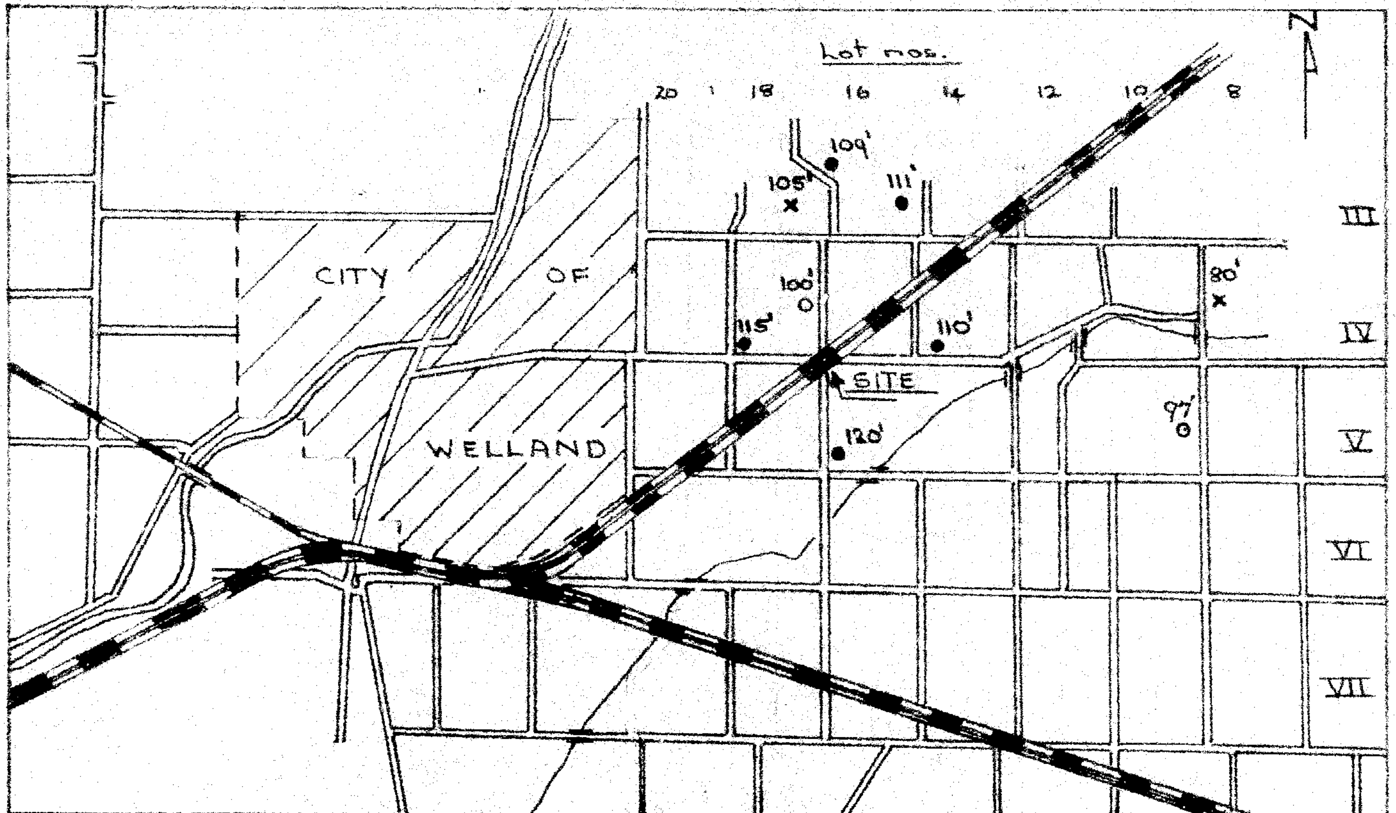


SITE PLAN - SHOWING PROVEN BEDROCK LEVELS.

1" = 1 mile.

(Locations within lots estimated only)

o Shale

x Limestone

● Not recorded.

DRAWING NO. 1(a)

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS   SOIL MECHANICS CONSULTATION

DRAWING NO. 2
PROJECT NO. J608

LEGEND

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

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

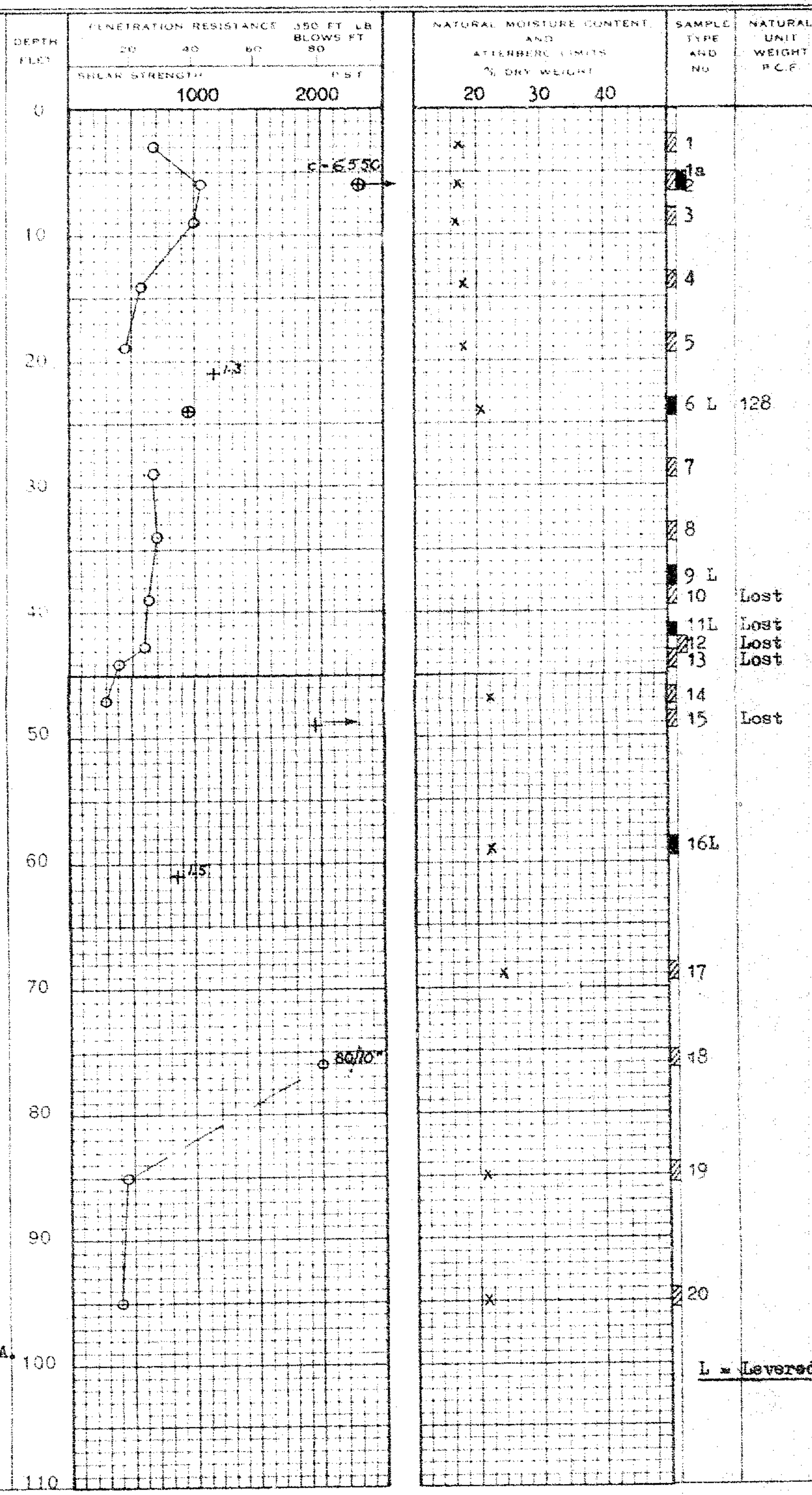
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS
LIQUID LIMIT
PLASTIC LIMIT

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

BOREHOLE NO. A
PROJECT Proposed Grade Separation
LOCATION County Road No. 11 & N.C.R.R., Welland
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 606.5 ft.
DATUM See Dwg. 1.

SOIL DESCRIPTION	ELEV FEET
	606.5
Topsoil ~9 ins. (W.L.)	604.5
SILTY CLAY - stiff red brown -frequent fine to coarse gravel sizes and small silt lenses throughout -very stiff, oxidized and slightly fissured above 12 ft. -very occasional root hairs down to 9½ feet.	
	~582
COARSE SILT - medium dense red brown -thin clay seam noted at 34½ feet.	
	560.5
SILTY CLAY - medium stiff & brown -frequent fine and medium gravel sizes throughout -large stone or boulder at 68 feet. -red brown medium silt layer noted from 76 to 80 ft. -thin gravel layers noted at 80 ft.	
End of Hole	511



L = Levered

Notes:
1) Hole A uncased to 50 ft.
2) Hole A-1 put down 10 ft. south of hole A.
3) Samples 1a, 9, 11, 12, 14 & 16-20 obtained from hole A-1.
4) Hole A-1 cased to 68 ft.
5) Water level observations:
   El 584 ft. 0 hrs.
   El 598.5 ft. 15 hrs.
   El 602.5 ft. 39 hrs.
   El 604.5 ft. 80 hrs.

