

72-F-86

W.O.

40-70-01

W.P.

HWY. 11 & ENGLEHART RIVER

LOCATION

31X-36

GEOCRES NO.

• DATA ON FILE IN SOIL MECHANICS SECTION

REFER TO:

CONTRACT 74-105

REMARKS

**GEOCRES**

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GI-20 AUG. 74

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## MEMORANDUM

To: Mr. R. H. C. Kohn,  
 Bridge Maintenance Section,  
 Maintenance Office,  
 Lab. Bldg.

From: Mr. J. A. G. Smith,  
 Materials & Testing Office,  
 Room 107, Lab. Bldg.

ATTENTION: Mr. R. H. C. Kohn

DATE: September 17, 1969

OUR FILE REF.

IN REPLY TO

## SUBJECT:

Blanche River Bridge at Sargents  
 Highway 511 -- Site Index 7-03  
 District 516 (New Liskeard)

W.P. 40-70

As requested by you, we have inspected the above mentioned site with the object of determining whether or not the condition of the timber piles in the foundations could be assessed. It is understood that the piles were not 'treated' initially to prevent decay. If it could be established with certainty that the groundwater level has always been higher than the tops of the piles, then it would be reasonable to assume that the piles have not decayed. Unfortunately, there is no way to obtain this information as fluctuations could well have occurred over the last 37 years since the bridge was built, even though the present groundwater level appears to be close to the ground surface on the lower two-thirds of the slopes. It is possible that inspection pits could be excavated adjacent to the footings and the pile conditions ascertained by visual observation. A number of these pits would, however, have to be dug in order to obtain the average condition and this would probably prove to be expensive.

Of equal importance to the 'decay' aspect, would be structural damage to the piles due to lateral earth movements. This type of damage could be at levels well below the footings, and it is almost impossible to ascertain whether such damage has actually occurred, although there are definite signs that some earth movements have taken place.

To sum up, therefore, it is our opinion that it would be extremely difficult - if not actually impossible - to assess the condition of the piles in the foundations by direct inspection. The only alternative which seems practical, is to thoroughly inspect the various structure members to determine whether signs of distress exist which could have been caused by foundation movements. If no such signs exist, then it would be reasonable to assume that the foundations are performing satisfactorily. The present geometry of the bridge may not be too much of a guide in this, since no plans of the "as constructed" condition are available; however, signs of distress in the form of cracked concrete, sheared-off bolts, buckled girders and closed-up expansion joints, could all be indicative of

Mr. W. D. Wilson,  
Bridge Maintenance Engineer,  
Maintenance Office, Lab. Bldg.  
Attention: Mr. R. W. T. Rahn

October 19, 1967

Re: Blanche River Bridge at Englehart - Exp. #11  
Site Index 47-32 -- District #14 (New Diskeard)

foundation movements. Other causes, however, should be eliminated  
before concluding that the foundations are at fault.

RGS/KdeP

*L. Selby*  
A. J. Selby  
SUPERVISING FOUNDATION ENGR.  
For:  
A. G. Boermeester  
PRINCIPAL FOUNDATION ENGR.

cc: Messrs. G. J. Orr  
B. R. Davis  
D. A. G. White

Foundations Files ✓  
Gen. Files

Mr. W. D. Birch,  
Bridge Maintenance Engineer.

G. P. Wilson

E. Van Beilen

August 17, 1970.

W 40-70

Blanche River Bridge, Englehart  
Highway 11, Site Index 47-32, District 14, New Liskeard

I refer to your telephone conversation with W.R. Bennett today, and to your letter of the 6th of January with regard to the above named structure which was built in 1932 and is 810 feet long and 30 feet wide.

The report of May 8th which you refer to was included in a Province-wide survey searching for signs of alkali-silica reactivity in the concrete of Department structures and certainly this structure had evidence of considerable alkali-silica deterioration in the piers, wing walls, abutments and extensively in the deck and side-walks.

The comprehensive investigation of the structure was carried out by the Bridge Deck Investigation crew, and submitted to you in August, 1967, but in the light of more recent information, particularly the report of the Foundation section, dated September 19, 1969, I would revise our report as follows:

First, I would recommend that this structure at Englehart should be one of the first to be investigated by the Bridge Research Engineers' proposed investigational equipment as soon as it becomes operational, hopefully this year. Over the years, since 1932, the steel bridge truss has become twisted, bolts have sheared off and the structure has had to be jacked up and wedged in place with metal shims. The integrity of the timber piles is in question and also the possible pushing of the piers by earth movement is of more importance at this time than any consideration of alkali-silica reactivity. Extensive epoxy repairs to the deck of the structure were carried out by K. Langhammer in 1961, the record of which is set out in Report No. 26, dated April 1962.

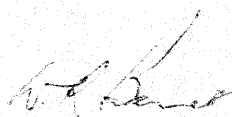
With regard to the last paragraph of your letter about the practical methods of repairing reactivity; there are many things we can do but I would like to see how effective my proposals, made in my letter dated April 17, 1969 which you are going to carry out at Wanapitae, are, over the next couple of years, before we attempt reactivity repairs at Englehart.

To sum up, before we spend \$500,000 on building a new structure



at Englehart, or half that much on a new deck and casing the piers in 12" of new concrete, we should check the integrity of the whole structure by this new Research equipment, finding out if the dynamic modulus of the existing structure is within reason, and also if the piles move when the structure is heavily loaded. Once that decision is made, and experience is gained at Wanapitae, then we can make further knowledgeable suggestions for repairing reactivity with some guarantee of success!

I enclose a copy of Mr. Woda's petrographic examination report on two pieces of concrete taken from the structure in January 1970. You will note that alkali-silica reactivity is considered to be of minor significance.



G. P. Wilson,  
Sr. Materials Engineer (Concrete).

GPW:nr  
Enc.

c.c. P. Gsagoly  
A.C. Stermac ✓

MEMORANDUM

To: Mr. D. A. Barr,  
Program Engineer,  
West Bldg., Downsview.

FROM: Structural Planning,  
North Bay.

74-105

ATTENTION:

DATE: April 7, 1972.

OUR FILE REF.

IN REPLY TO

SUBJECT:

W.P. 40-70 - Site 47-32  
Englehart River (Blanche River  
West Branch), Hwy. #11,  
District #14, New Liskeard.

The feasibility study to consider replacement rather than repair of the above structure has now been completed. I am attaching a copy of my letter to Mr. T. G. Smith, dated 31st January 1972, which details the existing conditions and compares the cost of replacement of the bridge, at the present location, with the cost of repairs.

Since that report, costs have been revised in the light of further information and cost estimates obtained for a new structure both upstream and downstream of the existing bridge.

Following is a summary of the various possibilities presented at our meeting on 30th March 1972 and the recommendations arising from that meeting.

a) Maintaining the existing structure.

Since the early 1960's, repairs to the existing bridge have cost in the neighborhood of \$150,000.00.

The structure has deteriorated considerably since repairs were first costed in 1969. The original estimate of \$230,000.00 could, when deterioration and rising construction costs are taken into consideration, well be in the area of \$325,000.00.

Even if repaired, we are still left with a sub-standard cross-section at a location where there is a high percentage of truck traffic and a fairly high accident rate.

It is most unlikely that the life of the structure can be extended beyond a further ten years, even with extensive repair.

More.....

b) Replacement

Two preliminary plans were prepared and costed. No great difference could be determined until foundations, etc., are completed. The same structure costs have been applied to alignments immediately upstream and immediately downstream of the existing crossing. These two alignments use the existing bridge to maintain traffic thus eliminating the cost of a bailey detour.

Upstream\_(south)\_alignment\_

Structure	700,000.00	) = 795,000.00
Grading	68,000.00	
Property	27,000.00	

Existing\_alignment\_

Structure	700,000.00	) = 910,000.00
Bailey	210,000.00	

Downstream\_(north)\_alignment\_

Structure	700,000.00	) = 786,000.00
Grading	66,000.00	
Property	20,000.00	

In view of the low cost benefit in repairing the structure compared with replacement, I have recommended the latter. In view of the extremely poor condition it is recommended that it be programmed for 1974 reconstruction. The Structural Maintenance Section will maintain the structure on this basis, with particular attention to the concrete pedestals. When the pre-engineering schedule is available, consideration should be given to advancing the project to 1973.

It should be noted that although agreement to reconstruct has been reached, no decision has been made on the alignment. There are other factors involved which will be discussed in the Region very shortly. When a decision has been made, I will let you know so that the program value can be revised if necessary.

*J. C. McAllister*  
J. C. McAllister,  
Regional Structural  
Planning Supervisor.

JCMcA/les

c.c. S. McCombie  
A. Radkowski  
W. D. Birch  
A. Stermac ✓  
T. G. Smith  
R. Murphy

Mr. T. G. Smith,  
Reg. Functional Planning Eng.,  
North Bay.

Bridge Planning,  
North Bay.

31 January 1972

Re: W. P. 40-70      Site 47-32  
Englehart Ri. Bridge @ Englehart  
Hwy. # 11      District # 14

The existing structure at the above site consists of a series of steel deck trusses supported on steel towers and concrete pedestals. Two steel beam approach spans at either end are supported on concrete piers and abutments. The structure, which was built in 1932, has a total length of 800' and a roadway width of 24' between a 5'-6" sidewalk and a 6" curb.

Some years ago there was considerable deterioration in the deck adjacent to the sliding plate expansion joints. The joints were replaced and the deck repaired and reasphalted. Since then the concrete pedestals under the towers show severe cracking. In May 1969, Materials and Testing Section carried out an inspection to assess the degree of alkali-silica reaction in the concrete. A very extensive reaction was found to be present in the abutments, wing walls, piers, deck and sidewalk.

The need for extensive repairs was evident and a preliminary cost estimate for replacement of the deck and sidewalk, using precast prestressed concrete planks, repairs to piers and abutments, was of the order \$230,000. The type of structure is such that there will be a continuing need for maintenance of the trusses, bearings, etc. even after replacement of the deck and other recommended repairs. The life of the structure would be extended by 10 years+ and existing substandard section maintained. With the high cost of repair, it was decided to investigate the feasibility of replacement.

When considering complete replacement of the structure, there are two basic alignments to be considered: -

- a) replacement on an alignment immediately upstream or downstream of the existing bridge, where the existing facility can be used to maintain traffic during reconstruction.

With this scheme, reconstruction of the approach curves to the north and south will be required and will cause considerable property damage to the small commercial development on either side of the bridge at the north end.

January 1972

b) replacement on the existing alignment, in which case it will be necessary to provide a detour to maintain traffic during reconstruction.

There are no existing structures that could be used to detour traffic. Therefore, the use of temporary bailey detour at three possible locations was investigated.

Detour #1 via Hwy. #624 turning north at the Martyr Twp. Line and crossing the river 1.3 miles downstream from Hwy. #11. The total length of detour is 5.6 miles of which approximately 40% is paved highway, the remainder is township road with exit to Hwy. #11 via the local street system in Englehart.

This detour is not satisfactory because of its length and because of the north access being through a local street system.

Detour #2 via Township road coming off the south approach curve of Hwy. #11, to the existing bridge, turning north and crossing the river 0.4 miles downstream of Hwy. #11. The total length of detour is 1.3 miles, all unpaved.

Although this detour is considerably shorter than #1, and by cutting at the approaches to the detour crossing, a shorter and lower bailey might be used. The north approach is, again, through the town of Englehart with a 90° turn required to approach the bailey. The skew of the river at this crossing is a further drawback.

This detour alignment should not be completely abandoned. An old timber structure was originally located on this line, however, approach grades are very poor. A profile of this alignment will be requested and the site also investigated by Materials and Testing when the foundation report for the structure site is undertaken. When the maximum cut and fill is known, a bailey at this site can be costed.

Detour #3 is a high level, single lane, standard wide bailey with one sidewalk, controlled by traffic lights and located parallel to and on either side of the existing structure. The cost of this bailey, approximately 800' long and supported on bailey piers on timber piles, is estimated at \$210,000.00.

This solution does offer the best horizontal and vertical alignment for the detour on what is the main route to Northern Ontario. Although temporary easements might be required through the commercial section at the north approach, it avoids routing highway traffic through the local street system in Englehart.

Proposed Replacement Structure.

One of the problems inherent in this area are the poor foundation conditions which limit the height of approach fills and/or cut. Estimates for the foundation work have been based on previous structures recently built in this area. A flat grade at the same elevation as the existing structure is recommended.

A preliminary design has been completed and costed. It consists of seven spans, 90'/120'/120'/150'/120'/120'/90', totaling 810'. The concrete piers are supported on either timber piles or H-piles driven to bedrock (if feasible). The superstructure is a plate girder design with a deck cross section, south 2', 3', 22', 3', 5' north. The cost of this design will be of the order \$670,000.

A second preliminary design with somewhat different span arrangement is being prepared. I do not believe, however, that there will be much variation in cost.

The cost of replacement on the existing alignment will be:

Structure	\$670,000.00
Bailey Detour	210,000.00
Total	\$880,000.00 + grading

The cost of construction on a revision alignment will exclude the Bailey cost but include extensive property and grading costs. Until these costs are available, this study cannot be finalized. However, on the basis of the costs now available, consideration should be given to maintaining the existing structure at minimum cost in order to use it for as much as possible of its normal life span. It is my estimate that it can be maintained for a further five years before maintenance costs become prohibitive and major repair can no longer be postponed.

Therefore, my tentative recommendation would be to start the pre-engineering for this struction, stock pile the design and program reconstruction for 1977.

When total costs for the project are available and alignment can be decided, I will arrange to discuss the project with Mr. W. Birch and the Program Section before making a final recommendation.

JCMcA/bn

J. C. McAllister,  
Regional Bridge  
Planning Supervisor.

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

MEMORANDUM

TO: Mr. A. G. Stermac,  
Principal Foundations Engineer,  
Foundations Office,  
Central Bldg.

FROM: Structural Services Section,  
West Building.

ATTENTION:

DATE: May 5, 1972.

OUR FILE REF.

IN REPLY TO

SUBJECT: Englehart River Bridge,  
Site 47-32,  
Highway 11, District 14.

As you are aware there have been a number of discussions recently concerning the repair vs. replacement of this structure.

At a recent Bridge Evaluation Committee meeting a question was raised concerning the stability of the banks of the valley at the bridge site.

Could you please let me have the opinion of your office on this matter by May 31?

SMcC/im

*S McCombie*

S. McCombie,  
Secretary,  
Bridge Evaluation Committee.

NOTE:

COPY OF LETTER DATED SEP. 19. 1969 TO  
W.D. BIRCH SENT TO MR. HOLCOMBE  
AS REPLY TO ABOVE MEMO.

MAY 8. 1972

*AGS.*



MEMORANDUM

72-11036  
74-105  
72-1086

TO: Mr. A. G. Stermac  
Principal Foundations Engineer  
West Bldg., Downsview

FROM: Structural Planning  
North Bay

ATTENTION:

DATE: 17th July 1972

OUR FILE REF.

IN REPLY TO

SUBJECT:

W.P. 40-70, Site 47-32,  
Englehart River Bridge  
(Blanche River West),  
Hwy. #11, District # 14,  
New Liskeard.

Attached are three prints of site plan E-5023-1, and a print of the general plan for the existing structure.

A number of alignments are available here:

- a) Reconstruct on the existing alignment with traffic maintained on a bailey detour offset from the line.
- b) Maintain traffic via the existing structure and reconstruct on a line immediately upstream.
- c) Maintain traffic via the existing structure and reconstruct on a line immediately downstream.

The decision on which alignment to use has not yet been made. Scheme a) is, however, considerably more expensive and would, with detour construction, demolition and reconstruction, etc., disturb the sensitive clay subsoil.

It is most unlikely that there will be any difference in the sub-soil over the three alignments; however, your comments regarding choice of alignment would be welcome.

Should you require any further information, please do not hesitate to call.

*J. C. McAllister/les*  
J. C. McALLISTER  
REG. STRUCTURAL PLANNING SUPVR.

JCMca/les  
Att.

cc. Mr. C. S. Grebski



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

31M-36

GEOCRE No.

TO: Mr. J. McAllister, (2)  
Regional Structural Planning Supervisor,  
Northern Region,  
North Bay, Ontario.

FROM: Foundations Office,  
Design Services Branch,  
West Bldg., Downsview.

ATTENTION: DATE: November 8, 1972.

OUR FILE REF. IN REPLY TO NOV 13 1972

SUBJECT:

FOUNDATION INVESTIGATION REPORT  
For  
The Proposed Englehart River Bridge  
(Blanche River West) of Hwy. #11  
District #14, New Liskeard  
W.O. 72-11086 -- W.P. 40-70-01 ✓  
*cont 74-105*

Attached we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above-mentioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

*A. G. Stermac*

A. G. Stermac,  
PRINCIPAL FOUNDATIONS ENGINEER.

AGS/ao  
Attach.

cc: E. J. Orr  
B. R. Davis  
A. Rutka  
H. McArthur  
T. A. Sharpe  
B. J. Giroux  
J. E. Graspier  
G. A. Wrong  
B. A. Singh

Foundations Files ✓  
Documents

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FOUNDATION INVESTIGATION REPORT  
For  
The Proposed Englehart River Bridge  
(Blanche River West) of Hwy. #11  
District #14, New Liskeard  
W.O. 72-11086    --    W.P. 40-70-01

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1. INTRODUCTION:

A foundation investigation was requested at the site of the Englehart River bridge on Hwy. #11 by Mr. J. C. McAllister, Regional Structural Planning Supervisor, in a memo dated July 17, 1972.

Three alignments are being considered for the proposed new structure: (a) reconstruction on the existing alignment, (b) reconstruction on a new alignment immediately upstream, and (c) reconstruction on a new alignment immediately downstream. This office was requested to comment on the above alignments from the point of view of foundation considerations.

Subsequently, a field investigation was undertaken, the results of which are presented in this report, together with recommendations concerning foundations.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is situated at the easterly entrance of Englehart along Hwy. 11. The Englehart River valley at this location is approximately 800-900 ft. wide and some 100-110 ft. deep. The valley is bush and tree covered. The overall slopes of the banks are about 4 horizontal to 1 vertical; on shorter distances, however, 2.5 to 1 and 3 to 1 slopes were also measured.

There is ample evidence, along the slopes, of past movements which show local lunar shape failures and bulges so typical of this particular region.

Geologically the site lies within the region known as the Little Clay Belt. The characteristic soil type forming the overburden in this region is varved and laminated clay. It is postulated that these banded deposits were originated in lake basins in which there were cyclic changes in temperature and sedimentation environment. The varved clays within the Little Clay Belt area are believed to be deposited by glacial lake Barlow-Objibway.

### 3. FIELD AND LABORATORY INVESTIGATIONS:

Some seven sampled boreholes were carried out during the field work, using a hollow stem continuous flight auger and a conventional diamond drill rig, the latter being adapted for soil sampling purposes. Samples were taken at frequent intervals, mainly by means of 2 inch diameter thin-walled Shelby tubes, which were pushed manually into the relatively undisturbed soils. Occasionally split-spoon samples were taken and Standard Penetration Tests performed by a 140 lbs. hammer falling a distance of 30 inches. The number of hammer blows required to advance the sampler 1 ft. into the soil was recorded as a penetration 'N' value.

Undrained shear strength of the overburden was measured by field vanes, usually 18 inches below soil samples. Remoulded strengths were calculated by turning the vanes 6 full revolutions and again measuring the torque. In order to observe groundwater conditions, piezometers were installed near certain borings at various depths. Porous brass (Geonor) instruments were used, being attached to E and A size drilling rods. Measurements were taken by inserting an electric probe through a 1/4 inch I.D. Polyflow tube.

Laboratory testing of the samples consisted of natural moisture contents, Atterberg limits, grain size analyses, unconfined and quick triaxial compressions, and consolidation tests. Two

series of consolidated undrained triaxial tests were carried out with pore pressure measurements in order to compute shear strength parameters in terms of effective stresses.

All the field and laboratory test results are compiled on the attached borelog sheets, while the estimated soil stratigraphy is shown on Drawing #72-11086A in the Appendix.

#### 4. SUBSOIL CONDITIONS:

##### 4.1) General:

Two principal soil strata were encountered throughout the investigation. The uppermost deposit was identified to be sandy silt to silt with some clay, underlain by a thick deposit of varved clay. Bedrock was recorded between elevation 483 ft. and 456 ft., generally dipping from east to west.

A detailed description of the various deposits is as follows.

##### 4.2) Sandy Silt to Silt with Some Clay:

This deposit was noted in every borehole. The thickness of the layer was measured to be 88-98 ft. in the borings placed at general ground level and only some 8 ft. at the bottom of the valley. The bottom of the layer thus varies between elevation 579 ft. and 598 ft. sloping westerly. The light grey coloured material was identified to be predominantly silt, and as such was found to be a borderline case between cohesive and non-cohesive soils. At certain depths faint stratification was visible without noticeable difference in the soil fabric. Standard Penetration 'N' values range from 2 blows per ft. to 24 blows per ft. indicating very soft to very stiff consistencies and very loose to compact relative densities. The average natural moisture content of the samples was estimated to be 25%, with plastic limits of 16-21% and liquid limits of 21-29%. Penetration 'N' values as well as undrained shear strengths increased with depth, denoting a normally consolidated deposit. Mean undrained shear strengths above elevation 630 ft. may be taken to be between 500 p.s.f. and 1000 p.s.f.; below

elevation 630 ft. they range from 1,000 p.s.f. to 2,000 p.s.f. Field vanes resulted in the highest strength values as usual. The range of constituent particles was computed to be as follows: gravel - 0%, sand - 0-83%, silt - 17-85% and clay - 11-20%. The average bulk density of the silt is 122 p.c.f.

#### 4.3) Varved Clay:

Below the sandy silt and silt a deep deposit of varved clay was observed in the boreholes extending to bedrock, around elevation 456-480 ft. The full thickness of the overburden is 203-221 ft., measured from existing pavement level. The depth of overburden at the bottom of the valley is 130-139 ft. Within the upper portion the varved material has very thin and faint seams of silt, rather irregularly distributed. The middle portion is a well developed varved sediment with approximately 1 inch thick light silt (summer layer) and 1 inch dark clay (winter layer) bands. Within the lowest 30-40 ft. there is a gradual increase of the silt portion, forming bands and layers of several inches thick. At least in one location (B.H. #7) this lower zone was identified as a separate deposit consisting predominantly of silt. The stratification is usually horizontal, in B.H. #6, however, inclined lamination was recorded at and below elevation 500 ft. The latter was likely the results of some ancient failure. Physical properties were computed independently for the silt and clay seams. Silt seams were found to have 19-22% plastic limits and 24-29% liquid limits, with average moisture contents of 24%. The plastic limits of clays range from 21% to 30%, the liquid limits from 40% to 61%, natural moisture contents being near the liquid limits.

The range of grain size distribution within the silts were found to be 0% sand and gravel 84-85% silt and 15-16% clay. Within the clay seams the corresponding figures were 1-5% sand, 16-26% silt and 73-82% clay. The bulk density of the varved clays averages some 112 p.c.f.

Laboratory and field undrained shear strengths vary between 500 p.s.f. and over 2,000 p.s.f. For stability analyses of the slopes mean strength values were assigned in each borehole.

Several consolidation tests were carried out in the laboratory, using incremental loading, with increments of  $\frac{\Delta P}{P} = 1$ . Typical consolidation curves are presented on Fig. #1. All the tests indicated that the deposit is overconsolidated by some 0.5 - 2.5 t.s.f. above the existing effective overburden pressures.

#### 4.4) Bedrock:

Bedrock was proven by diamond drilling in B.H.'s #2, 4, 5 and 7. Rock cores were recovered by means of an AXT size core barrel. The rock surface in B.H. #2, near the east bank, was hit at elevation 478.4 ft. and in B.H. #7 on the west bank at elevation 456.5 ft. All the boreholes indicated that the rock surface slopes from east to west.

The rock cores were identified to be dolomitic limestones of the Lockport formation, having shaley intrusions and seams.

#### 5. GROUNDWATER CONDITIONS:

The groundwater level was measured by piezometers, installed at various depths beside B.H.'s #1, 2, 3, 6 and 7. The depth of piezometers and the observed equilibrium water levels are marked on the borelog sheets. Water levels generally lie some 3-17 ft. below ground levels. The highest hydrostatic gradient measured by the piezometers may be taken to be some 80 ft. of head within an average length of 400 ft. The hydraulic gradient induces a downward seepage towards the river.

#### 6. DISCUSSION AND RECOMMENDATIONS:

##### 6.1) General:

The existing bridge over the Englehart River is a nine span steel deck truss structure. The superstructure is supported on steel towers and concrete pedestals. The four approach spans are carried on concrete piers and abutments. According to the bridge design drawing, the foundation of the two centre piers consists of 50 ft. long timber piles, while the rest of the footings



are on concrete piles, varying in length between 35 ft. and 45 ft. The overall length of the bridge is 810 ft. The structure was erected in 1932.

Considerable deterioration of the structure was reported during recent years, necessitating frequent and expensive repair. There appears to be some indications that at least some of the occurred distresses might have been caused by ground or foundation movements. Such damages as snapped off bearing bolts, cracked concrete, closed up expansion joints and buckled girders might well be the results of differential settlements or slow creep of the slopes.

Three alignments are being considered for the proposed new structure. The bridge may be constructed along the existing alignment or immediately upstream or downstream from the present line.

The uppermost deposit at the site was identified to be silt, sandy silt and clayey silt of soft to very stiff consistency, underlain by a thick deposit of varved and laminated clay. Dolomitic limestone bedrock was encountered between elevation 456 ft. and 478 ft. slipping from east to west.

#### 6.2) Stability of the River Banks:

The banks of the Blanche River have a long history of instability with periodic slip failures and landslides, occasionally involving large masses of soil. In view of this a detailed study of the stability of the existing slopes was carried out.

Stability analyses of the overall slopes as well as the partial slopes were implemented for both banks of the three proposed lines. Analyses were carried out by use of an electronic computer. Computations were performed according to Bishop's method, assuming that failure occurs along a circular arc. Both the undrained and long term (steady seepage) stabilities were investigated, assigning stress parameters to the soil layers in terms of total and effective stresses respectively.

The assumed undrained shear strength parameters ( $C_u$ ) were as follows: upper sandy silt and silt stratum  $C_u = 1,000 - 2,000$  p.s.f.; varved clays - upper zone  $C_u = 1,500 - 1,750$  p.s.f. - lower zone  $C_u = 2,000 - 2,250$  p.s.f.



Parameters used for the long term stability were: upper sandy silt and silt  $C' = 504$  p.s.f.  $\phi' = 35^\circ$ , varved clays  $C' = 620$  p.s.f.  $\phi' = 25^\circ$ . Long term stability of the slopes were further checked with  $C' = 0$ , postulating that in the stiff clay fissures may develop, which in turn may lead to a progressive reduction of  $C'$  to an eventual value of zero.

As a result of the study the overall stability of the existing slopes was deemed to be acceptable. It is recommended that the natural slopes of the river banks be kept intact and undisturbed. The present slopes are assumed to be in a limiting equilibrium, thus any steepening of the slopes or parts of it may trigger a slip failure. In those local areas, where construction will necessitate the disruption of natural slopes, they should be reshaped to not steeper than 3.5 horizontal to 1 vertical slopes, immediately upon completion of construction. Erosion control by seeding or sodding of the newly formed slopes will be essential.

#### 6.3) Choice of Alignment:

Stability of the slopes as well as subsoil conditions were found to be comparable along the three proposed alignments. As a consequence no preference can be made regarding choice of lines, based on geotechnical considerations. It is, however, recommended that in the case of maintaining traffic via existing structure during construction of the new bridge, no pile driving should be carried out within a distance of 25 ft. from any of the existing footings.

#### 6.4) Foundations:

The existing structure is supported on concrete and timber friction piles. On account of the suspected ground movements, mentioned earlier, and the possible much higher standard of the proposed bridge - after careful consideration - the use of friction piles under the piers was ruled out. It is, therefore, recommended that the entire bridge be supported on end bearing steel H piles, driven to refusal on bedrock. The elevation of bedrock under the east abutment is around 480 - 482 ft. dipping westerly and lying around elevation 456 ft. - 457 ft. under the west abutment. Since

the bedrock was noted to be sound dolomitic limestone, it is assumed that 12" BP 74 steel H piles driven to sound rock will support safe loads of 130 tons/pile.

Very long piles will be necessary especially under the abutments, exceeding 200 - 220 ft. length. Considerable savings could, however, be realized by driving shorter friction piles beneath the two abutments. It is estimated that very small differential settlements would occur between abutments and the first piers. In the case of adopting friction piles for the abutments, the use of 60 ft. long No. 14 treated timber piles are recommended, driven to the silt stratum. The estimated design loads on such piles, with a minimum embedded length of 50 ft. may be taken to be 25 tons/pile. Consolidation settlements under the pile groups are predicted to be less than 2 inches.

