B.A. 2924
Site 27-126

REPORT ON FOUNDATION CONDITIONS

AT

LITTLE CASTOR RIVER

TOWNSHIP OF RUSSELL, RUSSELL COUNTY
ONTARIO

TO

RUSSELL TOWNSHIP COUNCIL

BY

MCNEELY AND LECOMPTE

CONSULTING CIVIL ENGINEERS

ROCKLAND, ONTARIO

JUNE 26, 1968

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TOWNSHIP OF RUSSELL, RUSSELL COUNTY

ONTARIO

TO

RUSSELL TOWNSHIP COUNCIL

1. INTRODUCTION

This report presents an appraisal of the foundation conditions at the site of a proposed crossing over the south branch of the Little Castor River, between Concessions 8 and 9 in the Township of Russell, Russell County, Ontario. The appraisal and report were requested by the Russell Township Council.

2. LOCATION OF THE SITE

The site of the crossing is about two miles south of County Road 3 near Embrun, Russell County, Ontario. A drawing showing the general dimensions of the site and a profile along the centre line of the Concession Road at the site is included as Figure 1 in the appendix.

3. GEOLOGIC CONDITIONS

The location of the site is on a Pleistocene marine clay plain approximately two miles south of County Road 3, between Concessions 8 and 9, Russell County, Ontario. Limestone bedrock is approximately 48 feet below the road surface at this site at a geodetic elevation of 167.9 feet.

The marine clay is in the order of 45 feet thick and is very soft. It has a high water content and when remoulded it loses a large portion of its strength. If loads applied to this material are large enough to break down its structure, large settlements can take place.

This clay deposit is part of what is known as the Leda marine clay deposit which occurs along the Ottawa and St. Lawrence Rivers. Its engineering properties are not well understood and standard laboratory techniques do not always provide answers which are consistent with field behavior. As a result, much reliance is placed on the work of the National Research Council which has been carrying out research on this material for the past fifteen years. This work is of much assistance in estimating the shear strength and compressibility of the clays in this area.

4. FIELD INVESTIGATIONS

The field investigation consisted of two boreholes to identify the subsoil, obtain samples, and to make field vane shear tests; also, one cone penetration test hole to measure the consistency of the subsoil was made. Two locations were tested with a hand auger and small vane shear apparatus.

The two boreholes and the cone penetration hole were made on May 23rd and 24th, 1968. The remaining two holes were made on June 18th, 1968.

4.1 Boring Program

Borehole Nos. 1 and 2 were made to the full depth of the deposit and ended in bedrock. The thickness of the overburden at the location of the boreholes is approximately 48 feet. Standard split spoon penetration tests were made every 5 feet in borehole No. 1 and jar samples were taken. In borehole No. 2, two inch Shelby tube samples were taken every 5 feet in the clay and 5 feet of rock core was taken in the underlying shale and limestone. Field vane tests were taken in the clay to determine its undisturbed and remoulded undrained shear strengths.

In boreholes Nos. 3 and 4, which were made in the stream delta, field vane tests were made to depths of 8 feet below the surface.

The locations of the boreholes and their surface elevations are shown on the plan, figure 1.

4.2 Results of the Field Investigation

The properties of the clay deposit underlying the site were found to be uniform in the horizontal direction once the fill behind the existing bridge had been penetrated.

The borings which were made away from the existing approach fill on the original ground surface indicated about 3 feet of dessication below the organic topsoil.

All samples of the clay taken below fill and dessicated layers were found to be very soft to soft in consistency and high water contents and low shear strengths were indicated. The field vane apparatus provided values of undisturbed undrained shear strengths varying between 500 and 1700 pounds per square foot in the weathered clay, and between 300 and 500 pounds per square foot in the underlying soft clay. In the soft clay, disturbed undrained shear tests led to sensitivities in the order of 3.

The bottom $2\frac{1}{2}$ feet of the deposit consists of soft clay with some sand and small pieces of shale. Bedrock was encountered at a depth of about 48 feet and was composed of a thin layer of shale changing to very sound, fine grained limestone.

A detailed description of the soils encountered at the site is given in the borehole logs in the appendix to this report.

4.3 Ground Water Levels

At the time of the field investigation the ground water level in the boreholes was about 6 feet below the surface.

Since the ground water level is at the surface during the spring season, this level is assumed to ecist when calculations involving shear strength are made.

5. LABORATORY INVESTIGATIONS

The clay was identified and classified by determining its water content and Atterberg Limits at various levels. Unconfined compression tests were carried out to confirm the field test results for undisturbed undrained shear strength and to determine the total stress parameters of the clay. Consolidation tests were done to evaluate the compressibility characteristics of the soil.

5.1 Water Contents and Atterberg Limits

The results of the water content and Atterberg Limit tests are shown on the borehole logs.

The water content of the fill above the soft clay is in the order of 25%. The water content of the soft clay varies from 57% to 85% and in all cases is higher than the liquid limit of the soil. Void ratios in excess of 1 are indicated which generally means that the clay is susceptible to remoulding and loss of strength. High water contents would also imply that the soil mass is highly compressible if its structure is broken down.

5.2 Shear Strength

Unconfined compression tests were made in the laboratory to verify the undrained shear strength values found from the field vane tests. The results of these tests gave values of shear strength slightly larger on average than the field tests. Since the vane apparatus used in the field exploration was crude in its application and since the unconfined samples will be disturbed by the sampling process it is considered safe to use the results of the laboratory tests and a value of 400 pounds per square foot is used for design.

5.3 Compressibility

The compressibility of the clay was evaluated on the basis of three laboratory consolidation tests. These tests were made on undisturbed 2 inch Shelby tube samples taken from borehole No. 2.

The tests indicated overconsolidation ranging from 1530 p.s.f. to 1900 p.s.f. The clay indicated high compression indices but did not demonstrate the concave upward consolidation curves usually associated with highly sensitive clays. This point lends support to the low sensitivities found from the field vane tests.

The e-log p curves are included in the appendix and the results of the tests are summarized as follows.

Sample	Depth (ft)	e_o	p_o (psf)	p_o' (psf)	C_c
1	25'	1.74	1370	2900	0.84
2	30'	1.84	1540	3100	1.09
3	35'	1.83	1700	3600	1.15

e_o - initial void ratio

p_o - existing overburden pressure

p_o' - preconsolidation pressure

C_c - compression index

6. SUMMARY OF FOUNDATION CONDITIONS

The foundation conditions at the crossing site are not good. The limiting factor in the design and construction of any structure over the Little Castor River at this location is the low shear strength of the soil. Thus the stability of any slopes around and approaching any such structure must be analyzed.

The slope stability analyses which were done indicated that the highest bank which could stand with a side slope of 2:1 and a safety factor of 1.3 is 13 feet high. In order to exceed this height and maintain the 1.3 safety factor, berms are required.

The safety factor of 1.3 is that which is in current use by the Department of Highways of Ontario for structures on major roads.

The compressibility of the clay is high. Hence for any structure built at the site, maintenance problems due to settlement will be a factor in design.

Since the proposed structure is not on a major road, it is feasible that smaller safety factors than prescribed by the D.H.O. can be tolerated in the interests of economy;otherwise, the crossing may involve excessive costs.

Several alternatives are presented in the next section. Cost studies of these alternatives should be made so that a reasonable balance between cost and calculated safety can be reached.

7. ALTERNATIVES

7.1 Culvert and Fill

The construction of a culvert and fill structure at this site is probably the simplest and least expensive solution available. This solution will also permit diversion of the stream with fewer engineering problems than those involved with the construction of a bridge at the site. If such a structure is chosen it should be constructed so that its final grade is no more than 2 feet above that of the existing bridge.

7.1.1. Settlements

If the stream is diverted to a position as shown on the site plan as "Alternate No. 1" then a fill of 17 feet will be required at the location of the old stream bed. This fill will be placed directly on the soft clay layer and the settlement which can be anticipated here is in the order of 18 inches most of which will occur over a period of about 10 years. This estimate is based on the NRC technique for the calculation of settlements in the Leda clay. If the standard technique is used the estimate of settlement becomes about 2 feet 8 inches. The NRC approach is probably more reliable since it is based on extensive research and field experience in this deposit.

The settlement of the culvert itself will be small since the loads on the clay will in fact be less at the culvert location after construction than before, and any movement of the culvert will be due to drag from settlement of the fill over the old stream bed nearby.

7.1.2. Slope Stability

Because of the low shear strength of the soft clay the stability of the side slopes of the fill must be analyzed. There are several alternate configurations which are possible. The cost of each of these alternatives increases with its safety.

a) 2:1 side slopes.(fig.9). The minimum factor of safety against shear failure for this case is 0.97 as found from a slope stability analysis.

b) 2:1 side slopes with a berm one-half the height of the fill. (fig.10). Safety factor - 1.1.

c) 3:1 side slopes. (fig. 11). The minimum safety factor for this configuration was found to be 1.05.

d) 2:1 side slopes with a berm 12 feet high. (fig.12). The safety factor of this configuration is 1.3.

7.1.3. Other Factors

If a culvert and fill configuration is used it will be unnecessary to use piles, hence the short term safety of the structure is surer than for a structure which requires piles.

7.2 Bridge Configuration

In view of the difficulties presented by the stability of the slopes at this site, it is recommended that if a bridge structure is desired, the elevation of its deck be kept at or near that of the existing bridge. Also, the bridge should be designed so that its total length spans a distance of about 100 feet. If these conditions are met, then no appreciable amounts of additional fill will be required to meet the new road grade except for that needed to widen the road as it approaches the bridge. If the centre span of the bridge is in the order of 40 feet and the end spans are in the order of 30 feet in length, with the new south abutment at the location of the existing south abutment, then slopes of 2 horizontal to 1 vertical can be constructed and the factor of safety against shear failure will be about 1.25 (see fig.13).

This configuration will essentially eliminate any serious problems due to the compressibility of the underlying clay. Some settlement of the fill used to widen the approaches can still be expected however.

7.2.1. Bridge Foundation

7.2.1.1. Foundation Type

Since bedrock is quite close to the bases of the proposed new abutments, bridge foundations should be carried down to this material. Driving of piles will disturb the soft clay increasing pore water pressures. Thus, minimum displacement piles should be used to keep this disturbance as small as possible. The piles will be end bearing on sound limestone.

Piles will probably be driven before any required fill is placed. Since excess pore pressures will exist in the clay after the driving of piles, the stability of the soil mass may be at a critical stage. Therefore, before any appreciable amount of fill is placed, pore pressures must be measured and an effective stress stability analysis made to determine whether fill can be placed at this time or whether excess pore water pressures must be given time to dissipate, otherwise, it is possible that a bearing capacity failure could occur in the clay.

7.2.1.2. Negative Skin Friction

Consolidation of the clay will be occuring after pile driving; thus, the piles will be subjected to the downward force of the clay. This force is called negative skin friction and its magnitude depends on the surface area of the pile, the depth of clay, and the effective shear strength parameters of the clay.

The piles should be designed for the largest of: -

a) full dead load plus full live load of the bridge

or

b) full dead load plus full negative skin friction.

7.3.1.3. Horizontal Forces on Piles

The magnitudes of stresses in the soft clay may reach such a level that the piles are subjected to large horizontal forces.