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No. 3
PROJECT No. J608

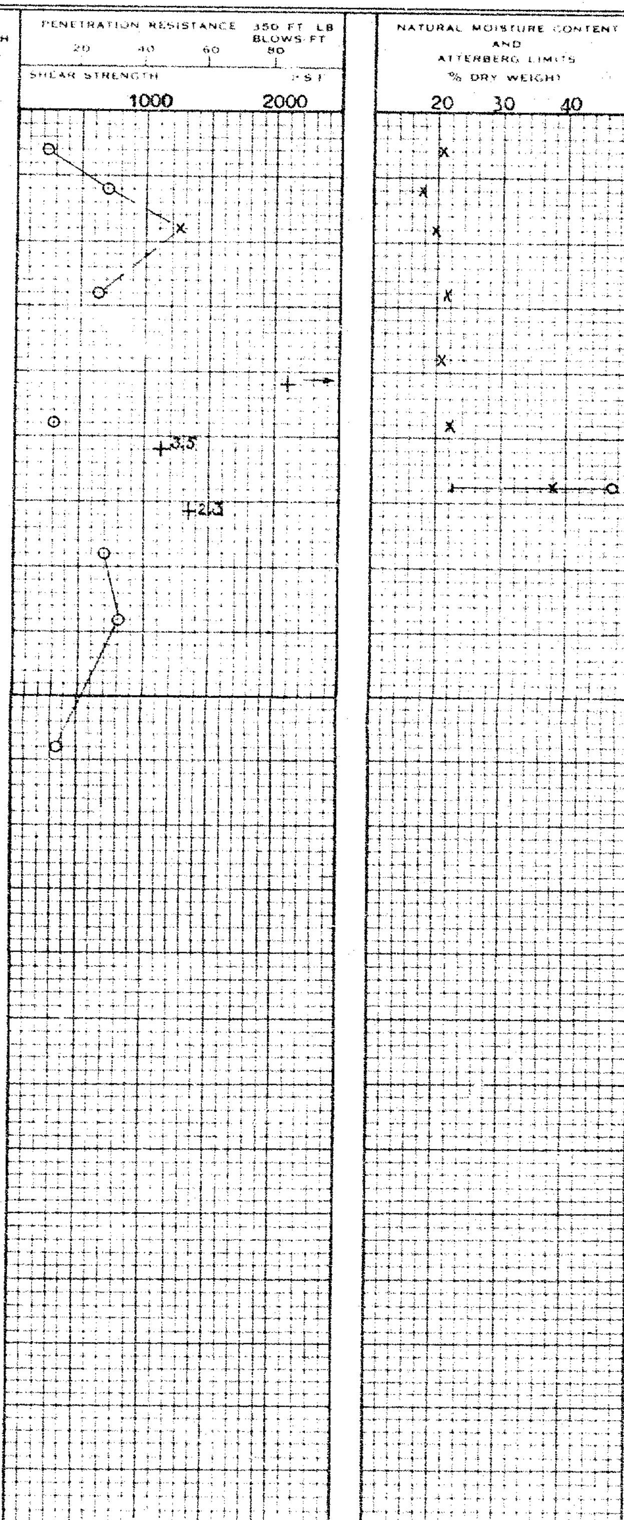
LEGEND

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

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

BOREHOLE No. B
PROJECT Proposed Grade Separation
LOCATION County Rd. No. 11 & M.C.R.R., Welland
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 608.6 ft.
DATUM See Dwg. 1.

SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
Ground surface	608.6	0		
Gravel fill & topsoil	606.6			
SILTY CLAY – very stiff red brown (V W.L.)	604.6		1	
-frequent fine to coarse gravel sizes & small silt lenses throughout			2	
-high silt content 8 – 10 ft.		10	3 D	
-friable & oxidized down to 10 ft.				
-faint varving noted at 14 ft.			4	
-fine root remains noted at 19 ft.		20	5 L	
-becomes medium stiff below 22 ft.			6	
-more plastic clay noted at 29 ft.		30	7 L	
	574.5		8	Lost
COARSE SILT – medium dense red brown				
-occasional fine gravel sizes		40	9	
-thin clay partings noted at 49 ft.				
End of Hole	559	50	10	



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

Notes: 1) Hole uncased for full depth.

2) Hole caved at 30 ft.

3) Water level observations:

El 585.5 ft. 0 hrs.
El 588.5 ft. 1½ hrs.
El 589.5 ft. 4½ hrs.
El 600.6 ft. 20 hrs.
El 604.6 ft. 44 hrs.

D= Driven
L= Levered

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SITE INVESTIGATIONS   SOIL MECHANICS CONSULTATION

DRAWING No. 4

PROJECT No. 3608

LEGEND

PENETRATION RESISTANCE		NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX	X
2" O.D. SPLIT TUBE	—o—o—o—	ATTERBERG LIMITS	
2" I.D. SHELBY TUBE	—x—x—x—	LIQUID LIMIT	—o
2" DIA. CONE	———	PLASTIC LIMIT	⊢—
SHEAR STRENGTH		SAMPLE TYPE	
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE	⊕	2" O.D. SPLIT TUBE	
UNCONFINED COMPRESSION	⊗	2" I.D. SHELBY TUBE	
VANE TEST AND SENSITIVITY (S)	$+^S$	3" O.D. SHELBY TUBE	

BOREHOLE No. C

PROJECT Proposed Grade Separation

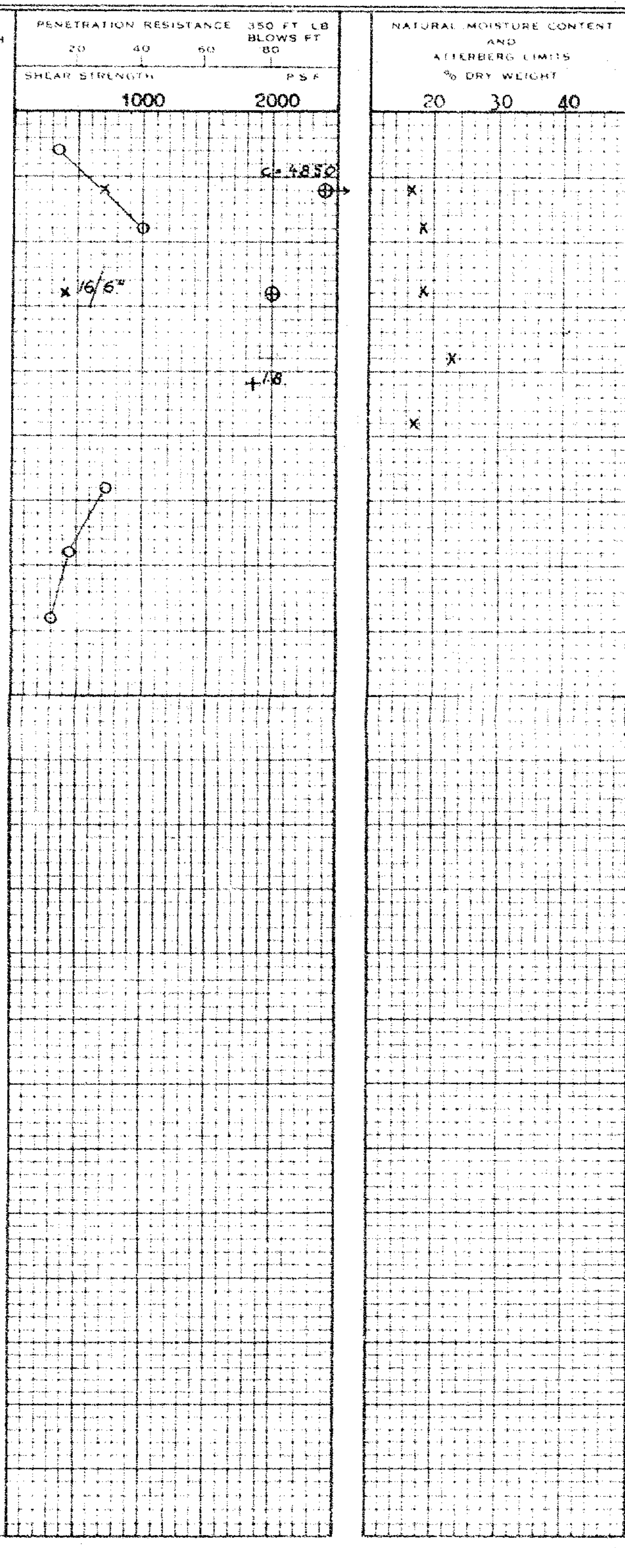
LOCATION County Rd. No. 11 & M.C.R.R., Welland

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 608.7 ft.

DATUM See Dwg. 1.

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
	Ground level	608.7	0		
	Topsoil ~7 ins. ▽ W.T.	606.7		1	
	SILTY CLAY – stiff red brown			2	L=6" D=12" 137
	–frequent fine to med. gravel sizes throughout			3	
	–high silt content at 5 – 7 ft.		10		
	–interbedded fine cohesive silt seams noted at 13 and 18 ft.			4	L=12" D= 6" 137
	–oxidized and very stiff to about 12 ft.		20	5	L
		584.2		6	L
	COARSE SILT – med. dense red brown			7	
	–very occasional gravel sizes at 29 ft. and 34 ft.		30	8	
	–thin clay seams noted at 29 ft., 34 ft. and 37 ft.		40	9	Lost
	End of Hole	559.2	50	10	L



Notes: 1) Hole uncased for full depth.

2) Hole caved to 32 ft.

3) Water level observations:

El 590.7 ft. 1 hr.
El 604.7 ft. 20 hrs.
El 606.7 ft. 60 hrs.

(believed to be effect of surface runoff)

DEPTH FEET: 60, 70, 80, 90, 100, 110

L = Levered
D = Driven

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO. 5
PROJECT NO. T608

LEGEND

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

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

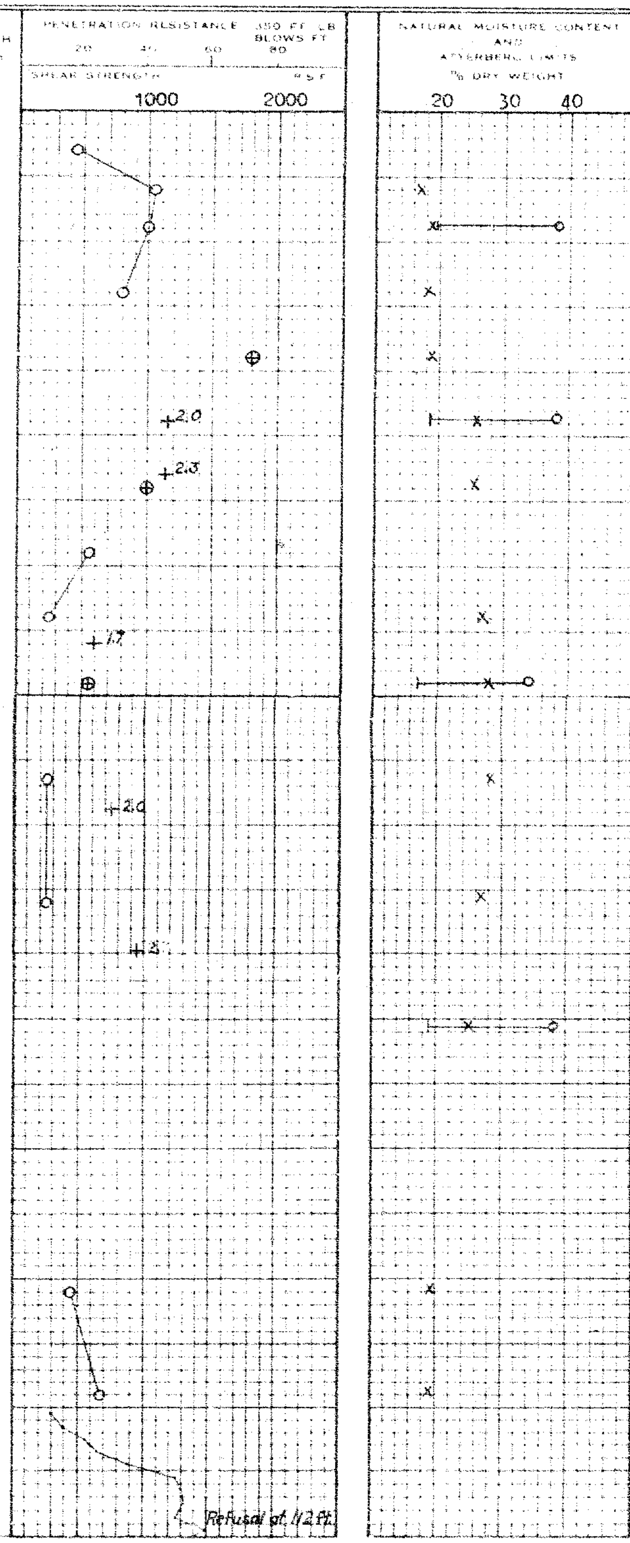
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS
- LIQUID LIMIT
- PLASTIC LIMIT

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

BOREHOLE NO. D
PROJECT Proposed Grade Separation
LOCATION County Rd. No. 11 & M.C.R.R., Welland
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 608.7 ft.
DATUM See Dwg. 1.

