



**DOMINION SOIL INVESTIGATION LIMITED**  
CONSULTING SOIL & FOUNDATION ENGINEERS

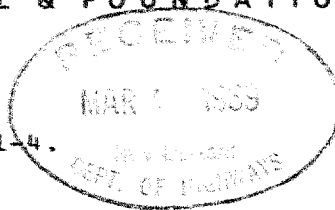
**HEAD OFFICE**

104 CROCKFORD BLVD.  
SCARBOROUGH, ONT.  
CANADA  
TELEPHONE: 751-5565

**BRANCH OFFICE**

369 QUEENS AVE.  
LONDON, ONT.  
TELEPHONE: 433-3731

Our Ref. No. 8-11-4.  
February, 1969.



**ASSOCIATED COMPANY**

SOIL TESTING AND ENGINEERING LTD.  
39 BRENTFORD ROAD  
KINGSTON 5, JAMAICA  
WEST INDIES

FOUNDATION STUDY  
FOR THE  
PROPOSED RECONSTRUCTION OF  
UNO PARK BRIDGE OVER WABI RIVER  
TOWNSHIP OF HARLEY.

Prepared for:  
THE SUTCLIFFE COMPANY.  
Consulting Engineers,  
NEW LISKEARD.

**Distribution:**

6 copies - The Sutcliffe Co.  
2 copies - Harley Township.  
2 copies - Dominion Soil Investigation Limited.



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INTRODUCTION

Dominion Soil Investigation Limited was retained by the Sutcliffe Company, Consulting Engineers, acting on behalf of the Township of Harley, to prepare a consultative report on the foundation conditions at Uno Park Bridge over the Wabi River. It is considered to repair or reconstruct the existing structure due to excessive differential settlement of its pile supports.

For the purpose of the study the following documents have been made available:

Department of Highways, Ontario - Drawing No.D2681-1 dated 27th June, 1940, General Plan and Details.

Sutcliffe Company - Plan and Elevation of Existing Structure, dated November, 1968.

Letter dated January 7th, 1969 by Mr. J.W. Neelands of Sutcliffe Company.

Several photographs showing the failure of the west approach fill in 1942 and the reconstructed present structure, 1968.

The results of borings put down by the Department of Highways in 1940.

On the basis of the above information and additional field and laboratory tests performed by Dominion Soil Investigation Limited between November 1968 and January 1969, the following report is submitted.

HISTORY OF THE BRIDGE

The 115 ft. long and 20.3 ft. wide highway bridge comprising a central 75 ft. long steel truss span and two 20 ft. long steel beam end spans was designed by the D.H.O. in 1940 to replace an old bridge lying just upstream from this site. The reasons of this replacement are not known and some of the notes on the D.H.O. Drawing No. D2681-1 indicate that some of the structural members of the old structure have been incorporated in the new construction.

The sub-structure of the new bridge consisted of two 25 inch diameter octagonal Algoma steel sheet, reinforced concrete piles at each end of the truss which according to the design should have been driven to refusal at elevation +54 ft. The two piles were connected by a reinforced concrete cap beam offering direct support for the trusses and for the I girders of the end spans. The other end of the end span was supported by two 20-3/4 inches diameter hexagonal Algoma steel sheet piles tied together with a reinforced concrete beam.

The axis of the new bridge crossed the stream line of the river at a sharper angle than the old one, thus exposing its supporting piles on the west bank to increased scour and erosion effects.

The construction of the bridge was completed in 1941. In the early summer of 1942, following three days of rain, the west embankment slid into the river, the toe stopping about mid-stream. The failure was probably due to the excessive overburden weight of the approach fill, the unsuitable nature of the fill material (fine sand which became heavily water-logged during the heavy rainfall), and possibly due to erosion and scour of flood waters.

Following shortly the failure in 1942, the approach fill on the west side was replaced by three 15 ft. long timber spans supported on timber piled trestles and a rock filled timber crib to reinforce the inclined south west pier.

Excessive settlement of the south west pier and the reconstructed end span at the junction with the timber approach span was noticed about 2 years ago and the movement appears to be continuing since then.

Following this, Dominion Soil Investigation Limited was requested to conduct a comprehensive soil investigation to determine the reason of this settlement and to make recommendations for the possible repair or replacement of the bridge.

### SOIL CONDITIONS

Prior to the construction of the original structure in 1940, three borings were put down by the Department of Highways. These borings designated as Boreholes S1, S2 and S3 showed that the site is underlain by soft clay deposits to a depth of about 35 to 38 feet, at which depth, that is at elevation 54 to 59 ft., hardpan was encountered. Therefore, on the design drawing for the structure it was indicated that the Algoma steel sheet piles shall be driven to refusal to the approximate level of 54 ft.

Exploratory borings (Boreholes No.1 and 2) effected in November, 1968 by Dominion Soil Investigation Limited have indicated however, that although some increase in the shear strength may be expected at this depth, the soft to stiff clay deposit extends to a depth of about 145 ft. or deeper. Detailed results of the borings are shown on Enclosure Nos.1 and 2 indicating also the other relevant index and engineering properties of the subsoil as established in the laboratory.

Below approximately 9 feet of heterogeneous surficial deposits, consisting predominantly of silty gravelly sand, to the full depth of the exploration the subsoil consists of a grey varved clay, typical of the post glacial lake deposits found in the New Liskeard area.

The clay has a medium to high plasticity and compressibility as indicated by the Atterberg tests (liquid limit 35 to 60%; plastic limit 18 to 20%) the high natural moisture contents which were at or above the liquid limit, and void ratios ranging between 0.8 and 2.0.

Except within the thin desiccated and stiff crust the undrained shear strength of the clay in the top 40 ft. of the deposit ranges only between 360 and 500 pounds per square foot. Below this soft zone, the shear strength of the clay gradually increases and at a depth of about 80 ft. it reaches a value of 1200-1500 pounds per square foot.

In Borehole No.1 the groundwater level was observed at elevation 88.2 ft. which corresponds with the river level at that time of the investigation. In Borehole No.2 located at the top of the west bank the water level was at elevation 99 ft. that is, 5 feet below the ground surface.

ANALYSIS OF THE EXISTING FOUNDATIONS

Analysing the results of Borehole No.1 put down adjacent to the south west pier it is obvious that the supporting pile could not have been driven to refusal and that the load on the pile is carried by the skin friction between the pile and the surrounding soft plastic clay.

The maximum load on the pile under total dead and live load conditions was calculated to be 65.2 tons. For the 40 ft. length of embedment shown on D.H.O. Drawing No. D2681-1, the average skin friction on the pile is:

$$f_s = \frac{P}{A_s \times L} = \frac{65.2 \text{ tons}}{2.1 \times \pi \times 40 \text{ ft.}} = 0.25 \text{ t.s.f.} = 500 \text{ p.s.f.}$$

According to the logs of Boreholes 1 and 2, the undrained shear strength value of the clay stratum above elevation 50 ft. ranges between 330 and 700 lbs. per square foot with an average value of less than 500 lbs. per square foot.

When considering that even these low shear strength values could not be mobilized because of the small specific adhesion between the soil and the steel mantle of the Algoma pile, it is clear that under the given soil conditions the bearing capacity of the pile was not sufficient to carry even the deadload (51.4 tons) transmitted by the bridge.



Levelling records taken by the Sutcliffe Company have indicated however, that excessive settlements were noticed only at the downstream south west pier. The vertical displacement of the centre truss span from the original, as designed, plane is shown on Fig.1.