If the imposed stresses approach the shear strength of the clay, large shear strains will occur which will cause movement of the clay away from the higher regions either toward the stream bed or away from the approach areas in an east-west direction. This movement will bend the piles and move the structure in the direction of bending.

Since movements of this nature cannot be predicted accurately, measures should be taken to minimize the possibility of this type of failure. Batter piles can be used as a step to resist such movements.

7.2.2. Other Factors

If a bridge structure is chosen as a crossing at this location, it is not recommended that the stream be diverted. The valley is not wide enough to permit a new bridge structure to be built far enough away from the existing stream location to avoid a shear failure through the old stream bed. The factor of safety against shear failure for such a configuration is 1.0 (see fig.14).

In order to increase the safety of the slopes for a bridge, the structure would have to be made lower and longer, hence increasing the cost.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1

The site is underlain by a soft to very soft clay with a low shear strength and quite low sensitivity. The shear strength which should be used for design purposes is 400 p.s.f.

8.2

The clay has a high void ratio and the water contents are greater than the liquid limits. It is overconsolidated by 1500 to 1900 p.s.f. and has a high compression index; hence, maintenance costs due to settlement may be a factor in design.

8.3

The low shear strength of the clay is the limiting factor in the design and construction of a crossing over the Little Castor River at the site in question. Since the road is a secondary route several alternative structures with safety factors lower than 1.3 as used by the Department of Highways of Ontario have been presented for consideration.

8.4

If a culvert and fill structure is to be built, the following points should be noted: -

8.4.1.

To prevent a bearing capacity failure any fill greater than 13 feet in depth should be constructed using either berms or stage construction for a safety factor of 1.3 to be maintained. If lower safety is acceptable, then one of the alternatives in section 7.1 can be used.

8.4.2.

If it is necessary to build head walls for the culverts and if the loads that they impart to the soil mass requires the use of piles, pore pressures must be measured before any substantial fill is placed after pile driving, particularly if one of the alternatives with a low factor of safety is chosen.

8.4.3.

Fill must be properly compacted in order to achieve as much shear strength as possible in the structure.

8.5

If a bridge structure is chosen, the following points should be noted: -

8.5.1.

The foundation for the bridge piers should be supported by minimum displacement piles such as steel H piles. The piles will be end bearing on sound limestone below the soft clay.

8.5.2.

Piles must be designed for the larger of: -

a) full dead and live load of the bridge.

or b) full dead load and full negative skin friction.

8.5.3.

Pore pressures resulting from pile driving should be measured and an effective stress stability analysis made to evaluate the stability of the side slopes before any appreciable amount of fill is placed.

8.5.4.

To prevent a bearing capacity failure the bridge will have to be designed so that it spans such a distance that minimum amounts of fill are required and so that all slopes are constructed no steeper than 2 horizontal to 1 vertical for a safety factor of 1.25 to be maintained. Larger safety factors can be attained by making the slopes less steep or by using berm construction.

8.6

Because of the nature of the clay, care must be taken to prevent excessive disturbance during construction. If, during excavation, the clay is found to be more sensitive than has been found by this investigation, special procedures will have to be used. If such sensitivity is found, excavation should be carried 1 foot more than required for the structure, with the last foot excavated by hand, and a 1 foot sand cushion should be placed in the overexcavated portion.

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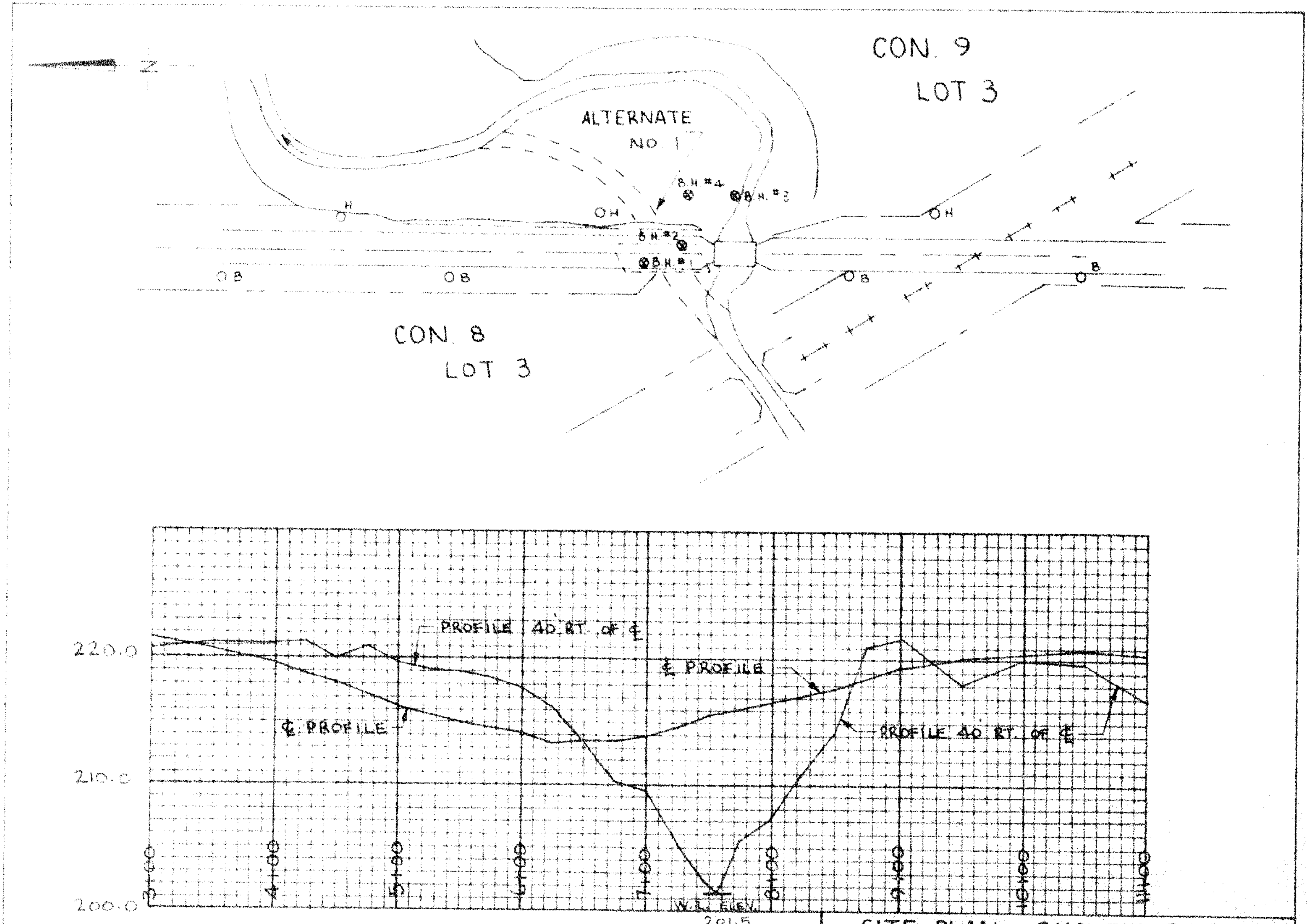
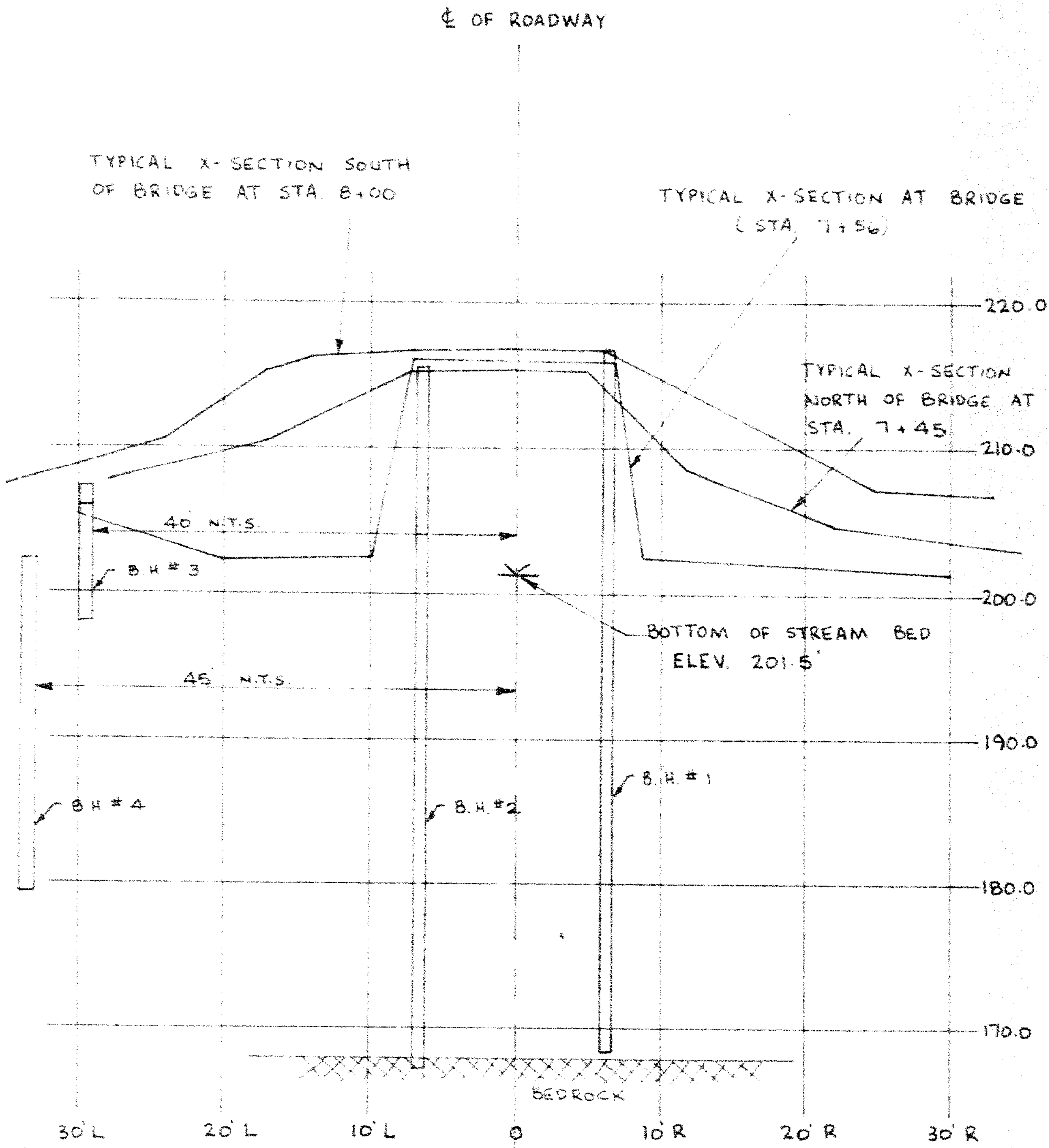


SITE PLAN - RUSSELL BRIDGE

SCALE - HOR. 1" = 100' VERT. 1" = 10'	FIGURE 1



TYPICAL X-SECTIONS AT EXISTING BRIDGE & VERTICAL REPRESENTATION OF BORE HOLES
- RUSSELL TOWNSHIP BRIDGE -

SCALE - 1" = 10'	FIGURE 1-A

P. A. McNEELY AND ASSOCIATES LTD.
TESTING LABORATORIES

FIG. 2

RECORD OF BORE HOLE	STRUCTURE BRIDGE ACROSS LITTLE CASTOR – RUSSELL TWP	DATE MAY 23, 1968	HOLE NO. 1
	LOCATION BETWEEN CON. 8 & CON. 9, LOT 3	BORING TECH. F.E. JOHNSTON	

BORING RECORD NO.	DRAWN BY	CHECKED BY	APPROVED BY
	D. A.	B. F.	P. M.