SOIL DESCRIPTION	ELEV. FEET
Ground surface	608.7
Topsoil ~ 7 ins.	
SILTY CLAY - stiff red brown	
V.W.L.	602.2
-frequent fine to coarse gravel sizes throughout	
-very stiff oxidized to 15 ft., very occasional root remains noted to this depth.	
-black & grey embedded coarse silt specks noted at 23 & 28 ft.	
	577
COARSE SILT-med. dense red brown.	
	574.4
SILTY CLAY-med. stiff red brown.	
-frequent fine to coarse gravel sizes throughout	
-fine silty sand partings noted at 39 ft.	
-small fine to coarse grey silt specks and lenses noted throughout.	
	531
COARSE SILT - red brown	
	520
SILTY CLAY - med. stiff red brown.	
-gravel sizes throughout - silt partings noted 99 ft.	
End of Hole	509



DEPTH FEET	SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.
0		
	1	
	2	
10	3	
	4	
20	5 L	135
	6 L	
30	7 L	127
	8	
40	9	
	10 L	127
50	11	
60	12	
70	13 L	
80	14 L = 9" D = 9"	
90	15	
100	16	
110		

L = Levered
D = Driven

Notes:
1) Hole uncased to full depth.
2) Vane tests at 24 and 28 ft. performed in hole 10 ft. north west of borehole.
3) Water level observations:
El578.7 ft. 2 hrs.
El602.2 ft. 64 hrs.

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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

LEGEND

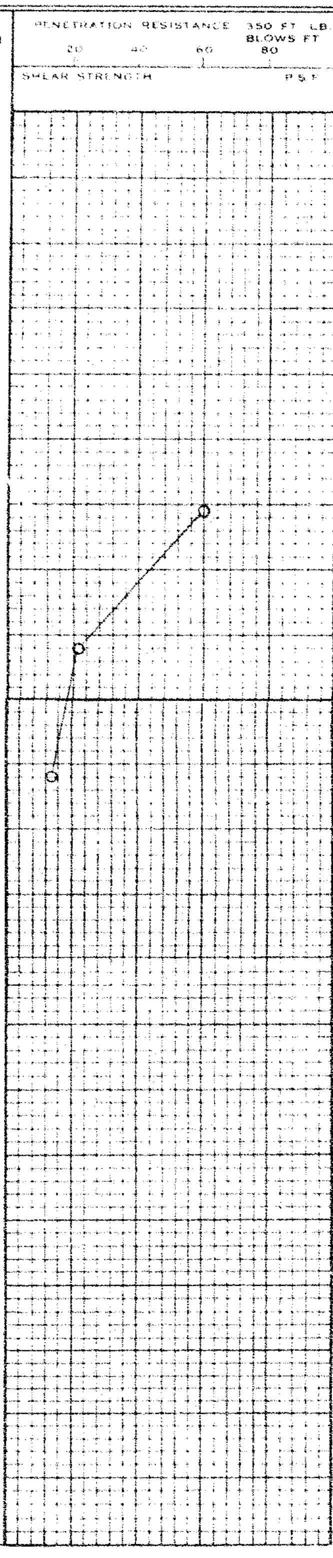
DRAWING NO. 6
PROJECT NO. J608

BOREHOLE NO. E
PROJECT Proposed Grade Separation
LOCATION County Rd. No. 11 & M.C.R.R., Welland
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 606.7 ft.
DATUM See Dwg. 1.

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

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

SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	SAMPLE TYPE AND NO.
Ground surface	606.7	0	
SILTY CLAY - stiff red brown -some gravel sizes throughout			
	~586	20	
COARSE SILT - dense red brown -becoming medium dense with depth			
		30	1
-thin clay seam noted at 41½ ft.			2
	~561		
SILTY CLAY - med. stiff red brown			
End of Hole	556	50	3

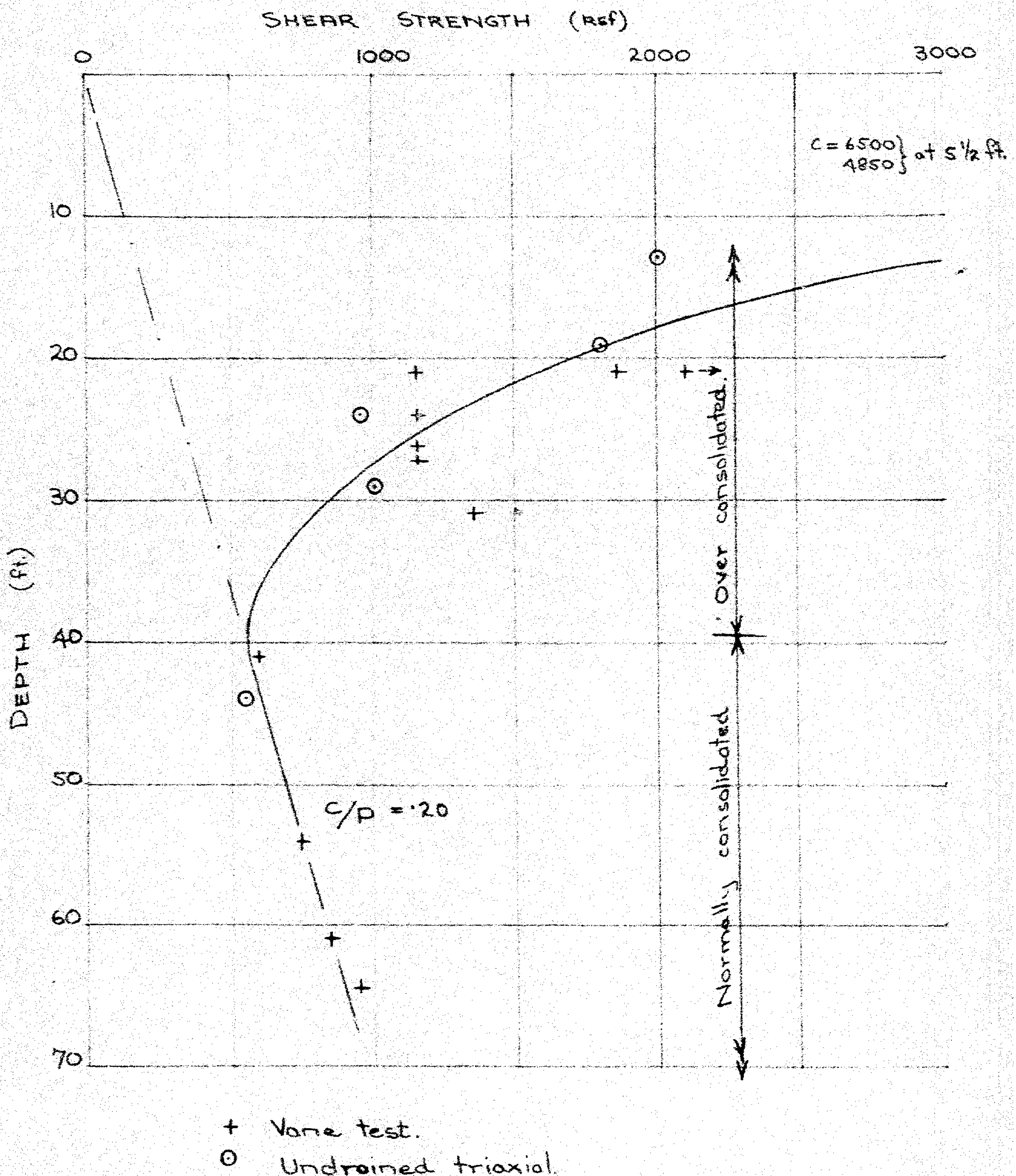


NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT

NATURAL UNIT WEIGHT P.C.F.

Notes: 1) Hole uncased for full depth.
2) Hole caved at 20 ft.
3) No sampling above 30 ft.