#### 6.5) Excavations:

A minimum cover of 7 ft. should be provided for the pile caps for frost protection. Footing excavations will be in the sandy silt and silt deposit. This material is very sensitive to conditions of unbalanced hydrostatic heads and will boil at the excavation bottoms if advanced below the groundwater level. Since the locations of the proposed footings are not decided as yet, it is difficult to determine whether the footing excavations will extend below the groundwater level or not. As a precaution it should be mentioned in the contract that a dewatering scheme may be necessary in order to keep the excavation bottoms dry and stable. Such scheme may consist of interlocking sheet piles, driven along the perimeter of the excavations, to a depth below the excavation bottom equal to or greater than the distance of the prevailing water level above it. The accumulated water could then be pumped out inside the sheet piles without inducing instability.

#### 7. MISCELLANEOUS:

The field work carried out during the period of

August 2 - August 30 was supervised by Messrs. P. Martin and J. Hodge, Engineering Students.

Equipment used was owned and operated by Dominion Soil Investigation Ltd., North Bay, Ontario.

Stability analyses of the slopes were carried out by Mr. E. Wood, Project Foundations Engineer. This report was prepared by Mr. A. K. Barsvary, Senior Foundations Engineer and reviewed by Mr. K. G. Selby, Supervising Foundations Engineer.

*A. K. Barsvary*  
A. K. Barsvary, P. Eng.



*K. G. Selby*

K. G. Selby, P. Eng.

AKB/ao  
Nov. 6, 1972.

APPENDIX I

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

## RECORD OF BOREHOLE NO 1

JOB 72-11086

LOCATION Sta. 279 + 05 (3' Rt. C

ORIGINATED BY LJR

W.P. 10-70

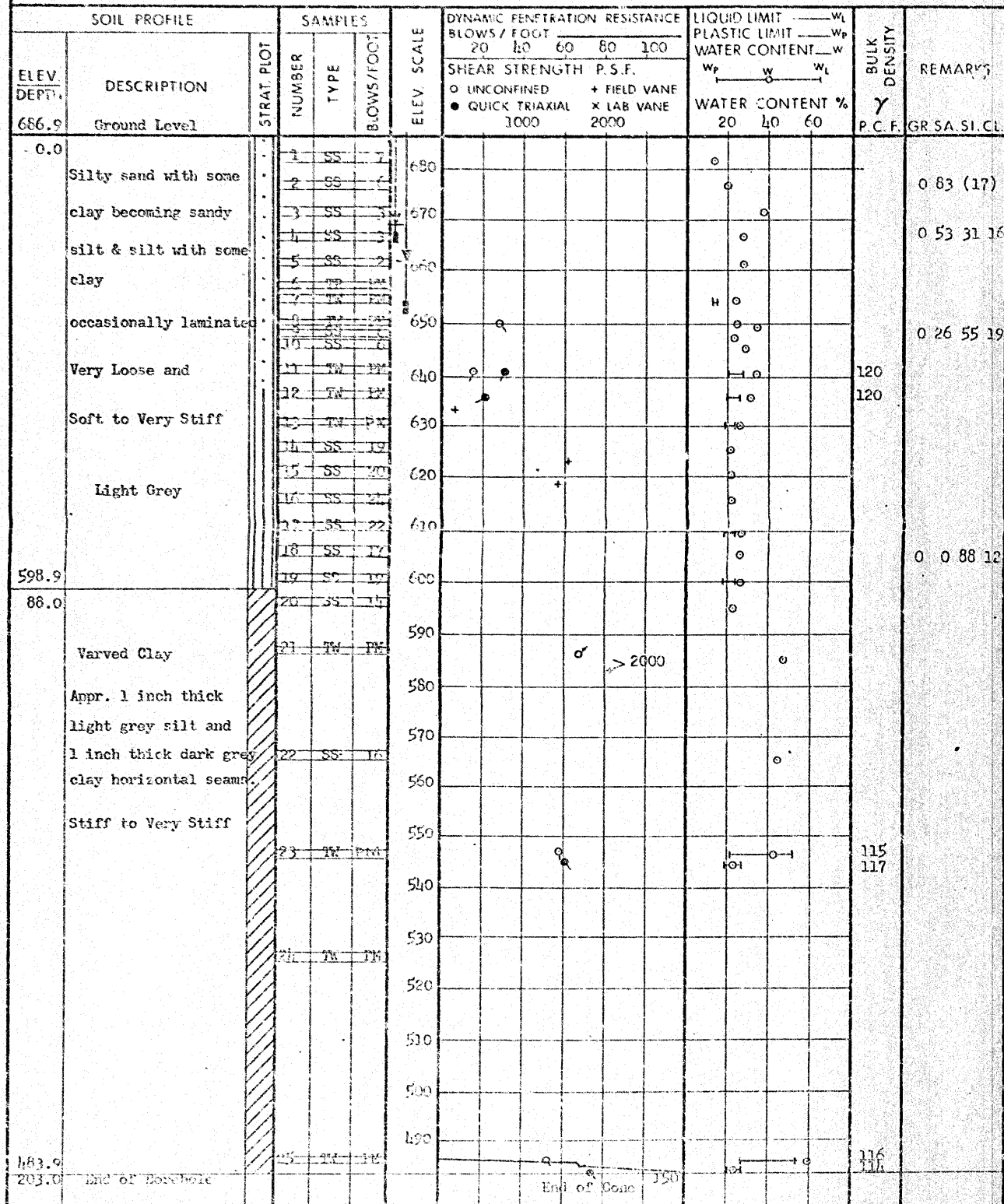
BORING DATE August 2, 1972 - Aug. 7/72

COMPILED BY LJR

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger &amp; Washbore

CHECKED BY LJR



DESIGN SERVICES BRANCH

## RECORD OF BOREHOLE NO 2

FOUNDATIONS OFFICE

JOB 72-11086

LOCATION Sta. 279 + 76 76' Lt. C

ORIGINATED BY L.J.H.

W.P. 40-70

BORING DATE Aug. 22 - 26, 1972

COMPILED BY L.J.H.

DATUM Geodetic

BOREHOLE TYPE Washbore, NX Casing

CHECKED BY J.L.

SOIL PROFILE			SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ——— $w_L$ PLASTIC LIMIT ——— $w_p$ WATER CONTENT ——— $w$		BULK DENSITY $\gamma$ P.C.F.	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS/FOOT	SHEAR STRENGTH P.S.F.		$w_p$ ——— $w$ ——— $w_L$				
							○ UNCONFINED ● QUICK TRIAXIAL 1000	+ FIELD VANE x LAB VANE 2000	WATER CONTENT % 20 40 60				
662.8	Ground Level												
0.0	Clayey silt to silt, occasional faint lamination.  Soft to Very Stiff  Light Grey		1	SS	3	660						122        0 0 80 20	
			2	TV	10	650		+					
			3	SS	11	640							
			4	TV	PM	630							
			5	SS	PM	620							
			6	TV	PM	610							
			7	TV	PM	600							
			8	TV	PM	590							
592.8	Varved clay with increasing thickness of dark clay and light silt seams.  Stiff to Very Stiff		9	TV	PM	580						117	
70.0			10	SS	13	570							111 113
			11	TV	PM	560							117
			12	TV	PM	550							
			13	SS	9	540							115
			14	TV	PM	530							
			15	TV	PM	520							110
			16	SS	11	510							
478.4	Bedrock		17	RO	10	470							
186.4	End of Borehole												



DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

## RECORD OF BOREHOLE NO 3

JOB 72-11086

LOCATION Sta. 281 + 50 70' Rt. 6

ORIGINATED BY L.H.

W.P. 40-70

BORING DATE Aug. 9 - 18, 1972

COMPILED BY L.H.

DATUM Geodetic

BOREHOLE TYPE Washbore, NX Casing

CHECKED BY L.H.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$		BULK DENSITY $\gamma$ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F.		$w_p$ — $w$ — $w_L$			
							○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE	WATER CONTENT % 20 40 60			
627.2	Ground Level											
0.0	Sandy silt becoming silt, some clay.		1	SS	15	620						0 34 53 13
	Compact		2	SS	22							
	Light Grey		3	SS	14	610						
			4	SS	12							0 0 85 15
599.2			5	SS	22	600						
28.0			6	SS	11							
			7	TW	FM	590					116	
			8	TW	FM						111	
			9	TW	FM							
			10	TW	FM	580						
			11	TW	FM						107	
			12	SS	8	570					109	
			13	TW	FM							
			14	TW	FM	560						
			15	TW	FM	550					114	
			16	TW	FM						110	
			17	TW	FM	540						0 0 84 16
			18	TW	FM							0 1 26 73
			19	TW	FM	530						
			20	SS	6	520						
			21	TW	FM	510					112	
			22	TW	FM						115	
			23	TW	FM	500						
			24	TW	FM	490						
			25	TW	FM	480						
465.4	Bedrock		26	RO	100%	460						
156.8	End of Borehole											

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

## RECORD OF BOREHOLE NO 4

JOB 72-11686

LOCATION Sta. 282 + 95 70' Lt. B

ORIGINATED BY L.J.H.

W.P. 40-70

BORING DATE August 4 - 8, 1972

COMPILED BY L.J.H.

DATUM Geodetic

BOREHOLE TYPE Washbore, BX Casing

CHECKED BY L.J.H.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE	LIQUID LIMIT	PLASTIC LIMIT	WATER CONTENT	BULK DENSITY	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		BLOWS / FOOT	SHEAR STRENGTH P.S.F.	w <sub>p</sub>	w			w <sub>L</sub>
601.3	Ground Level												
0.0	Sandy silt, some clay		1	SS	3	600						0 37 45 18	
593.3			2	SS	2	590							
8.0	Varved clay		3	SS	3	580					107		
	Increasing thickness of dark clay and light silt seams.		4	SS	3	580					108		
			5	SS	7	570							
			6	SS	7	570							
			7	TW	PM	560					113		
			8	TW	PM	550					112	0 5 16 72	
			9	TW	PM	540					112	0 8 85 15	
	Stiff to Very Stiff		10	TW	PM	530					113		
			11	SS	9	530							
			12	TW	PM	520							
			13	TW	PM	510					110	0 0 (100)	
			14	TW	PM	500					112	0 2 28 70	
	Increasing thickness of silt.		15	SS	24	490							
			16	TW	PM	490							
			17	SS	30	480							
			18	TW	PM	480							
469.8			19	SS	21	470							
451.5	Dolomitic Limestone Bedrock.		20	RC	98%	460							
459.6	Sound		21	RC	96%	460							
441.7	End of Borehole					450							



DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

## RECORD OF BOREHOLE NO 5

JOB 72-11086

LOCATION Sta. 284 + 25 35' Rt. 6

ORIGINATED BY L.H.

W.P. 40-70

BORING DATE Aug. 28 - 30, 1972

COMPILED BY L.H.

DATUM Geodetic

BOREHOLE TYPE Washbore, NX Casing

CHECKED BY

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT ——— W <sub>L</sub>		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BLOWS / FOOT	PLASTIC LIMIT ——— W <sub>P</sub>	WATER CONTENT ——— W		
604.8	Ground Level										
0.0	Silt with some clay and sand.		1	SS	8	600					0 13 69 18
584.8	Loose		2	SS	4	590					
20.0			3	SS	4	580	q				
			4	TW	PM	570	e			102	0 0 16 82 0 0 48 52
	Varved Clay		5	TW	PM	560	q			116 114	
	1/2 - 1" thick dark clay seams,		6	TW	PM	550					
	thickness of silt seams increases with depth		7	TW	PM	540	q			112 114	
	(1/2 inch to several inches).		8	TW	PM	530					
			9	TW	PM	520	q			112	
			10	SS	9	510					
			11	TW	PM	500					
	Firm to Very Stiff		12	TW	PM	490	q			115	
			13	TW	PM	480					
			14	SS	24	470				120	
465.6	Bedrock		25	SC	100						
139.0	End of Borehole										

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

## RECORD OF BOREHOLE NO 6

JOB 72-11036

LOCATION Sta. 285 + 65 60' Lt. B

ORIGINATED BY L.J.H.

W.P. 40-70

BORING DATE Aug. 25 - 30, 1972

COMPILED BY L.J.H.

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Auger &amp; Washbore

CHECKED BY L.J.H.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$		BULK DENSITY $\gamma$ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F.		$w_p$ — $w$ — $w_L$			
							○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE	WATER CONTENT % 20 40 60			
635.4	Ground Level											
0.0	Sandy silt to silt with some clay.	.	1	SS	11					○		0 26 63 11
			2	SS	12					○		
			3	TW	PM				Q	○		
			4	SS	10					○		
	Compact										126	
	Light Grey	.	5	TW	PM					○		0 1 86 13
			6	SS	10					○		
585.4	Clay	.	7	TW	PM					○		110
50.0			8	SS	13					○		
	Stiff Grey	.	9	TW	PM					○		107
564.9			10	SS	9					○		
70.5	Varved Clay	.	11	TW	PM					○		115
			12	TW	PM					○		
			13	TW	PM					○		
			14	SS	21					○		
	horizontal seams of dark clay and light silt											
	appr. 1" thick seams	.	15	TW	PM					○		112
			16	SS	21					○		
	Stiff to Very Stiff	.	17	TW	PM					○		112
			18	SS	21					○		
	Inclined lamination of varves.	.	19	TW	PM					○		112
			20	SS	21					○		
461.8	End of Borehole Probable Bedrock: Rods bouncing;	.	21	SS	21					○		
173.6			22	SS	21					○		

# RECORD OF BOREHOLE NO 7

JOB 72-11086	LOCATION Sta. 287 + 82 Ch. Rt. C	ORIGINATED BY L.H.
W.P. 40-70	BORING DATE Aug. 17 - 23, 1972	COMPILED BY L.H.
DATUM Geodetic	BOREHOLE TYPE Hollow Stem Auger & Washbore, BX Casing	CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$			BULK DENSITY $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F.			WATER CONTENT %				
							○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE		$W_P$	$W$	$W_L$		
677.5	Ground Level						1000	2000		20	40	60	P.C.F.	GR SA SI. CL
0.0	Silt with some clay  Soft to Very Stiff  Light Grey		1	SS	14	670				○				
			2	TW	12	660				—○—			122	0 1 (99)
			3	SS	6	650				○				
			4	TW	12	640	q			—○—			121	
			5	SS	20	630				○				
			6	TW	12	620				—○—				
			7	SS	20	610				○				0 0 87 13
			8	SS	18	600	p			○ N.P.			125	
			9	SS	6	590				○				0 2 80 16
579.5					10	SS	6	580			○			
98.0	Varved Clay  Increasing thickness of dark clay and light silt seams.  Stiff		11	TW	12	580	●			—○—		109		
			12	SS	0	570								
			13	TW	12	560				○				
			14	TW	12	550								
			15	TW	12	540	○			—○—			113	
			16	TW	12	530								
			17	TW	12	520								
			18	TW	12	510	p			—○—			114	
			19	SS	20	500								
172.5	Silt with some clay.  Hard		20	SS	20	470							0 1 86 13	
205.0														
156.5														
221.0														
151.5	Bedrock Sound		19	RC	90%	450								
226.0	End of Borehole													

# VOID RATIO - PRESSURE CURVES

JOB NO. 72-11086

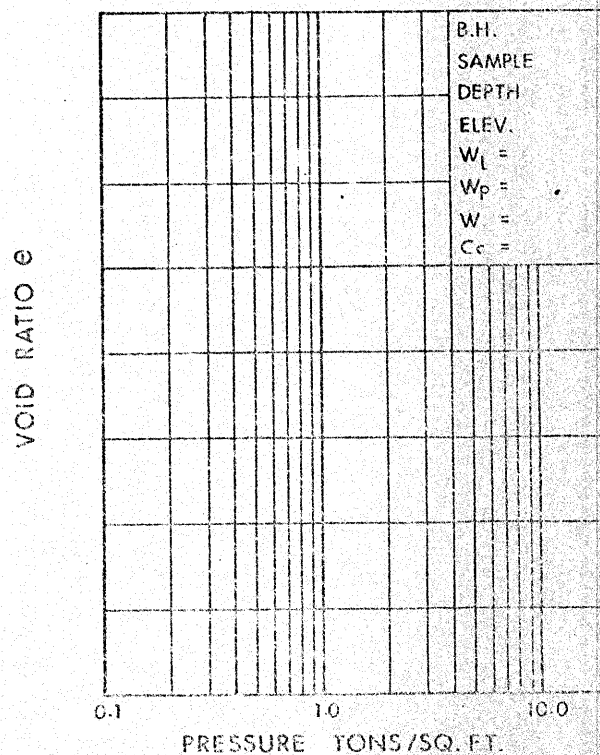
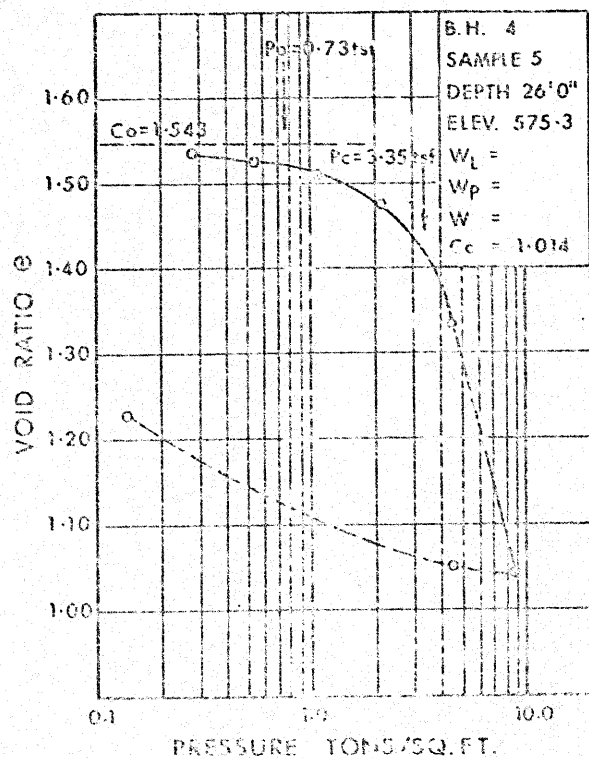
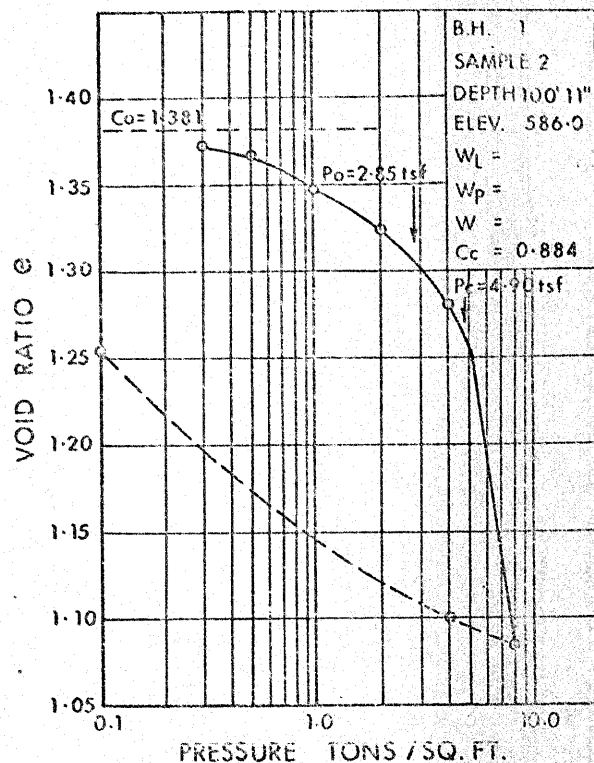
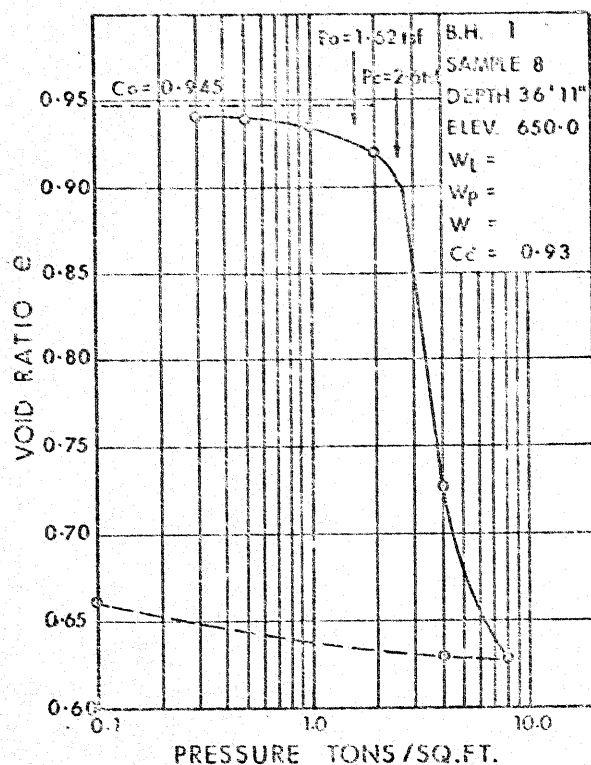


FIG. 1

## ABBREVIATIONS USED IN THIS REPORT

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

### DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

### TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

### SOIL TESTS

Q <sub>u</sub>	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Q <sub>cu</sub>	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q <sub>d</sub>	DRAINED TRIAXIAL	S	SENSITIVITY

# ABBREVIATIONS USED IN THIS REPORT

## SOIL PROPERTIES

$\gamma$	UNIT WEIGHT OF SOIL (BULK DENSITY)
$\gamma_s$	UNIT WEIGHT OF SOLID PARTICLES
$\gamma_w$	UNIT WEIGHT OF WATER
$\gamma_d$	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
$\gamma'$	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
$S_r$	DEGREE OF SATURATION
$w_L$	LIQUID LIMIT
$w_P$	PLASTIC LIMIT
$I_P$	PLASTICITY INDEX
s	SHRINKAGE LIMIT
$I_L$	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
$I_C$	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
$e_{max}$	VOID RATIO IN LOOSEST STATE
$e_{min}$	VOID RATIO IN DENSEST STATE
$I_D$	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY $D_r$ IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
$m_v$	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma}$
$c_v$	COEFFICIENT OF CONSOLIDATION
$C_c$	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma}$
$T_v$	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
$\tau_f$	SHEAR STRENGTH
$c'$	EFFECTIVE COHESION
$\phi'$	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
$c_u$	APPARENT COHESION
$\phi_u$	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
$\mu$	COEFFICIENT OF FRICTION
$S_t$	SENSITIVITY

## GENERAL

$\pi$	$= 3.1416$
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

## STRESS AND STRAIN

u	PORE PRESSURE
$\sigma$	NORMAL STRESS
$\sigma'$	NORMAL EFFECTIVE STRESS ( $\bar{\sigma}$ IS ALSO USED)
$\tau$	SHEAR STRESS
$\epsilon$	LINEAR STRAIN
$\gamma$	SHEAR STRAIN
$\nu$	POISSON'S RATIO ( $\mu$ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
$\eta$	COEFFICIENT OF VISCOSITY

## EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
$\delta$	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
$K_0$	COEFFICIENT OF EARTH PRESSURE AT REST

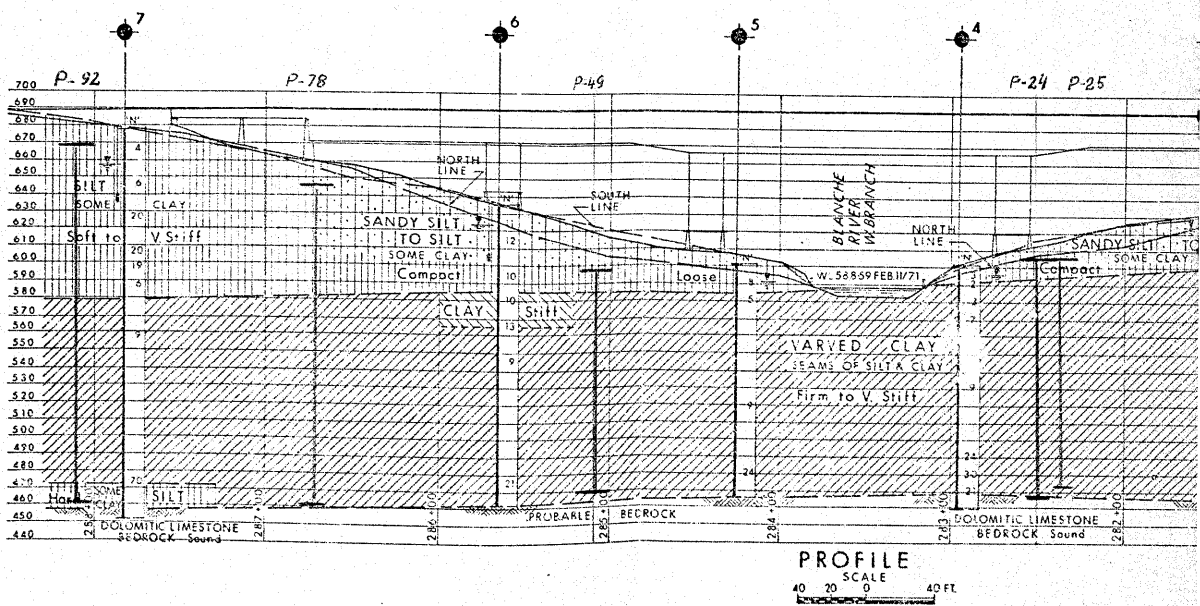
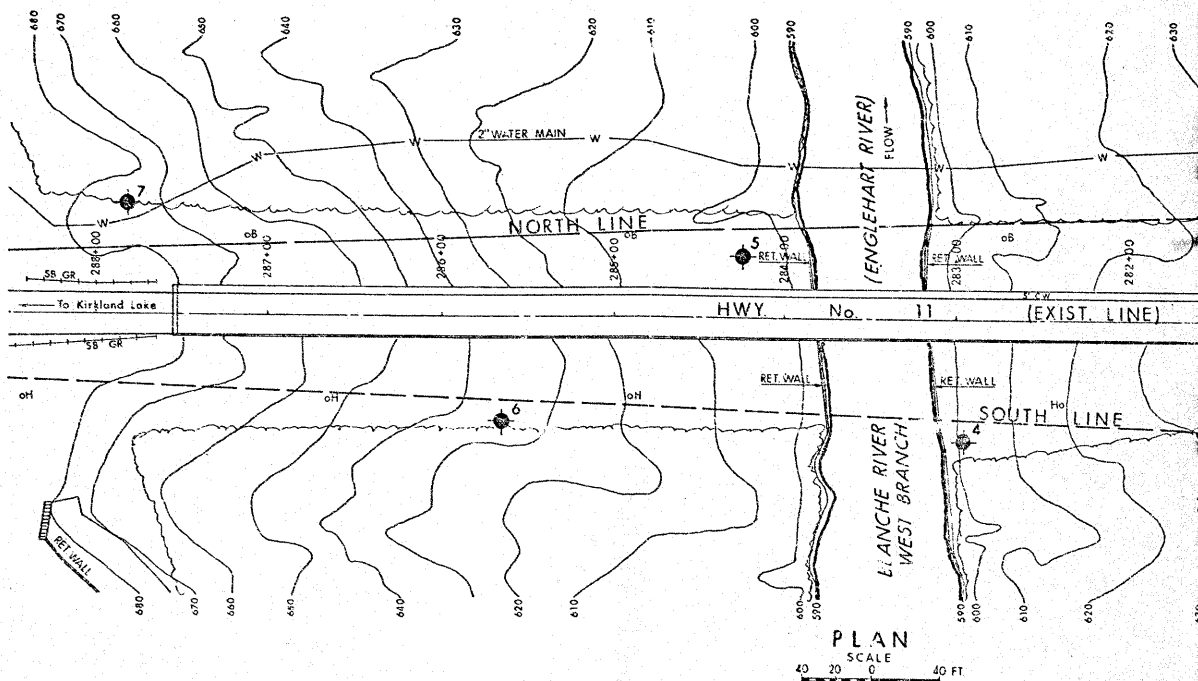
## FOUNDATIONS