ELEVATION IN FEET OF:

DATUM	GEODETIC	GROUND SURFACE	216.75
GROUND WATER	210.0	BEDROCK SURFACE	168.5
WATER SURFACE		BOTTOM OF HOLE	168.5

SAMPLING METHOD
- 2" DIAM. SPLIT TUBE
- 2" DIAM. SHELBY TUBE
- ROCK CORE

PENETRATION RESISTANCE
- STANDARD PEN. RESISTANCE (N)
- 2" DIAM. CONE
- CASING

STRENGTH
- UNCONF. COMP. STRENGTH (S_u)
- VANE SHEAR STRENGTH LAB (S_v) X
- FIELD (S_v) *

CONSISTENCY
- NATURAL MOISTURE CONTENT (ω)
- LIQUID LIMIT (ω_l)
- PLASTIC LIMIT (ω_p)
- BULK DENSITY (γ) □
- MAX. PRECONSOL. LOAD (P_c') △

SAMPLE NO. / TYPE / SYMBOLS	DESCRIPTION OF SUBSOIL AND BEDROCK	ELEV. FT.	DEPTH FT.
	FILL (SAND, CLAY, & SILT)		
	G.W.L. 24 HRS AFTER COMPLETION OF HOLE	210.0	
		204.7	
	SOFT TO VERY SOFT SILTY CLAY		
	REFUSAL	168.5	

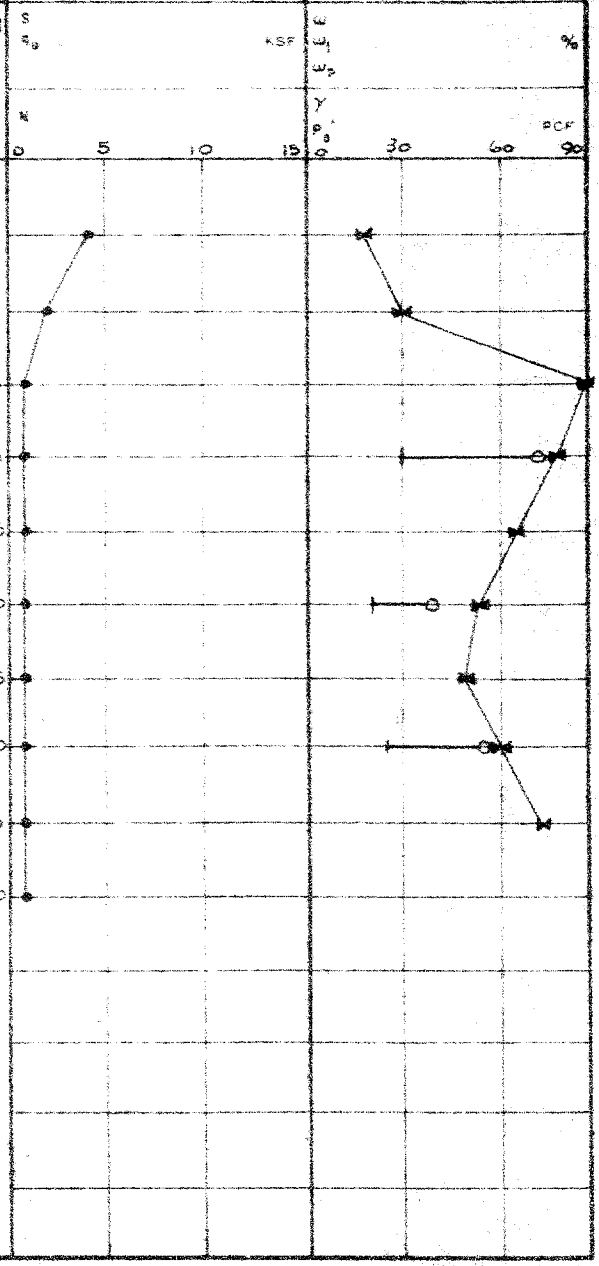


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FIG. 3

RECORD OF BORE HOLE	STRUCTURE: BRIDGE ACROSS LITTLE CASTOR – RUSSELL TWP.	DATE: MAY 23, 1968	HOLE NO. 2
	LOCATION: BETWEEN CON. 8 & CON. 9, LOT 3	BORING TECH.: F.E. JOHNSTON	

BORING RECORD NO.	DRAWN BY	CHECKED BY	APPROVED BY
	D.A.	B.F.	P.M

SAMPLING METHOD
- 2" DIAM. SPLIT TUBE
- 2" DIAM. SHELBY TUBE
- ROCK CORE

ELEVATION IN FEET OF:

DATUM	GEODETIC	GROUND SURFACE	215.8
GROUND WATER	209.0	BEDROCK SURFACE	167.3
WATER SURFACE		BOTTOM OF HOLE	167.3

PENETRATION RESISTANCE
- STANDARD PEN RESISTANCE (N)
- 2" DIAM. CONE
- CASING

STRENGTH
- UNCONF. COMP. STRENGTH (q_u)
- VANE SHEAR STRENGH LAB (S_L) X
- FIELD (S_F)

CONSISTENCY
- NATURAL MOISTURE CONTENT (ω)
- LIQUID LIMIT (ω_l)
- PLASTIC LIMIT (ω_p)
- BULK DENSITY (γ)
- MAX PRECONSOL LOAD (P_c')

SAMPLE NO.	TYPE	SYMBOLS	DESCRIPTION OF SUBSOIL AND BEDROCK	ELEV. FT.	DEPTH FT.
			FILL (SAND, SILT, & CLAY)	209.0	
			SILTY CLAY TO CLAY		
			MEDIUM TO SOFT TO VERY SOFT CLAY		
			MOTTLED SOFT TO VERY SOFT CLAY		
			MEDIUM STIFF CLAY WITH FINE TO COARSE SAND	167.3	
			LIMESTONE		

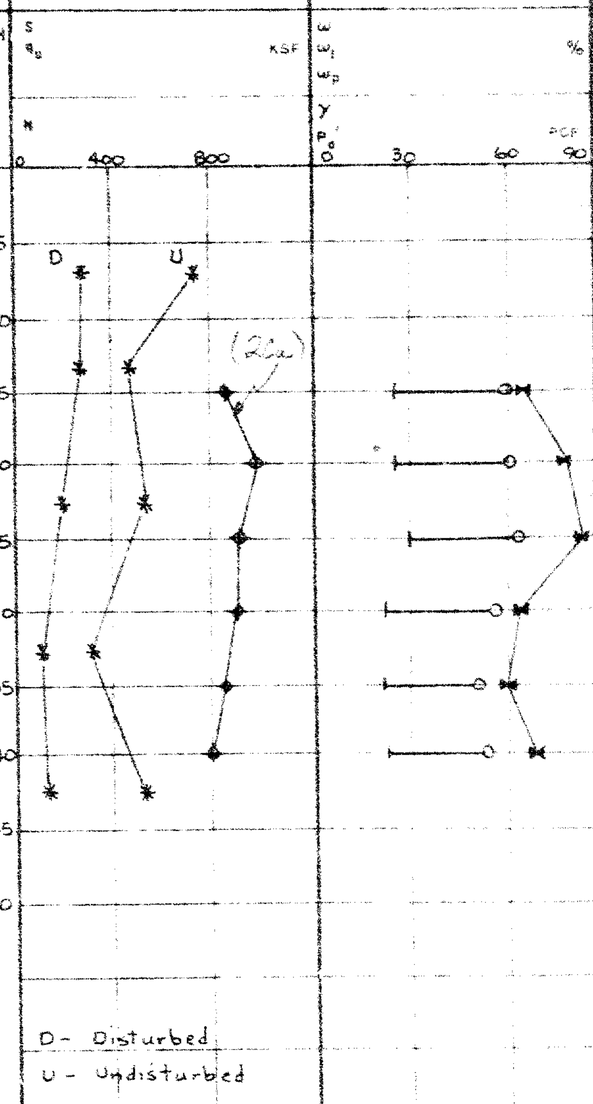

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FIG. 4

RECORD OF BORE HOLE	STRUCTURE BRIDGE ACROSS LITTLE CASTOR – RUSSELL TWP.	DATE MAY 23, 1968	HOLE NO. 3
	LOCATION BETWEEN CON. 8 & CON. 9, LOT 3	BORING TECH HAND AUGER	

BORING RECORD NO.	DRAWN BY	CHECKED BY	APPROVED BY
	D.A.	B.F.	P.M.

ELEVATION IN FEET OF:

DATUM	GEODETIC	GROUND SURFACE	207.3
GROUND WATER		BEDROCK SURFACE	
WATER SURFACE		BOTTOM OF HOLE	199.3

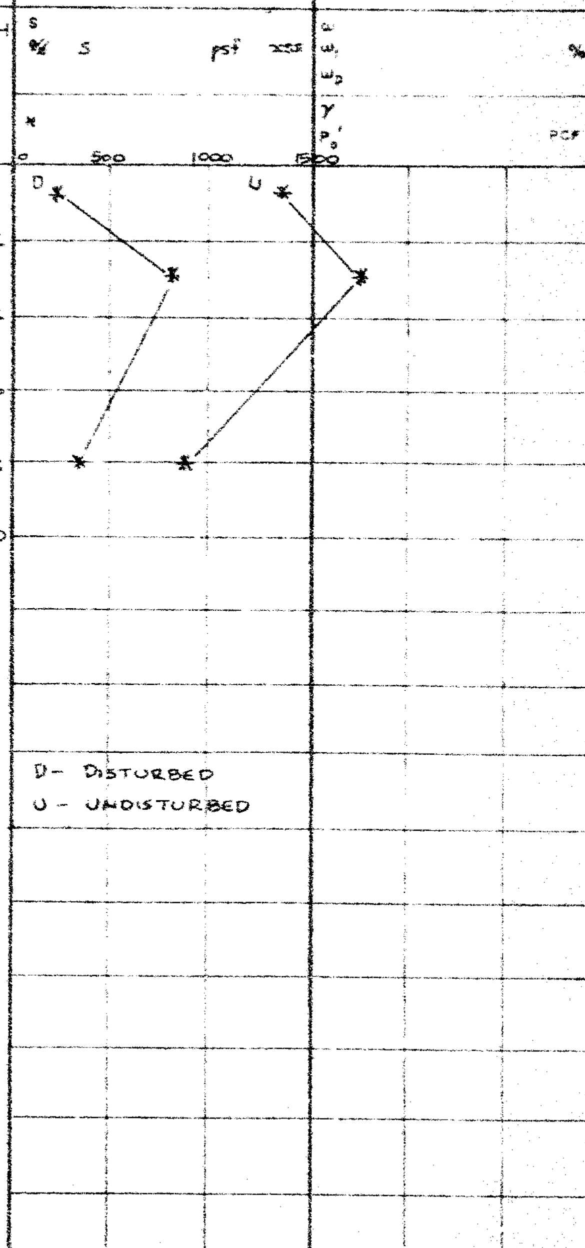
SAMPLING METHOD
- 2" DIAM. SPLIT TUBE
- 2" DIAM. SHELBY TUBE
- ROCK CORE

PENETRATION RESISTANCE
- STANDARD PEN RESISTANCE (N)
- 2" DIAM. CONE
- CASING

STRENGTH
- UNCONF. COMP STRENGTH (q_u)
- VANE SHEAR STRENGH LAB (S_L)
- FIELD (S_F)

CONSISTENCY
- NATURAL MOISTURE CONTENT (ω)
- LIQUID LIMIT (ω_L)
- PLASTIC LIMIT (ω_P)
- BULK DENSITY (γ)
- MAX. PRECONSOL. LOAD (P_o')

SAMPLE NO.	TYPE	SYMBOLS	DESCRIPTION OF SUBSOIL AND BEDROCK	ELEV. FT.	DEPTH FT.
			ORGANIC TOPSOIL		
			SANDY SILT		2
			CLAYEY SILT		4
			SILTY CLAY		6
			BOTTOM OF HOLE		8
					10



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P. A. McNEELY AND ASSOCIATES LTD. FIG. 5
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RECORD OF BORE HOLE	STRUCTURE BRIDGE ACROSS LITTLE CASTOR – RUSSELL TWP.	DATE MAY 23, 1968	HOLE NO. 4
	LOCATION BETWEEN CON. 8 & CON. 9 – LOT 3	BORING TECH. HAND AUGER	

BORING RECORD NO.	DRAWN BY	CHECKED BY	APPROVED BY
	D.A	B.F.	P.M.