As shown on Fig.1, piles A, B and B', that is the two east piers and the north west pier, have suffered relatively small and nearly uniform settlements (about 2 inches). The south west pier (A<sup>1</sup>) however, underwent a total vertical displacement of 0.83 ft., that is 10 inches, resulting in a differential settlement of about 0.64 ft. The difference in elevation between the curbs of the truss and the span is an indication that settlement is still in progress. The settlement pattern shown on Fig.1 also indicates that from the four pier supports of the truss, it was only the south west pier which suffered excessive settlements and the other three piers may be considered safe. It is therefore possible that the eastern and the north west piers were in fact driven to hardpan or till as indicated by the D.H.O. borings and the inferred subsoil profile shown on drawing No.1 attached to this report.

Analysing the settlement pattern shown on Fig.1 it is observed that points A', C', D' and B', all located on the lower chord of the south steel truss, can be connected with a straight line without considerable break in the lower chord axis of the truss. Such break would indicate excessive deformation and possible yielding of the steel superstructure. Naturally in the repair work all connections of the cross girders to the lower chord and primarily that of between A and A' must be thoroughly inspected and all loose and damaged connections must be repaired.

The original approach to the west abutment consisted of an approximately 50 ft. long and 14 ft. high earth embankment which, few months after the construction, failed and slid into the river. The failure damaged the end span of the structure and weakened the pile supports of the west pier.

Although a detailed analysis of the failure was not carried out, calculations based on total and effective stresses indicate that the failure of the approach fill was inevitable, as the weak subsoil was unable to support the weight of the embankment. Stability calculations also indicate that with an effective angle of shearing resistance of  $21^{\circ}$  (see enclosure No.5) the west bank of the river is stable at its present 7 to 10 degree slope (see enclosure No.6 and Drawing No.1).

FEASIBILITY OF REPAIR AND REMEDIAL MEASURES

The feasibility and the practicability of repairing and under-pinning the existing structure must be answered from the point of view of technical possibilities as well as economy.

In deciding on the present or possible alternate alignment for the structure, it must be considered that geological conditions in the entire region are similar and more favourable soil conditions are not likely to be found in the vicinity. In other words similar foundation problems will be encountered and must be solved at any alternate location.

Another circumstance which will have to be considered is that the structure carries a relatively light traffic and is apparently quite adequate to handle the traffic volume for some time in the future.

Both above considerations speak for the repair, that is for an adequate reinforcement of the present bridge and the postponement of the construction of a new structure to a later date when traffic demands will require a bridge of larger load carrying capacity. Until then it seems to be sufficient to replace the pile support for the south west pier A' by a safe and unyielding support while the other three piles A, B and B' may be retained as they are.

There is now a technical question how the settling Algoma pile A' could be replaced and the south west pier underpinned.

Spread footing foundations would have to be supported on a relatively big contact surface leading to the propagation of stresses over a large area and transmitting them to a considerable depth. This in turn considering the great thickness of the compressible soft clay would result in large settlements over a considerable time.

Therefore, some type of deep foundation, piles or caissons, will have to be considered. The pile or piles to be used to underpin the south west pier will have to carry a total load of 65 tons with a safety factor of at least 2. As a suitable dense bearing stratum is not to be expected within a depth of 145 ft. the bearing capacity of the piles will have to be derived from skin friction. The type of pile to be used should secure the greatest possible mantle resistance on one hand and also produce the least disturbance in the subsoils surrounding the existing piles. The first requirement excludes steel piles or steel sheet piles, whereas the second requirement excludes driven piles. Consequently bored cast in place concrete piles appear to be the most suitable. From these, preference should be given to the so-called precast piles

which are described in the following paragraphs.

For one type of prepack pile the borehole is established to the required depth with the protection of thin-walled steel casing, and the coarse fractions of the aggregates are poured into the borehole in steps of 4 to 6 ft. height surrounding a previously placed 2 to 3 inch diameter central grouting tube. Through this grouting tube a mixture of fine aggregates and cement is grouted under pressure into the borehole to fill the void between the coarse aggregate. The casing is pulled back in steps corresponding to the progress of grouting. It is important that the bottom of the casing is always 6 to 8 feet below the grouted section in order to prevent the inclusion of the surrounding plastic clay soil material in the voids of the coarse aggregates before these are filled with the grout. On the other hand, the time when the casing may be pulled back should be adjusted to the setting and hardening time of the grout.

In order to minimize the unfavourable negative friction due to the unavoidable remoulding of the surrounding clay, the construction of the prepack piles should preferably be phased out in a way that after the completion of the borehole the steel casing is left in place for about 7 to 10 days before the borehole is filled with the coarse aggregate and the grouting and the withdrawal of the casing is started. This will allow some of the consolidation to take place before the pile is loaded.

There is an alternative method of pile construction which would also be suitable. Patented under the name of "Intrusion Prepack piles" a 12 inch diameter continuous flight auger with hollow central stem is drilled down to the depth where the tip of the pile is to be established and by forcing grout or concrete under pressure through the hollow stem the auger flight together with the cuttings are forced out from the ground and the hole is filled with concrete. Apart from the construction methods both piles are similar and are expected to transfer the loads in an identical manner to the surrounding soil.

As the piles will be friction piles, their capacity will be determined by the length and the diameter of the pile. At first the required depth should be fixed.

According to the log of borehole No.1 and 2, the undrained shear strength of the subsoil, as determined by field vane tests and laboratory compression tests, will surpass the value of 1000 pounds per square foot o below elevation +30 ft. Considering that the surface level from where piles may be started is at about elevation +90 ft. the length of the piles will have to be at least 60 but preferably 70 ft. or more.

The ultimate carrying capacity of the pile is given by the following equation.

$$Q_u = L \times D \times \pi \times c_u \times \delta$$

in which expression:

- L = the embedded length of the pile
- D = diameter of pile
- $c_u$  = undrained shear strength of the pile
- $\delta$  = effective adhesion between the pile material and the surrounding soil

Considering that 'L' = 70 ft.;  $\delta$  = 0.8; and that  $c_u$  is represented by the shear strength profile shown on the log of borehole No.1, the ultimate bearing capacity of different diameter piles is as follows:

TABLE NO. I

Diameter of Pile	Ultimate Bearing Capacity	Safe Bearing Capacity (safety factor = 2)
12 inches	90 kips	45 kips
16 inches	120 kips	60 kips
20 inches	150 kips	75 kips

Thus to carry the estimated 130 kips (65 ton), load on the pier three 12 to 16 inch diameter piles or two 20 inch diameter piles will be required.

The best structural arrangement can be obtained when two piles are used, one on each side of the cap beam, which carries the steel superstructure and connects piles A and A'. Suggested arrangement of these piles and underpinning is shown on Fig. 2. The cut-off level for the piles would probably be at about elevation 96 feet to allow for the depth of the underpinning steel beam and the insertion of hydraulic jacks. Hydraulic jacks will make the adjustment of the deck level possible and could also be used for pre-loading the new piles. For the purpose of pre-loading and testing of the piles, a temporary surcharge load could be placed over the respective surface of the bridge deck. The connection between the existing Algoma pile head and the cap beam shall be carefully observed during the jacking. To avoid the over-stressing of the cap beam, once the load is transferred to the new piles, the Algoma pile shall be cut off. After the realignment of the bridge, the hydraulic

jacks will have to be replaced by cast in-situ reinforced concrete blocks, during which operation temporary support can be offered by the Algoma pile again.

The design of the cap beam shall be checked and if necessary the beam shall be reinforced to resist stresses produced by the jacking operations and the changed location of the final support.