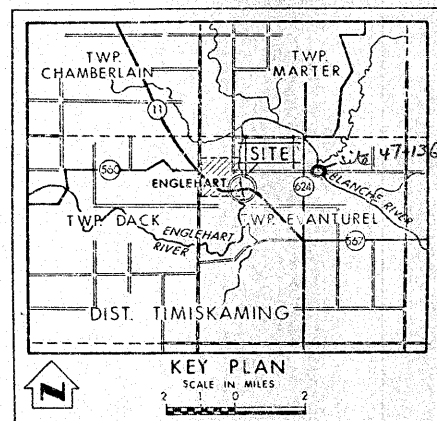
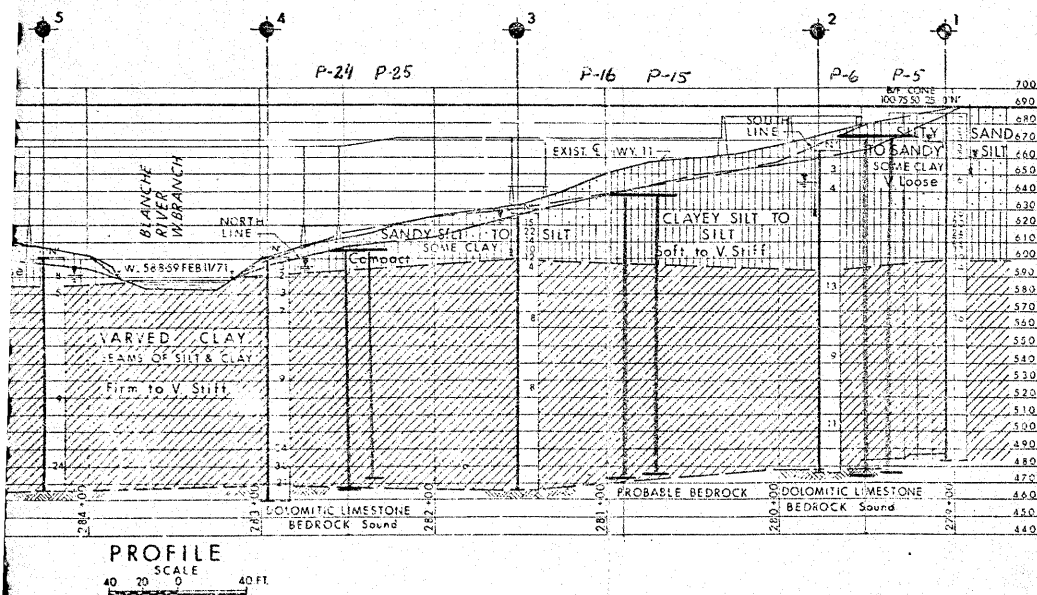
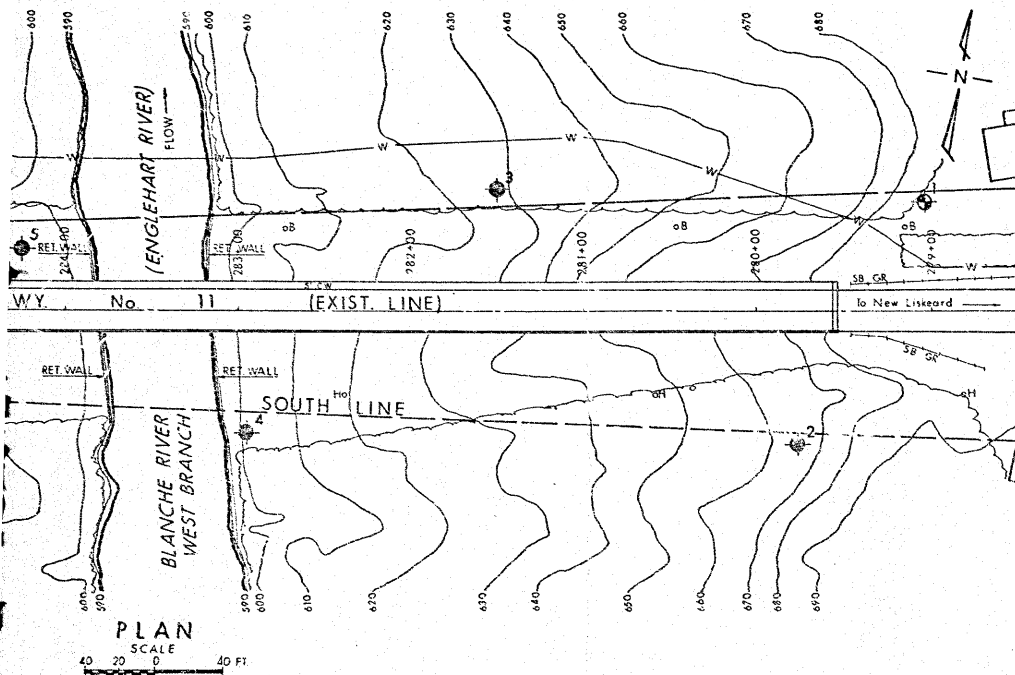
B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
$k_s$	MODULUS OF SUBGRADE REACTION

## SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
$\beta$	ANGLE OF SLOPE TO HORIZONTAL







LEGEND			
	Bore Hole		
	Cone Penetration Test		
	Bore Hole & Cone Test		
	Water Levels established at time of field investigation, Aug. 1972		
	Piezometer		
NO.	ELEVATION	STATION	OFFSET EXIST. HWY
1	686.9	279+05	63' RT
2	662.8	279+76	76' LT
3	627.2	281+50	70' RT
4	601.3	282+95	70' LT
5	604.8	284+25	35' RT
6	635.4	285+65	60' LT
7	677.5	287+82	64' RT

— NOTE —  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO  
DESIGN SERVICES BRANCH—FOUNDATIONS OFFICE

**BLANCHE RIVER WEST BRANCH  
(ENGLEHART RIVER)**

HIGHWAY NO. 11 DIST. NO. 14  
DIST. TIMISKAMING  
TWP. EVENTUREL LOT 10 CON. 5

**BORE HOLE LOCATIONS & SOIL STRATA**

SUBMD L.J.H.	CHECKED	WP NO. 40-70	DRAWING NO.
DRAWN O.L.J.	CHECKED	W.O. NO. 72-11086	<b>72-11086-A</b>
DATE 27 OCT 1972	SITE NO.	BRIDGE DRAWING NO.	
APPROVED	CONT. NO.		

PRINCIPAL DESIGNER

REF. No E-5023-1



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. J. McAllister,  
Reg. Structural Planning Supervisor,  
District #13, NORTH BAY.

FROM: Structural Office,  
West Building,  
Downsview, Ont.

ATTENTION:

DATE: June 22, 1973.

OUR FILE REF.

IN REPLY TO

SUBJECT:


Englehart River Bridge,  
At Englehart,  
W.P. 40-70-01, Site 47-32,  
Hwy. #11, District #14, New Liskeard.

72-11-086

Attached herewith are prints of the Preliminary Bridge  
Plan Drawing D-47-32-P1 for the above mentioned structure.

The estimated cost of the proposed structure is \$1,350,000  
which includes tender, materials, engineering and sundry construction.

Any comments or revisions you may have should be submitted  
within four weeks.



CSG/js

C. S. Grebksi  
Structural Design Engineer

Attach.

c.c. B. R. Davis  
W. Birch  
A. E. McKim  
A. Stermac  
J. Anderson  
R. Murphy  
J. Harris  
M. Stoyanoff  
W. McFarlane

No comments

K. L. Sully

# FOUNDATIONS OFFICE

REVIEW OF DESIGN DRAWINGS: W.P. .... 40-70-01  
W.O. .... 72-11086

Foundation Report By: ..... A.K. Balsary  
Review of Design Drawings By: ..... A. Prakash  
Design Drawing No.'s ..... 47-32-P1

1. Does footing design comply with our report or subsequent memos? yes
2. If answer to 1. is No, is present design acceptable? -
3. Has sufficient field work been done? YES
4. Are estimated pile lengths shown on Drawings correct? If not, make a new list. NO Lengths
5. If excavation of unsuitable soil is recommended, is this shown on Drawings? N A
6. Are approaches designed in accordance with our report? Check slopes and berm lengths. YES
7. Do you anticipate any construction problems? i.e., dewatering, stability of temporary slopes or excavations. yes
8. Summarize your comments; on separate sheet if necessary.

1. Piers. NO 2 & 3 are less than 20' away from the nearest existing footing. We recommended a minimum distance of 25'. Certain piles (within 25') will be preaugered - 1488
2. Check bearing capacity of piles. We recommended 130 T/Pile. Max loading is 135 - OK. 1488.

Drawings Received ..... June 25 ..... 1973  
Reviewed ..... July 11 ..... 1973

Signed ..... A. Prakash

NOTE . 1. North line chosen  
2. Entire structure supported on Steel H-piles driven to bedrock

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. A. Stermac  
Principal Foundation Engineer  
Room 107,  
West Bldg.

ATTENTION:

OUR FILE REF.

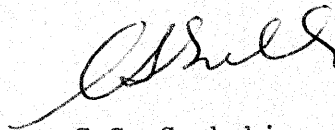
FROM: C.S. Grebski  
Structural Design Engineer  
Structural Office  
West Bldg.  
DATE: November 26th, 1973

IN REPLY TO

SUBJECT: Englehart River Bridge  
At Englehart  
W.P. 40-70-01, Site 47-32  
Hwy. #11, District #14

Attached herewith we are submitting the final bridge drawings which show the foundation design for this structure.

Kindly give us your comments at your earliest convenience.



C.S. Grebski  
Structural Design Engineer

CSG:AMF

Attached.

c.c. Foundation Office

*copy to B.O.  
Mon. 7/14.  
a.s.*

*Mar comments*

*ARakash*

*Dec 14/73*



FOUNDATIONS OFFICE

REVIEW OF DESIGN DRAWINGS:

W.P. ....40-70-01.....  
W.O. ....72-11086.....

Foundation Report By:

.....H. K. BURSARY.....

Review of Design Drawings By:

.....L. J. HODGE.....

Design Drawing No.'s

.....17-22.....

1. Does footing design comply with our report or subsequent memos? yes
2. If answer to 1. is No, is present design acceptable?
3. Has sufficient field work been done? yes.
4. Are estimated pile lengths shown on Drawings correct?  
If not, make a new list. yes.
5. If excavation of unsuitable soil is recommended,  
is this shown on Drawings? N/A
6. Are approaches designed in accordance with our  
report? Check slopes and berm lengths. yes.
7. Do you anticipate any construction problems?  
i.e., dewatering, stability of temporary slopes  
or excavations. yes.
8. Summarize your comments; on separate sheet if necessary.

No comments.

Drawings Received ....Nov 27.....19.73.  
Reviewed ....Nov 27.....19.73.

Signed

.....L. J. Hodge.....

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. J.C. McAllister,  
Reg. Struct. Supvr.,  
North Bay, Ontario.

FROM: Systems Design,  
North Bay, Ontario.

ATTENTION:

DATE: January 2, 1974

OUR FILE REF.

IN REPLY TO

SUBJECT: W. P. 40-70-01, Englehart River Bridge,  
Hwy. 11, Dist. #14, New Liskeard.

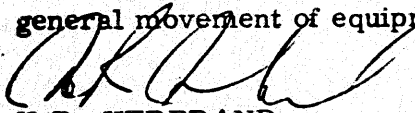
We are in receipt of the final bridge drawings and documents for this project and make the following comments for your consideration.

The Foundation Investigation Report placed considerable emphasis on slope stability and erosion control. Several features of this design will affect slope stability and erosion and have not, I feel, been covered adequately.

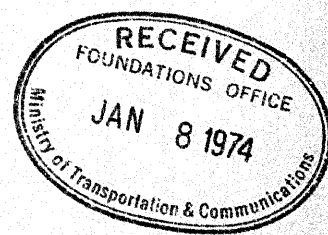
1) -Deck drains - The outlet pipes as indicated are to extend to a point just below the girders. The water must then free fall to a 20' square rip-rap pad. In the case of the drains adjacent to piers 2 and 3, this will mean a free fall of some 75'. The chances of the water being contained within the limits of the rip-rap pads is doubtful. In the case of the drains adjacent to piers 1 and 4, the chances of the water being contained within the initial rip-rap pad is better as the free fall is only some 25' to 30'. The water is then to be directed by means of 24"  $\emptyset$  half-pipe to 5' square rip-rap pads some 50' from the edge of the new structure. If the water can be concentrated in the half-pipe, this method will serve to move the potential erosion away from the bridge piers but not eliminate it. A possible alternative to the present design would be to a) move the deck drains closer to the piers -b) extend the outlet pipes, fastened to the piers, to a point within 2 or 3 feet of the ground -c) catch the water by means of dished rip-rap pads and ditch-inlet catch-basins and -d) direct the water to the river with storm sewer.

2) Excavation for pier footings - Some direction should be given to the Contractor regarding care and protection for this excavation work and the treatment of the finished slopes. Maximum of 3.5 to 1 reshaped slopes recommended in foundation report.

3) Work on river bank slopes - Some direction may be in order regarding the general movement of equipment on the slopes.

  
H.R. HERBRAND,  
PROJECT DESIGN SUPERVISOR,  
FOR: H. McARTHUR,  
SR. PROJECT DESIGN ENGINEER.  
HRH/HMcA/bb







MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

Copy for the information of K. Selby

72-11-086

Mr. C. S. Grebski,  
~~Structural Design Engineer,~~  
West Bldg., Downsview.

Structural Planning,  
Northern Region,  
North Bay.

4 January 1974.

W.P. 40-70-01, Englehart River,  
Highway #11, District #14

47-32

Attached is a copy of a memo received from Systems Design,  
Northern Region.

I had been led to believe that a much more effective form  
of deck drainage was under consideration some time ago and  
therefore recommend that consideration be given to the  
alternatives in Paragraph 1 of the attached memo.

Some warning or special provision should be included in  
the contract for pier excavation and work on the river bank  
slopes. I have seen comparatively light tracked equipment  
bogged down on these slopes in July when presumably at least  
the surface is dry.

JCMcA/les  
Att.

J. C. McALLISTER,  
REG. STRUCTURAL PLANNING SUPVR.

c.c. K. Selby ✓  
M. Stoyanoff  
R. Murphy

July 4/14 6"  $\phi$  Pipes @ 40' flow 0.08 cfs./inlet  
Meeting Total flow 0.114 cfs

This will disperse the water. Splash plates to be provided  
The dam nearest the abutment (40' from it) will  
drain about a quart of water/min. only JW

Mr. C.S. Grebski,  
Structural Design Engineer,  
West Building, Downsview.

Soil Mechanics Section,  
Geotechnical Office,  
West Building, Downsview.

Mr. A. Radkowski.

February 15th, 1974.

Englehart River Bridge,  
Hwy. #11, Dist. #14(New Liskeard),  
W.O. 72-11086 W.P. 40-70-01.  
\*\*\*\*\*

We have reviewed the final bridge drawings for this structure. The drawings indicate that excavations for pile caps will be up to 20 ft. in depth. In view of the above we have carefully reviewed the groundwater and subsoil conditions at this site. Our comments are as follows:

The subsoil, into which the excavations will be carried out, consists of very loose to compact silt to fine silty sand underlain by a relatively deep deposit of firm to stiff varved clay. Because of this sequence of soil types, perched water conditions are likely to occur. The level of perched water can be very high and may coincide with the ground surface. The ground water levels as shown on Foundation Drawing #72-11086A are 3-17 ft. below the ground surface. However, because of the non-cohesive material at the surface, ground water level will fluctuate and may be as high as the elevation of the ground surface itself. The silty to fine silty sand material is very dilatant and will 'boil' under an unbalanced hydraulic head. Furthermore, excavations with 1:1 side slopes in this type of soil will be unable to stand safely. Any failure of the side of an excavation may result in conditions detrimental to the existing slopes, existing structure and/or the new structure. This may cause permanent damage to the slopes or the present bridge.

Therefore, we recommend that the excavation be carried out within sheet piles driven into the ground. The sheet piles should be driven to a depth of 10 ft. below the footing base level prior to any excavation and should be adequately braced at all times, as excavation proceeds.

February 15th, 1974.

Mr. C.S. Grebski - RE: Englehart River Bridge.

The excavations may be backfilled using native earth in a suitable condition for compaction. However, if a granular type of backfill material is used, which will cause build-up of pressure thereby endangering the slope and the structure, then possible accumulations of water should be prevented by providing proper drainage. If it is decided to withdraw the sheet piles, extraction should be done only when the backfilling operation is completed.

These sheet piles should be designed and shown on the final bridge drawings. The fact that no open excavation can be permitted and that sheet piles are essential, should be emphasized by mentioning it in the Special Provisions to bidders. We would be pleased to advise you regarding suitable wording.

If you need additional information, please contact this Office.

AP/mj

C.C. G.A. Metcalfe  
A. Rutka  
T.A. Sharpe  
J.E. Gruspier  
S. McCombie  
J. McAllister  
Foundations Files  
Documents

*Anand Prakash*

Anand Prakash,  
Sr. Engineer-Soil Mechanics  
For:  
K.G. Selby,  
Supervising Engineer-  
Soil Mechanics.

File 1223

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: Mr. K. Selby ✓  
Soil Mechanics  
West Bldg.

FROM: Structural Office  
West Bldg.  
Downsview

ATTENTION:

DATE: March 18th, 1974

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 40-70-01, Site 47-32  
Englehart River Bridge  
Hwy. 11, District 14

The designer provided a drawing showing temporary protection for the foundation excavation for the above project. Lump sum payment for this work will be covered under a tender item of "Slope Protection" and we are proposing the following special provision for this tender item.

"Slope Protection"

In carrying out the construction of the footings the Contractor shall provide temporary interlocking sheeting as shown on the contract drawings. The sheeting shall be driven to a minimum depth of 10 feet below the bottom of the footings and shall be adequately braced at all times.

The Contractor shall submit drawings and details of his proposal to the Engineer for approval at least four weeks before carrying out the work.

Payment at the contract price for the above tender item will be full compensation for the supply of all labour, equipment and material necessary to complete the work described herein."

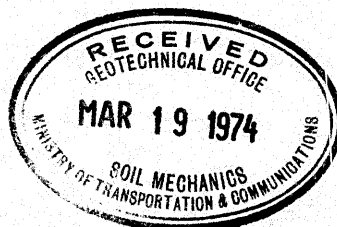
Would you please let me have your comments at your earliest conveniences.

*N. Zoltay*

N. Zoltay  
Contract Specification Engineer

NZ:AMF

c.c. C. Grebski  
A. Radkowski



## Slope Protection

The Contractor is advised that the subsoil at the pier and abutment locations and elsewhere on the slopes on both sides of the Englehardt River consists of very loose to compact silt to fine silty sand underlain by a deep deposit of firm to stiff varved clay. Because of this sequence of soil types perched water conditions are likely to occur. The level of perched water can be very high and may coincide with the ground surface. The groundwater levels shown on Drawing # 47-32-2 are 3 to 17 ft. below the existing ground surface, however if perched water conditions prevail water may be encountered right at the ground surface. The silt to fine silty sand subsoil is very dilatant and will 'boil' under conditions of unbalanced hydrostatic head. 'Boiling' of the subsoil in excavations could result in slope failures causing damage to the existing and new structures.

In carrying out the construction etc.

## MEMORANDUM

TO: Mr. J. Wear,  
Project Review Engineer,  
Systems Design Office.

FROM: M. Chamberlain,  
Landscape Development Planner,  
Landscape Development Section.

ATTENTION:

DATE: July 3, 1974

OUR FILE REF.

IN REPLY TO

SUBJECT: Englehart River Bridge - W.P. 40-70-01

As per your request, this office has completed a field inspection of the slope vegetation and soils of the above-mentioned site and the following comments are resultant from that investigation.

The vegetation in the area of proposed construction is of a fast growing nature and high regenerative capacity. Deciduous material includes *Populus* spp. and Black Cherry ranging in size from 1" saplings to 8" caliper trees. The understory is comprised of Horsetail, *Sarsaparilla*, ferns, various seedlings and grasses. Young Spruce trees up to 6 feet are also prevalent.

The soil type on the slopes is silt clay, a delicate material on the gentlest of slopes. As well as being highly susceptible to surface erosion, these silt clays have a past history of "slumping." We inspected several locations in the area where slumping had occurred previously (see Photo 1). The increase in weight resulting from the absorption and retention of water causes a mass movement of the material down the slope.

The slope areas where the vegetative cover is heavy are presently in a stable condition. This can be attributed to several factors including:

1. "umbrella" effect of plant foliage reducing surface erosion
2. anchoring influence of root systems
3. absorption of moisture by the roots and subsequent transpiration through the leaves, thus reducing the possibility of excessive water retention in the soil

The effects of removing this vegetation are exemplified by the recent installation of a water line approximately 100 feet north the existing bridge (see Photos 2, 3 and 4). The post construction corridor is eroding rapidly (see Photos 5 and 6), and it is our opinion that it is only a matter of time before slumping occurs.



The damage and problems resulting from the water line installation are indicative of what will happen if the slope is disrupted on a large scale to construct the proposed underground drainage system. It would appear that the preservation of existing vegetation is essential to retain the slope and that a minimal amount of clearing should be carried out by the contractor. This would suggest that an alternative drainage system be employed. The multiple drain, free fall system mentioned to us earlier seems the most feasible.

I trust this information is sufficient, and if there are any further questions please contact this office.



M. Chamberlain, B.Sc. (Agr.)  
Landscape Development Planner

For

H. Spence  
Chief Arboriculturist

MC:hs



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: DM. Hopper, Manager,  
Contract Control Office,  
Downsview, Ont.

FROM: Systems Design Branch,  
Downsview, Ont.

ATTENTION:

DATE: Aug. 15/74

OUR FILE REF.

IN REPLY TO

SUBJECT:

Cont: 74-105, WP.40-70-01, Dist.14, Hwy.11  
Location: Englehart Blanche River Bridge  
-----

Due to a misunderstanding between the Systems Design and the Foundation Office the Special Provision under title "Earth Excavation for Structure Foundations - Item No. 51" on page #29 of the Tender Form is in error.

It should be corrected by altering, in the first paragraph (under sub-heading "Operational Constraint"), the specified distance "...10 feet from any footing...." to "....20 feet ....".

EJW:kc

c.c. T.A. Sharpe  
S. McCombie  
B.J. Giroux  
J.M. Crannie  
C. Mirza ✓

E.J. Willis  
Project Review Supervisor  
for  
J.R. Wear  
Project Review Engineer

The id constraint applied only to a proposed sensor which was taken out of the contract. The 20 ft. constraint applies to the structure footing excavations and is shown on the structural drawings (Sheet 44).  
To K.G.S.  
etc.



## MEMORANDUM

TO: D.M. Hopper, Manager  
Contract Control Office  
Downsview, Ontario.

FROM: Systems Design Branch,  
Downsview, Ontario.

ATTENTION:

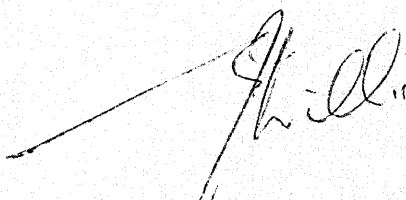
DATE: August 16, 1974

OUR FILE REF.

IN REPLY TO

SUBJECT: Contract 74-105 W.P. 40-70-01, Dist. 14, Hwy. 11  
Englehard Blanche River Bridge.

Further to our letter of August 15th 1974 an error was brought to our attention by Engineering Audit in regards to the Structure Drawings namely Contract Drawing No. 45. The dimension on elevations 1 and 2 indicating 10' below wale 'A' should be corrected to 11' as the centre of wale 'A' is placed 1' above the bottom of the footings.



EJW:mc

c.c. T.A. Sharpe  
S. McCombie  
B.J. Giroux  
J.M. Crannie  
C. Mirza ✓

E.J. Willis  
Project Review Supervisor  
for  
J.R. Wear  
Project Review Engineer



MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO: FILE

FROM: Soil Mechanics Section,  
Geotechnical Office,  
West Building, Downsview.

ATTENTION:

DATE: September 9th, 1974.

OUR FILE REF.

IN REPLY TO

---

SUBJECT: RE: Englehart River Bridge,  
Highway #11,  
Contract #74-105.

I received a call from Mr. Mac Budnick of C.A. Pitts Engineering this P.M. requesting information on the above Contract from a Soils viewpoint.

Checked with Mr. D. Hopper of Contract Control who saw no harm in passing out factual information upon the request of the Contractor.

I had Marie prepare a copy of the factual information portion of this Report.

Mr. Budnick advised he would be sending someone to pick up this information tomorrow A.M.

*L. L. Jordan*  
For: C. Mirza,  
Head, Soil Mechanics Section.

CM/mj

17/11/74

CONTRACT 74-105  
W.O. 72-11086

CHESTER :-

I phoned N.L. Office and spoke to Construction Supervisor (Mr. Dunn) I told him the following (to be also transmitted to Gos.)

(1) I agreed to the driving of one pile without preaugering as a test.

(2) While pile is being driven observations must be made on adjacent pier using a transit.

(3) Any permanent shift (either laterally or vertically) which occurs means that driving must stop and pile must be extracted and preaugered as called for in the contract.

(4) Results of the test to be telephoned in to me at which time further advice will be given.

(5) It was explained to Mr. Dunn that driving resistance is no indication of how much lateral displacement is taking place since this is dependent on the effective volume of the pile and not the strength of the soil. It was explained that plugs often form within the flanges of the piles and the resulting displacement may be considerable.

K. L. Guley



*File*  
**Memorandum**

*K. Selby*

To: Mr. J. I. McDougall,  
Construction Engineer,  
District #14, New Liskeard.

From: Structural Office,  
West Building, Downsview.

Attention: Date: December 20th, 1974.

Our File Ref. In Reply to

Subject: Englehart River Bridge,  
Contract 74-105, Site 47-32,  
Highway #11, District 14.

*W.P. 40-70-01*

This will confirm our telephone conversation of December 17th, 1974, in which permission was given to drive in Pier #3 the seven steel H piles (nearest to the existing Pier bent and for which pre-drilling is called for) in a usual manner. The driving was to be carried out on a trial basis with a driving force as small as possible for a full length of the pile.

This memorandum will also confirm that prior to giving the permission, this matter was discussed with Mr. K. Selby, who agreed in principle with our decision.

Mr. Selby further requested through Mr. R. Dunn, Construction Supervisor, that during driving operations a continuous observation for a possible movement of existing Pier bent using a transit be made and that the most critical pile be driven in my presence.

The driving of this critical pile was carried out in my presence as requested and no lateral or vertical movement of Pier bent was observed.

The instruction was left at the site with Mr. Dunn that similar precautionary measures are to be taken while driving remaining piles.

It is understood that the above permission applies to the driving of piles in Pier #3 only.

AR/cf

*A. Radkowski*

A. Radkowski,  
Regional Structural Design Engineer.

c.c. C. Grebski  
K. Selby  
P. O. Law  
R. Dunn  
A. Cripps



# TCU539 12231513  
LIS444

DNA

K G SELBY, SUPRG FOUNDATIONS ENG

RE: CONTRACT 74-105, STREAM BANK PROTECTION, BLANCHE RIVER  
HAVE HAD A QUERY FROM W SCHAEFFER OF DINEEN CONSTRUCTION ON WHETHER  
THE TWO FOOT THICK RANDOM RIP RAP AT 2 TO 1 SLOPE IS ADEQUATE FOR  
THE POOR SOILS CONDITIONS IN THIS AREA. A MUNICIPAL BRIDGE SITE 47-136  
ON THE SAME RIVER IN THE SAME TOWNSHIP HAS 4 TO 1 SLOPES. YOUR  
COMMENTS AND/OR RECOMMENDATIONS PLEASE.

J I MCDOUGALL, CONST/MUN ENG  
JVA

*Construction staff*  
*705-544-7440*  
*Charlie Caldwell*  
*Project Supervisor*



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P 6 JAN. 8/75 3:40 PM

J I MACDOUGALL CONSTRUCTION/MUNICIPAL ENGINEER

RE: CONTRACT 74-105

PLEASE DISREGARD LETTER OF JAN. 8/75 FROM PETER STEWART FOR  
K G SELVIE TO YOU IN REPLY TO YOUR TELETYPE OF DEC. 24/74  
UPON RECEIPT PLEASE DESTROY LETTER.

C. MARSA MECH SECT

CSS#

ACC TCI 356 01081553 ENR 358



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Return to  
J. I. MACDONALD  
CONST/MUN ENGR.

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**Mr. T.A. Sharpe,  
District Engineer,  
District #14, New Liskeard.**

**Soil Mechanics Section,  
Geotechnical Office,  
West Building, Downsview.**

**Mr. J. McDougall.**

**January 23rd, 1975.**

**Englehart River Bridge, Highway #11,  
District #14, New Liskeard,  
Contract 74-105.**

*W.F. 40-20-01*

Following is a summary of our discussions held at the abovementioned site on January 21st, 1975, in regard to slope drainage and to the shaping and placing of rip-rap at the river banks.

1. It was drawn to our attention by the Contractor that, as presently designed, surface water 'ponds' at a number of locations in depressions on the slope surface. This is due to the fact that the new structure follows the former drainage path and the new piers and associated fills dam this drainage path up. It was agreed that it would be desirable to construct a new drainage path to the north by placing sufficient fill in the depressions to achieve a gradient adequate for drainage, sloping downwards in the northerly direction. Where possible the fill to be used would be the material from the footing excavations, however, new fill material might have to be obtained. It was agreed that the District would do the actual design and would provide the Soil Mechanics Section with the following information in order that stability checks might be made:
  - (a) Original X Sections
  - (b) Present X Sections
  - (c) Proposed X Sections
2. The Contractor has requested more specific instructions regarding the treatment of the new river banks, and wishes to proceed with the work in the very near future in order to take advantage of the frozen ground for movement of machinery, etc. It was agreed that the District would provide the Soil Mechanics Section with X Sections (at least three) parallel to C<sub>1</sub> and extending from back of Pier 2 to the back of Pier 3. One X Section would show the old bridge pier footings. On receipt of this information the Soil Mechanics Section will review the present design with regard to stability and provide new recommendations if necessary.