SAMPLING METHOD
- 2" DIAM. SPLIT TUBE
- 2" DIAM. SHELBY TUBE
- ROCK CORE

ELEVATION IN FEET OF:

DATUM	GEODETIC	GROUND SURFACE	202.3
GROUND WATER		BEDROCK SURFACE	
WATER SURFACE		BOTTOM OF HOLE	179.3

PENETRATION RESISTANCE
- STANDARD PEN RESISTANCE (N)
- 2" DIAM. CONE
- CASING

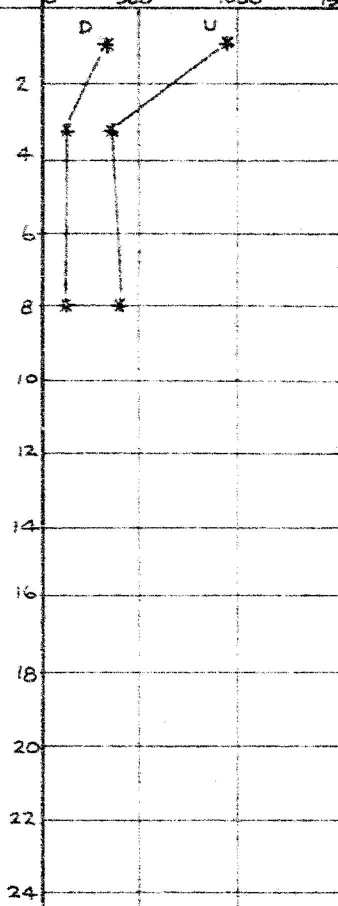
STRENGTH
- UNCONF. COMP. STRENGTH (q_u)
- VANE SHEAR STRENGH LAB (S_L) X
- FIELD (S_F)

CONSISTENCY
- NATURAL MOISTURE CONTENT (ω)
- LIQUID LIMIT (ω_L)
- PLASTIC LIMIT (ω_P)
- BULK DENSITY (γ)
- MAX. PRECONSOL LOAD (P_0')

SAMPLE NO.	TYPE	SYMBOLS	DESCRIPTION OF SUBSOIL AND BEDROCK	ELEV. FT.	DEPTH FT.
			SILTY CLAY		
			SILTY CLAY TO CLAY		
			BOTTOM OF HOLE		

S (psf): 0, 500, 1000, 1500 — N

ω, ω_L, ω_P % — γ, P_0' PCF



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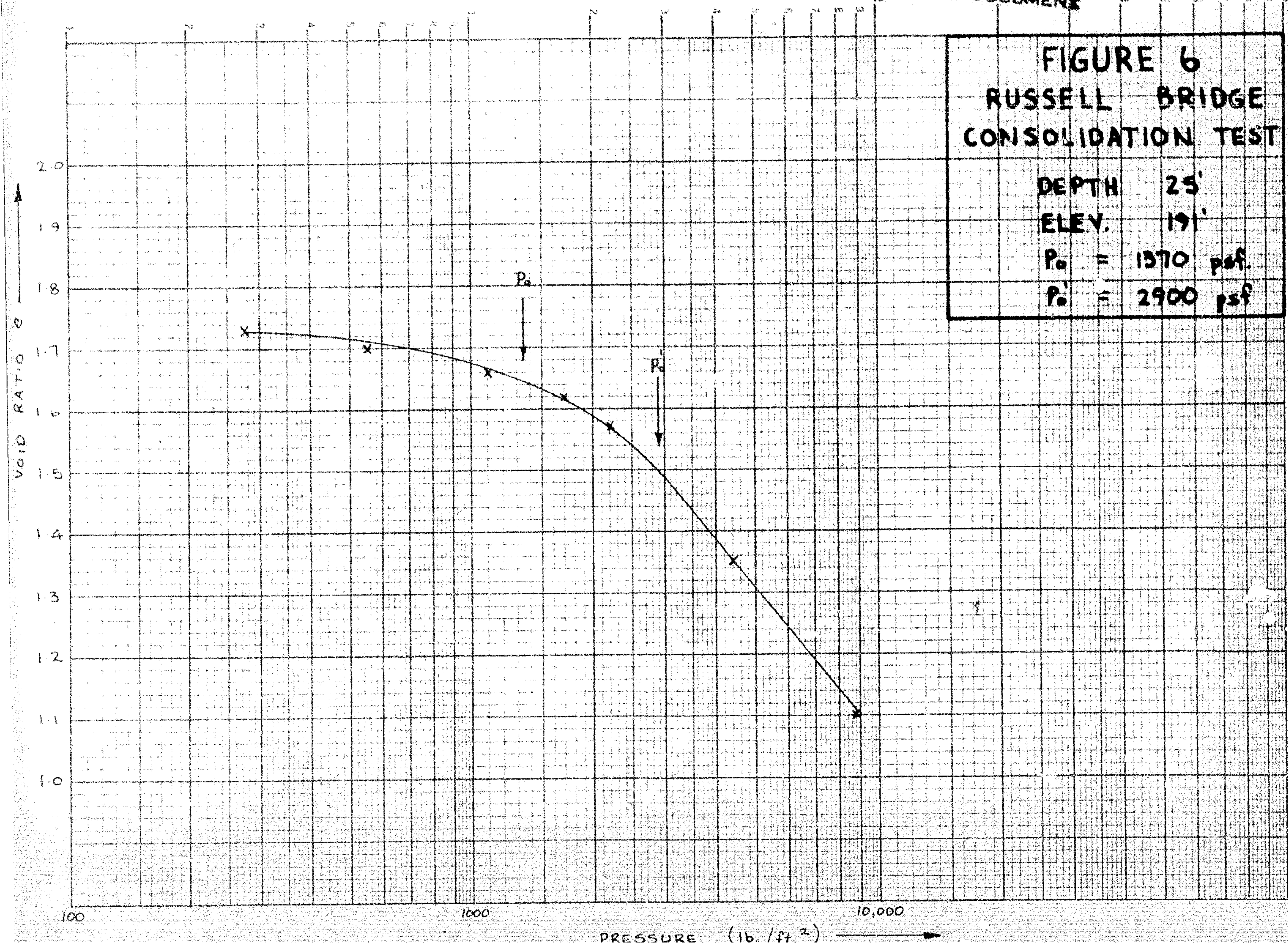

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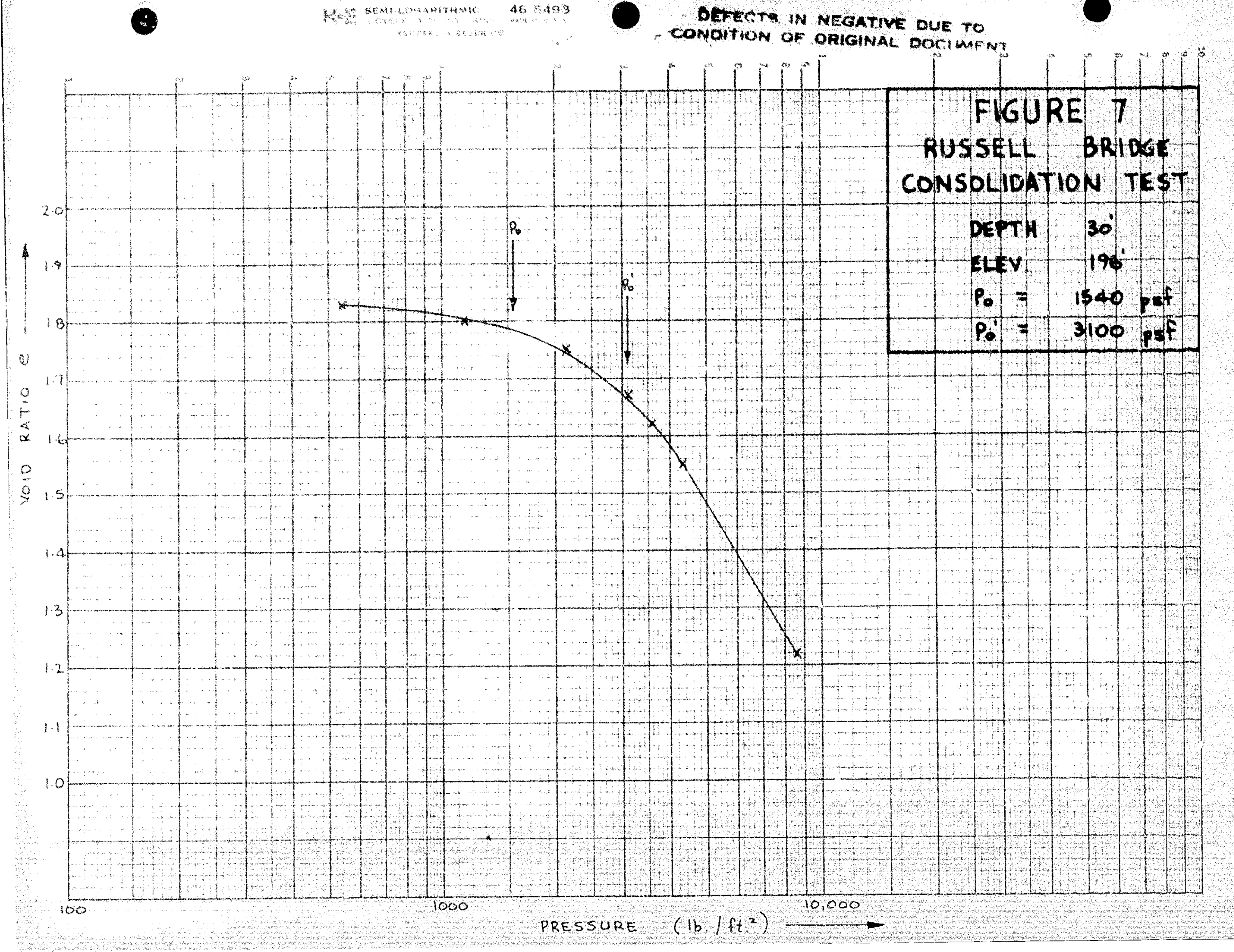