Because of the exposed location of the south west pier, it is recommended that the new pile group be protected against erosion and drifting ice.

When construction work is finished, the performance of the structure shall be checked at regular intervals by levelling in order to plot a consolidation curve and to check the safety of the structure.

DOMINION SOIL INVESTIGATION LIMITED

*I. P. Lieszkowszky*  
I.P. Lieszkowszky, P.Eng.,  
Chief Engineer.

IPL/jm





Enclosures.

## LOG OF BOREHOLE ..... 1.....

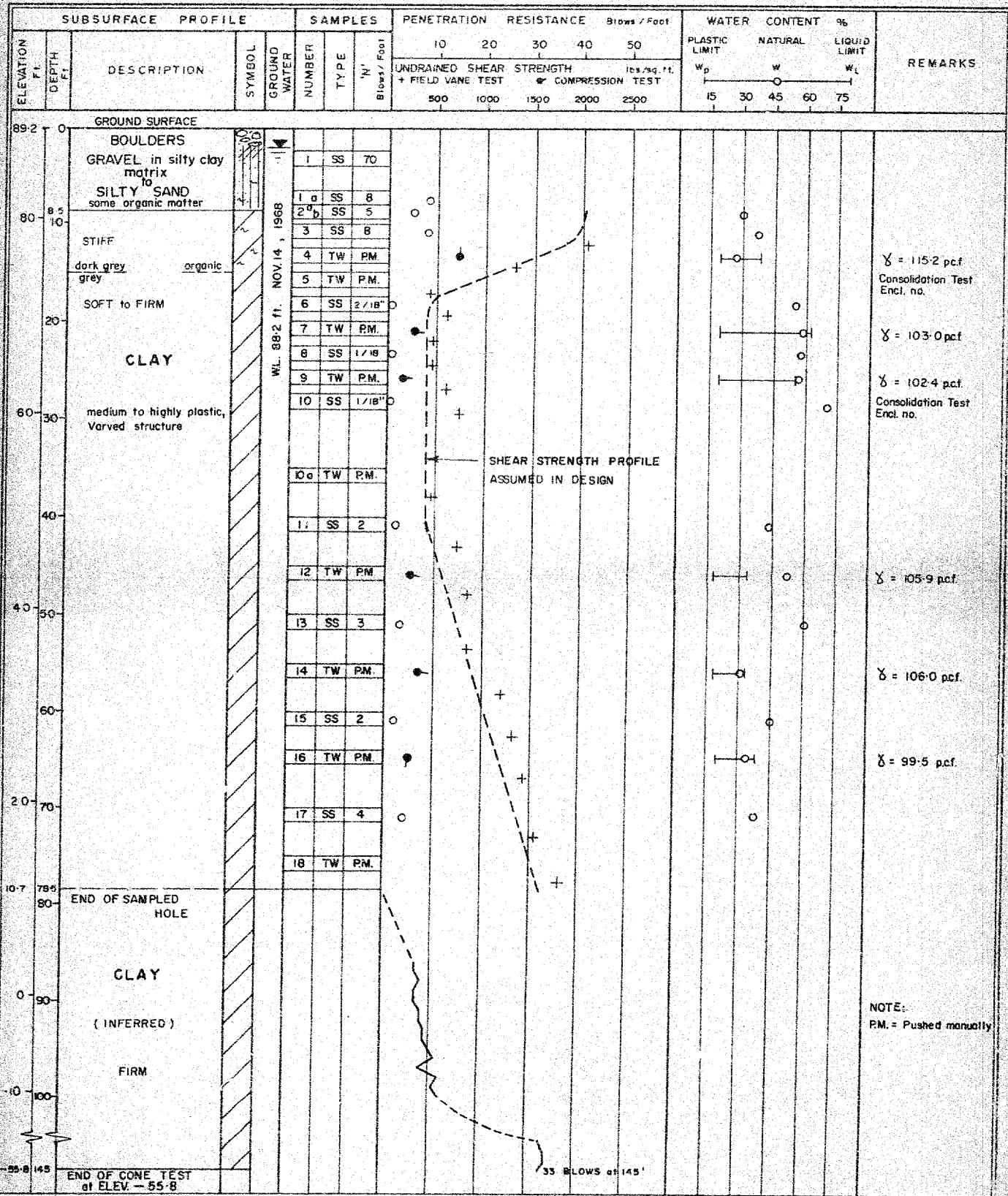
Our Reference No. 8-11-4

Enclosure № 1

CLIENT: THE SUTCLIFFE COMPANY  
PROJECT: UNO PARK BRIDGE OVER WABI RIVER  
LOCATION: TWP. OF HARLEY DYMOND  
DATUM ELEVATION: S.E. ABUTMENT 105.33'

### DRILLING DATA

Method: WASHBORING  
Diameter: 3" (BX)  
Date: NOV. 11 - 14, 1968



VERTICAL SCALE: 1 inch to 10 feet

DOMINION SOIL INVESTIGATION LIMITED

MADE: C.K. CHECKED:

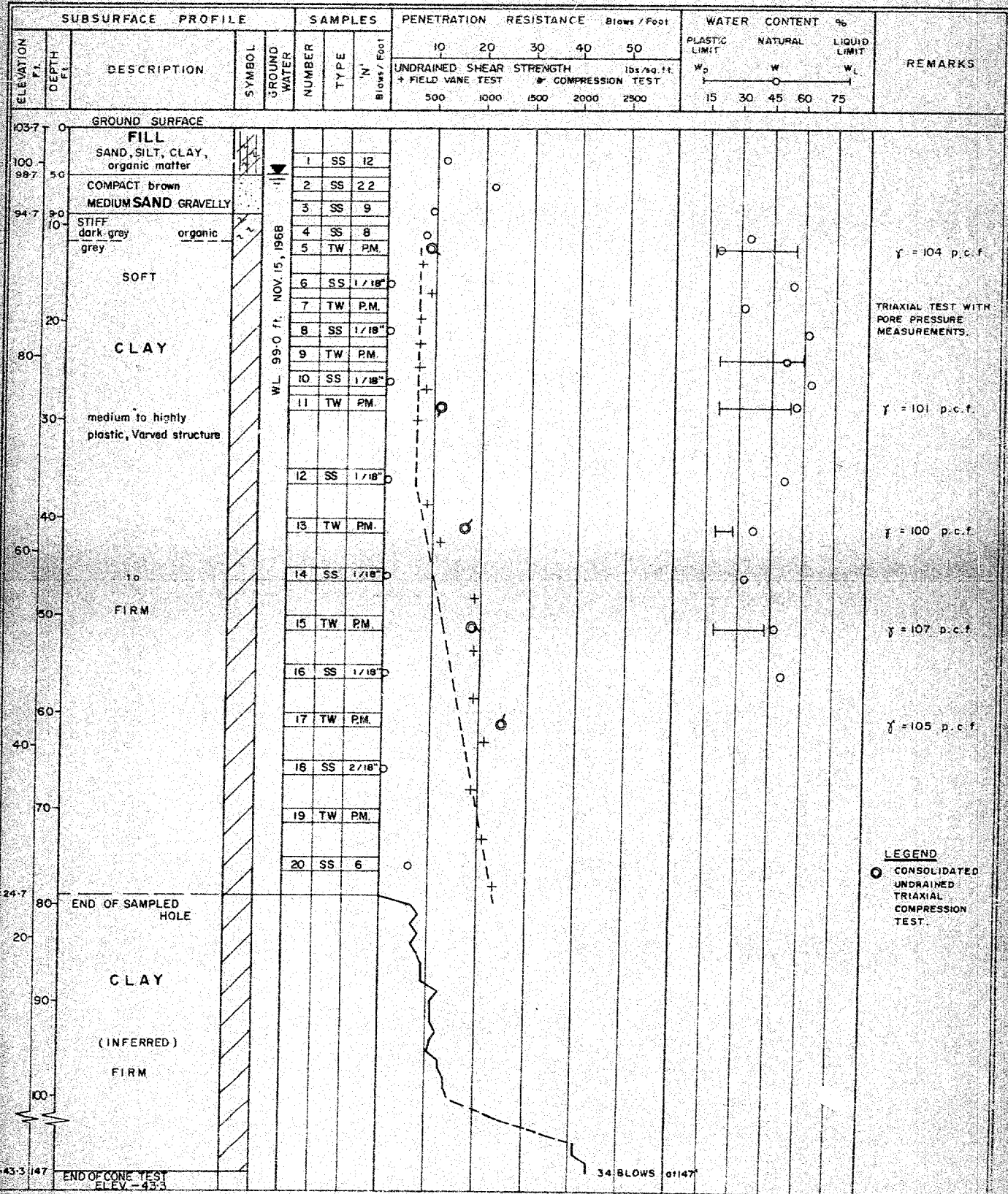
# LOG OF BOREHOLE 2

Our Reference No. B-11-4

Enclosure No. 2

CLIENT: THE SUTCLIFFE COMPANY  
PROJECT: UNO PARK BRIDGE OVER WABI RIVER  
LOCATION: TWP. OF HARLEY DYMOND  
DATUM ELEVATION: S.E. ABUTMENT 105.33'

DRILLING DATA  
Method: WASHBORING  
Diameter: 3" (BX)  
Date: NOV. 14, 1968



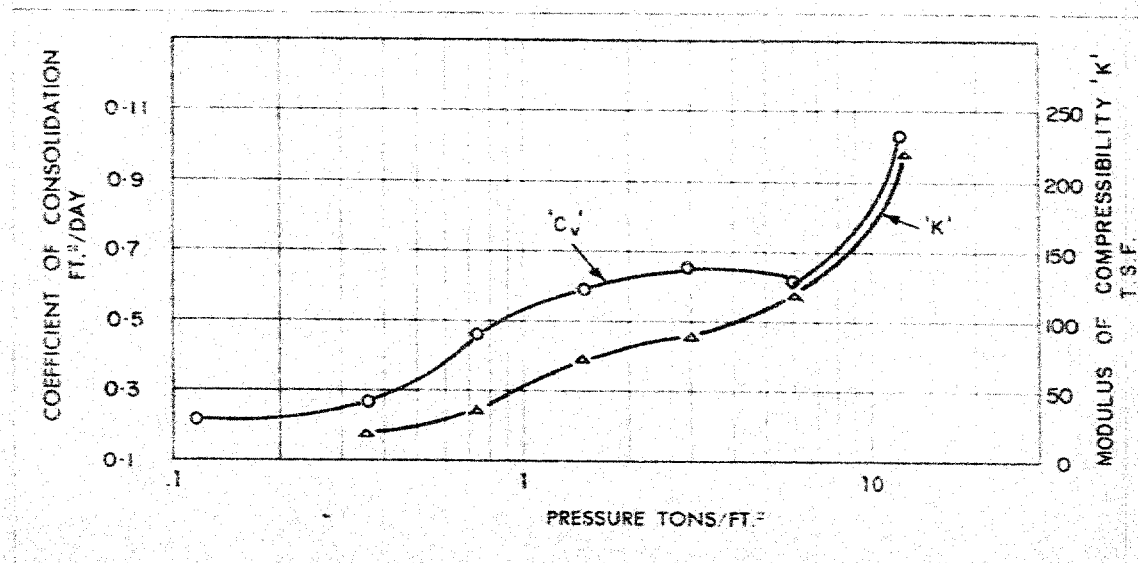
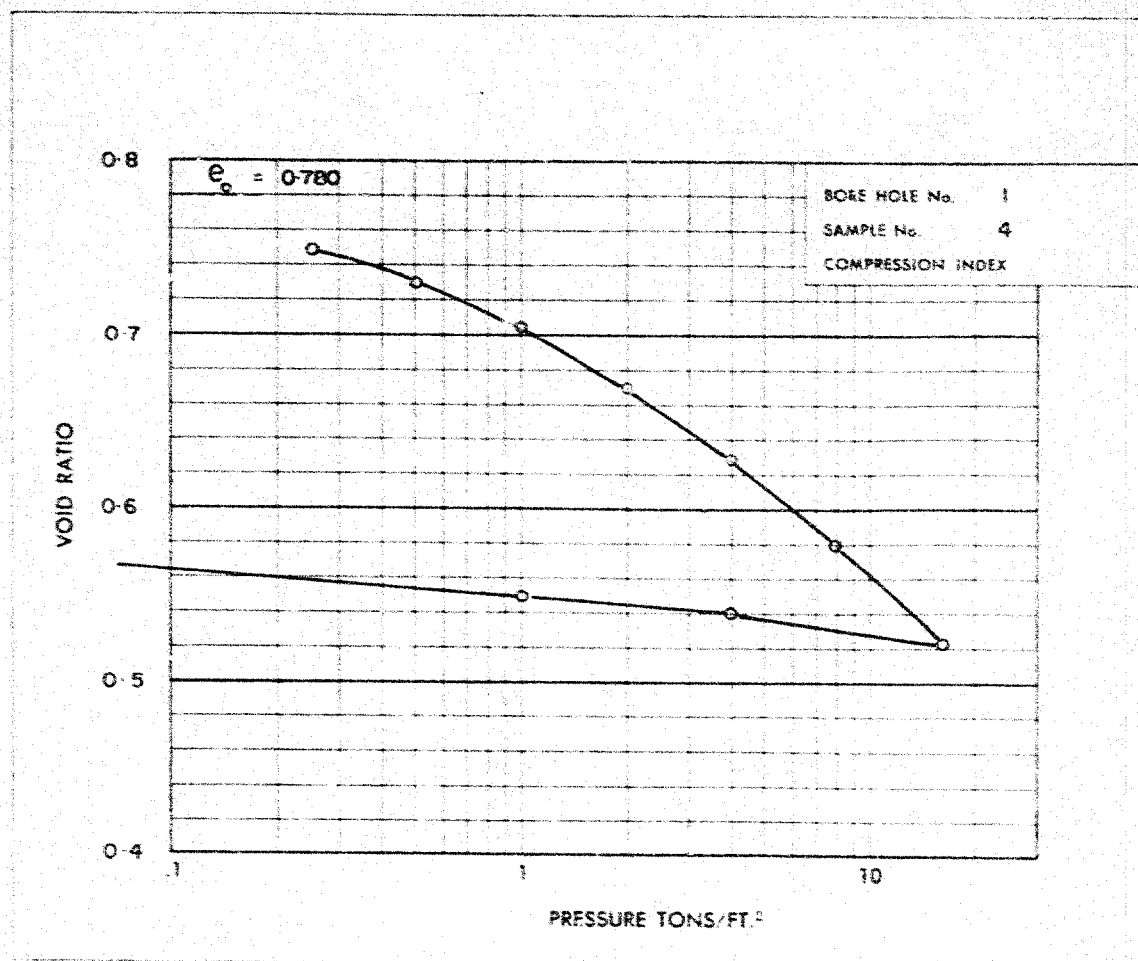
VERTICAL SCALE: 1 inch to 10 feet