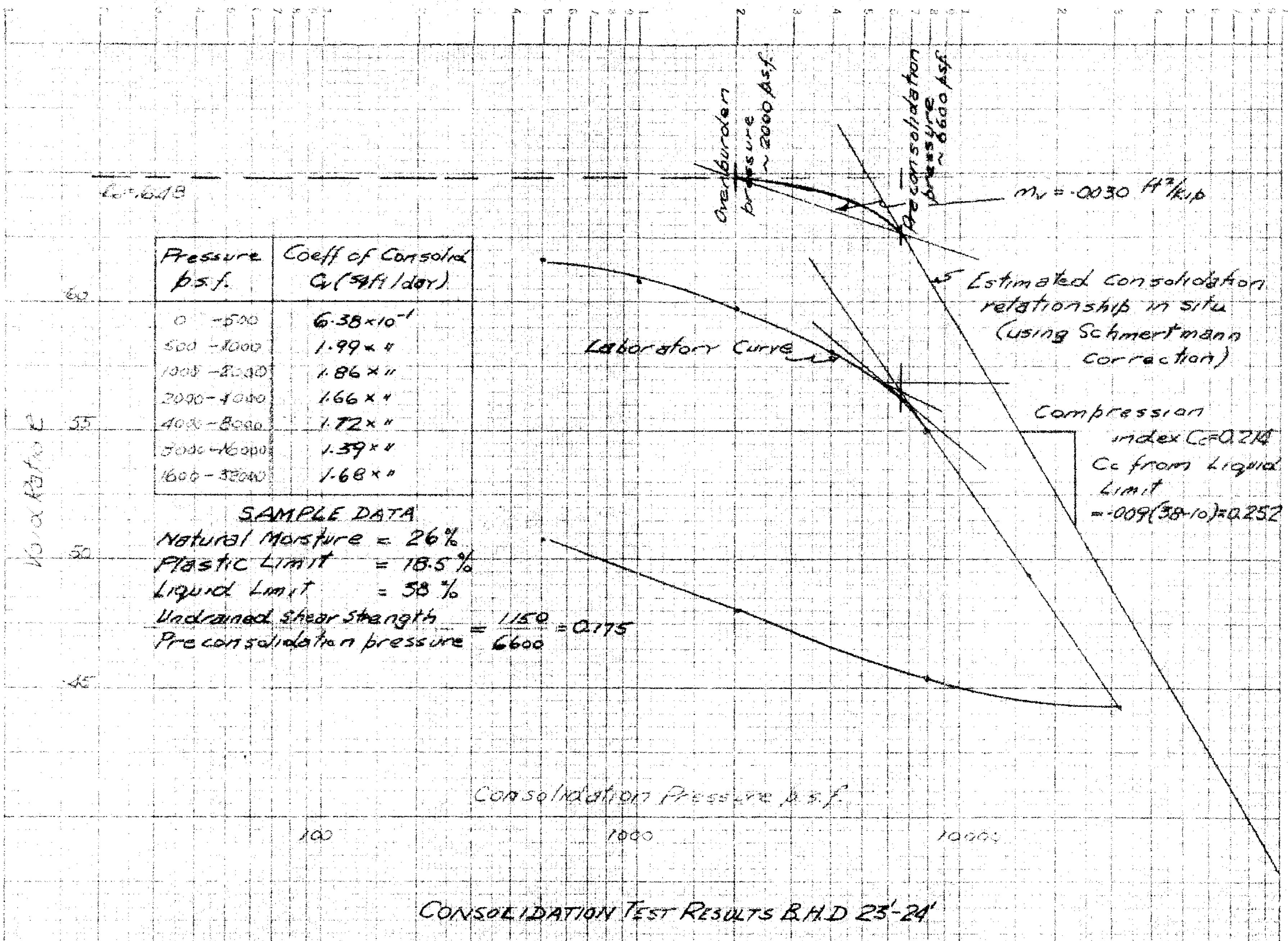
PROJECT NO. J608 DRAWING NO. 7



STRENGTH v. DEPTH RELATIONSHIP

WILLIAM A. TROW AND ASSOCIATES

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



Pressure p.s.f.	Coeff of Consolid C_v (sq ft/day)
0 - 500	6.38×10^{-1}
500 - 1000	1.99 × "
1000 - 2000	1.86 × "
2000 - 4000	1.66 × "
4000 - 8000	1.72 × "
8000 - 16000	1.39 × "
16000 - 32000	1.68 × "

SAMPLE DATA
Natural Moisture = 26%
Plastic Limit = 18.5%
Liquid Limit = 38%

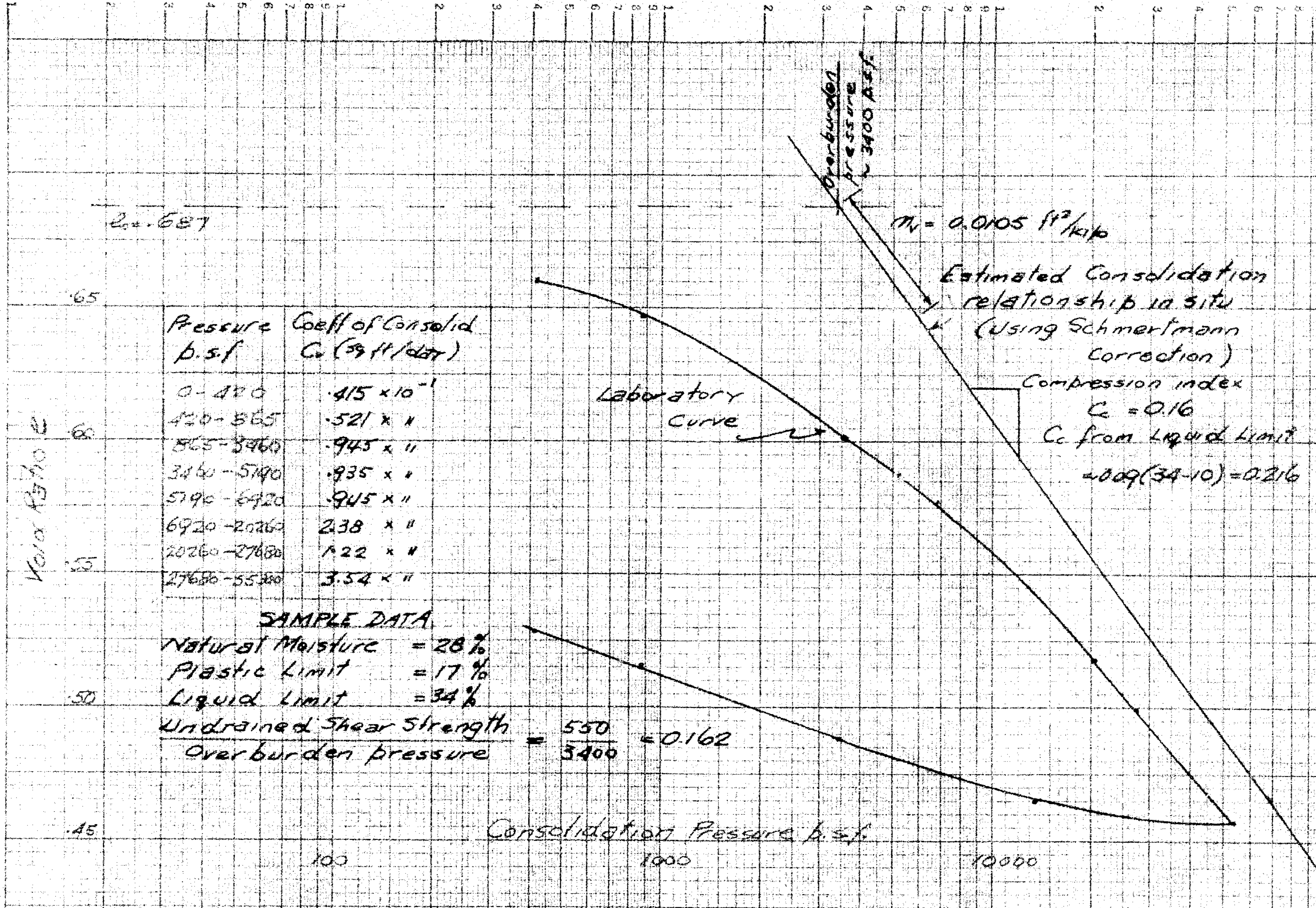
$$\frac{\text{Undrained shear strength}}{\text{Preconsolidation pressure}} = \frac{1150}{6600} = 0.175$$

CONSOLIDATION TEST RESULTS B.H.D 23'-24'

J 608

Dwg 8

K+E SEMI-LOGARITHMIC 359-81G
KEUFFEL & ESSER CO.
4 CYCLES X 70 DIVISIONS

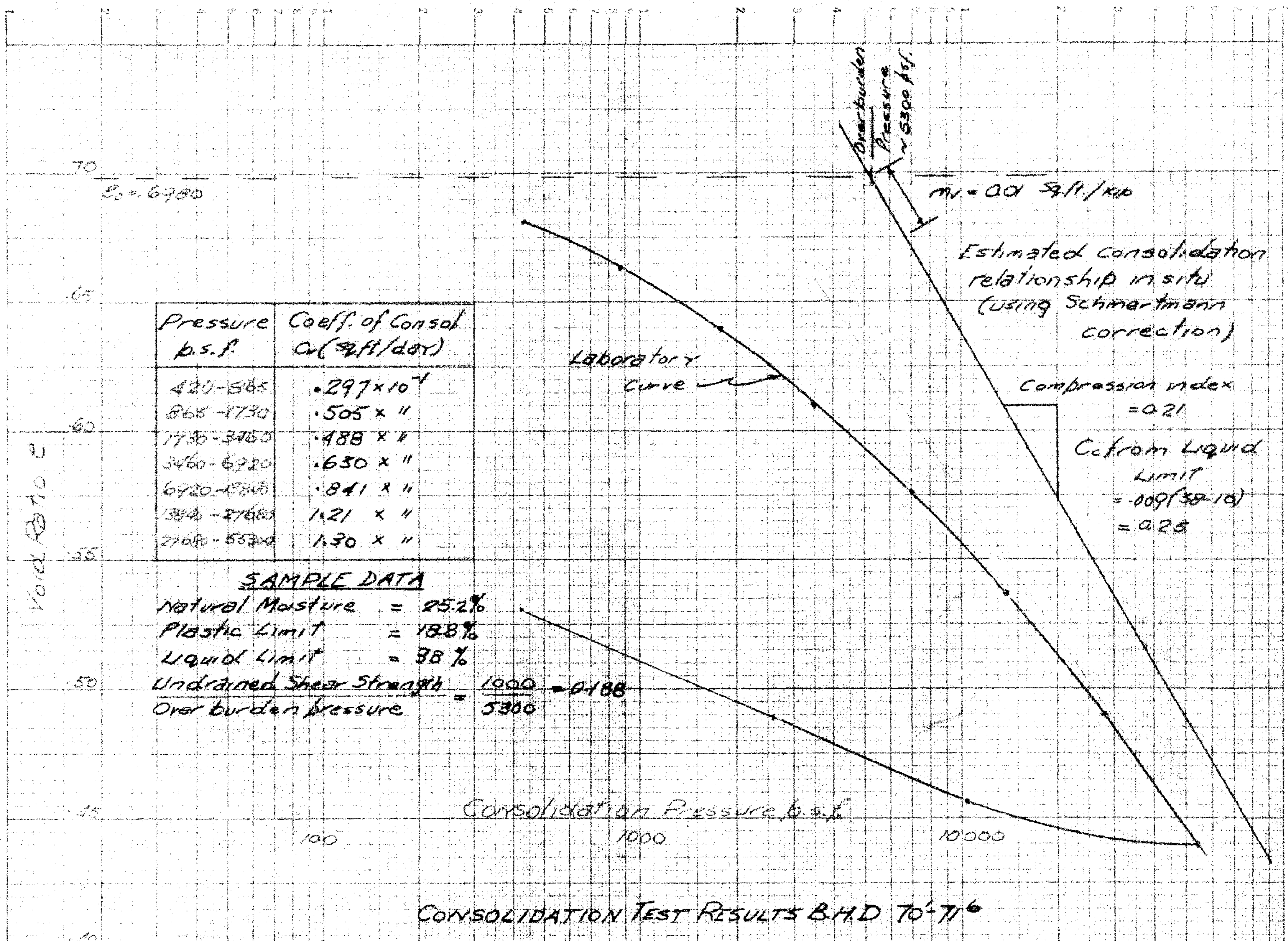


CONSOLIDATION TEST RESULTS B.H.D 43'-44'6"