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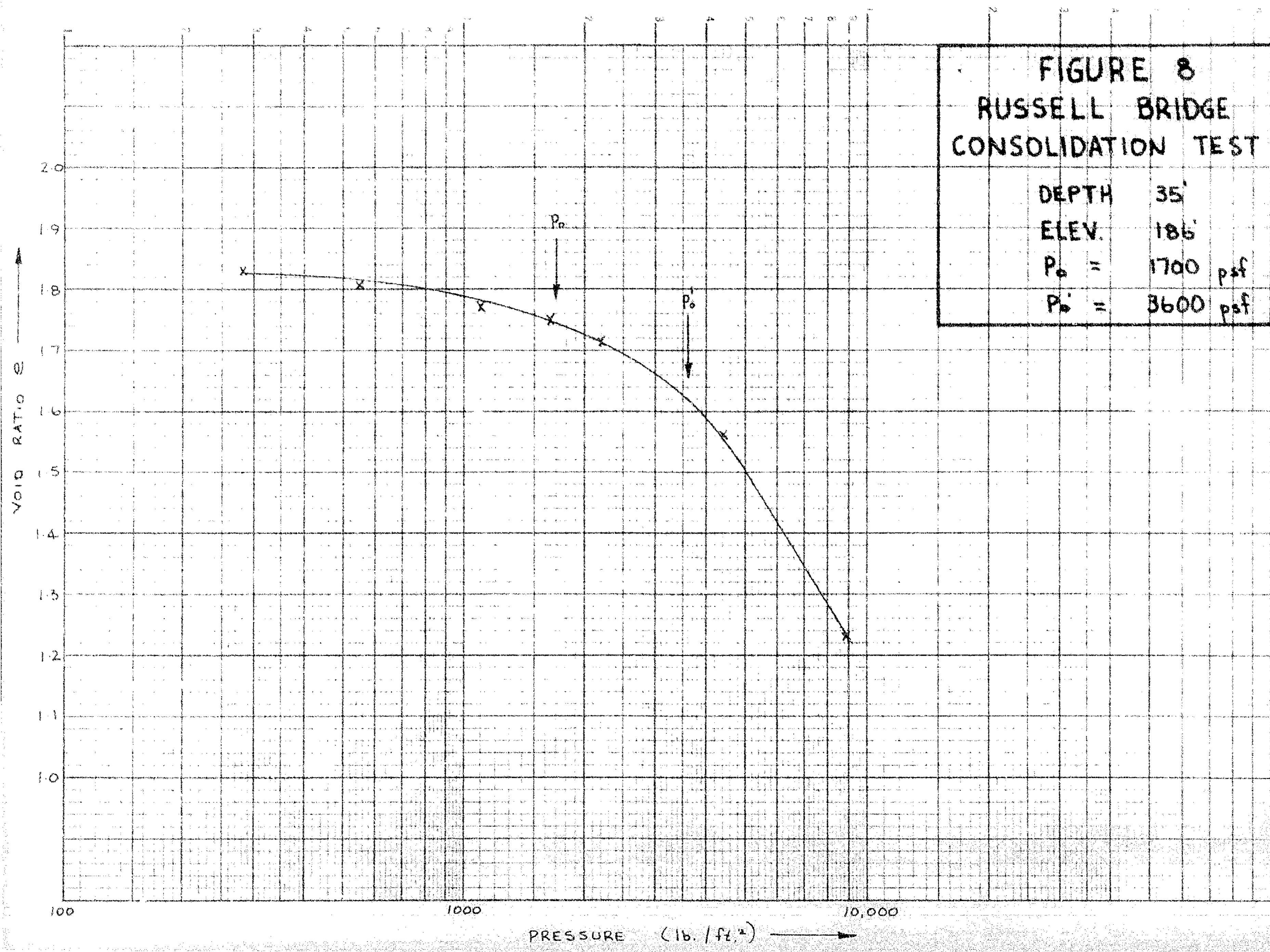
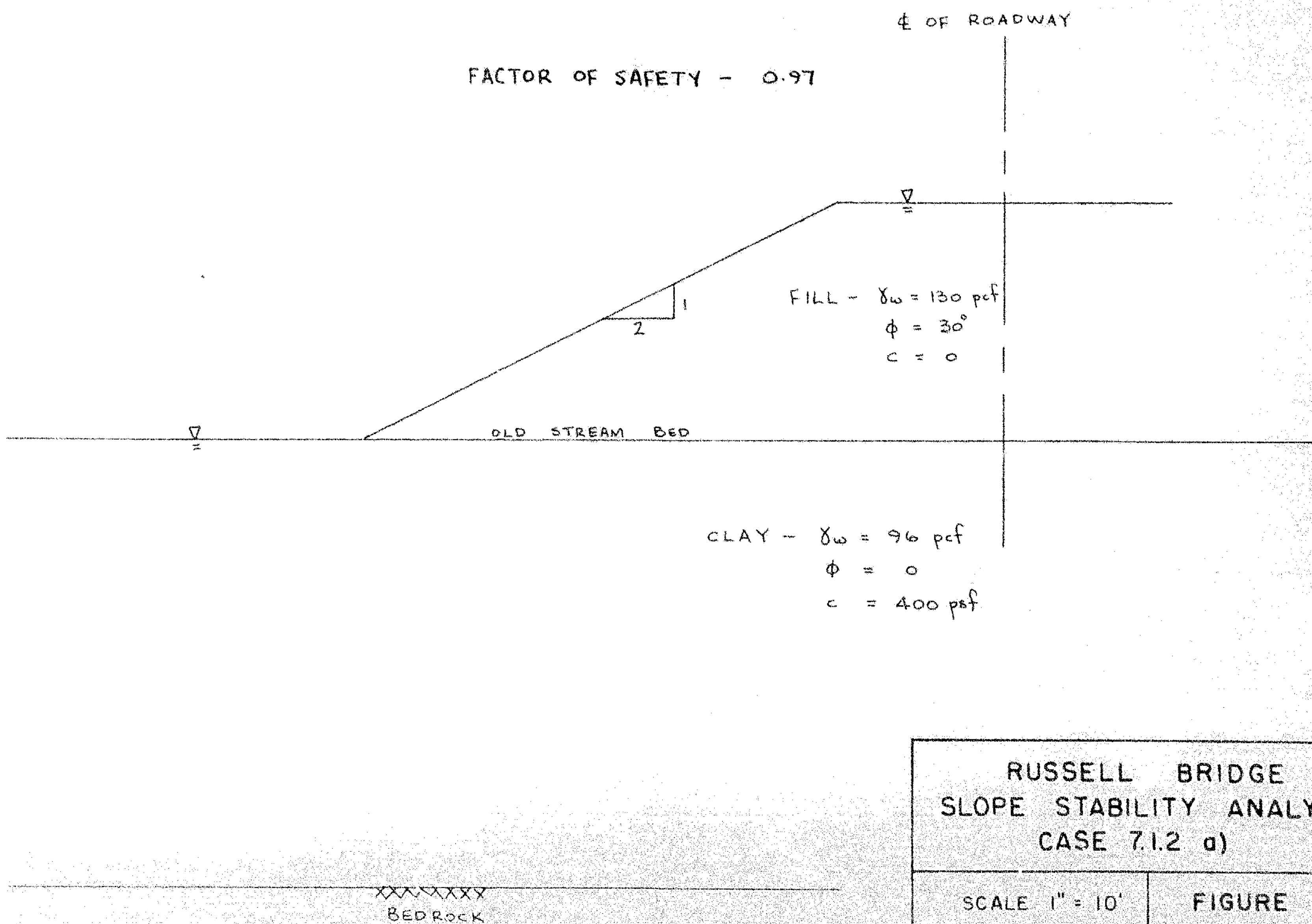
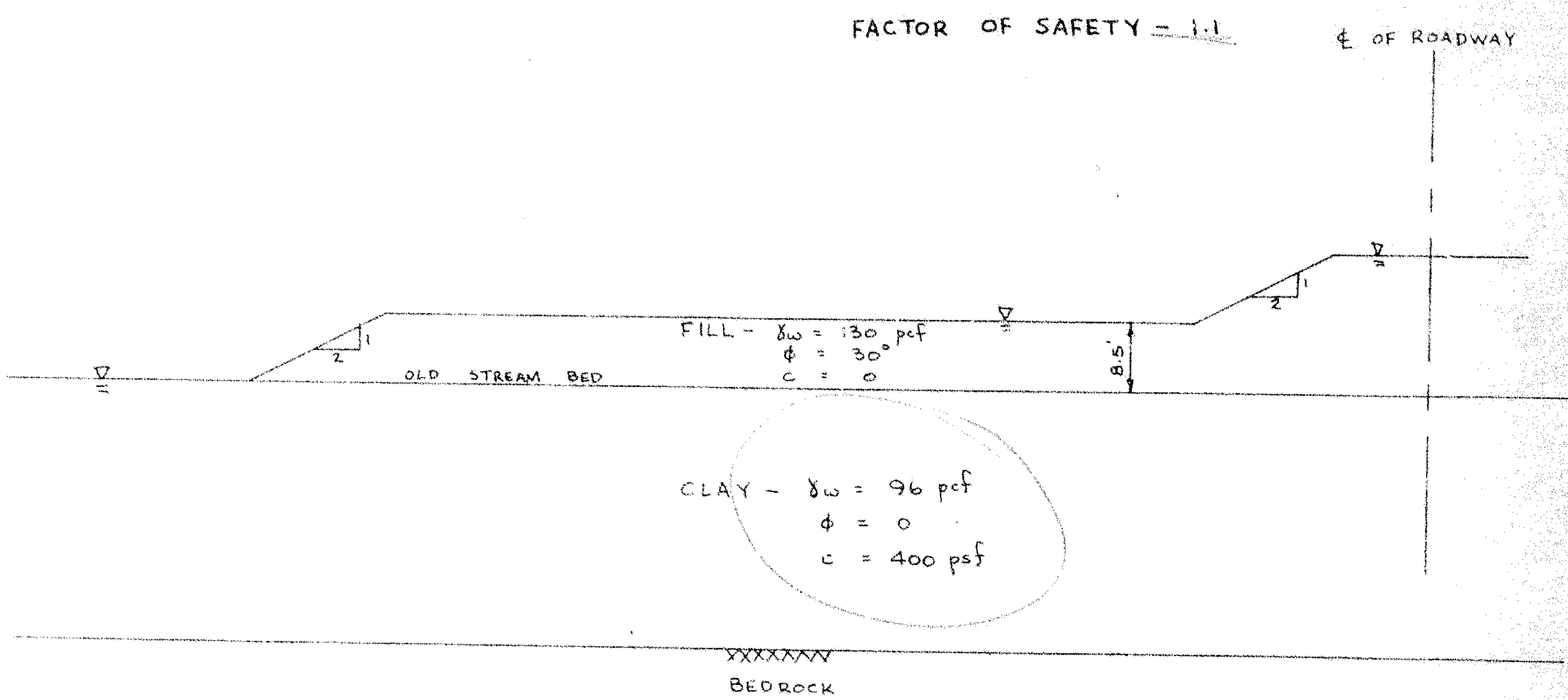