DOMINION SOIL INVESTIGATION LIMITED

MADE: C.K. CHECKED:

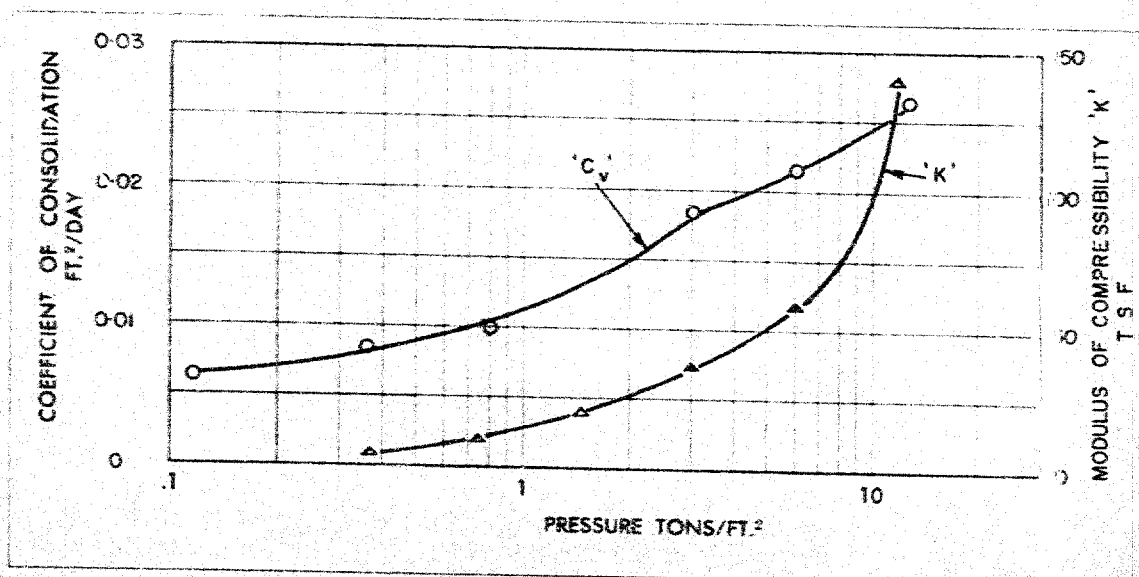
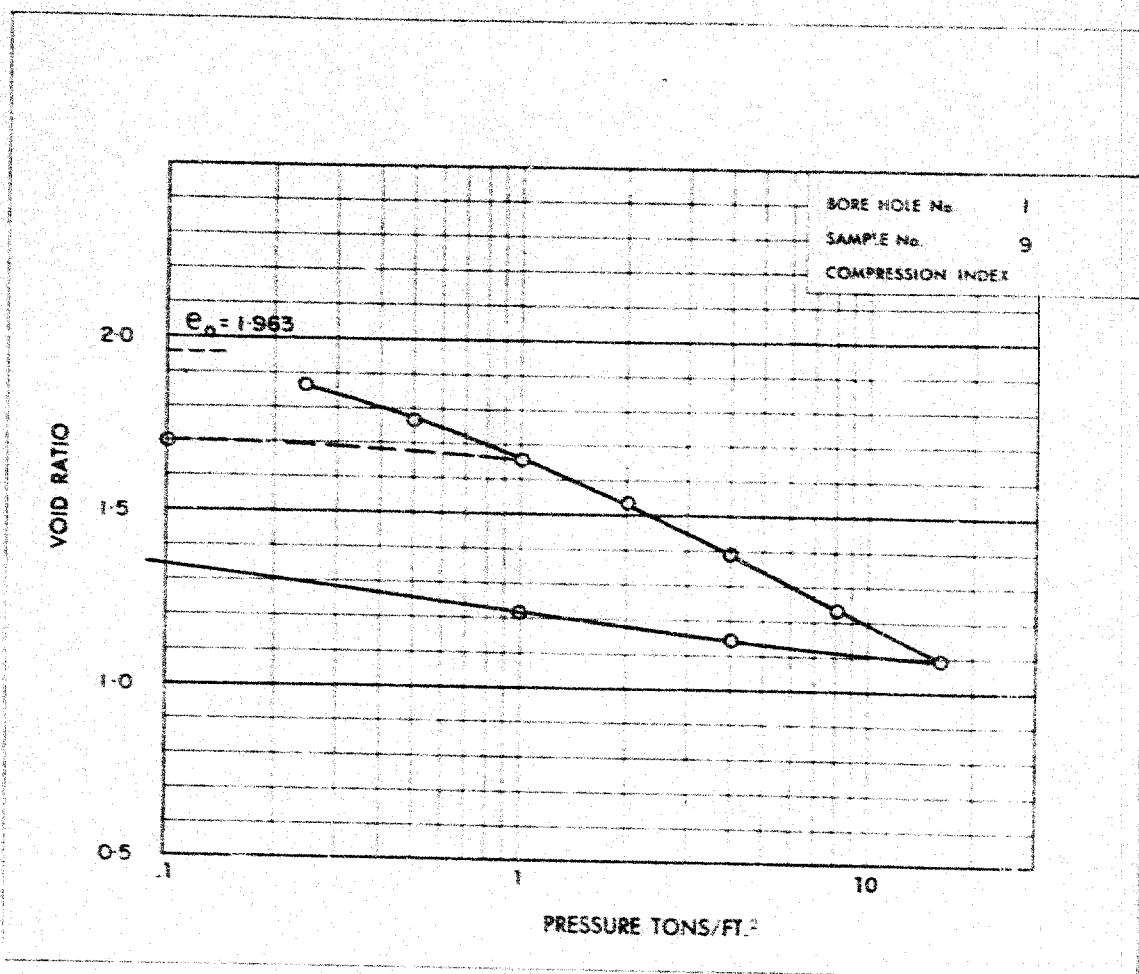
# DOMINION SOIL INVESTIGATION LIMITED