Dwg 9

J 608

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



Pressure p.s.f.	Coeff. of Consol. Cv (sq.ft/day)
420-865	.297 x 10^-1
865-1730	.505 x "
1730-3460	.488 x "
3460-6920	.630 x "
6920-13840	.841 x "
13840-27680	1.21 x "
27680-55360	1.30 x "

SAMPLE DATA

Natural Moisture = 25.2%
Plastic Limit = 18.8%
Liquid Limit = 38 %

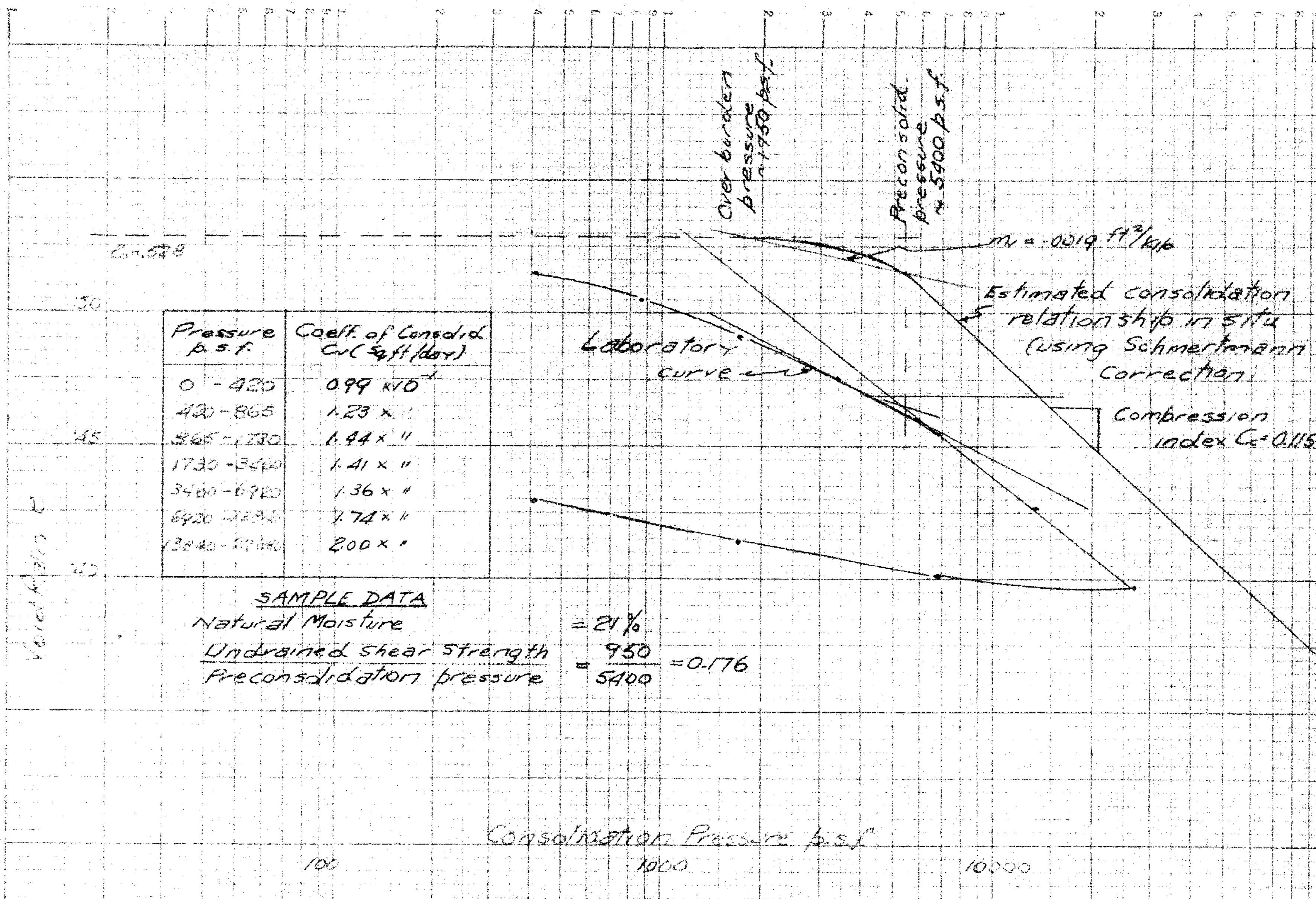
$$\frac{\text{Undrained Shear Strength}}{\text{Over burden pressure}} = \frac{1000}{5300} = 0.188$$

CONSOLIDATION TEST RESULTS B.H.D 70'-71'6"

J 608

Dwg. 10

J 608



Pressure p.s.f.	Coeff. of Consolid. Cv (sq.ft/day)
0 - 420	0.99 ×10^{-4}
420 - 865	1.23 × "
865 - 1730	1.44 × "
1730 - 3460	1.41 × "
3460 - 6920	1.36 × "
6920 - [illegible]	1.74 × "
13840 - [illegible]	2.00 × "

SAMPLE DATA

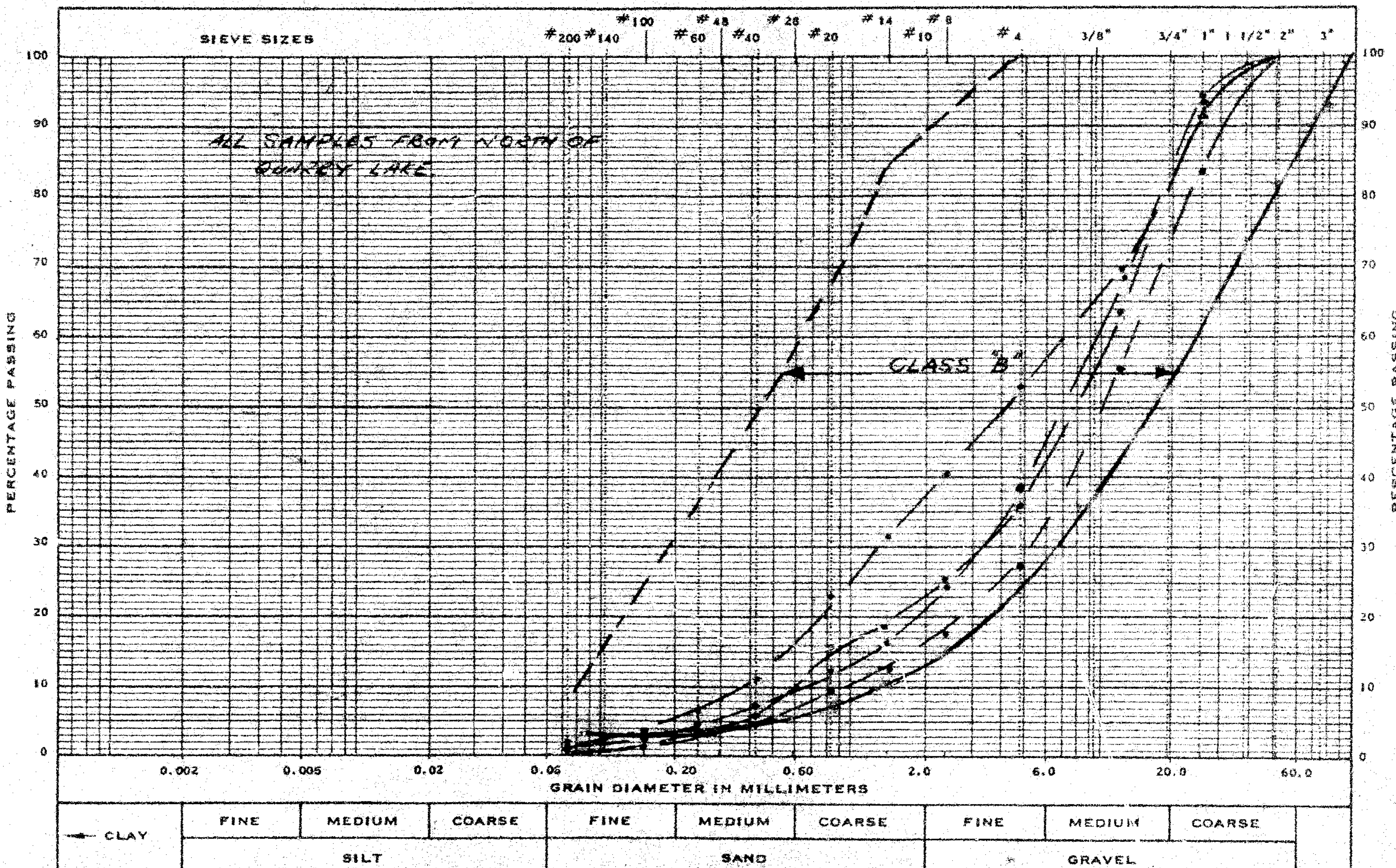
Natural Moisture = 21%

$\frac{\text{Undrained shear strength}}{\text{Preconsolidation pressure}} = \frac{950}{5400} = 0.176$