KGS/ma

c.c. H. Martens  
A. Radkowski  
A. Cripps

J. Crannie  
Files  
Record Services

**K.G. Selby,  
Supervising Engineer.**

Mr. J. MacDougall,  
Construction Engineer,  
New Liskeard.

Construction office,  
Third Floor, Central Bldg.

January 23, 1975.

Englehart River Bridge, District 14,  
Contract 74-105, Site 47-32.

---

After you had left the above site, August Radkowski, Ken Selby and myself discussed the backfilling of the footing excavations and the required dressing of the slopes under the new structure, quite extensively.

It was plain to see that the native material that was excavated for the footings would not be satisfactory to use as backfill due to its inability to be compacted in its present state and therefore it is recommended that the contractor use a borrowed material for backfill.

The material the contractor has used for his site haul roads should be satisfactory and upon discussing the availability of this material with Bill Schafer of Dineen Construction, he said that this material could be placed for the same as Item #6 of the Contract "Granular C - backfill to culverts."

The dressing of the slopes is in the process of being checked into. Ken Selby asked Charlie Caldwell the Project Supervisor, to send him cross-sections so that a further study can be carried out. The placing of the Rip Rap is also under reconsideration, and I will keep you informed as to progress.

C.A. Cripps,  
Bridge Construction Liaison Engineer.

CAC/JC

c.c. A. Radkowski  
K. Selby ✓  
B. Davis



Copy for the information of

K. G. Selby

C. Grebski  
Structural Design Engineer  
Structural Design Section  
Central Building, Downsview

T. A. Sharpe  
District Engineer  
#14, New Liskeard, Ontario

January 24, 1975

Contract 74-195 Blanche River Bridge  
Site 47-32, Highway 11, Englehart

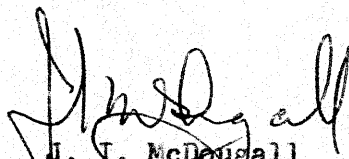
We were approached by W. Schaeffer of Dineen Construction the Contractor on the above noted contract re the rip-rap scheme as proposed.

As proposed it would be unwise to construct the rip-rap until the old bridge is out of service. An alternate scheme (indicated on the attached X sections) would:

- a) Allow the work to be carried out this winter
- b) Leave the existing root-map as is
- c) Protect the existing banks without excavating into the compact to loose saturated silt gully banks
- d) Construction during spring to fall of this work would be nigh impossible
- e) Contractor would not be able to trim up as he would have to leave his access road down the banks until the protection is completed.

The above has been discussed in the field with Messrs. Radkowski, Selby and Cripps who would be prepared to discuss this approach with yourselves and the Hydrology Section.

An early reply would be of great help.

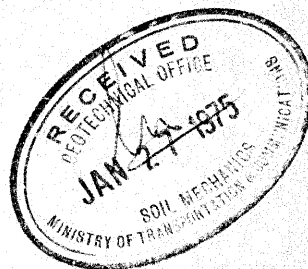
  
J. I. McDougall  
District Construction Engineer

For: T. A. Sharpe  
District Engineer

JIM/jva

Attach.

c.c. K. G. Selby ✓  
G. H. Martens





## Memorandum

To: Mr. J.I. McDougal,  
Construction Engineer,  
New Liskeard District.

From: Structural Office,  
West Building,  
Downsview, Ontario.

Attention:

Date: January 24, 1975.

Our File Ref.

In Reply to

Subject:

Englehart R. Bridge  
Cont. 74-105, Site 47-32  
Hwy. #11, District #14

This will confirm that following our discussion at the site on January 21, 1975 with Mr. K. Selby, Supervisory Foundation Engineer, and Mr. A. Cripps, Construction Liaison Engineer, present, it was agreed to drive the seven steel H piles in pier #2, for which pre-drilling is specified, in a usual manner. ?

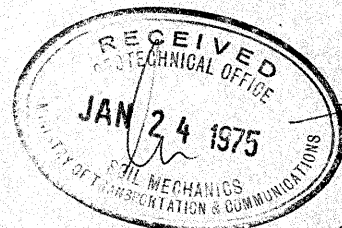
All precautionary measures that were requested while driving piles in pier #3 and outlined in my memo of December 20, 1974, are to be adhered to.

*A. Radkowski*

A. Radkowski,  
Reg. Structural Design Eng.

AR/ac

c.c. C. Grebski  
K. Selby ✓  
A. Cripps  
P.O. Law



KAS  
to  
MT  
Feb

Mr. J. McDougall,  
Construction Engineer,  
New Liskeard.

Construction office,  
Third Floor, Central Bldg.,

January 28, 1975.

Englehart River Bridge, District 14,  
Contract 74-105, Site 47-32.

---

This will confirm our discussion re backfill to footings and will correct my letter of January 23, 1975 wherein I stated that granular C was to be used.

At a meeting with Hardie Martens; Ken Selby and August Radkowski reconsidered the back fill problem and decided that granular backfill would pond water therefore native soil should be used.

It seems that this was considered at time of setting up the contract, see item 3A and page 16 of special provisions.

C.A. Cripps,  
Regional Bridge Construction Engineer.

CAC/JC

c.c. K. Selby  
A. Radkowski  
H. Martens  
B. Davis



T. A. Sharpe  
District Engineer  
District #14, New Liskeard

Soil Mechanics Section  
Geotechnical Office  
West Building, Downsview

Mr. J. McDougall

February 5, 1975

W.P. 40-70-01

ENGLEHART RIVER BRIDGE, Highway #11  
District #14, New Liskeard  
Contract 74-105

---

Further to our memo of January 23, 1975, this Section has looked into the question of treatment of the river banks at the toes of slopes on the above mentioned contract. In our view, taking into account the effect on the stability of (a) the overall slopes and (b) the partial slopes it would be advisable to change the present design and construct as follows:

1. Rip-rap (rock fill) should be placed in front of the existing sheeting to form a  $1\frac{1}{2}:1$  slope from top of sheeting (elev. 592.5<sup>+</sup>) forwards.
2. Where there is no sheeting rip-rap should be placed to form a  $1\frac{1}{2}:1$  slope from elev. 592.5 forwards. If the existing slope is flatter than  $1\frac{1}{2}:1$  from elev. 592.5 downwards, place 2 ft. of rip-rap on this part of the slope until the steepness is greater than  $1\frac{1}{2}:1$  then proceed forming a  $1\frac{1}{2}:1$  slope with rip-rap.
3. Form a suitable transition between 1 and 2 in a distance of about 10 ft.

The above recommendations have been discussed with Mr. J. Harris and Mr. A. Radkowski who are in agreement.

The foregoing will reduce the quantities and the amount of work to be done and will therefore result in a monetary saving.

When we receive the information previously requested, we will be in a position to make recommendations relating to the drainage of the overall slopes as referred to in paragraph 1 of our memo of January 23, 1975.

*K. G. Selby*  
K. G. SELBY  
Supervising Engineer.

c.c. H. Martens  
A. Radkowski  
A. Cripps  
J. Harris  
J. Crannie

Files  
Record Services



D. Laframboise  
Structural Steel Technologist  
Construction Office  
Central Building

Soil Mechanics Section  
Geotechnical Office  
West Building

April 16, 1975

**ENGLEHARDT RIVER BRIDGE**

**Structural Steel Erection  
Contract 74-105, District 14**

---

This is in reply to your request for an opinion as to the feasibility of a scheme by Dineen Construction Ltd. for slope stabilization to provide working platforms for two 110 ton crawler cranes to be used in the erection of structural steel. The scheme is outlined in two reports: Golder Associates Report #751019 and, a report by Canron Ltd. dated 20/3/75. Our comments are as follows:

1. Provided that the stabilization scheme as designed by Golder is built as designed and their recommendations followed, the scheme would be acceptable to us.
2. To construct the berms and associated drainage schemes, which involve some excavation, care must be taken not to create temporary unstable conditions. To achieve this it is our opinion that:
  - a) Work must proceed from the bottom of slope upwards.
  - b) Drainage trenches must be excavated and backfilled more or less simultaneously. Not more than 10 ft. in length of open trench should be permitted at any time.
  - c) Excavated material must be removed from the site as it is dug out. It should not be stockpiled on the slopes.
  - d) An experienced Foundation Engineer should be employed by the Contractor to supervise the work.
  - e) At completion of the steel erection work the slopes should be restored to the satisfaction of the Ministry.

K. G. SELBY  
Supervising Engineer.

c.c. H. Martens  
A. Radkowski  
J. McDougall  
J. McAllister  
  
Files  
Record Services





## Memorandum

To: K. G. Selby  
Soils Mechanics Section  
Engineering Services Branch  
West Building, Downsview

From: T. A. Sharpe  
District Engineer  
#14, New Liskeard, Ontario

Attention:

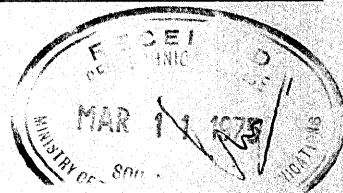
Date: February 7, 1975

Our File Ref.

In Reply to

Subject:

Contract 74-105,  
Englehart (Blanche) River Bridge



Attached please find one set of X-sections indicating original and January 31, 1975 elevations within the limits of the new structure. We have not shown a proposed laterally falling grade however we propose to contour grade probably by equipment rental this summer. Some of the materials which show up on the sections will be used for backfill to the footing excavations and thus until we find out how much material is available we will not be able to indicate any scheme. As work progresses we can keep you up to date on our proposals.

J. I. McDougall  
District Construction Engineer

For: T. A. Sharpe  
District Engineer

JIM/jva

Attach.

c.c. C. Caldwell

REPORT

TO

DINEEN ROADS & BRIDGES LIMITED

ERECTION PROCEDURE

ENGLEHART RIVER BRIDGE

ENGLEHART

ONTARIO

PREPARED

MARCH 20, 1975



EASTERN STRUCTURAL DIVISION 100 DISCO ROAD, REXDALE, ONTARIO M9W 1M1  
TELEX 86 968622 TEL. 677-2700

March 20, 1975

Dineen Roads & Bridges Limited  
70 Disco Road  
Rexdale, Ontario

Attention: Mr. W. Schafer, P. Eng.  
Project Manager

RE: Englehart River Bridge

Gentlemen:

Please find enclosed duplicate copies of Canron's erection scheme and the report prepared by Golder Associates Consulting Geotechnical Engineers on the soil stability, which outlines the access roads, we require for the erection of the structural steel.

Yours very truly

Eastern Structural Division

CANRON LIMITED

A handwritten signature in dark ink, appearing to read 'S. Weightman', written in a cursive style.

S. Weightman

Project Manager

SW/jb

Encl.



ENGLEHART RIVER BRIDGE  
ERECTION PROCEDURES

GENERAL

Erection to be done with two 110 Ton Crawler Cranes working at ground level below the bridge. Both cranes to work together to erect two spans from the east abutment to pier number 3, then move to the west side to erect two span from the abutment to pier number 2. The closing central span to be lifted by the two cranes with one on each side of the river. Girders will be delivered onto the existing bridge and picked from there, except for the spans adjacent to the piers where the girders may be delivered via the crane access roads.

SITE PREPARATIONS

Because of the nature of the site, special provisions must be made to stabilize the soil to permit the moving of cranes and to give firm foundations during the lifting of girders. These provisions are outlined in detail in the report "Stability of River Banks, Englehart River Bridge, Englehart, Ontario", dated March 14, 1975 prepared by Golder Associates, Consulting Geotechnical Engineers. In general the intention is to lower the ground water table by installing a drain down each bank. These drains will consist of ditches filled with graded sand along with tributary drains from the areas where the cranes will sit during the lifting of girders. Suitable fill is required at each crane location along with stabilizing berms on the steeper portions of the sloping banks. For crane locations see drawings ER1 and ER2, and the noted report.

March, 1975

- 2 -

STEEL ERECTION

As outlined below this scheme involves the installation of girders one at a time from the old bridge. As such the sequence of delivery of material to site is to be consistent with this scheme. If conditions should dictate a revision to the sequence of steps outlined herein or, if those girders in the end spans can be delivered to the cranes at ground level, the delivery sequence and these procedures must be altered accordingly. All anchor bolts at piers and abutments to be set before girder erection starts. All girders to be loaded on trucks with their "west" ends toward the front of the truck. Cranes to be Link Belt LS-418A with 63,000 lbs. upper counterweight AB and as per Link Belts capacity chart CRF 13023-9-73-table A for maximum capacities. Booms to be 150' except as noted in step 16.

1. Check location of bearing anchor rods.
2. With crane number 1 in position, number 1 deliver girder A30 on to the existing bridge, lift from the truck and set on the ground north of the new bridge's line.
3. With crane 2 in position 2 deliver girder C14 on to the old bridge, lift and set to the north of the new bridge line.
4. Repeat steps 1 and 2 alternately for girders A32, C16, B32 and D16.
5. Set bearing at east abutment and pier number 4 on the south girder line. Remove transit bolts from bearings.
6. Delivery girder B30, Crane number 1 to lift and land on the

March, 1975

- 3 -

---

east abutment bearing. Deliver girder D14, Crane number 2 to lift and land on pier number 4. With cranes holding loads make the splice with a minimum of 20 bolts in each flange and the web and sufficient drift pins to maintain joint alignment. Bolts to be distributed over the splices. With the cranes still holding their loads brace each girder to the existing bridge as indicated on Page ER-3. With bracing complete cranes can release loads.

7. Attach cross-frame G7 to girder B30 at the abutment and block to elevation. Attach cross-frame A8 to girder B30 at middle of span.

8. Attach cross-frame, C7 to girder D14 at pier number 1 and block to elevation.

9. Repeat steps 5, 6, 7, and 8 for bearings, girders and cross-frames to complete span. Lifting girders from the ground.

10. Fill in all bottom plan bracing and cross-frames.

11. Move crane number 1 to position number 3.

12. Repeat the operations for the second span with girders C18, C20 and D20 lifted from the old bridge to the ground by crane number 2. Girders C22, C24 and D24 lifted by crane number 1. Girders D18 and D22 go from the trucks to their positions in the bridge, are spliced and braced to the old bridge before the cranes release their loads.

13. Attach cross-frame D7 to girder D22 at pier number 3 and block to elevation. Attach cross-frame A8 to girder D18 at location closest to west end of D18.



March, 1975

- 4 -

- 
14. Repeat steps for bearings, girders and cross-frames to complete span.
  15. Fill in all top and bottom plan bracing and cross-frames.
  16. Move both cranes to the west side of the river. Crane number 1 to position number 4 and crane number 2 to position number 5.
  17. Repeat steps 1 to 14 inclusive from the west abutment to pier number 2 setting girders A10, A14, A12, A16, B12, B16, B10, B14, in the span from pier number 1 to the abutment and girders A18, A22, A20, A24, B20, B24, B18, B22 in the span from pier number 1 to pier number 2, along with all associated bracing, cross-frames and bearings.
  18. Move crane number 2 to the east side of the river to position number 8 and leave crane number 1 on the west side, but moved to position 7. Extend booms to 180'.
  19. Deliver girders B26, B28, A28 in sequence onto the existing bridge and lift with two cranes and set on the ground north of the new bridge's centre line. Note that maximum crane reaches will not permit girders to be set outside the width of the new bridge.
  20. Deliver girder A26 onto the old bridge, lift with two cranes and put into position. With cranes holding make splices and brace at mid span to the old bridge.
  21. Attach cross-frame A8 to girder A26 mid-span.
  22. Repeat steps 18 and 19 for girders A28, B28 and B26 and cross-frames A8 at mid span to complete the span. All girders lifted from the ground.





EASTERN STRUCTURAL DIVISION

March, 1975

- 5 -

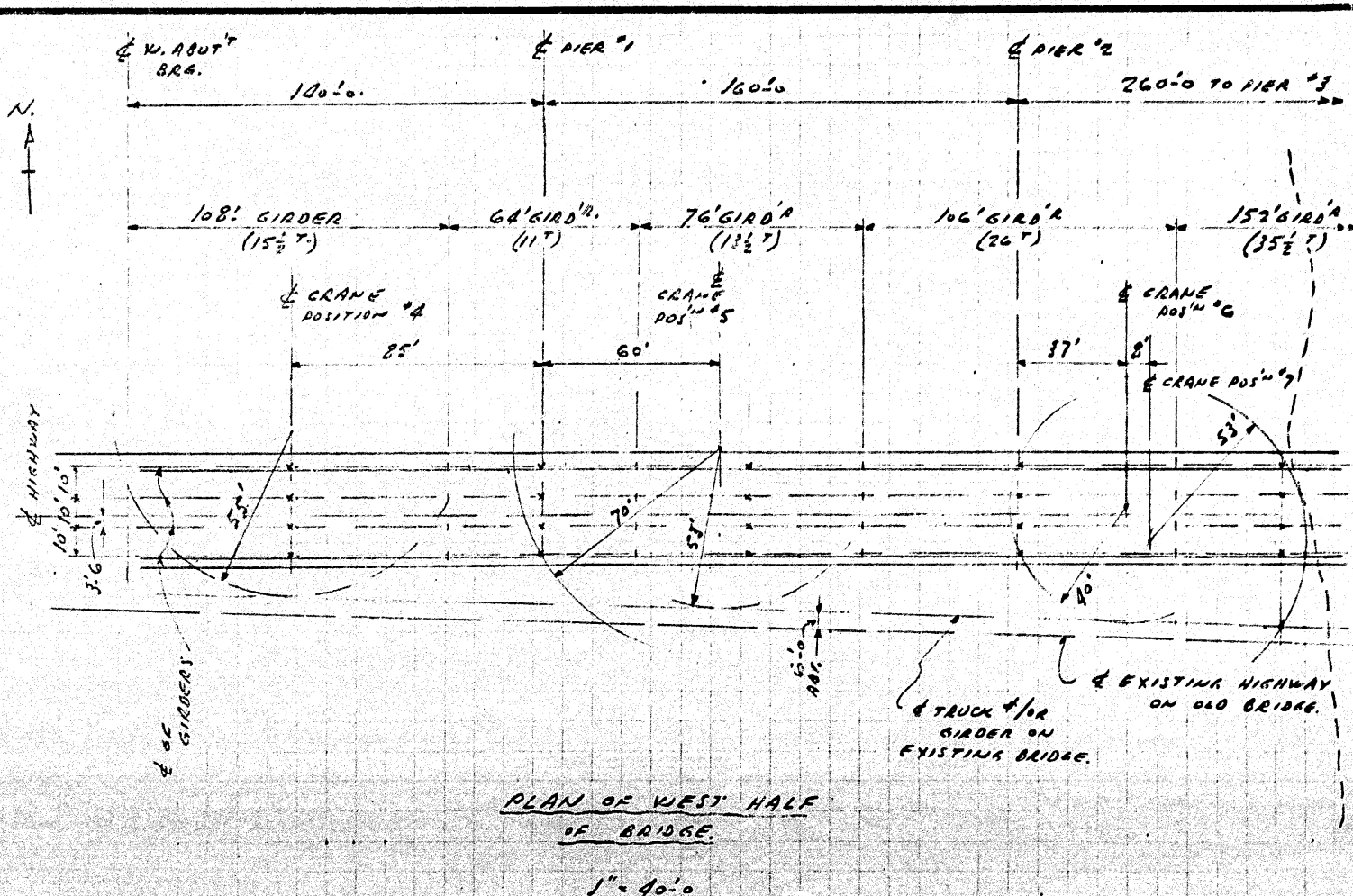
- 
23. Fill in all bottom plan bracing and cross-frames.
  24. Check setting of bearings, adjust as required as per drawing E1 and field weld.

MADE BY C.E.J. DATE .....

CHECKED BY .....

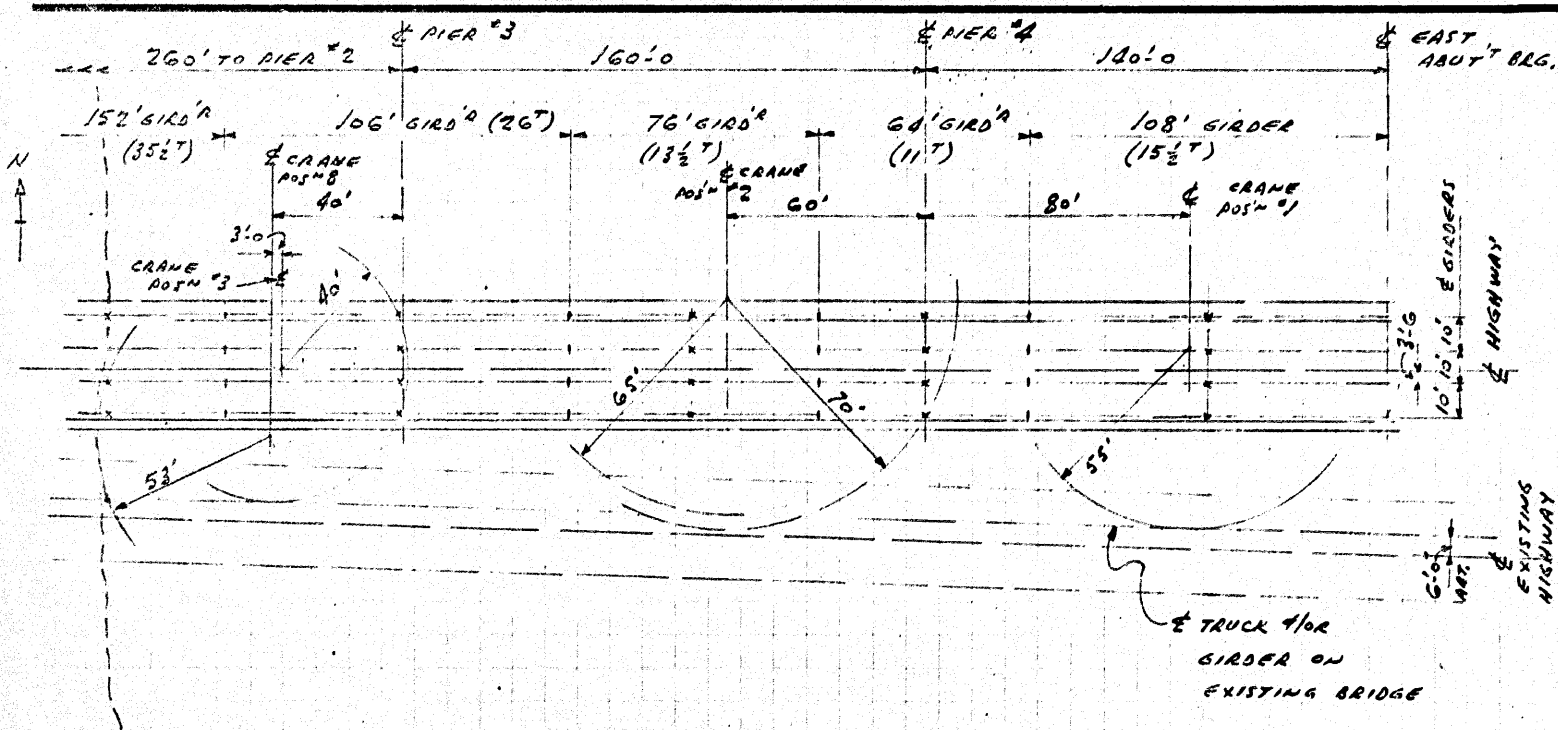
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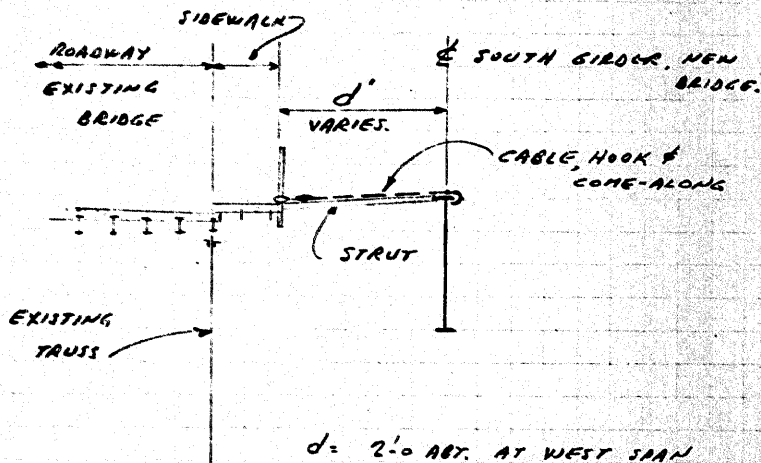


PLAN OF EAST HALF  
OF BRIDGE.

1" = 40'-0"

MADE BY G.R.S. DATE SEP. 2/15.CONT. No. 743-2685

CHECKED BY.....DATE.....

PAGE No. ER-3 OF.....

$d = 2'-0"$  ABT. AT WEST SPAN  
 $= 23'-0"$  ABT. AT EAST SPAN.

PART CROSS-SECTION THRU  
EXISTING & NEW BRIDGES.

BRACE NEW GIRDER TO EXISTING BRIDGE WITH DOUBLE ANGLE STRUT -  $2 - L^S 5 \times 3\frac{1}{2} \times \frac{3}{8}$  MINIMUM FOR EAST SPAN. ALSO, TIE NEW GIRDER TO EXISTING BRIDGE WITH CABLE & HOOK TIGHTENED BY A COME-ALONG, TURNBUCKLE OR SIMILAR MEANS.

CONNECT STRUT & TIE TO EXISTING HANDRAIL POSTS AT SIDEWALK LEVEL.

PROVIDE BRACES AT ABUTMENTS, EACH PIER AND MIDDLE OF EACH SPAN. BRACES TO BE REMOVED WHEN TWO GIRDERS ARE BRACED TOGETHER BY CROSS-FRAMES AT SUPPORT BEARINGS AND AT MID-SPAN.



**Golder Associates**  
CONSULTING GEOTECHNICAL ENGINEERS

REPORT

TO

CANRON LIMITED  
STABILITY OF RIVER BANKS  
ENGLEHART RIVER BRIDGE  
ENGLEHART                      ONTARIO

Distribution;

6 Copies - Canron Limited  
Rexdale, Ontario

2 Copies - H.Q. Golder & Associates Ltd.  
Mississauga, Ontario

March, 1975

751019



**Golder Associates**  
CONSULTING GEOTECHNICAL ENGINEERS

March 14, 1975

Canron Limited  
Eastern Structural Division  
100 Disco Road  
REXDALE, Ontario  
M9W 1H1

ATTENTION: Mr. Wayne W. Baigent, P. Eng.  
Manager of Engineering

RE: STABILITY OF RIVER BANKS  
ENGLEHART RIVER BRIDGE  
ENGLEHART, ONTARIO

Dear Sirs:

This letter reports the results of stability analyses carried out for the proposed Highway 11 bridge crossing over the Englehart River, near Englehart, Ontario.

TERMS OF REFERENCE

H.Q. Golder & Associates Ltd. have been retained by Canron Limited to assess the stability of the river bank slopes under the loading of a crawler crane which is to be used for the erection of the proposed bridge. The geotechnical aspects of the erection of the bridge were discussed at a meeting between Mr. K. Selby (M.T.C.), Mr. W.W. Baigent (Canron Limited) and Dr. H.Q. Golder (Golder Associates), held in the M.T.C. offices at Downsview, Ontario on January 31, 1975. During the meeting, details



of the proposed construction were provided to us, including the erection crane loading conditions and the soil and ground-water conditions along the river banks as outlined in a report prepared by M.T.C. (Foundation Investigation Report For The Proposed Englehart River Bridge (Blanche River West) of Hwy. #11, District #14, New Liskeard, W.O. 72-11086, W.P. 40-70-01, dated November 8, 1972). Based on this information, we were requested to determine whether the river bank slopes would be stable under the crawler crane loadings and to provide recommendations regarding methods of stabilizing the slopes, if required, during construction.

#### DESCRIPTION OF SITE

The site is located at the Highway 11 crossing over the Englehart River (also called the Blanche River) near Englehart, Ontario. We have not inspected the site; however, the M.T.C. report indicates that:

"The Englehart River valley at this location is approximately 800 - 900 ft. wide and some 100 - 110 ft. deep. The valley is bush and tree covered. The overall slopes of the banks are about 4 horizontal to 1 vertical; on shorter distances, however 2.5 to 1 and 3 to 1 slopes were also measured. There is ample evidence, along the slopes, of past movements which show local lunar shape failures and bulges so typical of this particular region."

#### DESCRIPTION OF PROJECT

It is understood that Canron Limited have been awarded the contract to erect the steel deck beams for the new bridge. The new bridge will be located immediately north of the existing bridge which will be dismantled following completion of the new structure.

The new bridge is to consist of an 860 foot long, 5 span structure having a 260 foot long central span, two intermediate spans of 160 feet each and two 140 foot end spans. The piers and abutments are supported on piles driven to the underlying bedrock; the piles have already been driven.