RUSSELL BRIDGE
SLOPE STABILITY ANALYSIS
CASE 7.1.2 a)

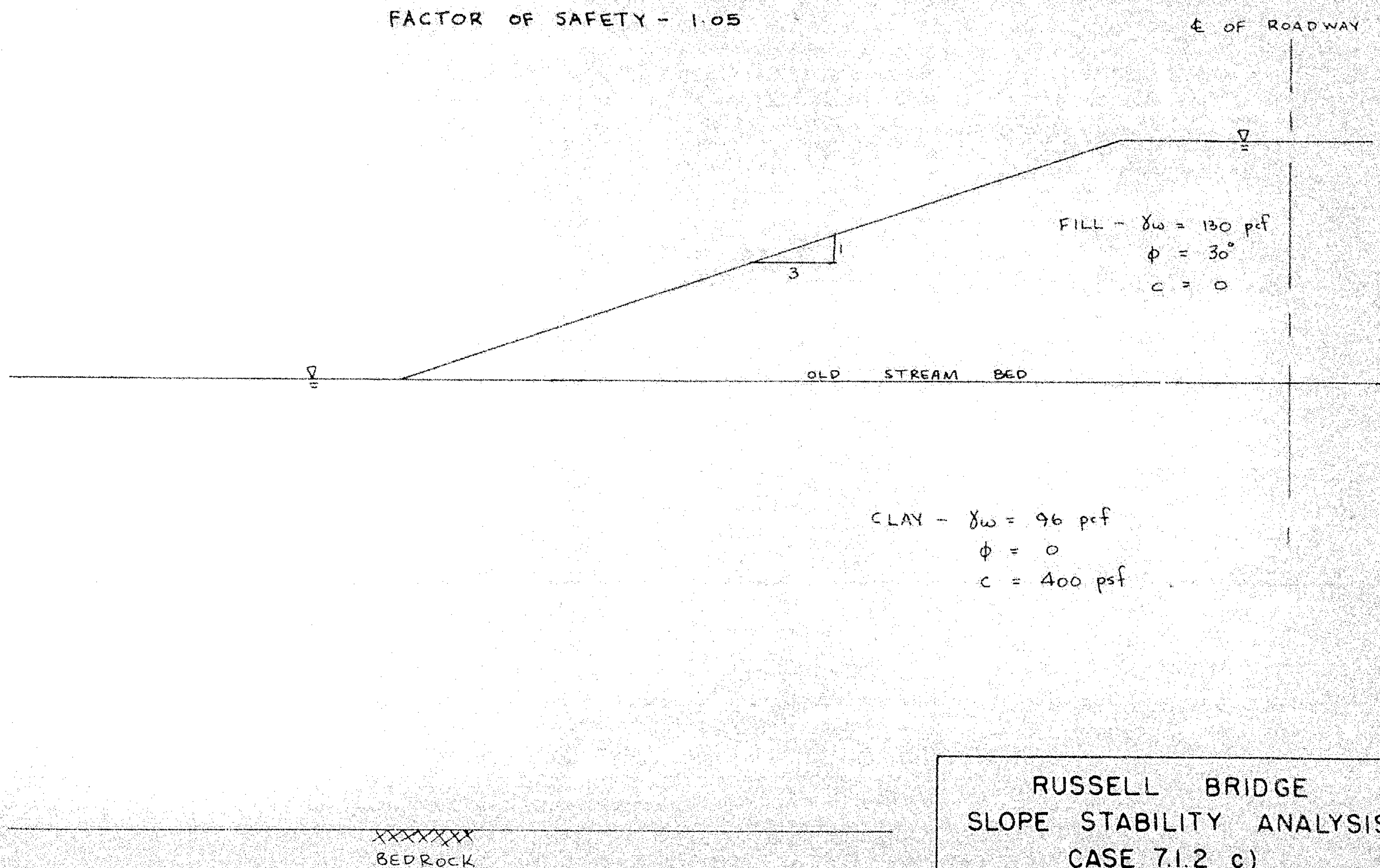
SCALE 1" = 10'	FIGURE 9



RUSSELL BRIDGE
SLOPE STABILITY ANALYSIS
CASE 7.1.2 b)

SCALE 1" = 20'

FIGURE 10

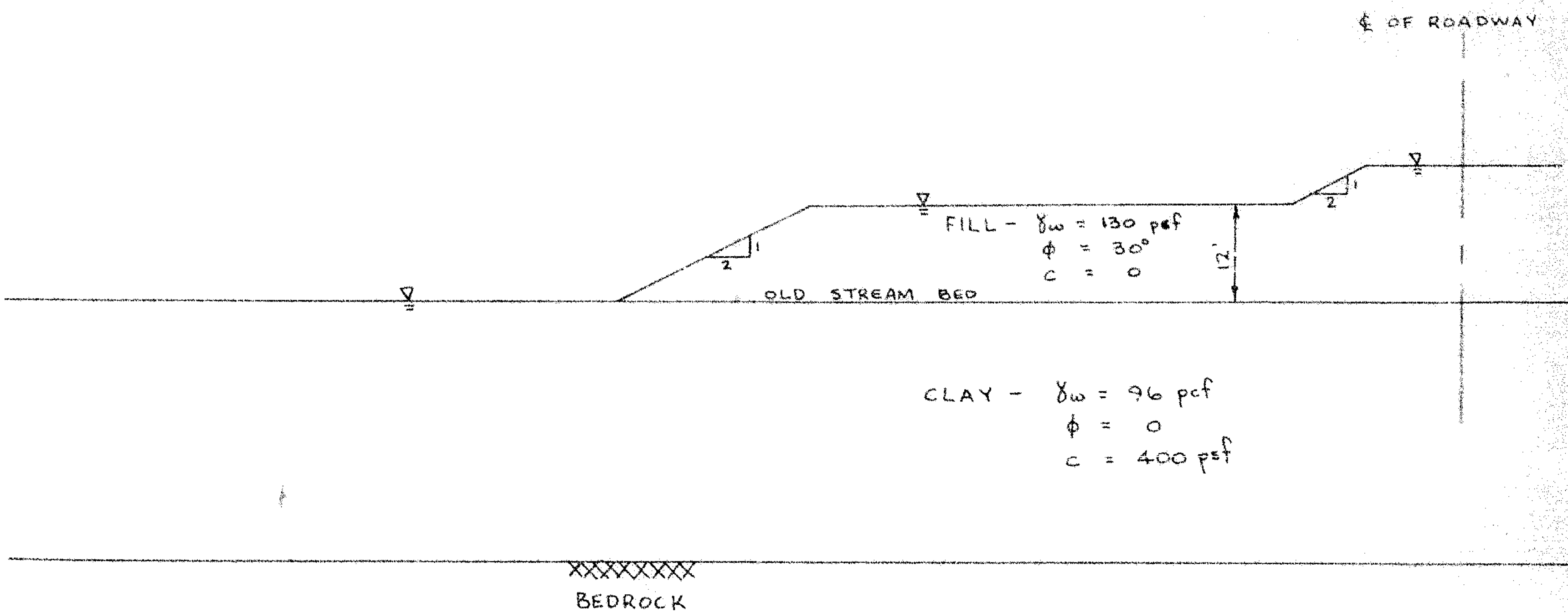


RUSSELL BRIDGE
SLOPE STABILITY ANALYSIS
CASE 7.1.2 c)

SCALE 1" = 10'