CONSOLIDATION TEST RESULTS B.H.A 23'-24'6

Dwg. 11

MECHANICAL ANALYSIS



MODIFIED M.I.T. CLASSIFICATION

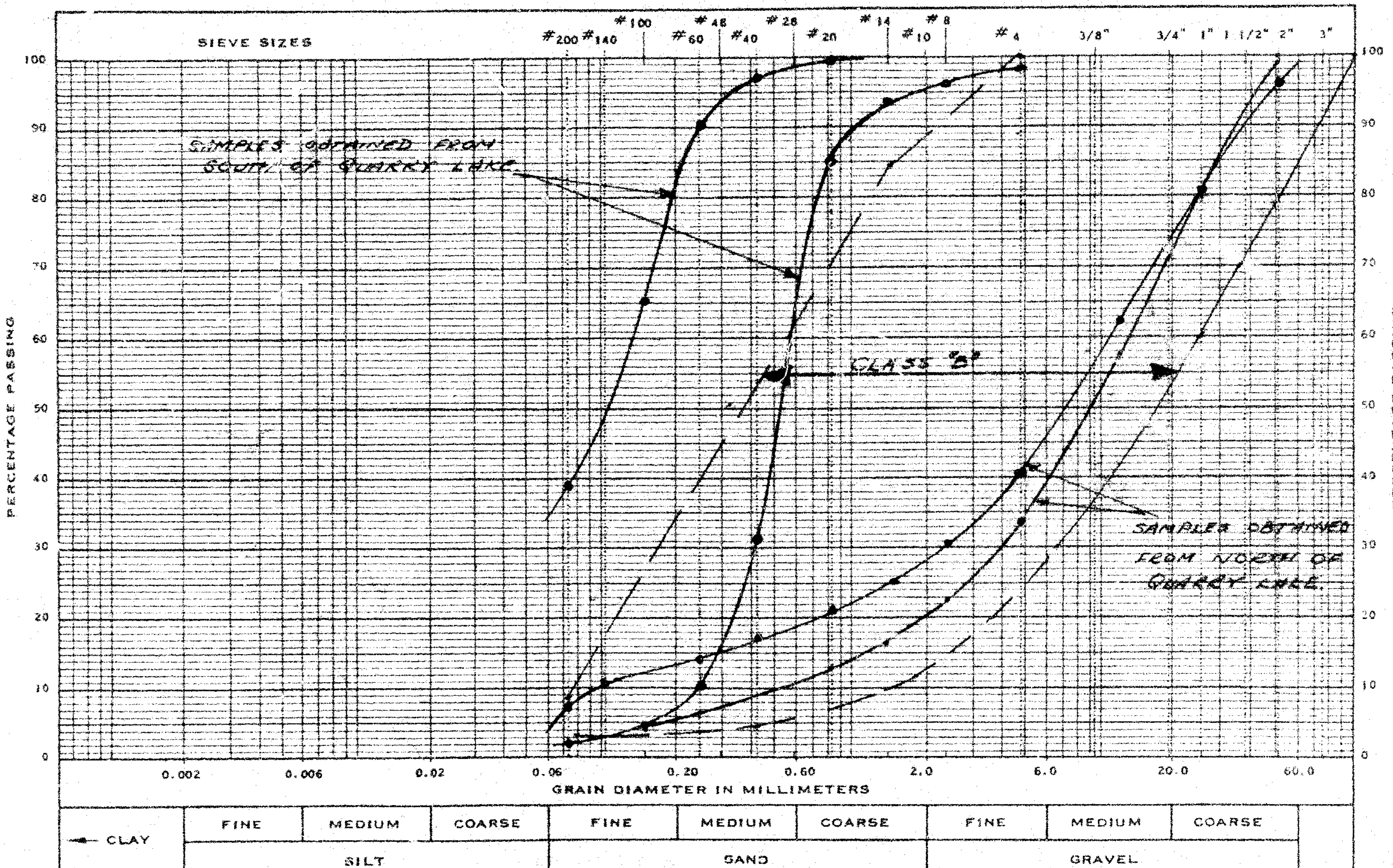
GRAIN SIZE DISTRIBUTION CURVES FOR SAMPLES OF GRAVEL FROM McKENNY ROAD PIT.

WILLIAM A. TROW AND ASSOCIATES LTD.

608

Dwg 18

MECHANICAL ANALYSIS



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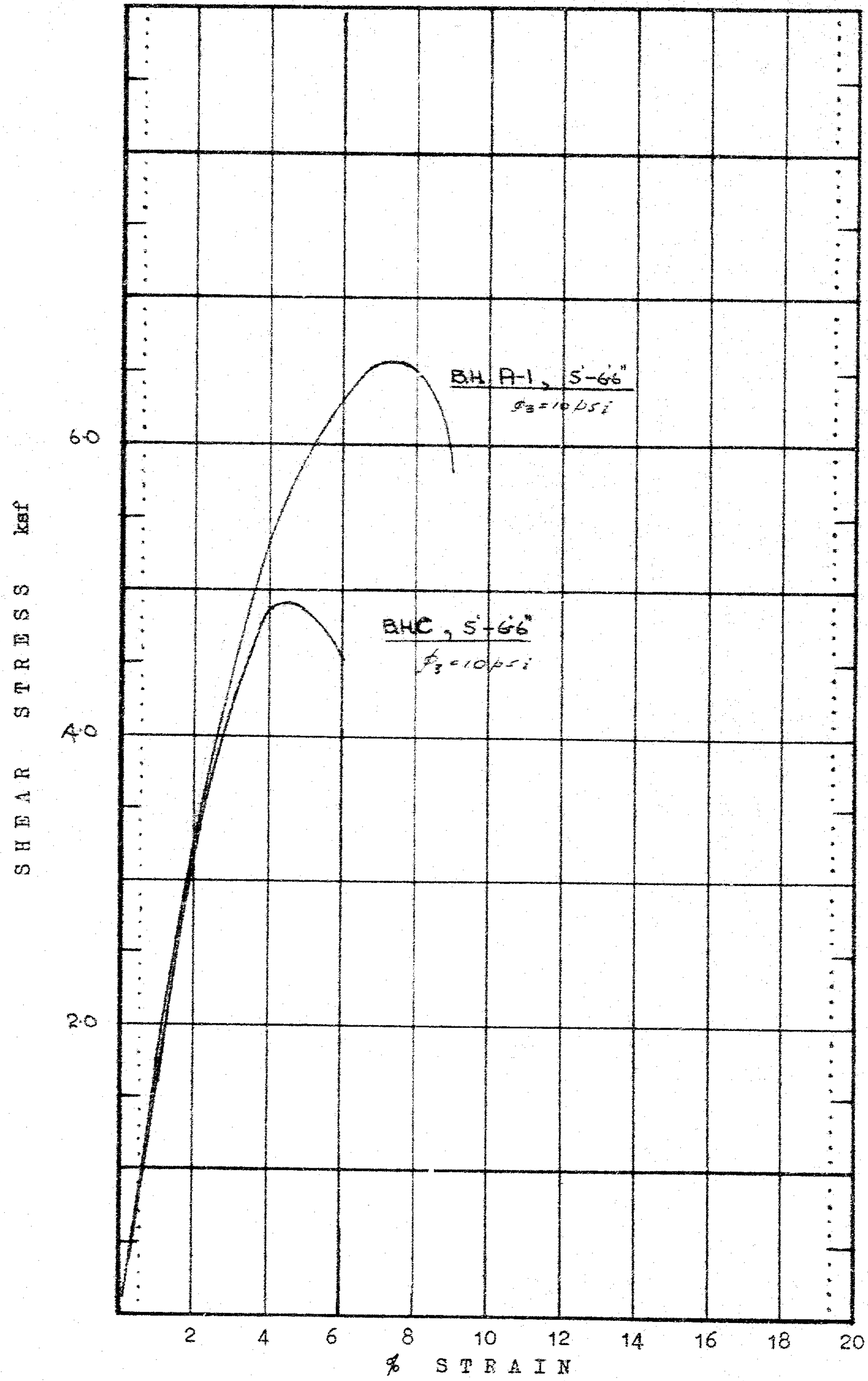
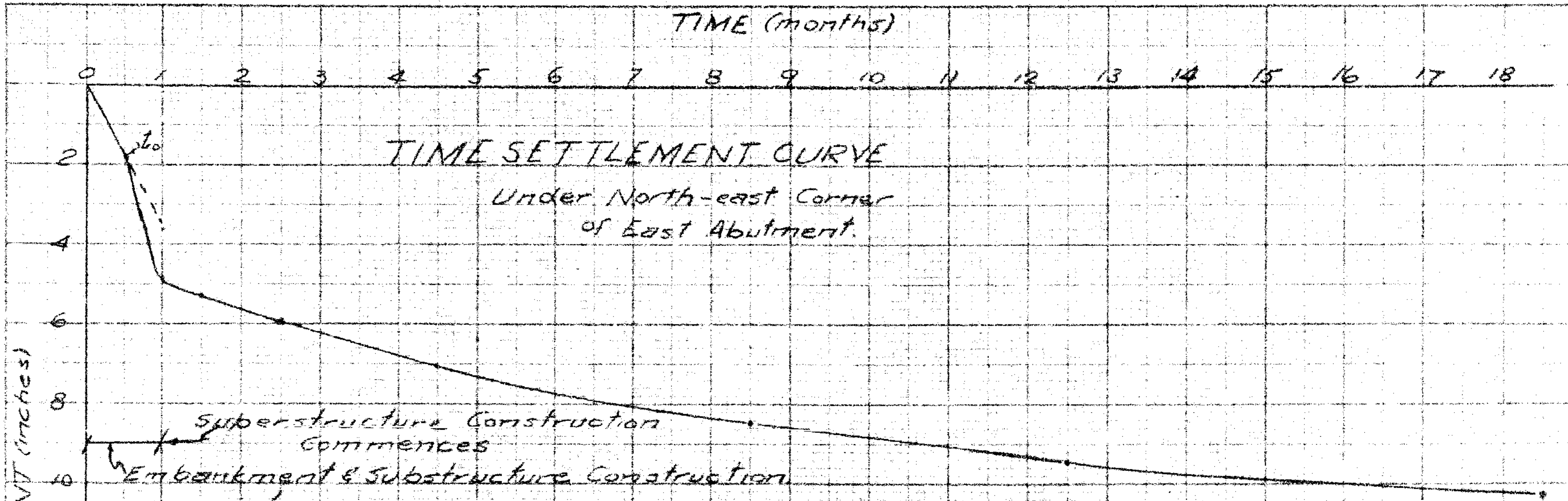
Dwg. 18



STRESS - STRAIN CURVES FOR UNDRAINED TRIAXIAL TESTS

WILLIAM A. TROW AND ASSOCIATES

K&E 20 X 20 TO THE INCH 359-101G
KEUFFEL & ESSER CO.

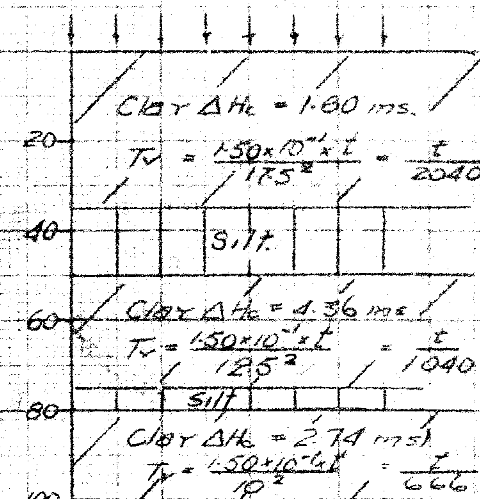


Final Settlement 12.47"

*t (days)	TOP LAYER T_v	TOP LAYER U_c	TOP LAYER ΔH_c (ins)	MIDDLE LAYER T_v	MIDDLE LAYER U_c	MIDDLE LAYER ΔH_c (ins)	BOTTOM LAYER T_v	BOTTOM LAYER U_c	BOTTOM LAYER ΔH_c (ins)	$\Sigma\Delta H_c$ (ins)
15	·007	·08	·13	·014	·15	·65	·022	·17	·47	1·25
30	·015	·15	·24	·029	·19	·83	·045	·24	·66	1·73
60	·029	·19	·30	·058	·26	1·14	·090	·33	·90	2·34
120	·059	·27	·43	·115	·38	1·66	·180	·49	1·35	3·44
240	·118	·39	·62	·230	·55	2·40	·360	·67	1·84	4·86
360	·176	·48	·77	·346	·66	2·88	·540	·79	2·17	5·82
540	·264	·57	·91	·520	·76	3·32	·810	·89	2·47	6·70

* Measured from time t_o or one halfway through Embankment construction period

U_c = Consolidation as a fraction.