It is further understood that Canron Limited propose to erect the structural steel using two crawler cranes working from granular pads placed on the face of the existing valley slopes. Each crane will work independently, beginning at either end span and working towards the centre of the bridge. The centre span beams will be positioned by the two cranes lifting at either end of each beam. The crane and maximum load to be lifted will weigh 150 tons. This load will be carried on a minimum area of 25 feet square.

A construction access road has been built down both slopes of the river valley along the north side of the proposed bridge by the General Contractor. The road provides access along the slopes to construction traffic including concrete trucks, bulldozers and the like.

#### SUBSURFACE CONDITIONS

Complete details of the subsurface conditions encountered in the borings put down at the site are given in the M.T.C. report. The following is a brief summary of the soil and groundwater conditions at the site.

The overburden at the site consists of two basic strata, namely, a surficial deposit of very loose to compact sandy silt to silt with some clay, and an underlying extensive deposit of firm to very stiff varved clay. The silt deposit

varies in thickness from about 8 feet along the floor of the valley to 90 to 100 feet at the proposed abutment locations. The varved clay deposit is generally about 120 feet thick and overlies dolomitic limestone bedrock.

The groundwater level along the valley slopes, as given in the M.T.C. report, varies between 3 and 17 feet below ground surface. It is understood, however, that water seeps out of the slopes throughout most of the year; the seepage probably occurs along the more pervious layers of the stratified silt deposit.

#### SLOPE STABILITY ANALYSES

We have not put down any borings at the site nor have we carried out any laboratory tests on samples from the site. However, we have carried out effective stress stability analyses (circular and non-circular arcs) for varying slope geometry, groundwater conditions and soil strength parameters as determined from the M.T.C. report, and surface loading conditions obtained from Canron Limited. The results of these analyses are presented on Figures 2 to 6 inclusive, and are summarized on Table I following the text of this report.

The effective stress parameters used by M.T.C. in assessing the stability of the slopes were as follows:

Upper sandy silt and silt	- $c' = 504$ pounds per square foot
	$\phi' = 35$ degrees
Varved clay	- $c' = 620$ pounds per square foot
	$\phi' = 25$ degrees

Based on the data provided in the report, it is our opinion that the above values represent the upper limit. Therefore, we have assumed the following parameters in assessing the slope stability:

Upper sandy silt and silt -  $c' = 0$

$\phi' = 30$  degrees

Varved clay

-  $c' = 200$  to  $400$  pounds per square foot

$\phi' = 25$  degrees

The analyses point out the marked effect of the groundwater level on the slope stability; if the groundwater can be lowered some 4 to 5 feet below the surface of the slope, the slope becomes appreciably more stable.

The analyses also indicate that the most critical portion of the slopes is between piers 1 and 2 where the slope is the steepest, that is, about 2.5 horizontal to 1 vertical. In this area, the computations show that the slope will not be stable under the proposed crane loading (see Figure 3). However, the slope can be stabilized by provision of a berm adjacent to the crane base and by lowering the groundwater as discussed under "Recommendations". The upper parts of the slopes (between the abutment and the first pier) are stable while the portion immediately adjacent to the river channel is marginally stable if the groundwater is at the surface of the slope.

#### RECOMMENDATIONS

Based on the results of our analyses of the river bank slopes, we recommend that the following measures be taken to stabilize the slopes during erection of the proposed bridge.

1. Drainage trenches must be provided along the slopes to ensure the groundwater level at the crane operating positions is maintained at least 4 feet below existing grade or the underside of the granular pad, whichever is lower. The drainage trenches could consist of a collector drain located along the north side of the proposed bridge and extending from above the abutment locations to the river with a series of diagonal drains leading to the collector at each crane operating position. A possible drain configuration is indicated on Figure 1. The drainage trenches should be at least 4 to 5 feet deep and should be backfilled with granular filter material meeting the gradation specification indicated on Figure 8. Since the trenches will extend below the groundwater level in silts and sands, (at least during periods of high groundwater levels) sloughing of the sides of the trenches must be anticipated. Therefore, it will probably be necessary to employ relatively wide trenches to achieve the required depth. The trenches should be installed as soon as possible to allow drainage to occur for at least several weeks prior to construction.
2. The granular pads for the crane at the upper position (between abutment and first pier) and adjacent to the river banks should have a minimum thickness of 3 feet (see Figure 7). It is imperative that the crane not be allowed any nearer to the crest of the river channel slopes than is indicated on Figures 6 and 7.
3. A stabilizing berm having a minimum crest width of 10 feet must be provided on the lower side of the crane base between piers 1 and 2 and between piers 3 and 4. The configuration of the required crane bases and stabilizing berms are given on Figures 5 and 7. The crane bases and berms should be constructed of well compacted granular fill and should be constructed as one unit beginning from the toe of the berm.

4. Although we have not seen the construction roads down the river valley slopes, it is assumed that they will be suitable for moving the cranes up and down the slopes.

5. The installation of the drainage trenches and the crane bases/berms should be inspected by a qualified engineer.

We trust that this report provides sufficient information for your present requirements. If you have any questions regarding any aspect of the report, or if we can be of any further service to you on this project, please call us.

Yours truly,

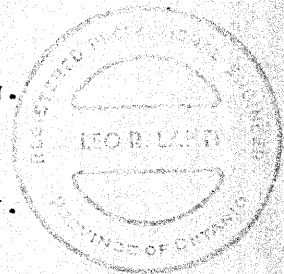
H.Q. GOLDER & ASSOCIATES LTD.

*L.R. Lahti*

L.R. Lahti, P. Eng.

*J.B. Davis*

J.B. Davis, P. Eng.



LRL:JBD:def  
751019

Enclosures - Abbreviations and Symbols  
Table I  
Figures 1 to 8



## LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

### I. SAMPLE TYPES

AS auger sample  
 CS chunk sample  
 DO drive open  
 DS Denison type sample  
 FS foil sample  
 RC rock core  
 ST slotted tube  
 TO thin-walled, open  
 TP thin-walled, piston  
 WS wash sample

### II. PENETRATION RESISTANCES

**Dynamic Penetration Resistance:** The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

**Standard Penetration Resistance, *N*:** The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

*WH* sampler advanced by static weight—weight, hammer  
*PH* sampler advanced by pressure—pressure, hydraulic  
*PM* sampler advanced by pressure—pressure, manual

### III. SOIL DESCRIPTION

#### (a) *Cohesionless Soils*

<i>Relative Density</i>	<i>N, blows/ft.</i>
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) *Cohesive Soils*

<i>Consistency</i>	<i>c<sub>u</sub>, lb./sq. ft.</i>
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

### IV. SOIL TESTS

*C* consolidation test  
*H* hydrometer analysis  
*M* sieve analysis  
*MH* combined analysis, sieve and hydrometer<sup>1</sup>  
*Q* undrained triaxial<sup>2</sup>  
*R* consolidated undrained triaxial<sup>2</sup>  
*S* drained triaxial  
*U* unconfined compression  
*V* field vane test

### NOTES:

<sup>1</sup>Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

<sup>2</sup>Undrained triaxial tests in which pore pressures are measured are shown as  $\bar{Q}$  or  $\bar{R}$ .

## LIST OF SYMBOLS

### I. GENERAL

$\pi$	= 3.1416
$e$	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of $a$
$\log_{10} a$ or $\log a$	logarithm of $a$ to base 10
$t$	time
$g$	acceleration due to gravity
$V$	volume
$W$	weight
$M$	moment
$F$	factor of safety

### II. STRESS AND STRAIN

$u$	pore pressure
$\sigma$	normal stress
$\sigma'$	normal effective stress ( $\bar{\sigma}$ is also used)
$\tau$	shear stress
$\epsilon$	linear strain
$\epsilon_{xy}$	shear strain
$\nu$	Poisson's ratio ( $\mu$ is also used)
$E$	modulus of linear deformation (Young's modulus)
$G$	modulus of shear deformation
$K$	modulus of compressibility
$\eta$	coefficient of viscosity

### III. SOIL PROPERTIES

(a) <i>Unit weight</i>	
$\gamma$	unit weight of soil (bulk density)
$\gamma_s$	unit weight of solid particles
$\gamma_w$	unit weight of water
$\gamma_d$	unit dry weight of soil (dry density)
$\gamma'$	unit weight of submerged soil
$G_s$	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
$e$	void ratio
$n$	porosity
$w$	water content
$S_r$	degree of saturation

#### (b) *Consistency*

$w_L$	liquid limit
$w_P$	plastic limit
$I_P$	plasticity index
$w_s$	shrinkage limit
$I_L$	liquidity index = $(w - w_P) / I_P$
$I_C$	consistency index = $(w_L - w) / I_P$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$D_r$	relative density = $(e_{max} - e) / (e_{max} - e_{min})$

#### (c) *Permeability*

$h$	hydraulic head or potential
$q$	rate of discharge
$v$	velocity of flow
$i$	hydraulic gradient
$k$	coefficient of permeability
$j$	seepage force per unit volume

#### (d) *Consolidation (one-dimensional)*

$m_v$	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
$C_c$	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
$c_c$	coefficient of consolidation
$T_v$	time factor = $cv/d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

#### (e) *Shear strength*

$\tau_f$	shear strength
$c'$	effective cohesion
$\phi'$	effective angle of shearing resistance, or friction
$c_u$	apparent cohesion*
$\phi_u$	apparent angle of shearing resistance, or friction
$\mu$	coefficient of friction
$S_i$	sensitivity

\*For the case of a saturated cohesive soil,  $\phi_u = 0$  and the undrained shear strength  $\tau_f = c_u$  is taken as half the undrained compressive strength.

TABLE I

Sheet 1 of 3

## SUMMARY OF STABILITY ANALYSES

Analysis	Loading on Slope	Depth to Groundwater (feet)	SOIL PARAMETERS						Factor of Safety	Reference Figure
			Silt			Varved Clay				
			$\phi'$ Deg.	c' psf	$\gamma$ pcf	$\phi'$ Deg.	c' psf	$\gamma$ pcf		
1.Deep seated stability a) Failure in varved clay b) Failure at varved clay/silt contact  2.Localized failure in silt (steepest part of slope - pier 1 to pier 2)	Nil	0	35	0	120	25	0	110	1.2	2
		0	35	0	120	25	200	110	1.3	
		0	35	0	120	25	400	110	1.5	
		9	35	0	120	25	0	110	1.3	
		9	35	0	120	25	200	110	1.5	
		9	35	0	120	25	400	110	1.7	
	Nil	0	35	0	120	25	0	110	1.0	2
		0	35	0	120	25	200	110	1.2	
		0	35	0	120	25	400	110	1.3	
		9	35	0	120	25	0	110	1.4	
		9	35	0	120	25	200	110	1.6	
		9	35	0	120	25	400	110	1.7	
	3' Gravel + Crane	0	35	0	120	-	-	-	0.9	3
		4	35	0	120	-	-	-	1.3	
		9	35	0	120	-	-	-	1.7	
		0	35	200	120	-	-	-	1.0	
		4	35	200	120	-	-	-	1.5	
		0	32.5	200	120	-	-	-	1.0	
		4	32.5	200	120	-	-	-	1.4	
		0	30	200	120	-	-	-	1.0	
		4	30	200	120	-	-	-	1.3	
		0	35	0	120	-	-	-	0.8	
		4	35	0	120	-	-	-	1.1	
		9	35	0	120	-	-	-	1.4	
		0	32.5	0	120	-	-	-	0.7	
		4	32.5	0	120	-	-	-	1.0	
		0	30	0	120	-	-	-	0.7	
		4	30	0	120	-	-	-	0.9	

TABLE I

Sheet 2 of 3

## SUMMARY OF STABILITY ANALYSES

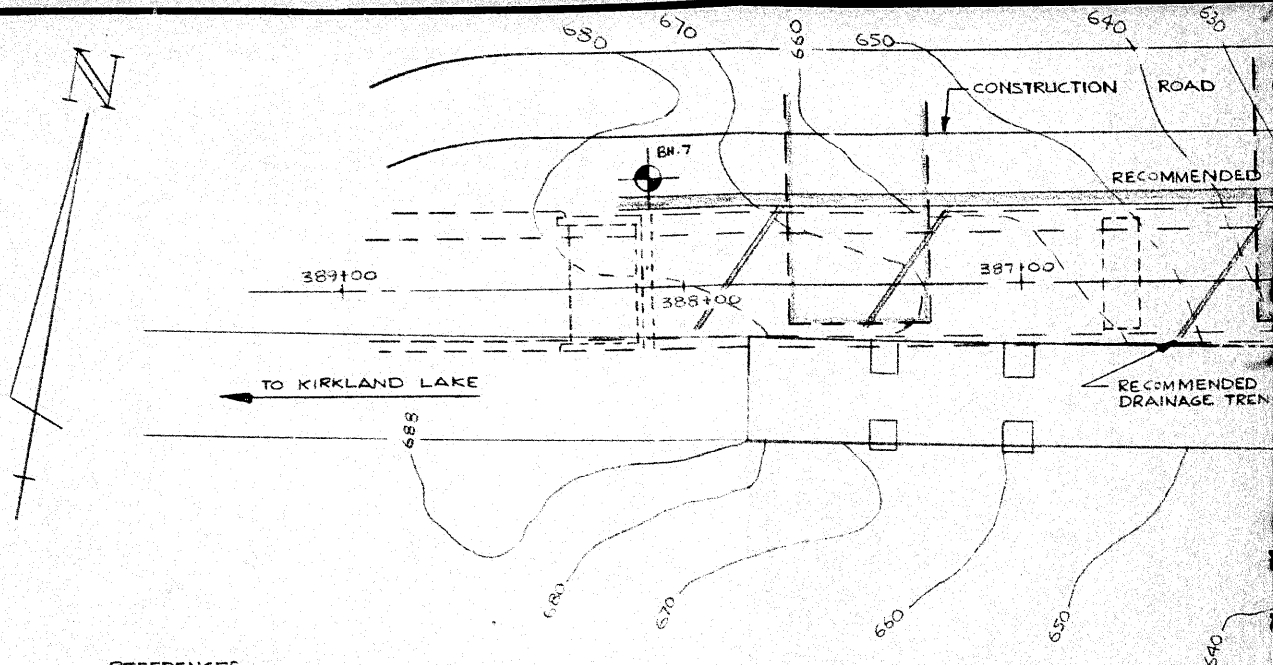
Analysis	Loading on Slope	Depth to Groundwater (Feet)	SOIL PARAMETERS						Factor of Safety	Reference Figure
			Silt			Varved Clay				
			Ø' Deg.	c' psf	γ pcf	Ø' Deg.	c' psf	γ pcf		
3.Localized failure in silt (upper part of slope-west abutment to pier 1)	Gravel + Crane	0	35	0	120	-	-	-	1.5	4
		4	35	0	120	-	-	-	1.9	
		0	30	0	120	-	-	-	1.2	
		4	30	0	120	-	-	-	1.5	
		0	27	0	120	-	-	-	1.1	
		4	27	0	120	-	-	-	1.3	
4.Localized failure in silt (steepest part of slope-pier 1 to pier 2)	Gravel Pad	0	35	0	120	-	-	-	1.0	5
		4	35	0	120	-	-	-	1.3	
		9	35	0	120	-	-	-	1.7	
		0	30	0	120	-	-	-	0.9	
		4	30	0	120	-	-	-	1.1	
		9	30	0	120	-	-	-	1.4	
		0	27	0	120	-	-	-	0.8	
		4	27	0	120	-	-	-	1.0	
		9	27	0	120	-	-	-	1.2	
	Gravel Pad + 10' Berm	0	35	0	120	-	-	-	1.2	
		4	35	0	120	-	-	-	1.5	
		9	35	0	120	-	-	-	1.8	
		0	30	0	120	-	-	-	1.0	
		4	30	0	120	-	-	-	1.2	
		9	30	0	120	-	-	-	1.5	
		0	27	0	120	-	-	-	0.8	
		4	27	0	120	-	-	-	1.1	
		9	27	0	120	-	-	-	1.3	

TABLE I

Sheet 3 of 3

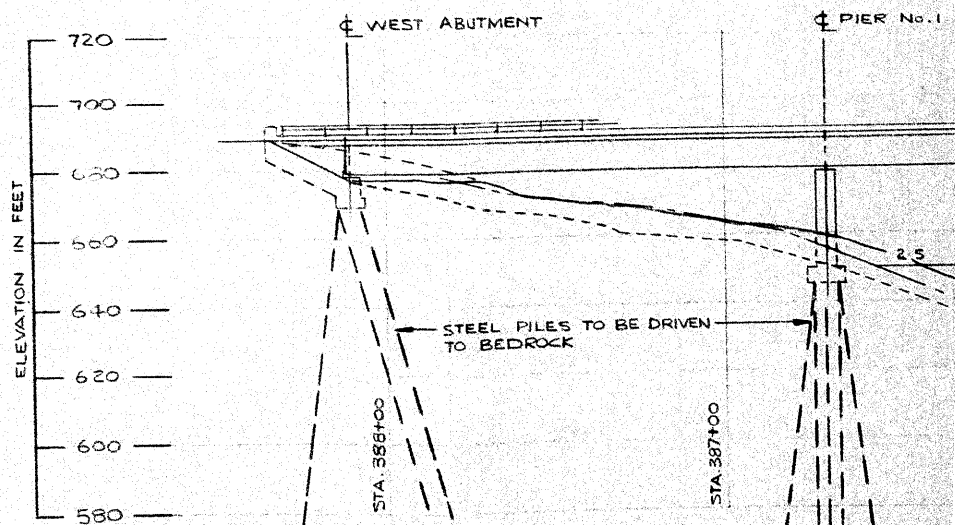
## SUMMARY OF STABILITY ANALYSES

Analysis	Loading on Slope	Depth to Groundwater (Feet)	SOIL PARAMETERS						Factor of Safety	Reference Figure	
			Silt			Varved Clay					
			Ø' Deg.	c' psf	γ pcf	Ø' Deg.	c' psf	γ pcf			
4. Localized failure in silt (steepest part of slope - pier 1 to pier 2)	Gravel Pad + Berm + Crane	0	35	0	120	-	-	-	1.2	5	
		4	35	0	120	-	-	-	1.4		
		9	35	0	120	-	-	-	1.7		
		0	30	0	120	-	-	-	1.0		
		4	30	0	120	-	-	-	1.2		
		9	30	0	120	-	-	-	1.4		
		0	27	0	120	-	-	-	0.9		
		4	27	0	120	-	-	-	1.1		
		9	27	0	120	-	-	-	1.3		
5. River channel slope stability a) West Bank	Gravel Pad + Crane	0	35	0	120	-	-	-	1.2	6	
		4	35	0	120	-	-	-	1.6		
		0	30	0	120	-	-	-	1.1		
		4	30	0	120	-	-	-	1.4		
		0	27	0	120	-	-	-	0.9		
		4	27	0	120	-	-	-	1.2		
	b) East Bank	Gravel Pad + Crane	0	35	0	120	-	-	-		1.3
			4	35	0	120	-	-	-		1.7
			0	30	0	120	-	-	-		1.1
			4	30	0	120	-	-	-		1.4
			0	27	0	120	-	-	-		0.9
			4	27	0	120	-	-	-		1.2
0	30	0	120	25	0	110	0.8				
0	30	0	120	25	200	110	1.2				
0	30	0	120	25	400	110	1.5				
4	30	0	120	25	0	110	1.0				
4	30	0	120	25	200	110	1.3				
4	30	0	120	25	400	110	1.7				

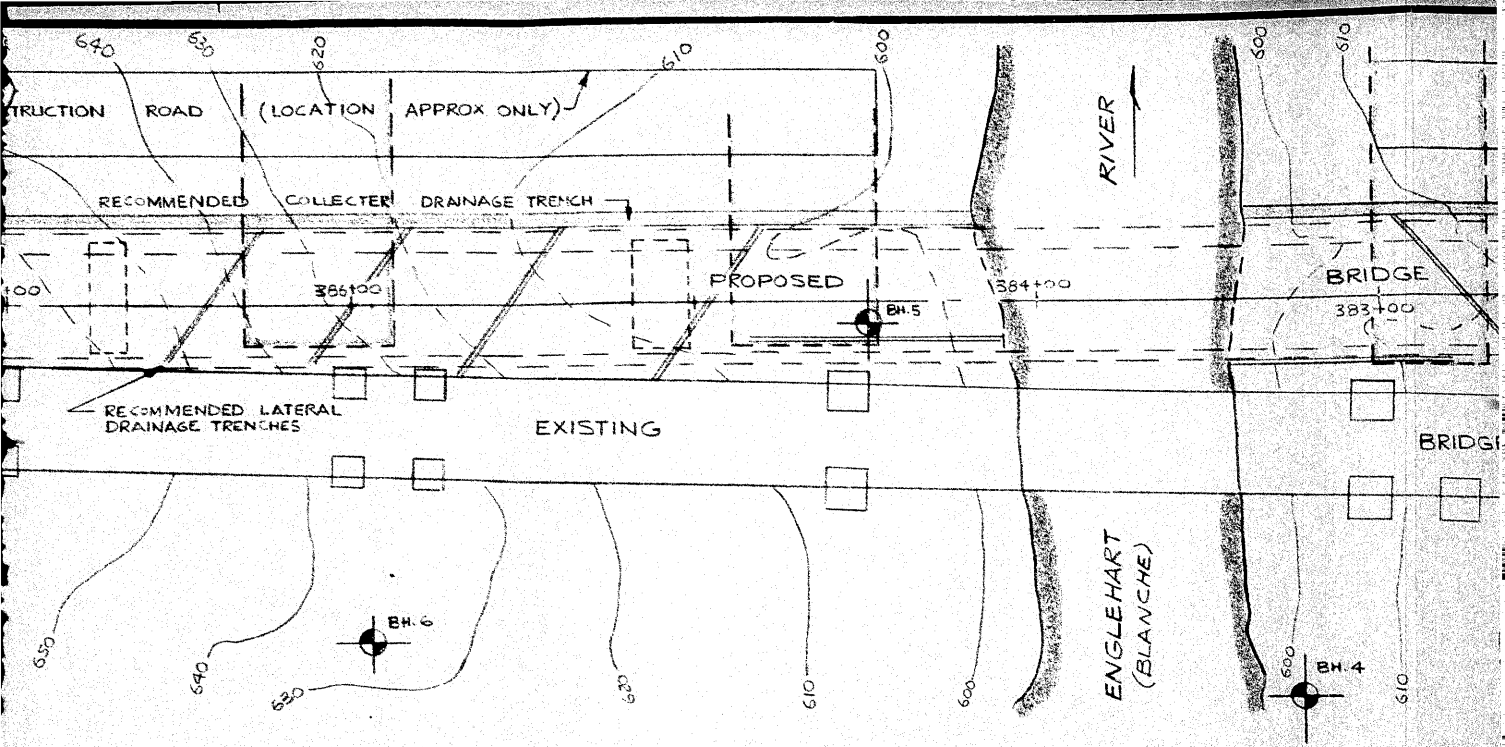


#### REFERENCES

- 1) GENERAL LAYOUT DRAWING SUPPLIED BY  
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO, TITLED "ENGLEHART RIVER BRIDGE AT ENGELHART"  
W. P. No. 40-70-01, SHEET 1, DATED NOV. 1973.
- 2) M.T.C. FOUNDATION INVESTIGATION REPORT FOR THE  
PROPOSED ENGELHART RIVER BRIDGE. W.O. 72-11086,  
W.P. 40-70-01, NOV. 8, 1972.

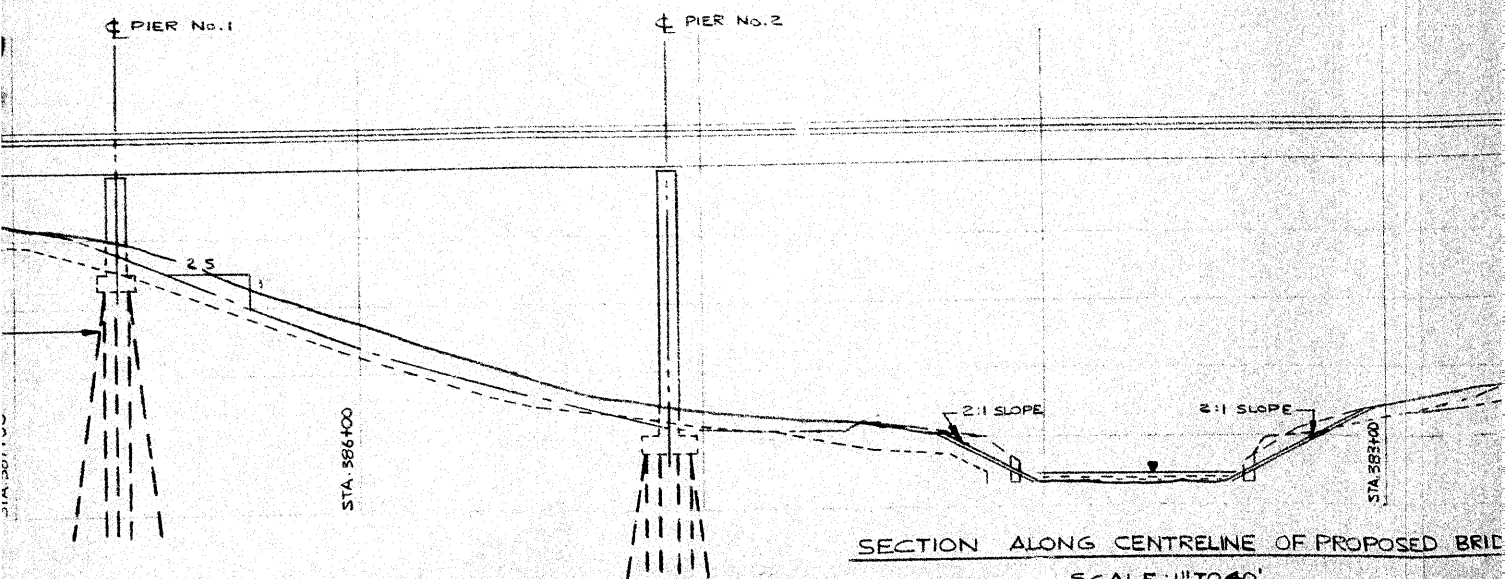






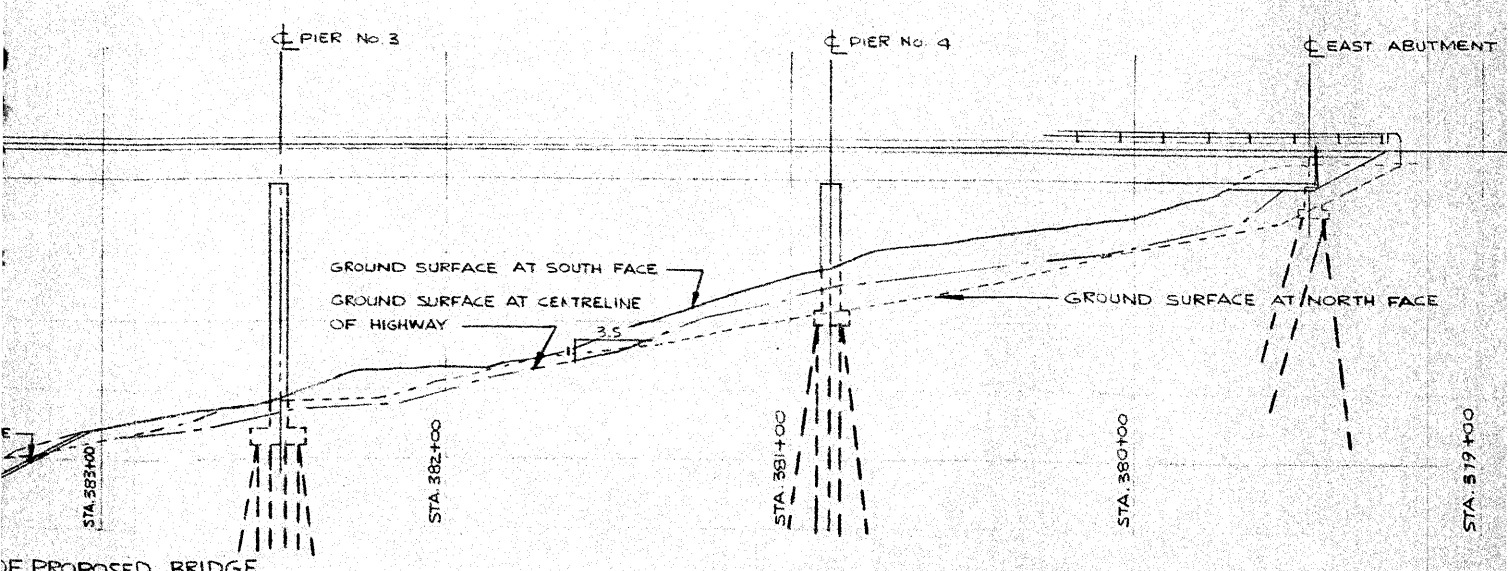
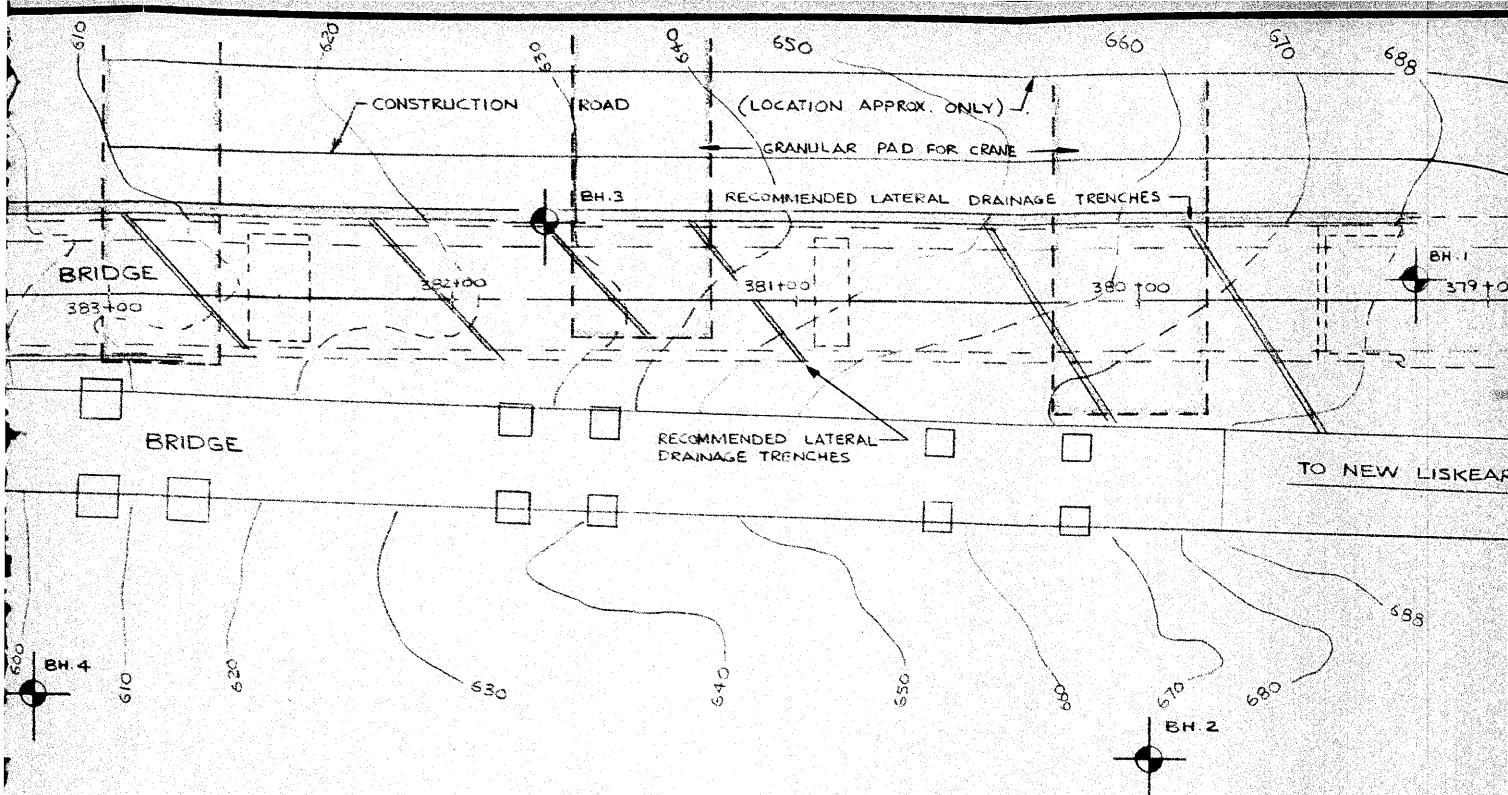
PLAN