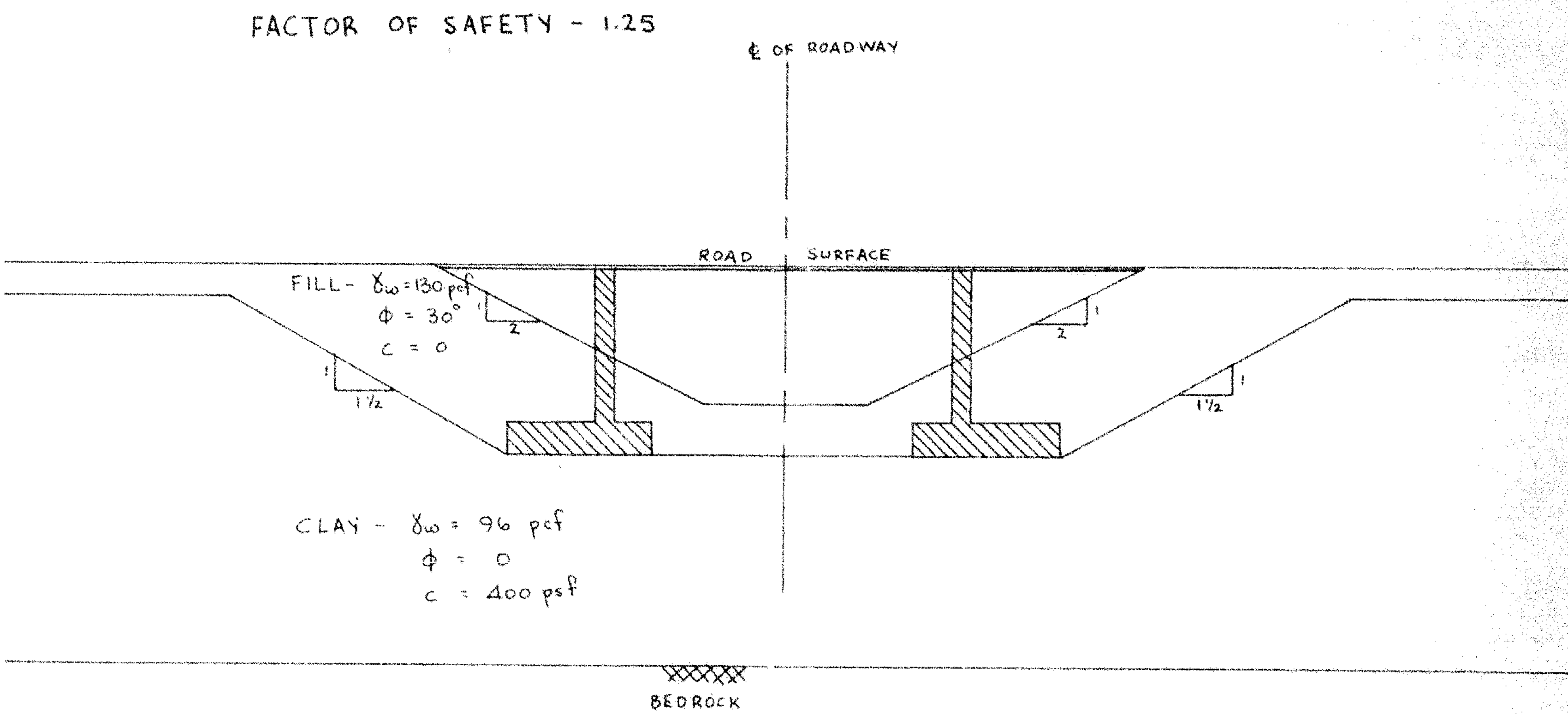
FIGURE 11

FACTOR OF SAFETY - 1.3



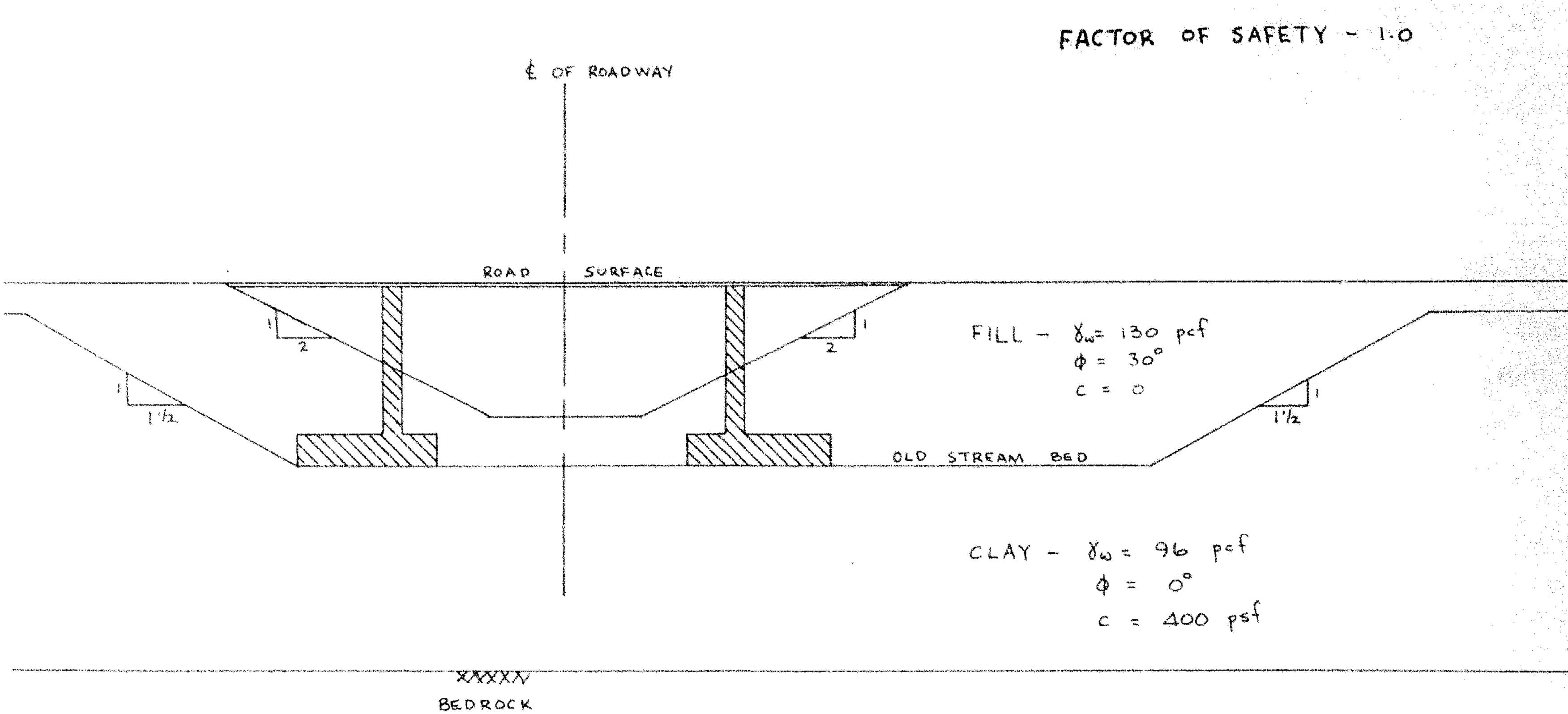
RUSSELL BRIDGE
SLOPE STABILITY ANALYSIS
CASE 7.1.2 d)

SCALE 1" = 20'	FIGURE 12



RUSSELL BRIDGE
SLOPE STABILITY ANALYSIS
CASE 7.2 a)

SCALE 1" = 20'	FIGURE 13



RUSSELL BRIDGE
SLOPE STABILITY ANALYSIS
CASE 7.2 b)

SCALE 1" = 20'	FIGURE 14

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. K. L. Kleinsteiber,
Municipal Bridge Liaison Engr.,
Bridge Office,
Admin. Bldg.

From: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

Attention:

Date: December 19, 1968

Our File Ref:

In reply to

Subject:

Little Castor River Crossing
Twp. Road Between Conc. 8 & 9
At Lot 3, Russell Township

We have reviewed the Report on Foundation Conditions for the above mentioned site, prepared and submitted by the Consultant, McNeely and Lecompte, and the letter of November 15, 1968 by the Consultant to Mr. L. M. Peverett, District Municipal Engineer, regarding the same subject. Below please find our comments:

The Consultant has investigated two basic alternatives for this site, one being twin culverts, and the other being a 100-ft. long bridge.

A number of stability analyses were carried out to determine the factors of safety for various fill configurations.

In the above mentioned letter the Consultant presents the preliminary cost estimates, giving a figure of $43,000.00 for the twin culverts, and $70,000.00 for the bridge.

From this presentation it would appear that the twin culverts have a distinct advantage over the bridge. However, in his further presentation the Consultant points out that the factor of safety for the twin culverts scheme is only 1.05 as compared with 1.3 for the bridge scheme.

We feel that such a direct comparison is not fair and does not present the true picture.

It is our approach to aim for a factor of safety of about 1.2 to 1.3 when analyzing embankment stabilities. It does not matter whether there is a bridge or a culvert, because practically always, the possible failure is a failure in the sub-soil, and this would affect any structure.

There is naturally a certain flexibility in choosing the appropriate factor of safety. A number of factors influence this choice. These are: the importance of the structure, consequences resulting from a possible failure, means and costs of repair work, political implications, etc. For the different alternatives considered for the same crossing, the most important factors are usually the same and, consequently, the same factor of safety should apply.

We would like to mention though, that sometimes a careful three-dimensional analysis of the problem discloses that certain failures are most unlikely to occur because of the changing geometry in the direction of the third dimension. This problem is encountered often when lateral stabilities of fills are analyzed, which cross ravines with steep side slopes. The analysis of the deepest cross-section is certainly not representative for the entire embankment, since a section as high as that may only be 20 ft. wide. A failure of a slice 20 ft. wide within a large embankment, is not very likely to occur.

It is suggested that the Consultant review his design in the light of the above mentioned.

AGStermac

AGS/MdeF

A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

P.S. -- Consultant's report returned herewith.

cc: Foundations Files
Gen. Files

PHONE 762-5426

P. O. BOX 536
ROCKLAND, ONTARIO

McNEELY & LECOMPTE

CONSULTING CIVIL ENGINEERS

November 15th, 1968.

Mr. L. M. Feverett, P.Eng.,
District Municipal Engineer,
Ontario Department of Highways,
530 Tremblay Rd.,
Ottawa, Ontario.

Re: Little Castor River Crossing,
Twp. Road between Con. 8 & 9
at Lot 3, Russell Township.

Dear Sir:

We have forwarded on November 4th, 1968, two copies of the soils report, one copy of the original cross sections and five copies of the plan and profile for this river crossing.

The preliminary cost estimate was sent to the Township of Russell on September 3rd, 1968. This estimate for the twin culverts and the reconstruction of 1,350 lin. feet of approach roads was $43,000.00. A less detailed estimate for a bridge at this location indicated a total cost of $70,000.00.

The basic reason for the high cost of the bridge and the large difference between the costs of the two types of crossings is due principally to the poor foundation soil at this site requiring low fills therefore, long approaches. We considered that a lower factor of safety could be tolerated with a metal culvert than that which would be required for a bridge.

The twin culvert design would have a safety factor of 1.05 against shear failure of the approaches while the estimate for the bridge was based on requiring a safety factor of 1.3.

From discussions with Mr. K.L. Kleinsteiber, from the Department of Highways Bridge Office, on November 7th, 1968, it would appear that the Department of Highways

..../2

would not accept a factor of safety less than 1.3 for either alternative.

Could you please forward our plans, profile etc. to the Bridge Office together with a copy of this letter and we will await their comments before carrying out additional work.

Yours very truly,

Jacques Lecompte

Jacques Lecompte, P.Eng.

JL/cc
c.c. Corporation of Russell Township.

PHONE 762-5486

P. O. BOX 536
ROCKLAND, ONTARIO

McNEELY & LECOMPTE

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Jacques Lecompte

Jacques Lecompte, P.Eng.

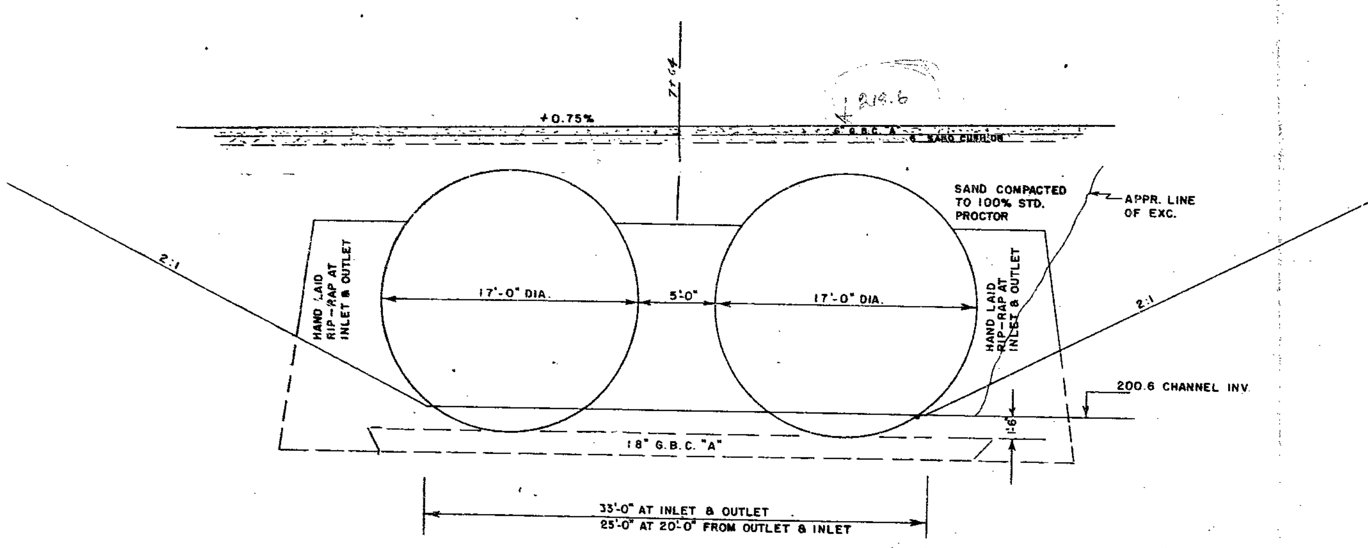
JL/cc
c.c. Corporation of Russell Township.

68-F-226 M

LITTLE CASTOR

RIVER

RUSSELL TWP.