Dwg. 16

Materials & Research Section

January 30, 1961.

De Leuw, Cather and Co., Ltd.,
1491 Yonge Street,
Toronto, Ontario.

<u>Attn: Mr. H. Van Bodegan</u>

Michigan Central Railway Overhead,
<u>County Road No. 11, City of Welland.</u>

Dear Sir:-

In response to your request, we have reviewed the soil conditions as reported for the above site (W.A. Trow & Assoc., Report No. 1J608) and submit for your consideration, the following comments pertaining to the design of overhead structure.

It appears that two proposals can be considered:-

(1) A continuous multi-span structure with piers and abutments supported on end-bearing piles driven to practical refusal in the underlying till or bedrock; or -

(2) A multi-span simply-supported structure with the piers resting on spread footings and abutments on spread footings founded in the compacted approach fill.

Proposal (2) above, can only be considered if it is possible to construct the bridge and approach fills in two stages such that the approach fills are placed first, followed by a minimum waiting period of 8 months prior to letting the structure contract.

A surcharge design and a relatively simple instrumentation program are required in connection with the adoption of proposal (2).

The Homer Skyway is presently under construction and the foundation design is identical with proposal (2) outlined in the preceding paragraph. Settlement observations of the fills and rate of pore pressure dissipation in the compressible subsoil have been made and these data show that the design is sound. That the stage construction procedure at the M.C. Rlway. will be equally successful is virtually assured because soil conditions at this site are more

cont'd. /2 ...

favourable than at Homer. The data obtained at Homer to date, are appended for your review.

It is our recommendation that the stage construction and surcharge proposal be adopted. The following details should be adhered to in design:-

(1) Pier spread footings should be designed using a net footing pressure not in excess of 2-1/2 tons/sq.ft.

(2) The abutments founded within the fill should be designed with a net footing pressure not in excess of 1 ton/sq.ft. Footing width should be not less than 10 feet.

(3) A berm dimension of not less than 8 feet should be maintained in front of the abutment breastwall.

(4) The embankment fill should be compacted by the 6" layer method to a specified 100% of the Standard Procter maximum density value for the particular fill material used. This degree of compaction will have to be confirmed during construction, by in-situ density tests.

(5) The fill need only be placed in layers and compacted from existing ground elevation to final grade elevation. The surcharge material which should be placed to a height at least 12 feet above final grade, need not be compacted or placed in accordance with D.H.O. standard layer construction procedures.

(6) The surcharge should be positioned so that it is entirely effective in pre-loading the abutment footing areas. This will necessitate a steepening of the spill-through slope for the pre-load period of 8 months. The surcharge should extend from the abutment area to a point along road centre line at least 200 feet distant from the abutment.

(7) A minimum number of 6 piezometers and 3 settlement plates should be installed under each surcharged fill. Details of these measuring units and specifications for installation, can be obtained from the Foundation Engineers' Office, D.H.O.

(8) In order to take care of the eventuality that the actual settlement-time relationships do not develop as theory and practice lead us to believe, it seems prudent to provide the last spans with end diaphragms to allow jacking, if necessary.

In considering the alternative proposals, it must be noted that a maximum settlement of 6 to 9 inches will result from the approach fill loadings of 30 feet. If proposal No. (1) is adopted with no stage construction, a differential movement between the fill and the pile-supported abutment of 6 to 9 inches can be virtually guaranteed. With proposal No. (2) utilizing stage construction,

cont'd. /3 ...

a specified differential movement will be insignificant.

If we can be of further assistance in this matter, please contact our Office.

Yours very truly,

L. G. SODERMAN,
PRINCIPAL FOUNDATION ENGR.

Per:

(M. Devata,
PROJECT FOUNDATION ENGR.)

MD/MdeF
Encls.

cc: Mr. K. Kleinsteiber

Foundations Office
Gen. Files.

cc: Mr. K. Kleinsteiber (2nd copy)

Materials & Research Section

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Per:

M. Devata

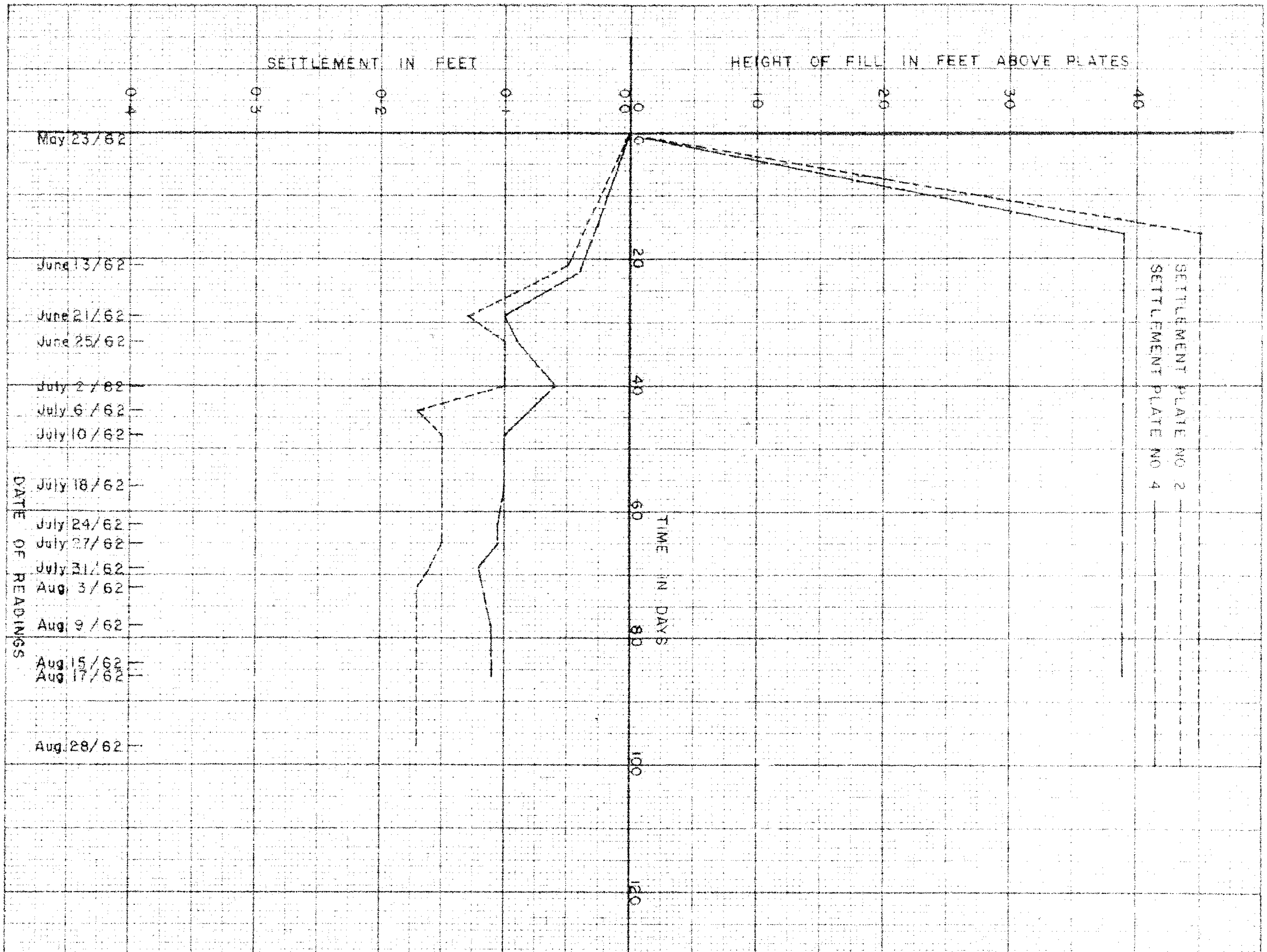
(M. Devata,
PROJECT FOUNDATION ENGR.)

MD/MdeF
Encls.

cc: Mr. K. Kleinsteiber

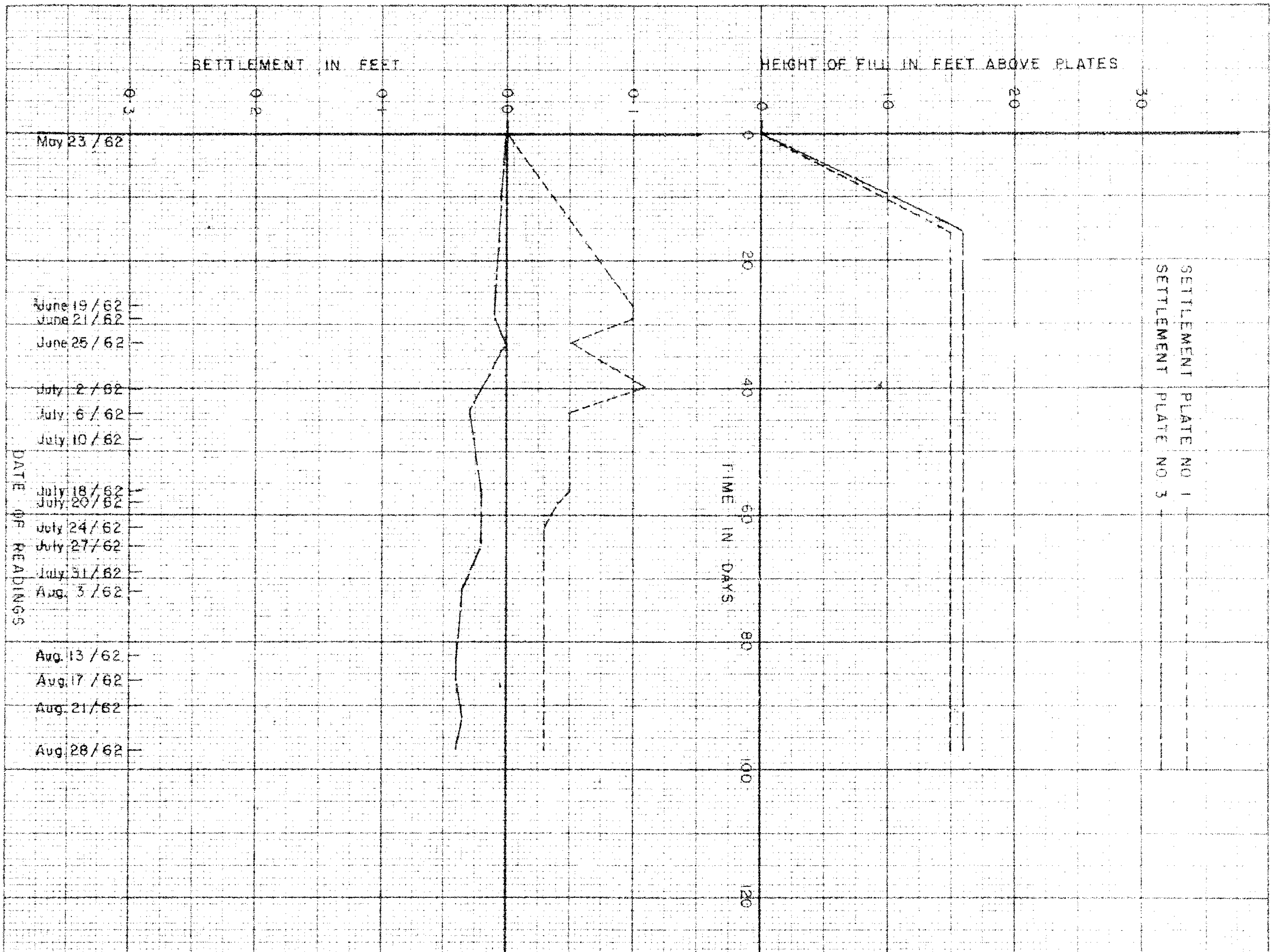
Foundations Office
Gen. Files.