SCALE: 1" TO 40'

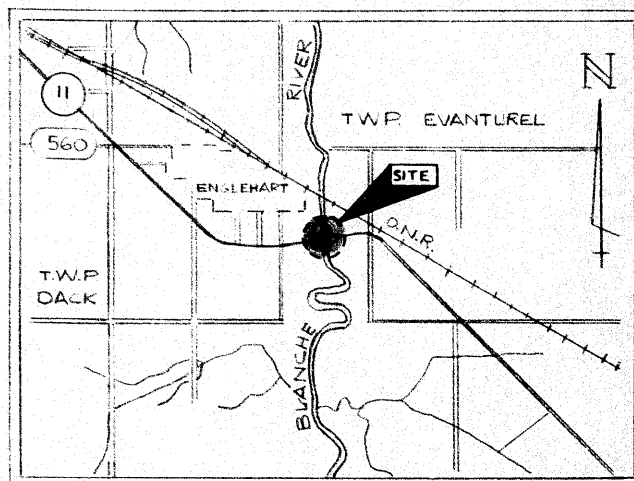
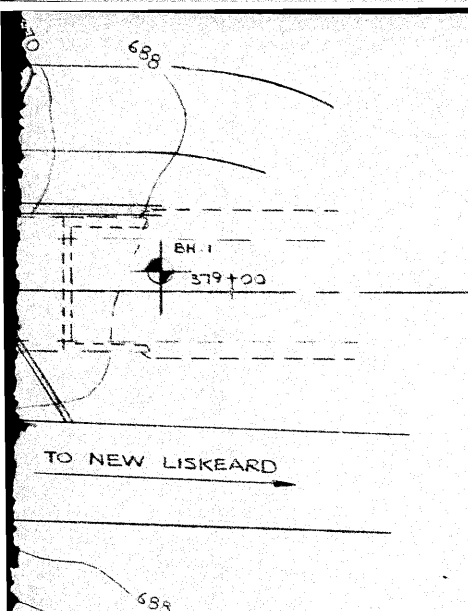


SECTION ALONG CENTRELINE OF PROPOSED BRIDGE

SCALE: 1" TO 40'



OF PROPOSED BRIDGE



KEY PLAN  
SCALE: 1" TO 4,200' (APPROX.)

### LEGEND

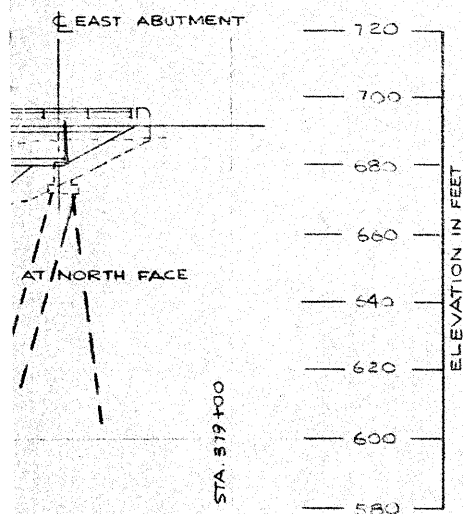


BOREHOLE LOCATION IN PLAN - M.T.C. FOUNDATION INVESTIGATION REPORT, W.O. 72-11086, W.P. 40-70-01, DATED NOV. 8, 1972.

NOTE: FOR DETAILED STRATIGRAPHY REFER TO M.T.C. FOUNDATION INVESTIGATION REPORT

#### SPECIAL NOTE

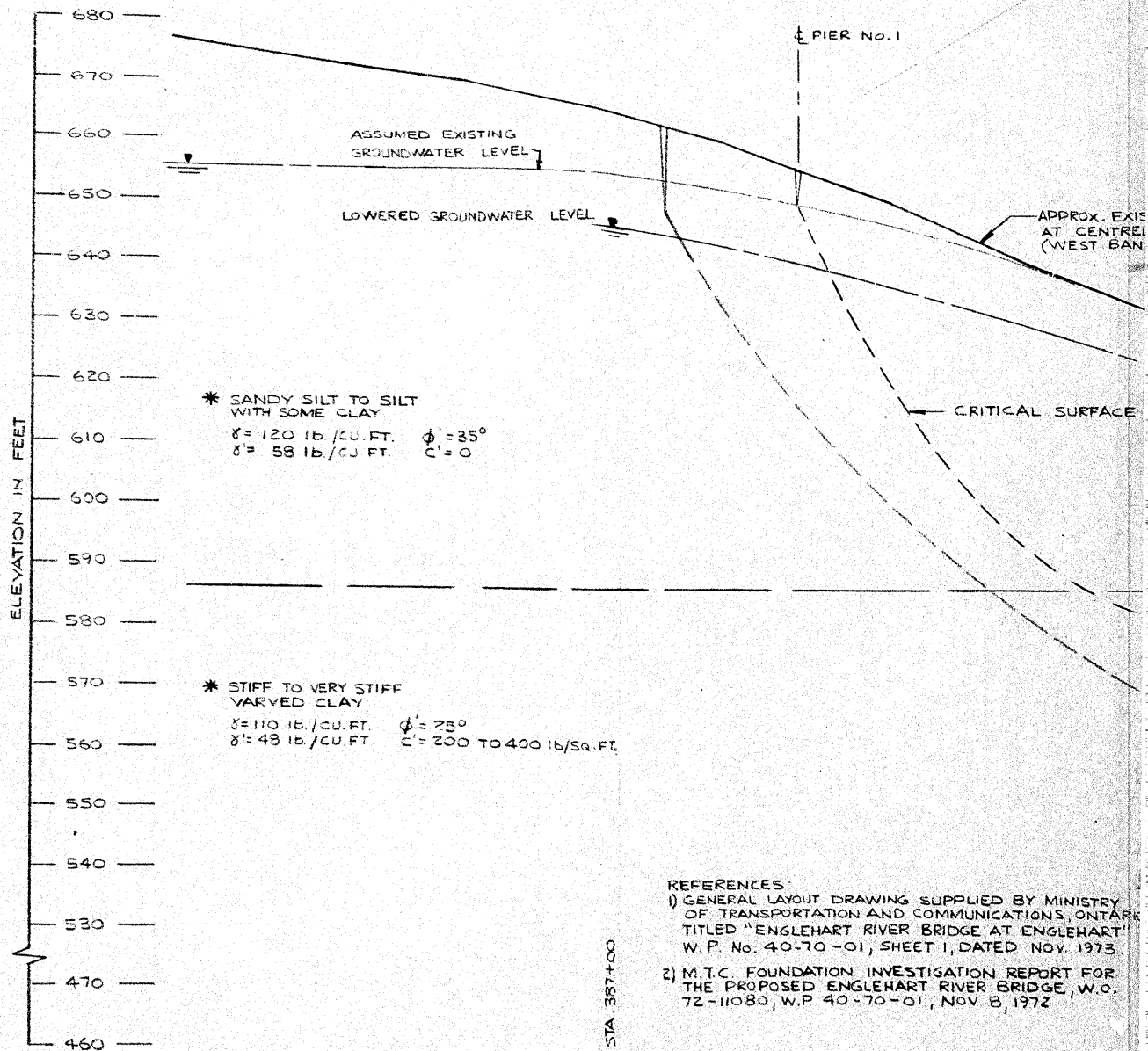
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.



Date MAR. 6, 1975

**Golder Associates**

Drawn D.M.  
Chkd. PHK  
Appd. PHK



#### REFERENCES

- 1) GENERAL LAYOUT DRAWING SUPPLIED BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO TITLED "ENGLEHART RIVER BRIDGE AT ENGELHART" W.P. No. 40-70-01, SHEET 1, DATED NOV. 1973.
- 2) M.T.C. FOUNDATION INVESTIGATION REPORT FOR THE PROPOSED ENGELHART RIVER BRIDGE, W.O. 72-11080, W.P. 40-70-01, NOV. 8, 1972

#### SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

APPROX. EXISTING GROUND SURFACE  
AT CENTRELINE OF PROPOSED BRIDGE  
(WEST BANK)

CRITICAL SURFACE 1

$d=9'$

PIER NO. 2

CRITICAL SURFACE 2

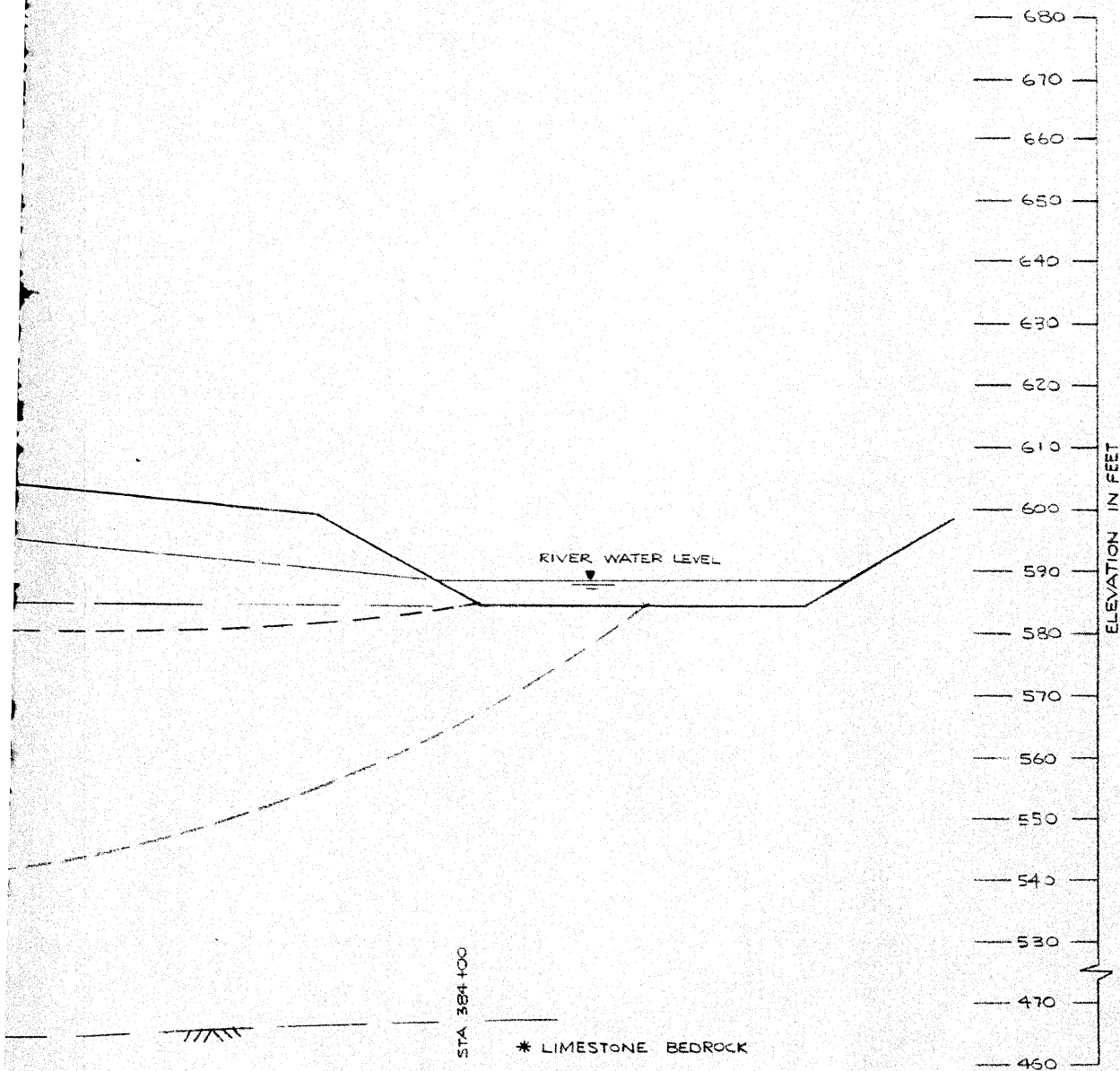
STA. 383+00

STA. 385+00

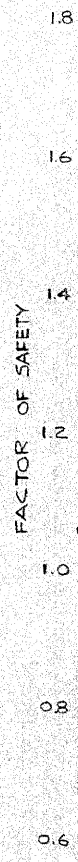
ING SUPPLIED BY MINISTRY  
D COMMUNICATIONS, ONTARIO.  
ER BRIDGE AT ENGLEHART"  
HEET 1, DATED NOV. 1973.  
ESTIGATION REPORT FOR  
ART RIVER BRIDGE, W.O.  
01, NOV 8, 1972

CONJUNCTION

SCALE: 1" TO 20'

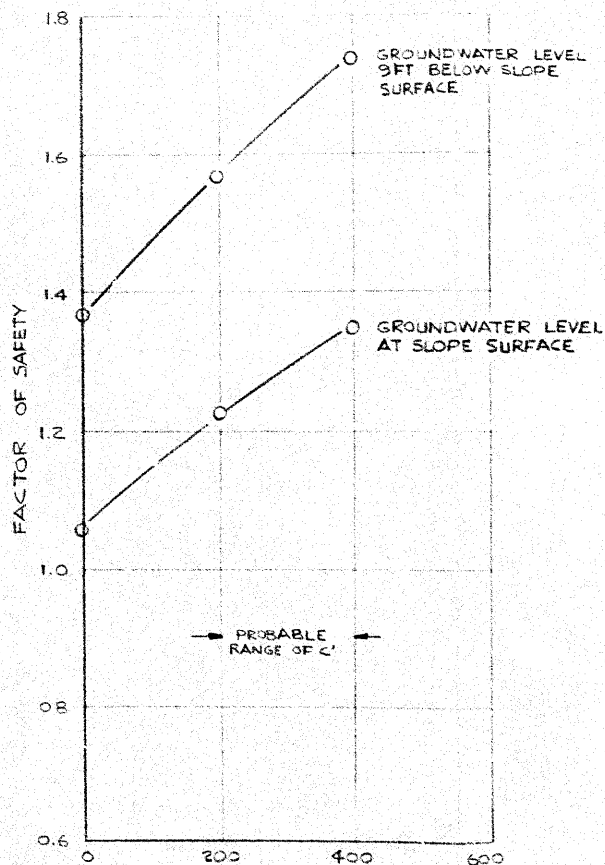


\* NOTE: FOR DETAILED STRATIGRAPHY  
REFER TO M.T.C. REPORT



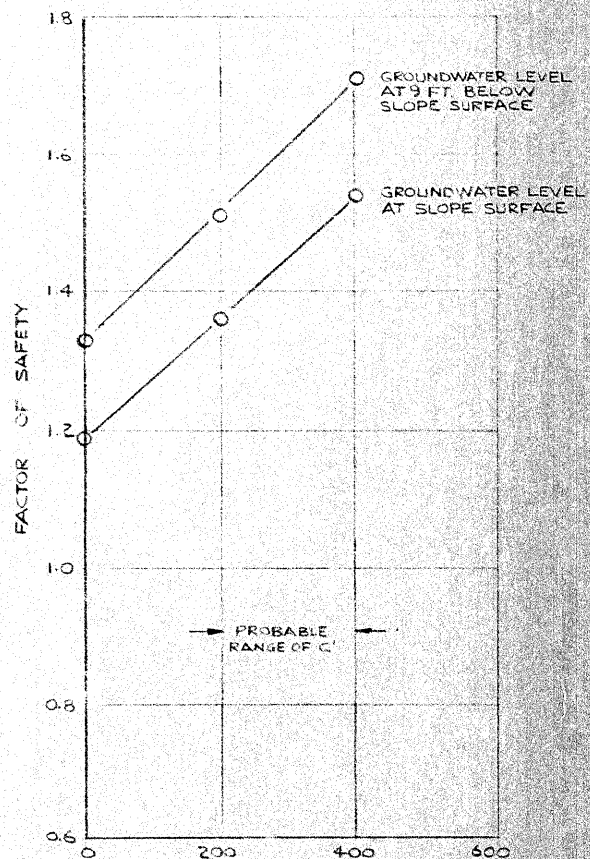


CRITICAL SURFACE 1  
NO SURCHARGE



$C'$  OF VARVED CLAY LB/SQ. FT.  
 $\phi' = 25^\circ$  (VARVED CLAY)

CRITICAL SURFACE 2  
NO SURCHARGE

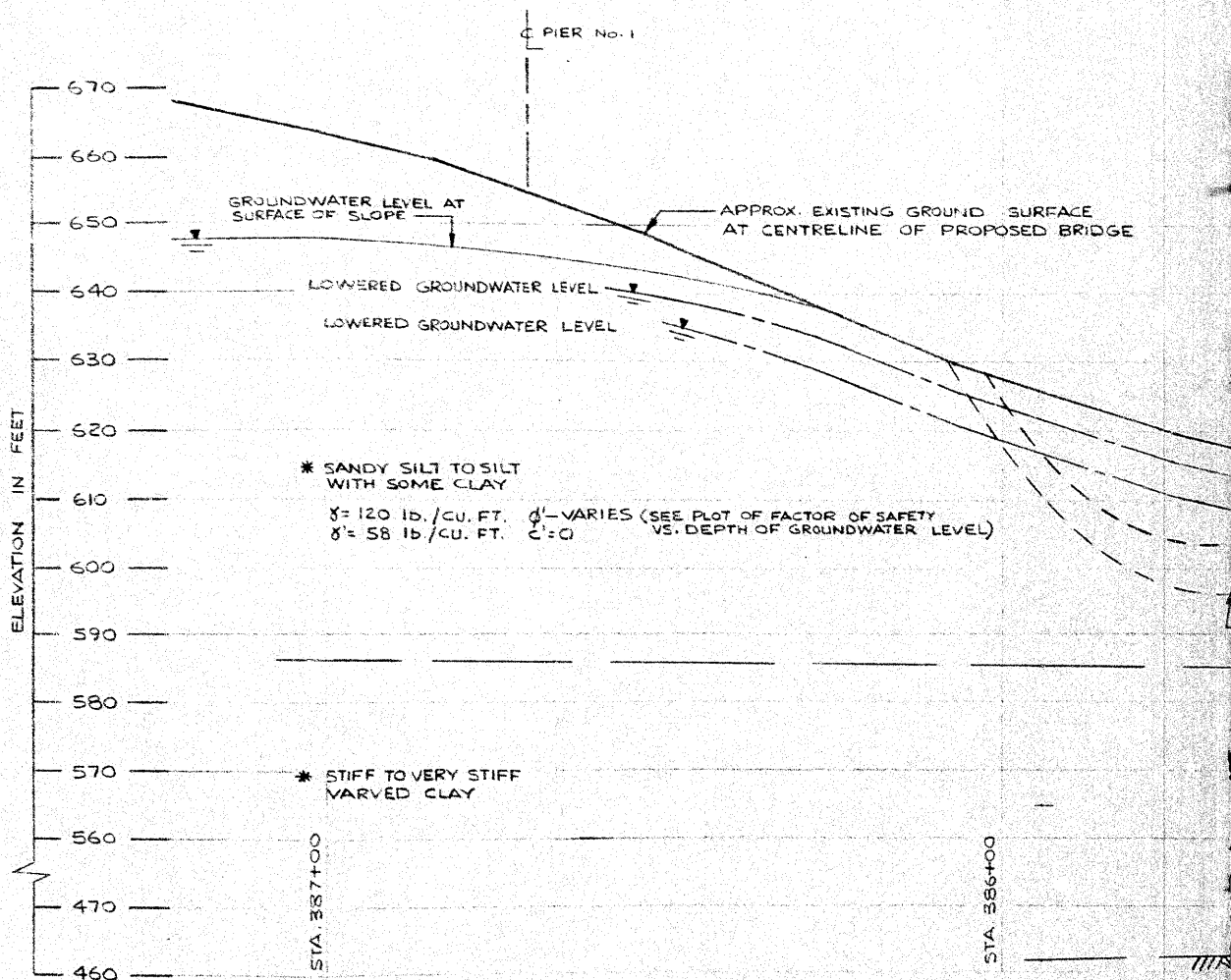


$C'$  OF VARVED CLAY LB/SQ. FT.  
 $\phi' = 25^\circ$  (VARVED CLAY)

Date MAR. 13, 1975

Golder Associates

Drawn D.M.  
Chkd. JAC  
Appd. JAC



**SPECIAL NOTE**

THIS DRAWING IS TO BE READ IN CONJUNCTION  
WITH ACCOMPANYING REPORT.

REFER  
1) GEN  
OF  
TIT  
W P  
2) M  
THE  
W. C

ND SURFACE  
PROPOSED BRIDGE

PIER NO. 2

LEVEL)

CRITICAL ZONES

d=4'  
d=9'

STA. 386+00

STA. 385+00

STA. 384+00

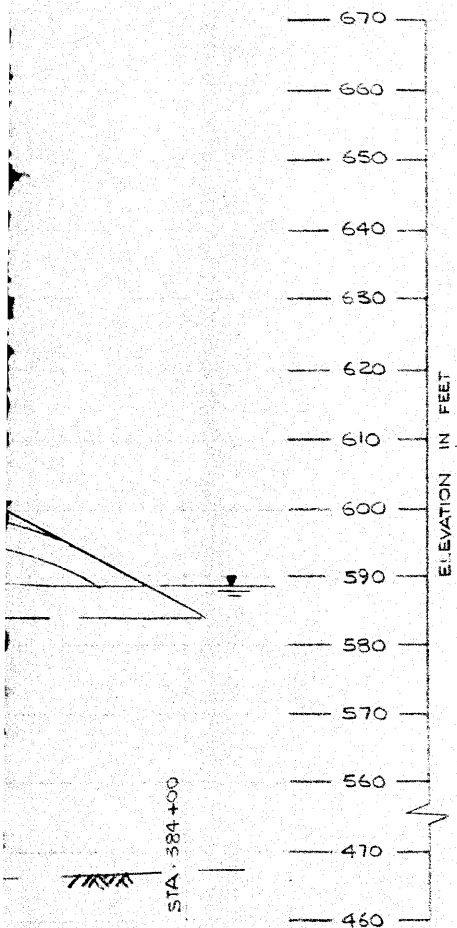
\* LIMESTONE BEDROCK

SCALE: 1" TO 20'

REFERENCES:

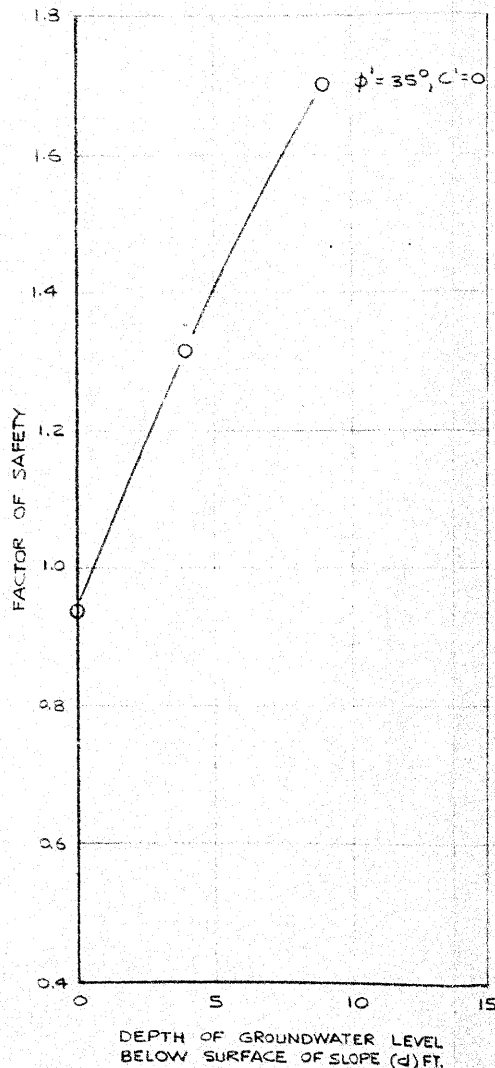
- 1) GENERAL LAYOUT DRAWING SUPPLIED BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO, TITLED "ENGLEHART RIVER BRIDGE AT ENGLEHART" W.P. NO. 40-70-01, SHEET 1, DATED NOV. 1973.
- 2) M.T.C. FOUNDATION INVESTIGATION REPORT FOR THE PROPOSED ENGLEHART RIVER BRIDGE, W.O. 72-11086, W.P. 40-70-01, NOV. 8, 1972

\* NOTE: FOR DETAILED STRATIGRAPHY REFER TO M.T.C. REPORT.