## CONSOLIDATION TEST



# DOMINION SOIL INVESTIGATION LIMITED

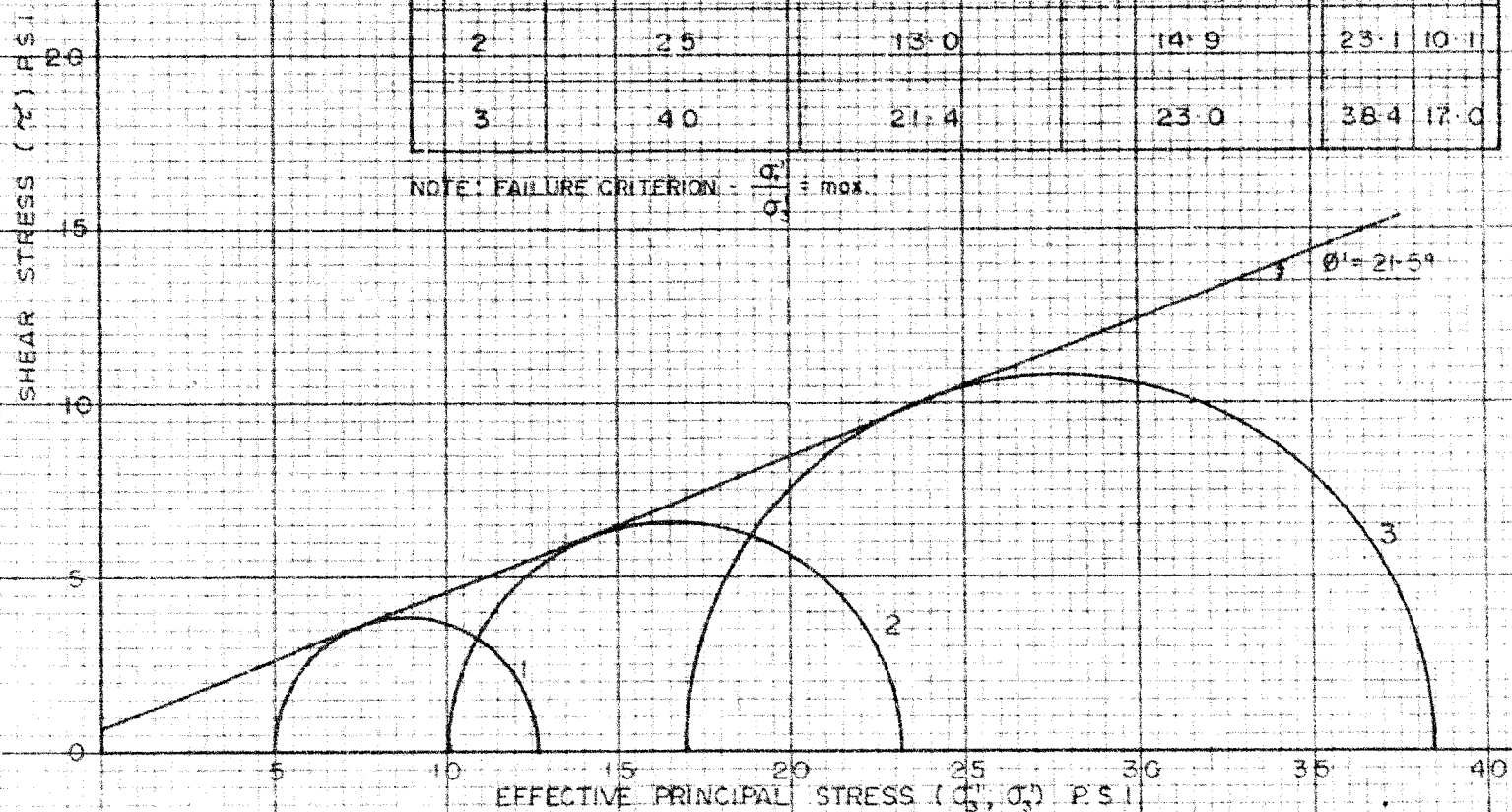
## CONSOLIDATION TEST



# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST WITH PORE PRESSURE MEASUREMENTS

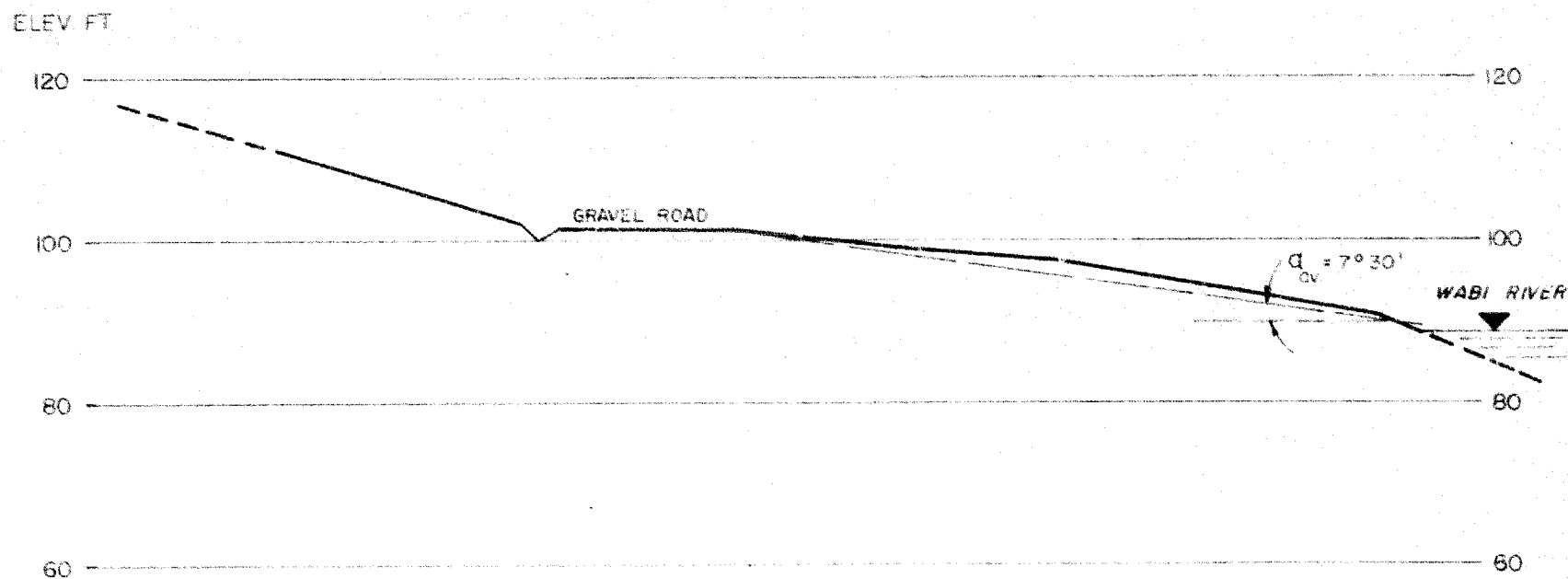
TEST NO.	CONSOLIDATION AND CELL PRESSURE P.S.I.	DEVIATOR STRESS AT FAILURE ( $\sigma_1 - \sigma_3$ ) P.S.I.	PORE PRESSURE AT FAILURE P.S.I.	$\sigma_1'$ P.S.I.	$\sigma_3'$ P.S.I.
1	10	7.7	5.0	12.7	5.0
2	25	13.0	14.9	23.1	10.1
3	40	21.4	23.0	38.4	17.0