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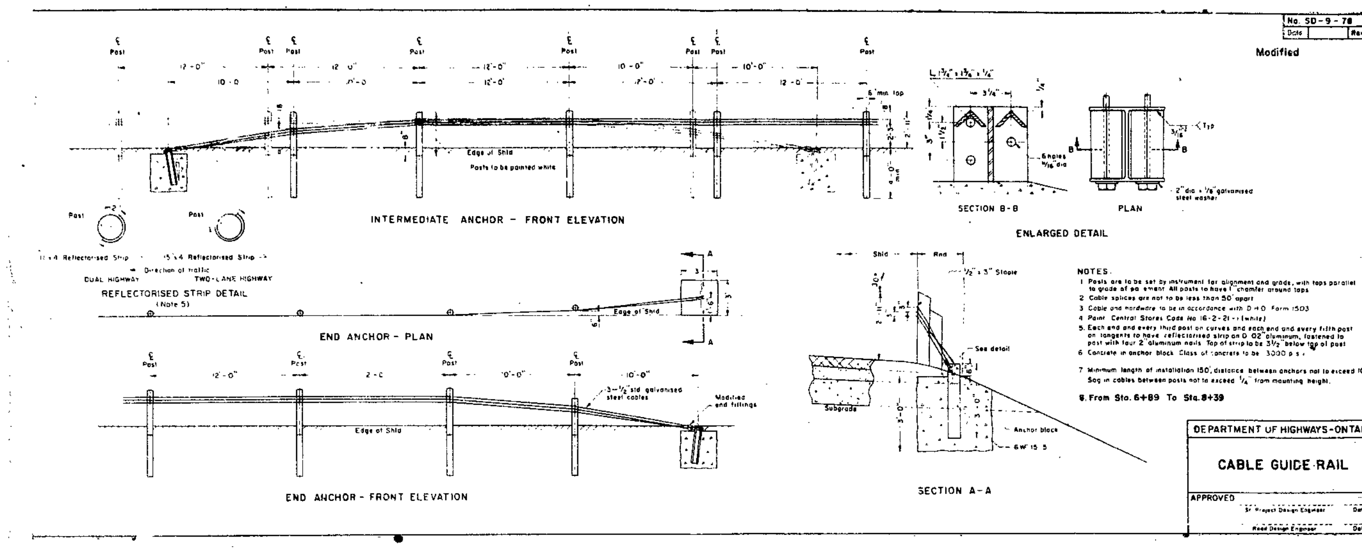
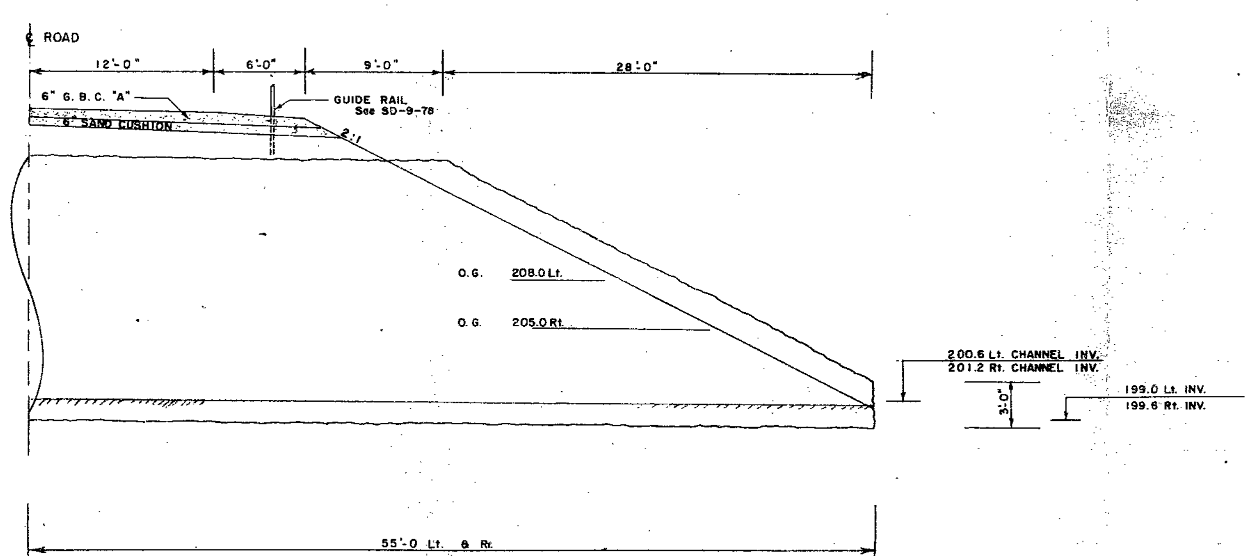
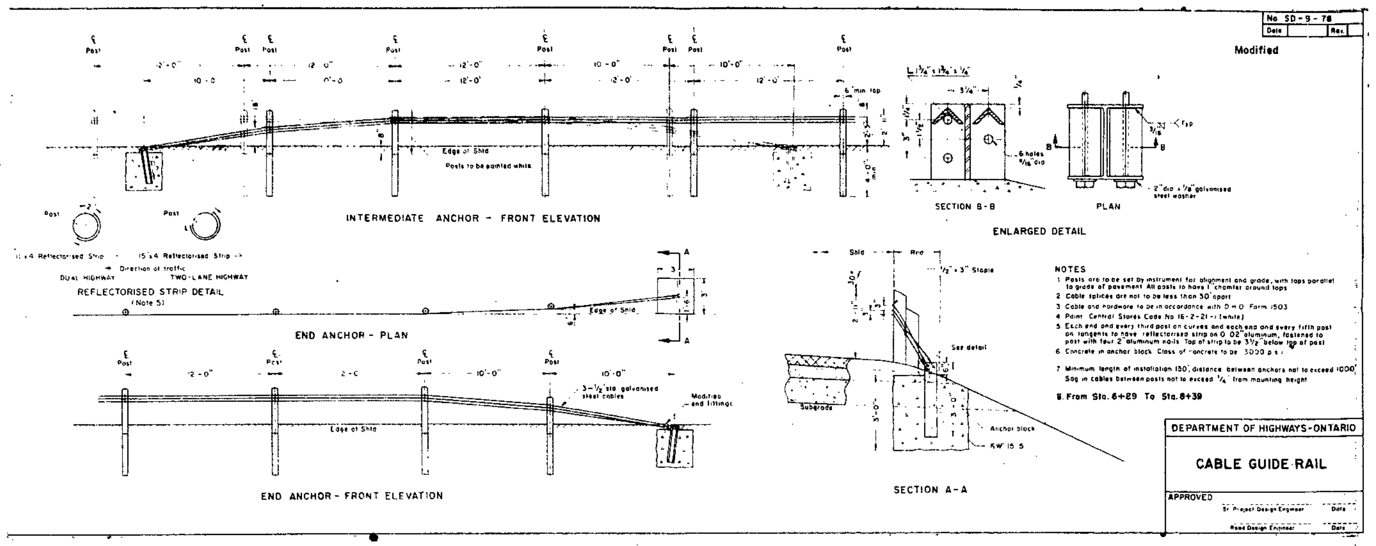


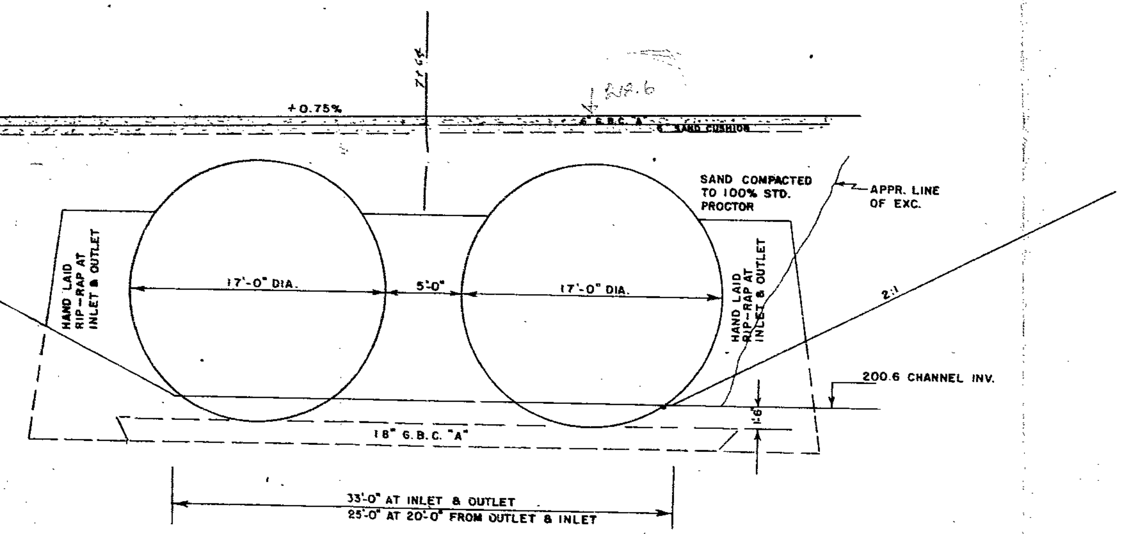


SCALE: 1" = 5'-0"

REGISTERED PROFESSIONAL ENGINEER
CIVIL
J. LECOMPTE
PROVINCE OF ONTARIO

STRUCTURE SITE No. 27-126





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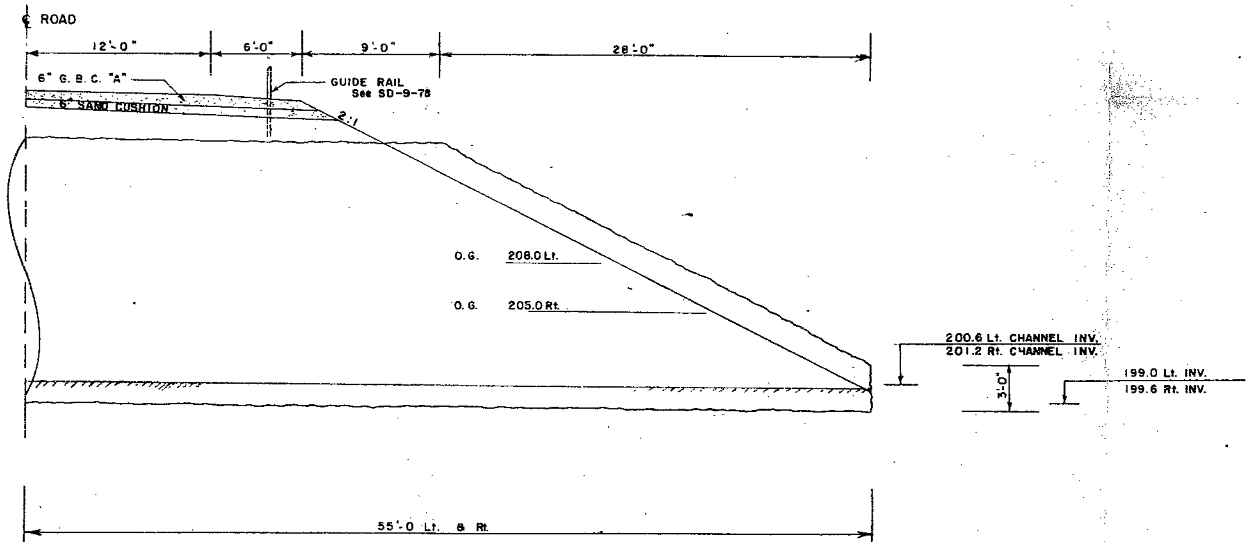

SCALE: 1" = 5'-0"

REGISTERED PROFESSIONAL ENGINEER
CIVIL
J. LECOMPTE
PROVINCE OF ONTARIO

STRUCTURE SITE No. 27-126

D. H. O.
TORONTO
RECEIVED
FEB 24 1969
BRIDGE OFFICE
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