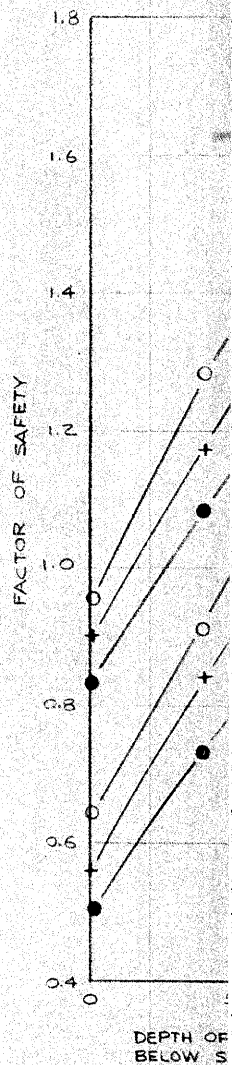


PHOTOGRAHY  
REPORT

3 FT. GRAVEL LAYER ON  
SLOPE WITHOUT CRANE



3 FT. G  
SL



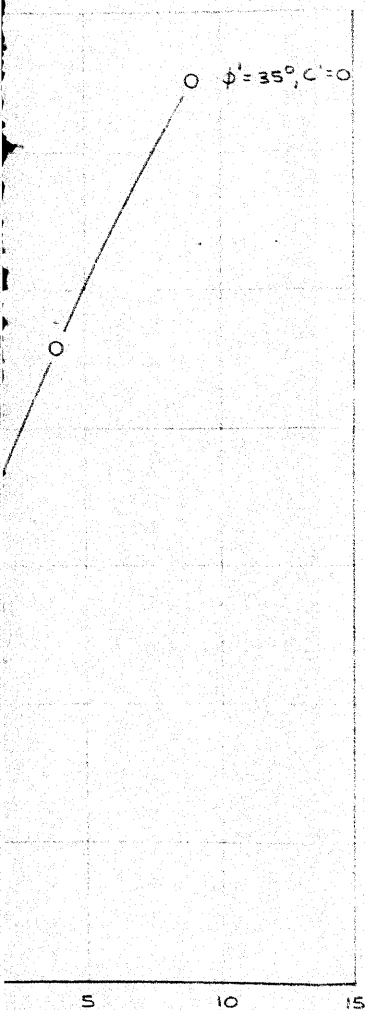
Date MAR 13, 1975

Golder A

# STABILITY ANALYSES LOCALIZED FAILURE IN SILT

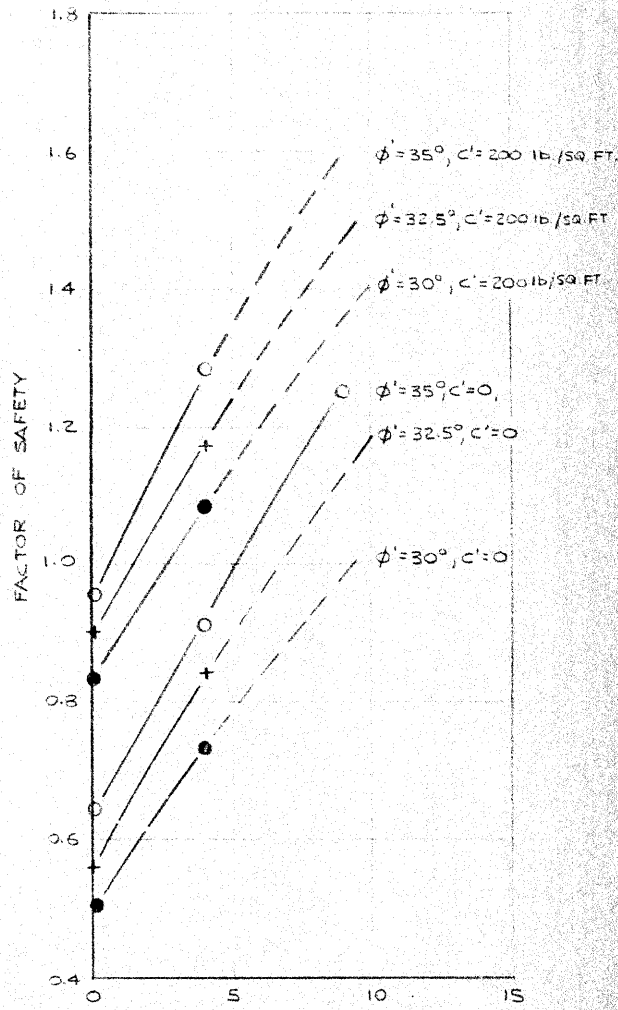
FIGURE 3

3 FT. GRAVEL LAYER ON  
SLOPE WITHOUT CRANE



DEPTH OF GROUNDWATER LEVEL  
BELOW SURFACE OF SLOPE (d) FT.

3 FT. GRAVEL LAYER ON  
SLOPE + CRANE

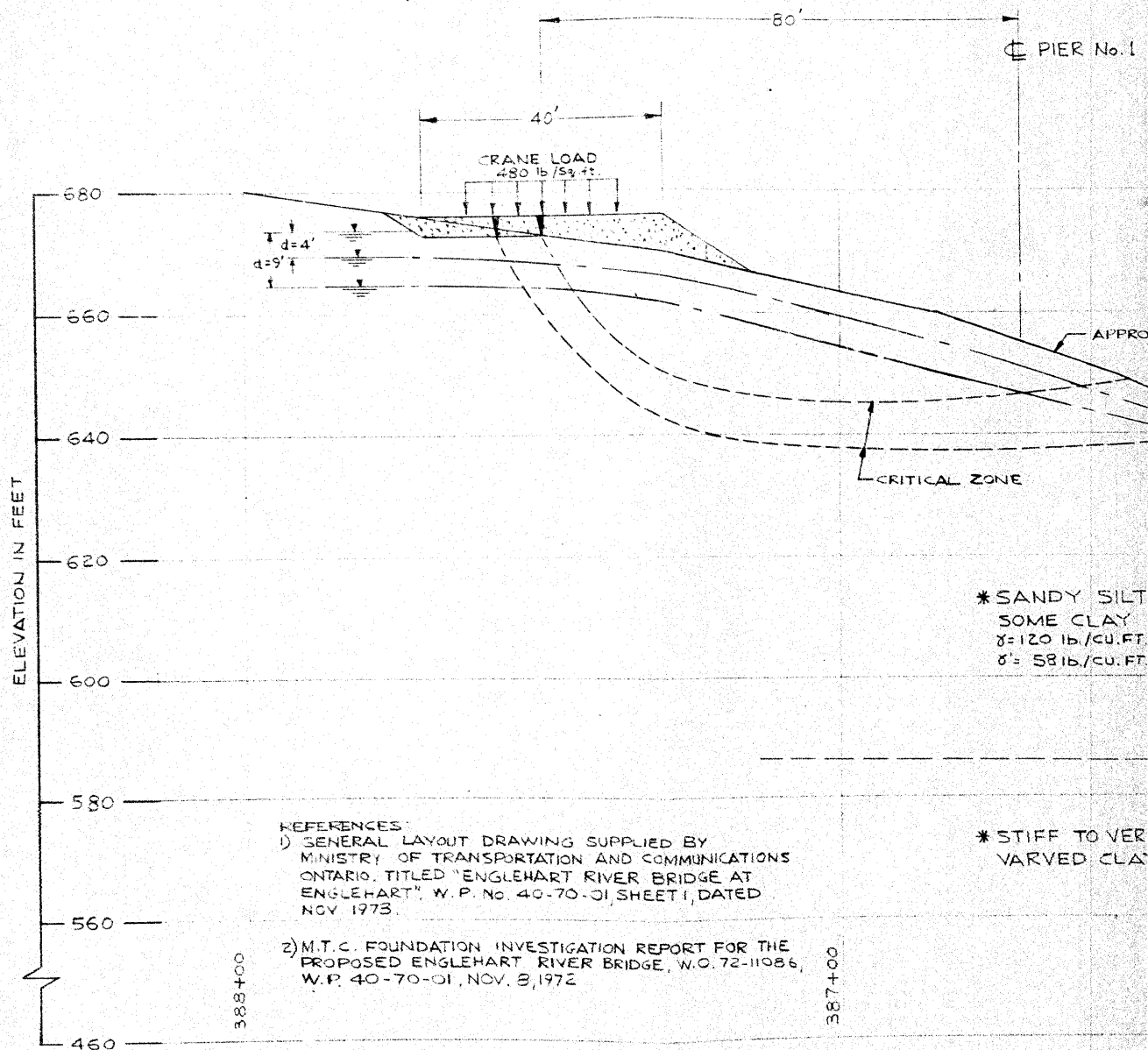


DEPTH OF GROUNDWATER LEVEL  
BELOW SURFACE OF SLOPE (d) FT.

Date MAR 13, 1975

Golder Associates

Drawn D.M.  
Chkd. INAC  
Appd. INAC



\* NOTE FOR DETAILED STRATIGRAPHY  
REFER TO M.T.C. REPORT.

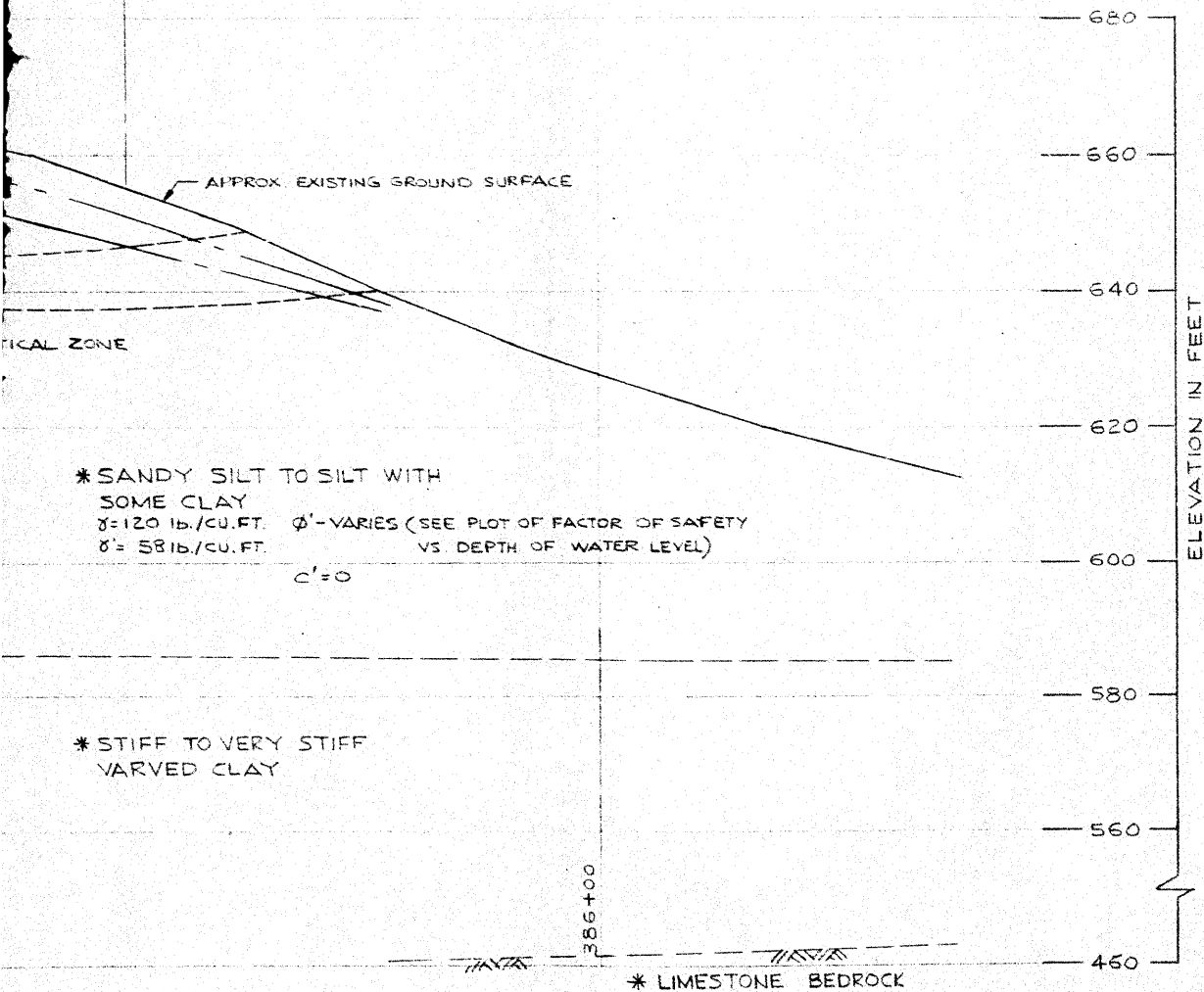
SCALE:



SPECIAL NOTE

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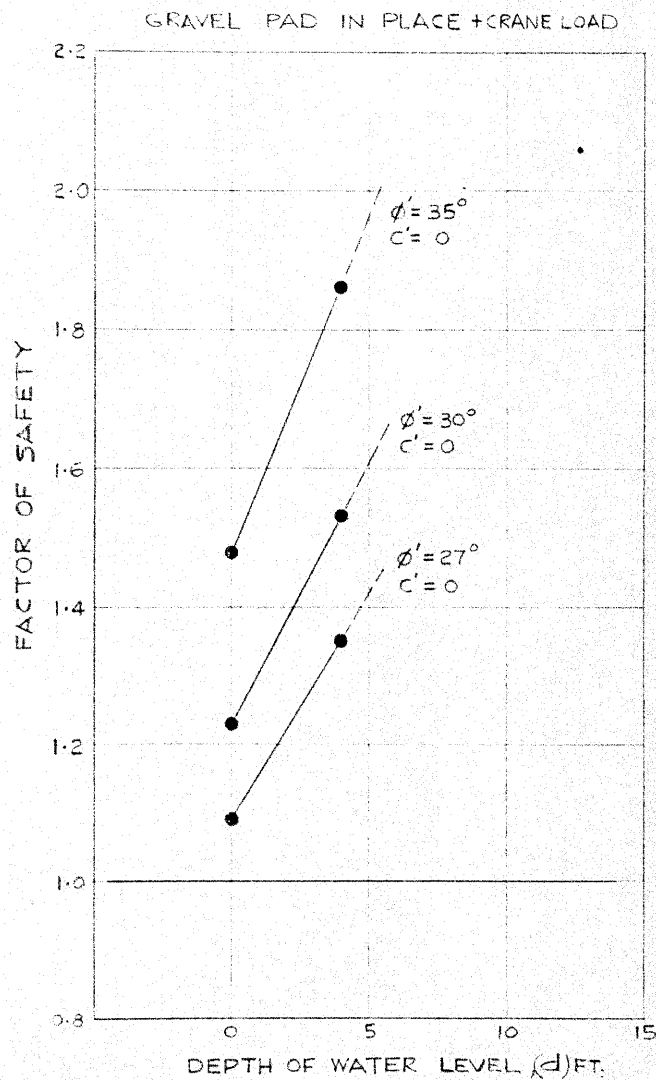
PIER No. 1



SCALE: 1" TO 20'

STABILITY ANALYSES - CRANE ON  
UPPER PART OF SLOPE  
(BETWEEN ABUTMENT AND FIRST PIER)

FIGURE 4

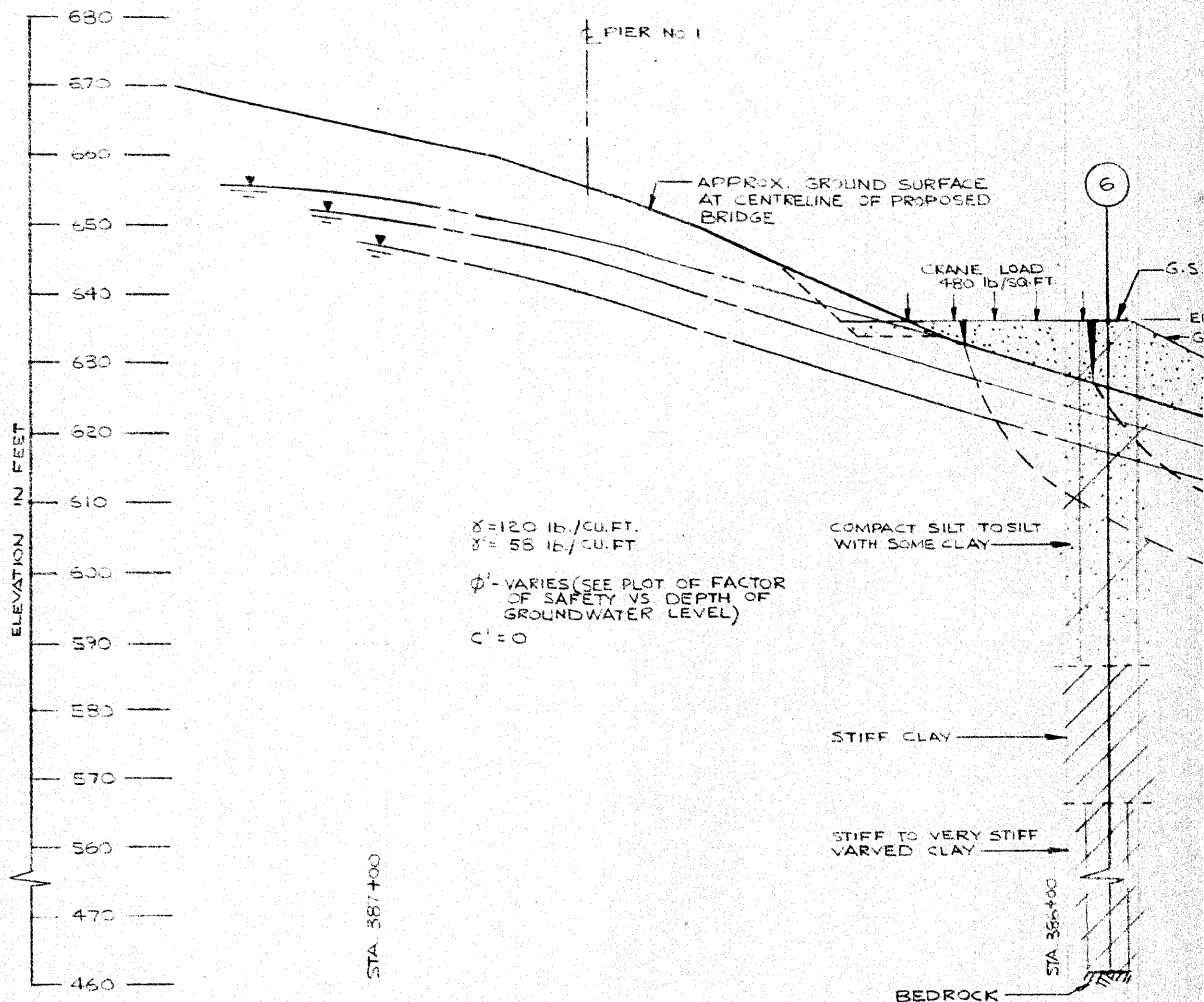


ELEVATION IN FEET

Date MAR. 13, 1975

Golder Associates

Drawn *m.j.B.*  
Chkd. *[Signature]*  
Appd. *[Signature]*



REFERENCES:

- 1) GENERAL LAYOUT DRAWING SUPPLIED BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO TITLED "ENGLEHART RIVER BRIDGE AT ENGLEHART" W.P. No. 40-70-01, SHEET 1, DATED NOV. 1973
- 2) M.T.C. FOUNDATION INVESTIGATION REPORT FOR THE PROPOSED ENGLEHART RIVER BRIDGE. W.O. 72-11086, W.P. 40-70-01, NOV. 8, 1972

BERM STABILITY (GROUNDWATER TABLE  
AT SLOPE SURFACE,  $\beta = 0.25$ )

SANDY SILT	F.S.
$\phi' = 35^\circ, c' = 0$	1.45
$\phi' = 30^\circ, c' = 0$	1.20
$\phi' = 27^\circ, c' = 0$	1.07

FACE  
CLOSED

RANE LOAD  
480 LB/SQ.FT.

G.S. AT B.H.

ELEV. 636.0  
GRAVEL PAD

PIER No. 2

ELEV. 622.0

BERM

SILT TO SILT  
CLAY

CRITICAL ZONE  
- GRAVEL PAD

CRITICAL SURFACE  
- BERM

LOOSE SILT WITH SOME  
CLAY AND SAND

FIRM TO VERY STIFF  
VARVED CLAY

VERY STIFF  
CLAY

BEDROCK

ROCK

SCALE: 1" TO 20'

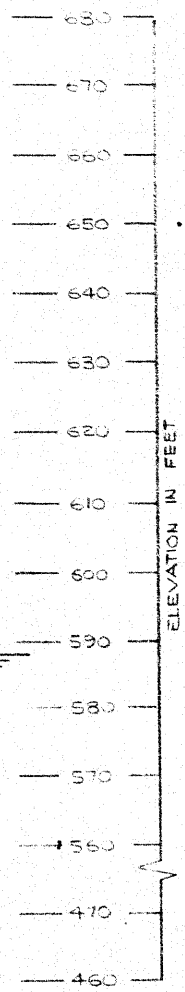
**SPECIAL NOTE**

THIS DRAWING IS TO BE READ IN CONJUNCTION  
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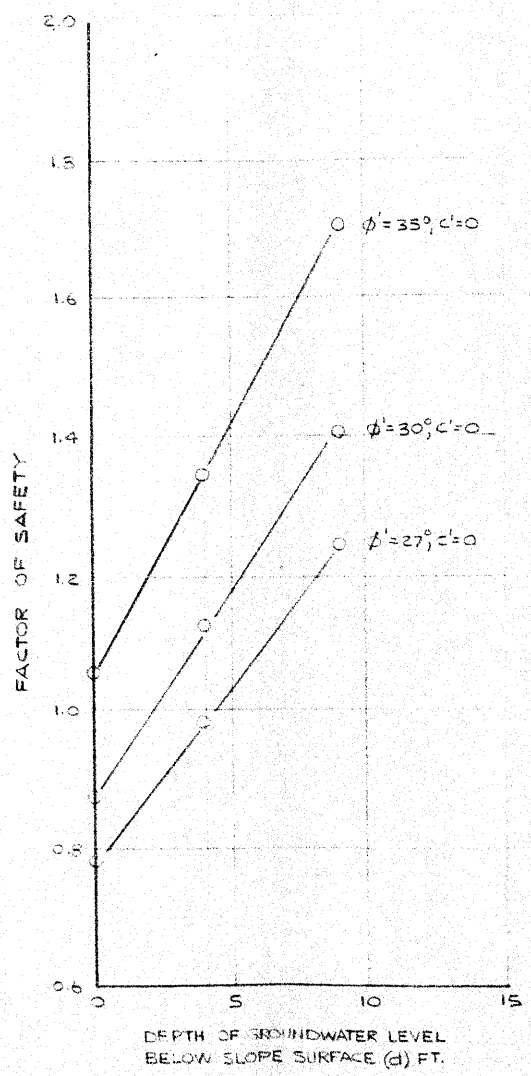
WATER TABLE  
(ES)

G.S. AT BH.

STA 384+00



CASE 1: GRAVEL PAD IN PLACE,  
WITHOUT BERM OR CRANE LOAD



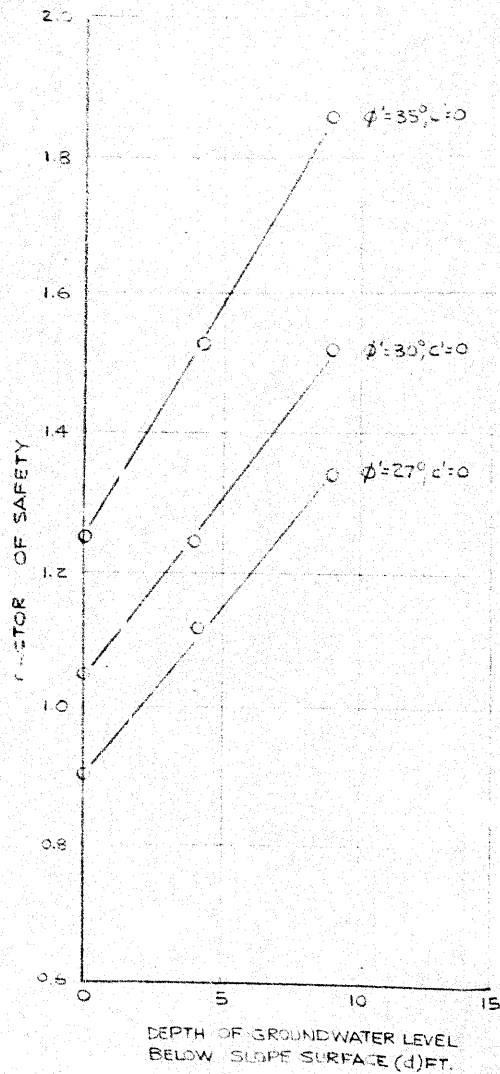
NOTE  
READ IN CONJUNCTION  
REPORT.

NOTE: FOR DETAILED STRATIGRAPHY  
REFER TO M.T.C. REPORT

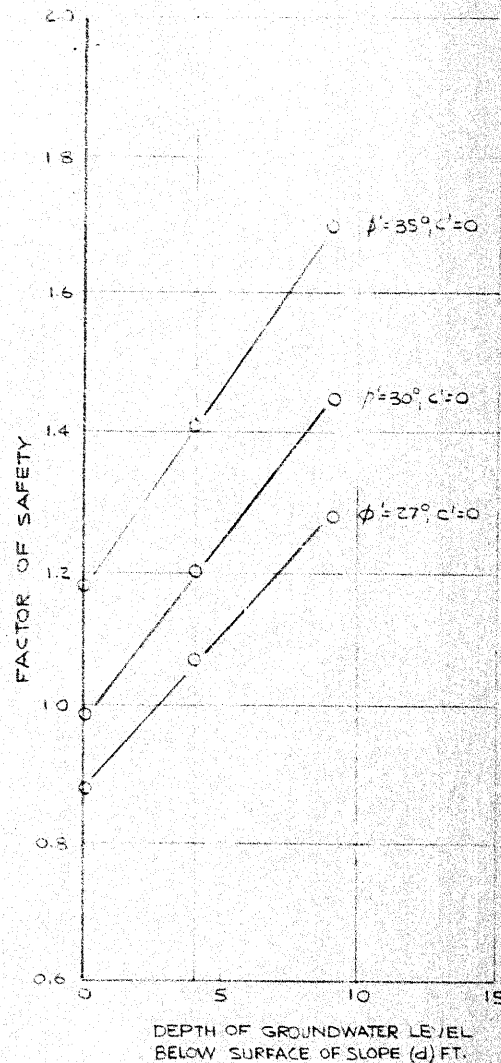
STABILITY ANALYSES  
CRANE ON STEEPEST PART OF SLOPE  
(BETWEEN PIER 1 AND PIER 2)

FIGURE 5

CASE 2: GRAVEL PAD IN PLACE WITH  
10' WIDE BERM, NO CRANE LOAD.



CASE 3: GRAVEL PAD IN PLACE WITH  
10' WIDE BERM + CRANE LOAD (430 LB/SQ.FT.)



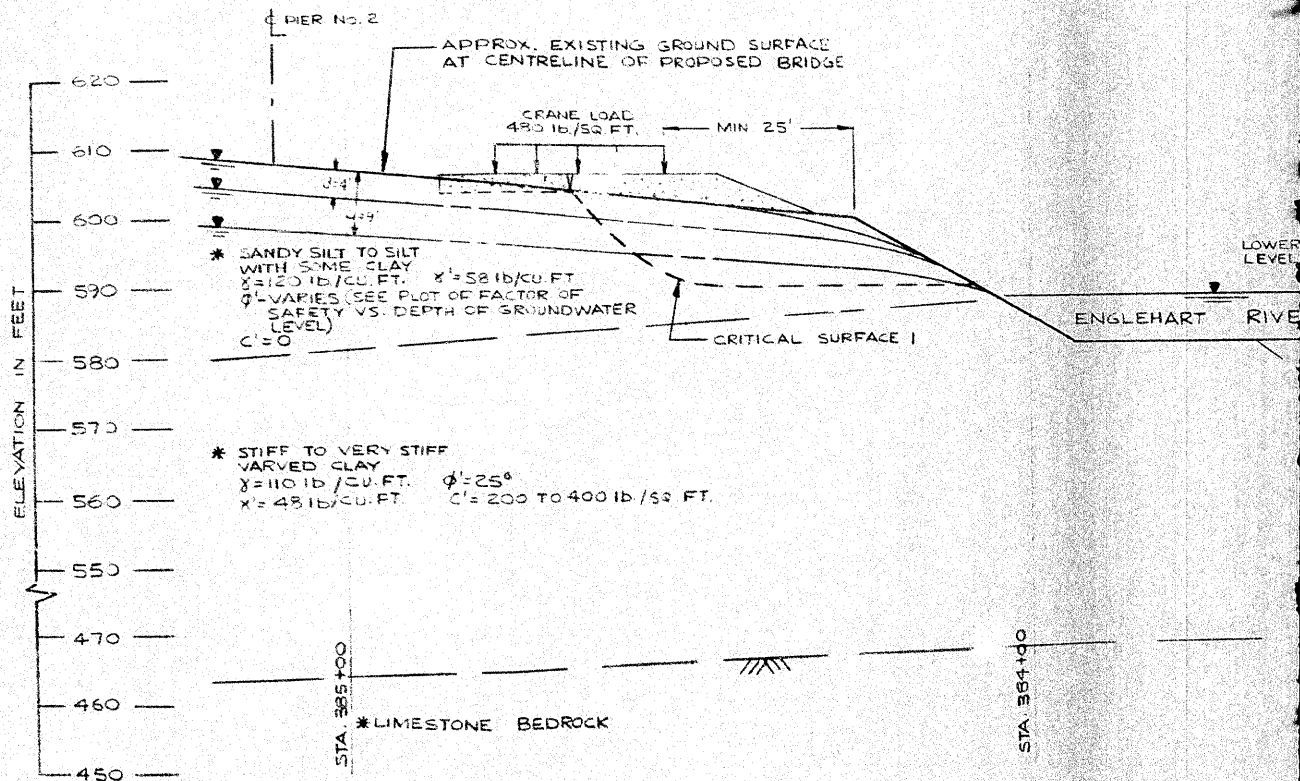
Date MAR. 13, 1975

Golder Associates

Drawn D.M.  
Chkd J.M.  
Appd J.M.