NOTE: FAILURE CRITERION -  $\frac{\sigma_1'}{\sigma_3'} = \max.$



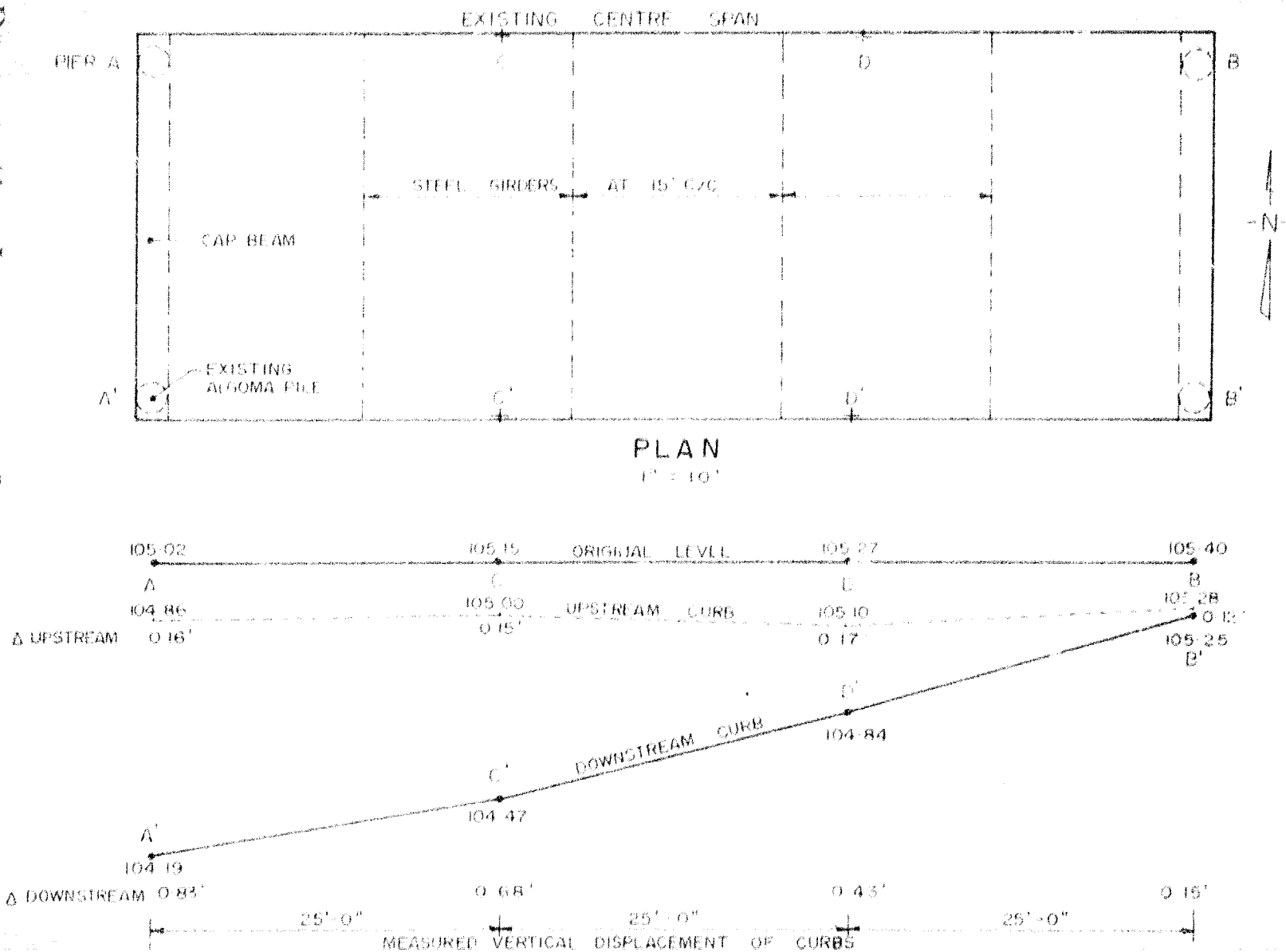
VARVED CLAY B.H.2 T.W.9

## TYPICAL CROSS-SECTION OF WEST BANK



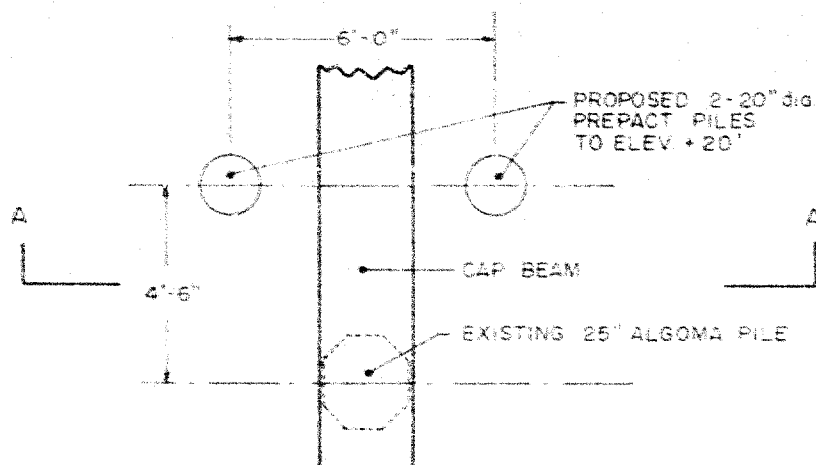
SECTION 'A'-A'

SCALE: vert. 1" = 20'  
hor. 1" = 20'

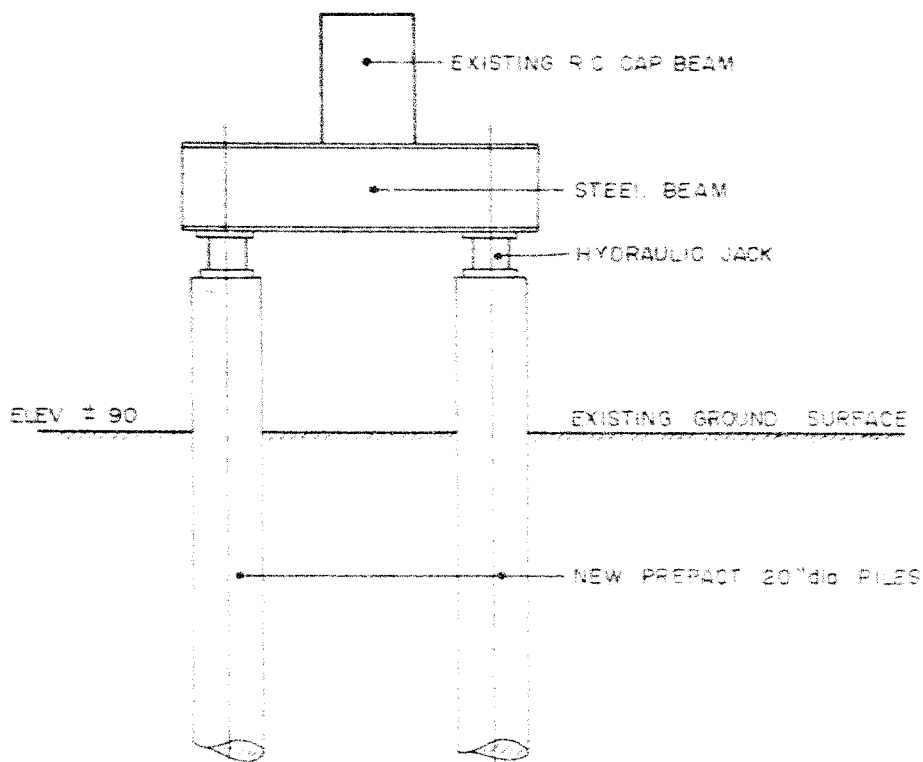




Prep. By S.O.