359-10½G

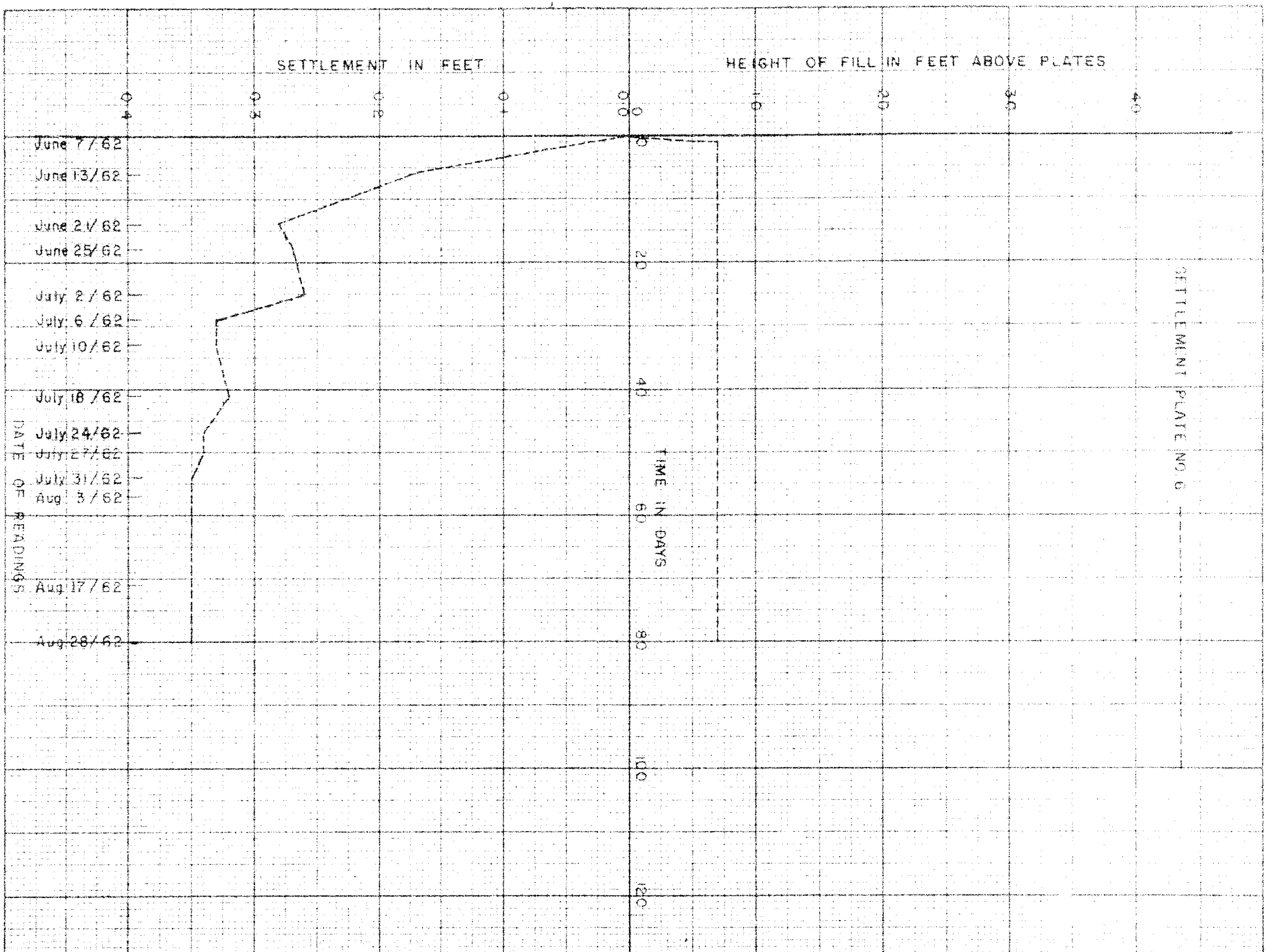


JOB 62-F-66

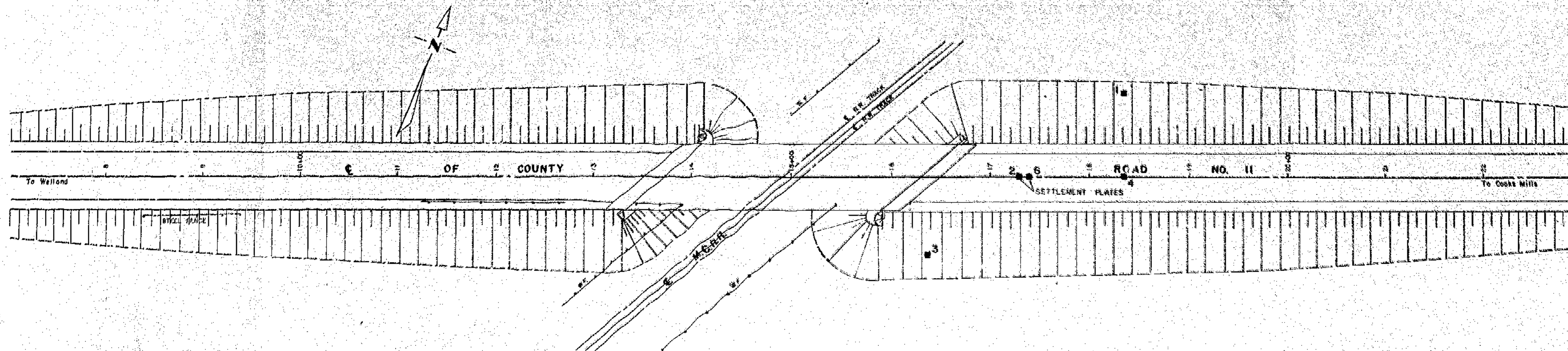
20 X 20 TO THE INCH 359-10½LG



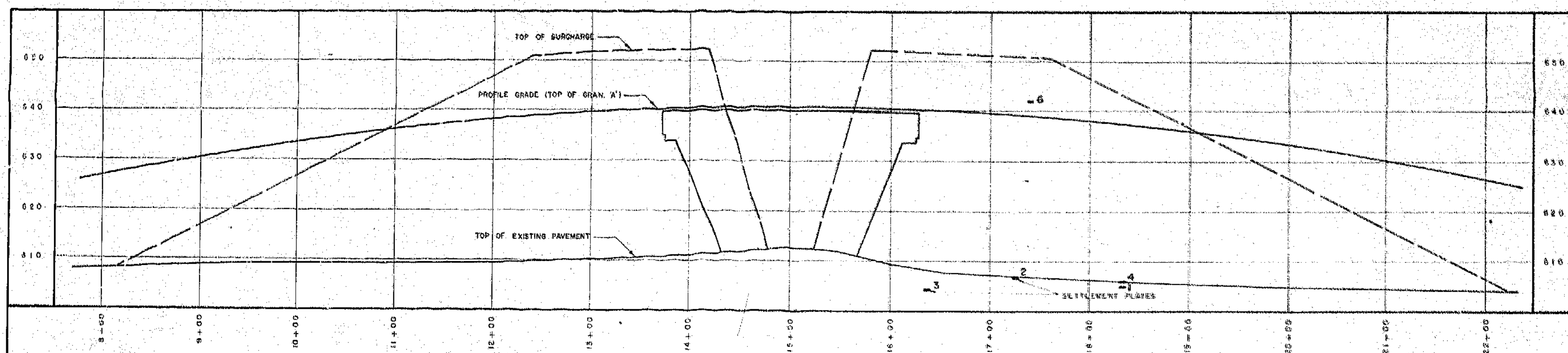
JOB 62-F-66


SETTLEMENT IN FEET
HEIGHT OF FILL IN FEET ABOVE PLATES
0.4
0.3
0.2
0.1
0.0
0
10
20
30
40
June 7/62
June 13/62
June 21/62
June 25/62
July 2/62
July 6/62
July 10/62
July 18/62
July 24/62
July 27/62
July 31/62
Aug. 3/62
Aug 17/62
Aug 28/62
DATE OF READINGS
TIME IN DAYS
20
40
60
80
100
120
SETTLEMENT PLATE NO 6

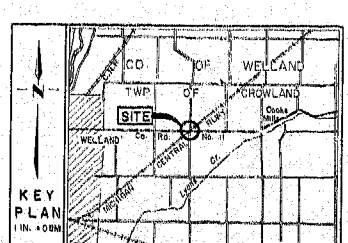
JOB 62-F-66



PLAN
SCALE: 1 in. = 50 ft.



PROFILE
SCALE: HOR. 1 in. = 50 ft.
VERT. 1 in. = 10 ft.



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION

SETTLEMENT PLATE LOCATIONS
AT
COUNTY ROAD NO. 11 & M.C.R.R.
COUNTY OF WELLAND

ORIGINATED M. DEVATA	DISTRICT NO. 4	DATE 21 SEPT. 196[illegible]
DRAWN C. MUMFORD	W.P. NO.	JOB NO. 62-F-66
CHECKED	SCALE AS SHOWN	DRAWING NO. 62-F-66A
APPROVED		