PLAN



A-A

SUGGESTED METHOD OF UNDERPINNING

SCALE  $\frac{1}{4}" = 1'-0"$ 

FIGURE 2

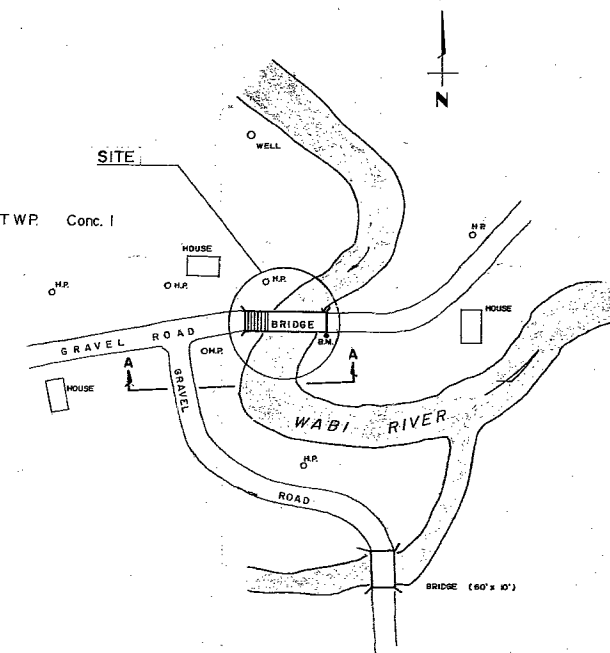
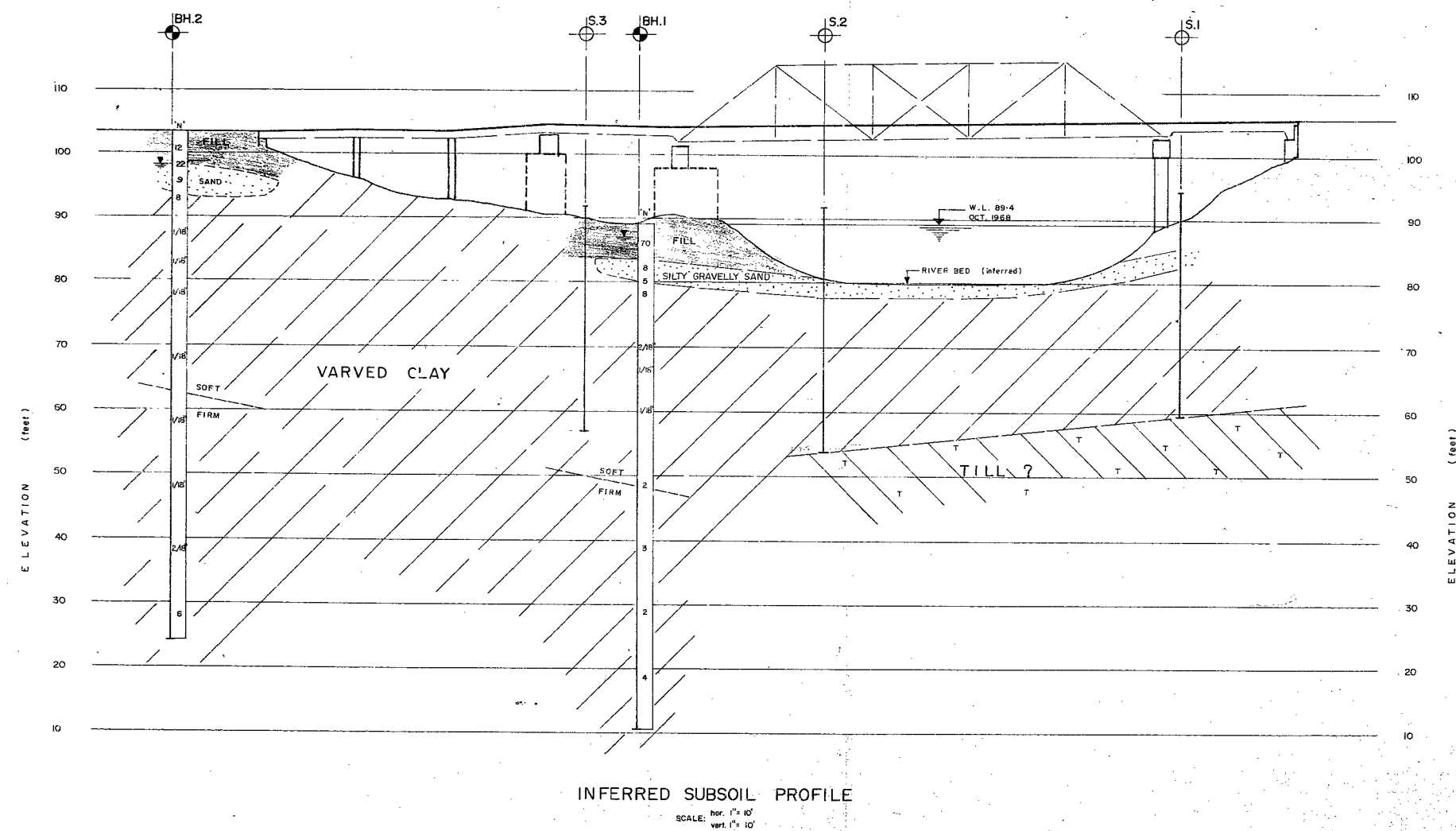
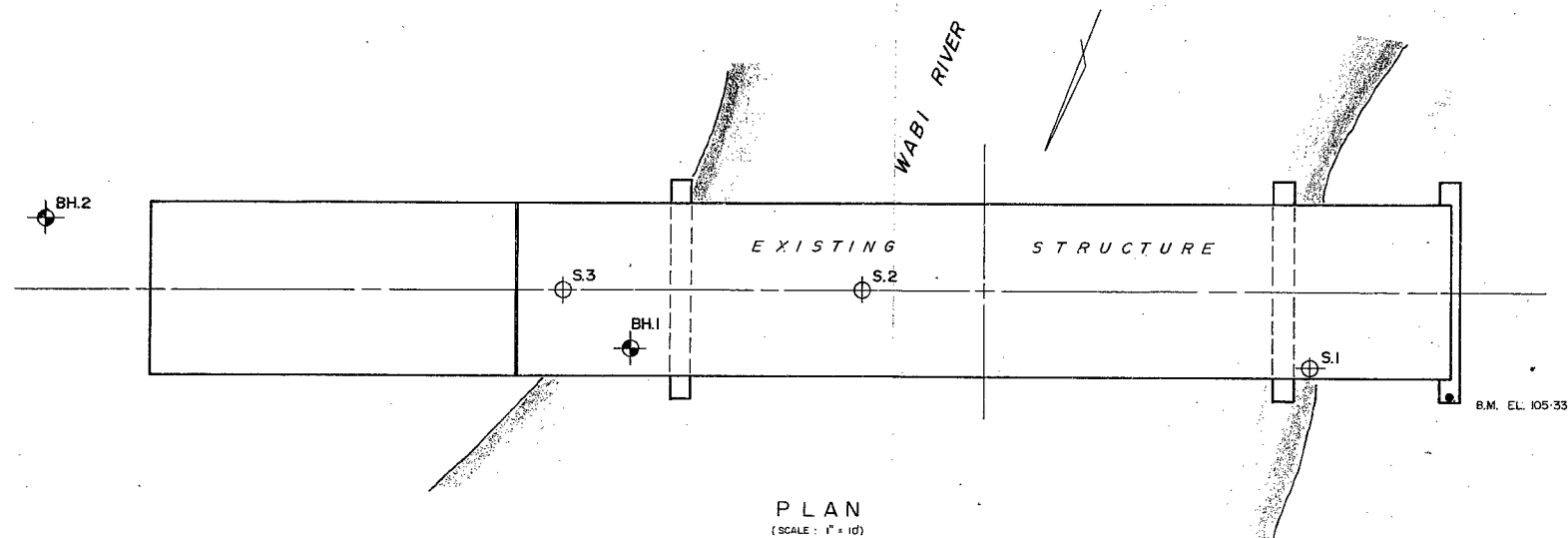
#69-F-237M

SITE 47-211

UNO PARK BRIDGE

RECONSTRUCTION.

WABI RIVER.



- LEGEND
- BOREHOLE & CONE TEST (by D.S.I.)
  - BOREHOLE BY D.H.O.

THE SUTCLIFFE CO.				
DOMINION SOIL INVESTIGATION LTD. CONSULTING ENGINEERS, TORONTO				
WABI RIVER BRIDGE AT UNO PARK TWP. OF HARLEY CONC. I				
BOREHOLE LOCATION & SUBSOIL PROFILE				
SCALE AS SHOWN	DRAWN BY C. K.	CHECKED	OUR REF. 8-11-4	DWG. NO. 1