WEST



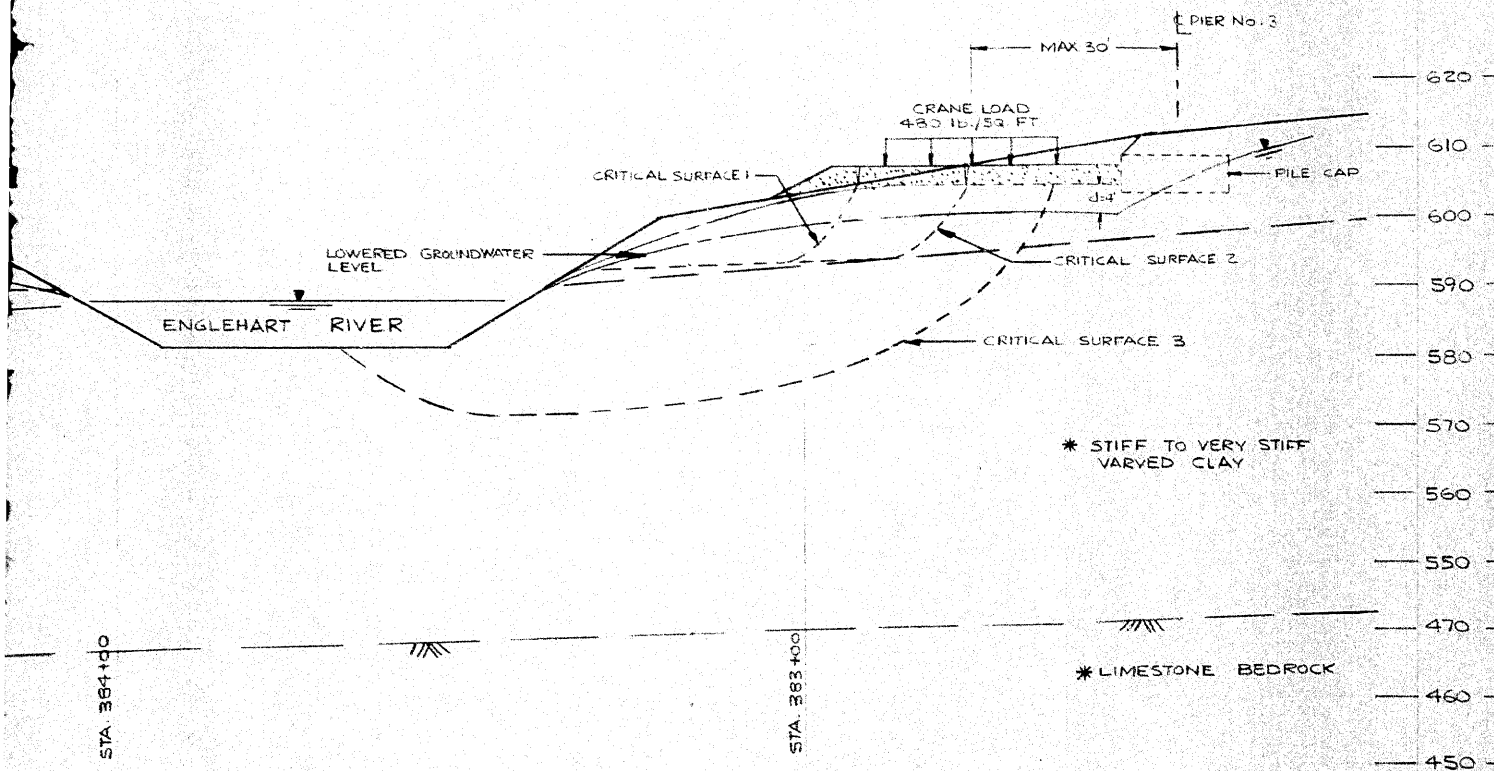
REFERENCES:

1) GENERAL LAYOUT DRAWING SUPPLIED BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO. TITLED "ENGLEHART RIVER BRIDGE AT ENGLEHART." W.P. No. 40-70-01, SHEET 1, DATED NOV. 1973.

2) M.T.C. FOUNDATION INVESTIGATION REPORT FOR THE PROPOSED ENGLEHART RIVER BRIDGE W.O. 72-11086, W.P. 40-70-01, NOV. 8, 1972

SCA

EAST

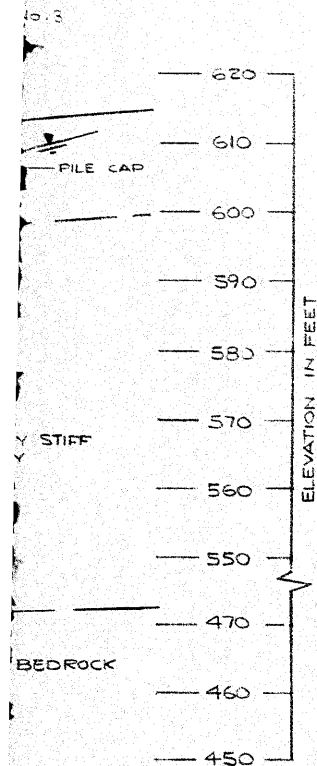


\* NOTE: FOR DETAILED STRATIGRAPHY  
REFER TO M.T.C. REPORT

SCALE: 1" TO 20'

SPECIAL NOTE  
THIS DRAWING IS TO BE READ IN CONJUNCTION  
WITH ACCOMPANYING REPORT.

EAST

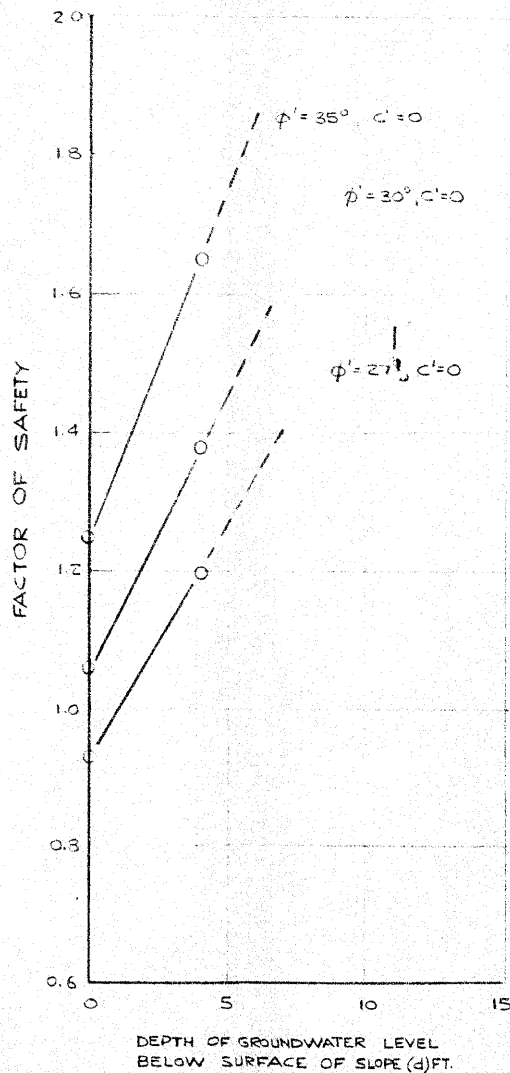


STRATIGRAPHY  
REPORT

NOTE  
E READ IN CONJUNCTION  
REPORT.

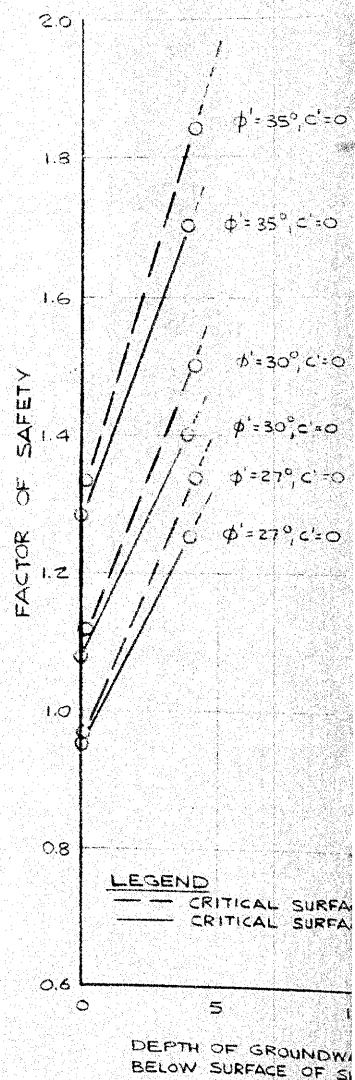
# WEST BANK

GRAVEL PAD IN PLACE  
+ CRANE LOAD



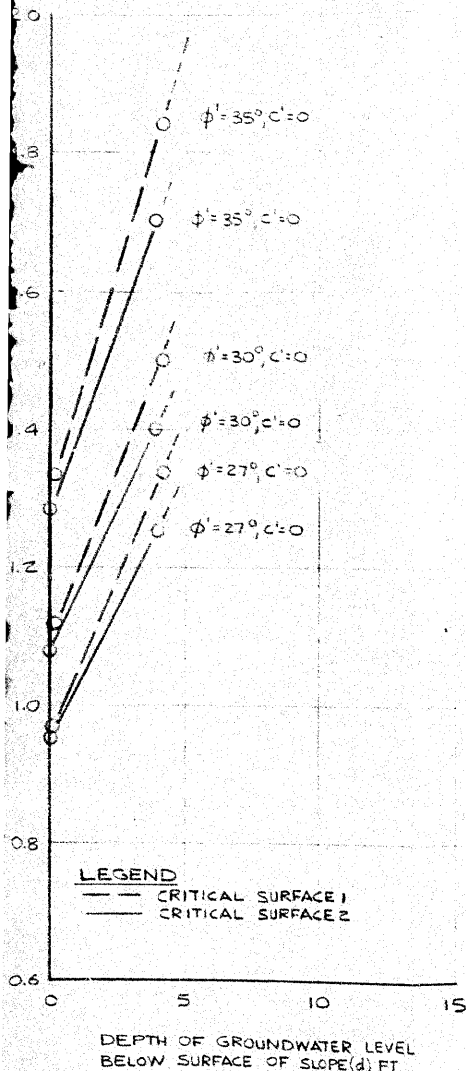
# EAST BANK

GRAVEL PAD IN PLACE  
+ CRANE LOAD



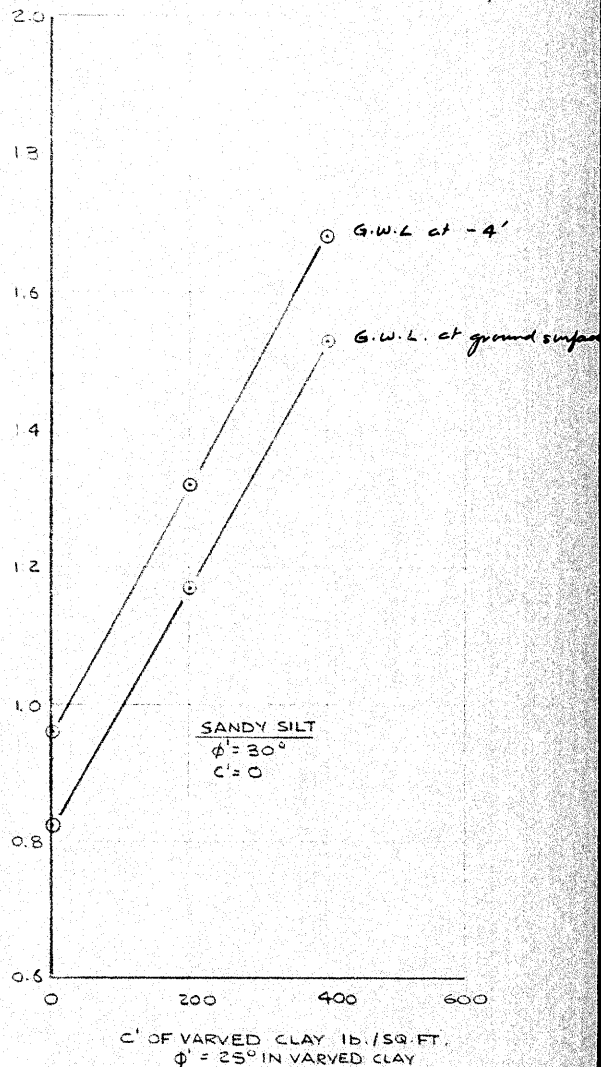
EAST BANK

GRAVEL PAD IN PLACE  
+ CRANE LOAD



EAST BANK

GRAVEL PAD + CRANE LOAD  
DEEP SEATED FAILURE (CRITICAL SURFACE 3)



Date MAR. 14, 1975

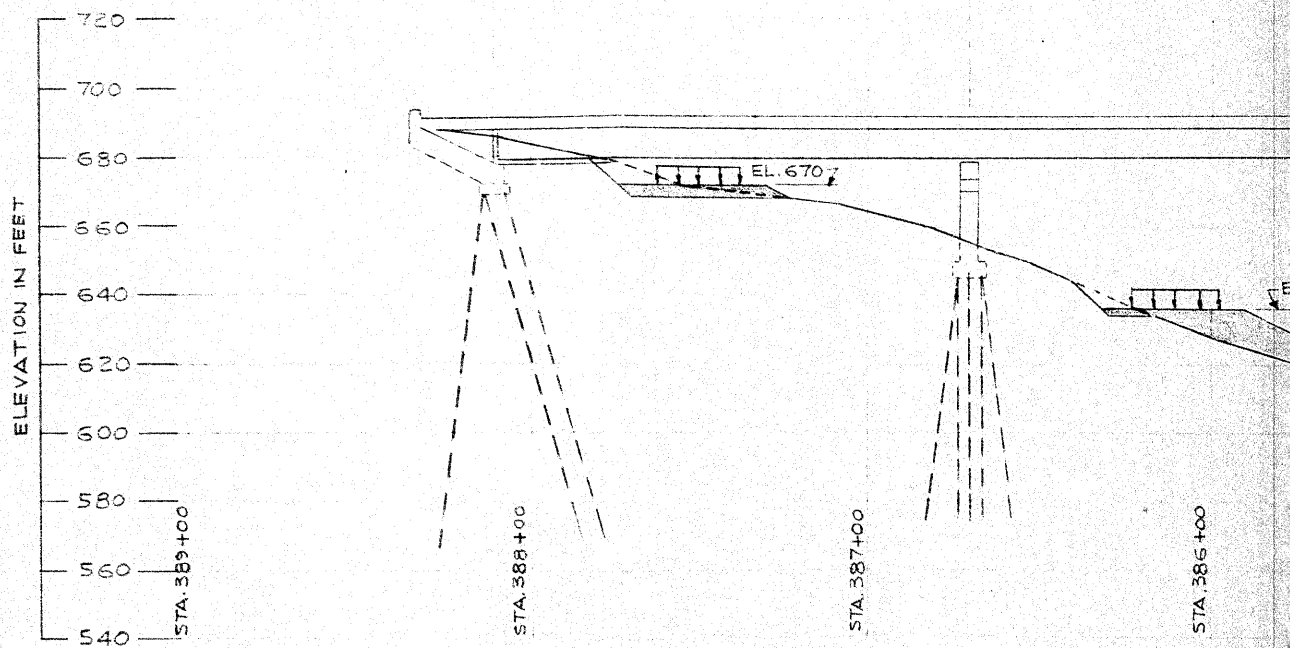
Golder Associates

Drawn D.M.  
Chkd. *[Signature]*  
Appd. *[Signature]*

TO KIRKLAND LAKE

WEST ABUTMENT

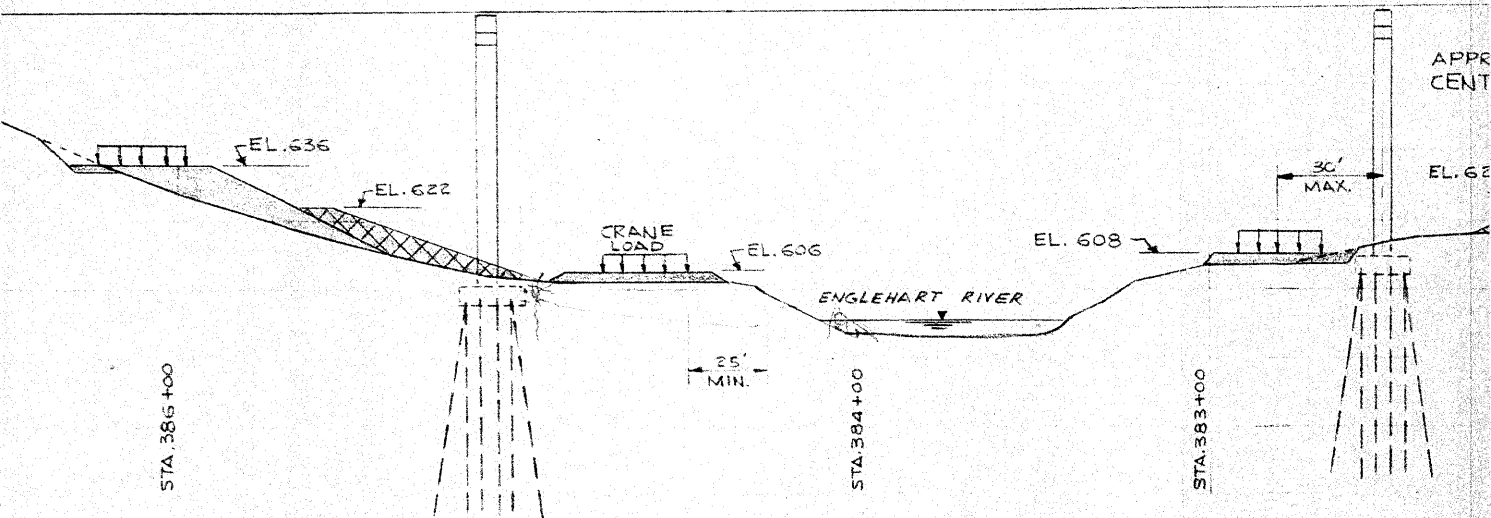
PIER No. 1



No. 1

PIER No. 2

PIER No.



SCALE: 1" TO 40'



TO NEW LISKEARD →

⊕ PIER No. 3

⊕ PIER No. 4

⊕ EAST ABUTMENT

APPROX. GROUND SURFACE AT  
CENTRELINE OF BRIDGE

30'  
MAX.

EL. 625 7

EL. 635 7

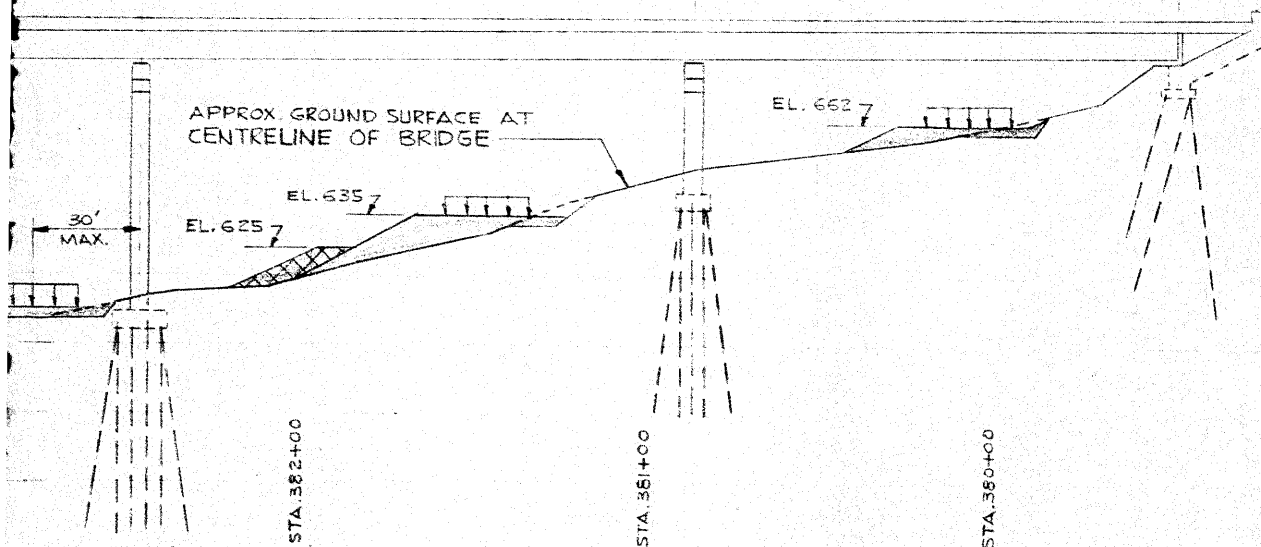
EL. 662 7

STA. 382+00

STA. 361+00

STA. 360+00

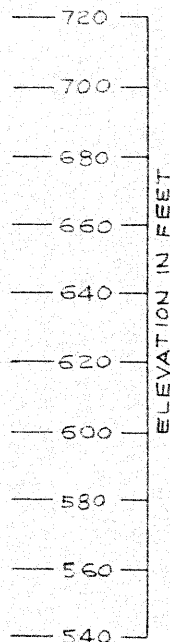
STA. 379+00



SECTION THROUGH RIVER BANK SHOWING  
GRANULAR PADS AND BERMS FOR CRANE

FIGURE 7

WARD  
ST ABUTMENT



STA. 379+00

REFERENCE: GENERAL LAYOUT DRAWING SUPPLIED  
BY MINISTRY OF TRANSPORTATION AND COMMUNICATIONS  
ONTARIO, TITLED "ENGLEHART RIVER BRIDGE AT ENGLEHART"  
W.P. No. 40-70-01, SHEET 1, DATED NOV., 1973

SPECIAL NOTE

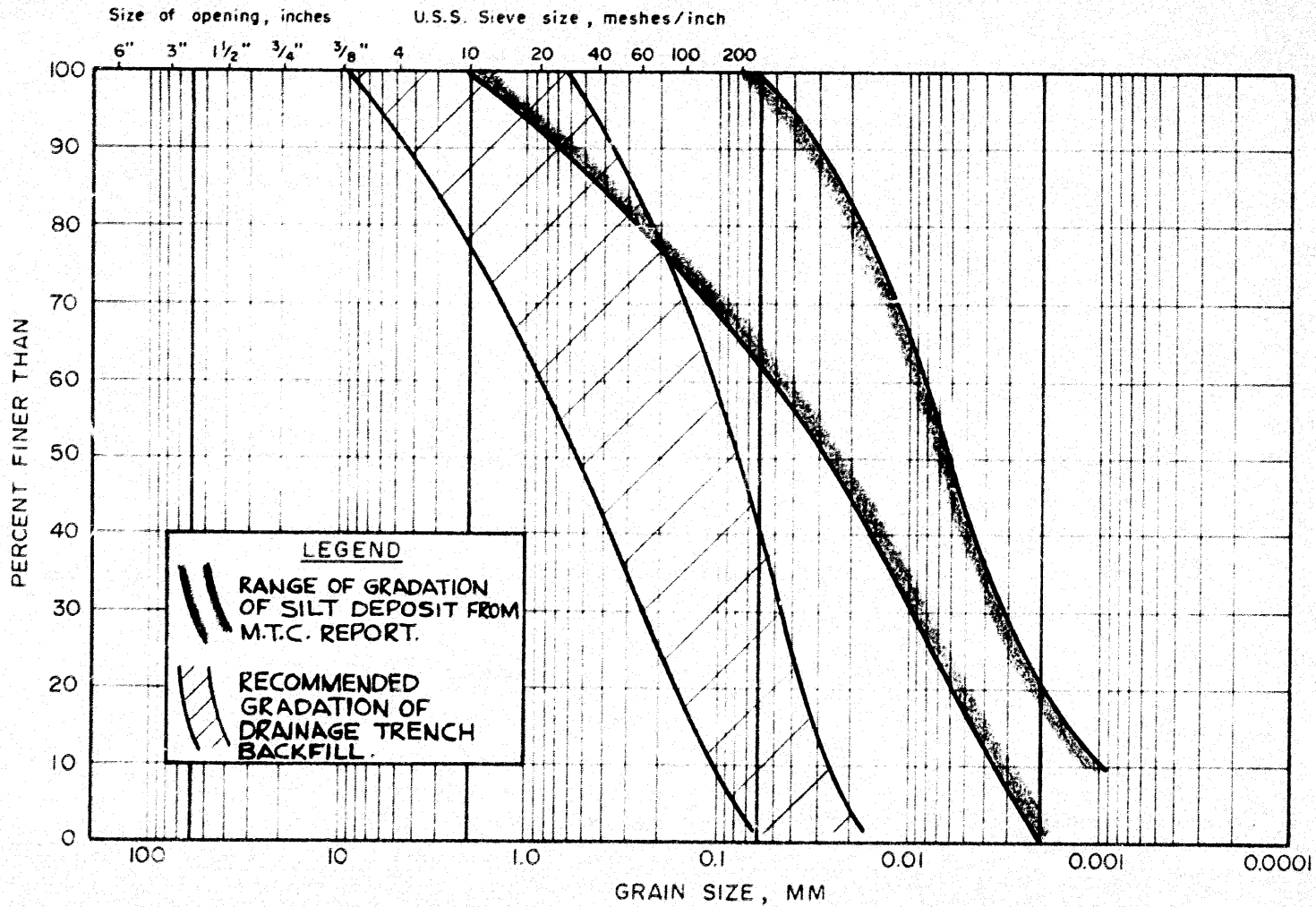
THIS DRAWING IS TO BE READ IN CONJUNCTION  
WITH ACCOMPANYING REPORT.

Date. MAR. 17, 1975

**Golder Associates**

Drawn *m.j.B.*  
Chkd. *[Signature]*  
Appd. *[Signature]*

## M.I.T. GRAIN SIZE SCALE



Golder Associates

GRAIN SIZE DISTRIBUTION  
RECOMMENDED DRAINAGE TRENCH BACKFILL

**D. Laframboise**  
Structural Steel Technologist  
Construction Office  
Central Building

**Soil Mechanics Section**  
Geotechnical Office  
West Building

**April 16, 1975**

**ENGLEHARDT RIVER BRIDGE**

**Structural Steel Erection**  
**Contract 74-105, District 14**

---

This is in reply to your request for an opinion as to the feasibility of a scheme by Bineen Construction Ltd. for slope stabilization to provide working platforms for two 110 ton crawler cranes to be used in the erection of structural steel. The scheme is outlined in two reports: Golder Associates Report #751019 and, a report by Canron Ltd. dated 26/3/75. Our comments are as follows:

1. Provided that the stabilization scheme as designed by Golder is built as designed and their recommendations followed, the scheme would be acceptable to us.
2. To construct the berms and associated drainage schemes, which involve some excavation, care must be taken not to create temporary unstable conditions. To achieve this it is our opinion that:
  - a) Work must proceed from the bottom of slope upwards.
  - b) Drainage trenches must be excavated and backfilled more or less simultaneously. Not more than 10 ft. in length of open trench should be permitted at any time.
  - c) Excavated material must be removed from the site as it is dug out. It should not be stockpiled on the slopes.
  - d) An experienced Foundation Engineer should be employed by the Contractor to supervise the work.
  - e) At completion of the steel erection work the slopes should be restored to the satisfaction of the Ministry.

**K. G. SELBY**  
Supervising Engineer.

**c.c. H. Martens**  
**A. Radkowski**  
**J. McLougall**  
**J. McAllister**  
**Files**  
**Record Services**



## Memorandum

To: Mr. A. E. McKim,  
Construction Office,  
Central Building, Downsview.

From: Structural Office,  
West Building, Downsview.

Attention:

Date: May 8, 1975.

Our File Ref.

In Reply to

Subject:

Englehart River Bridge,  
Contract 74-106, District #14.



This will confirm our recommendations on the erection of structural steel at this bridge site as we outlined after today's meeting with the contractor - W. Schaeffer of Dineen Construction.

It would appear there is a disagreement between the General Contractor and the Structural Steel Subcontractor, Canron, regarding the roadway and granular pads in the river valley which are required to sustain the heavy erection crane loads. We believe this dispute should be resolved by those two parties.

Regarding the proposed erection scheme which was submitted some time ago we would recommend that the Contractor be advised his scheme is approved in principal, subject to the conditions as outlined in the attached memo by Mr. K. G. Selby, Supervising Engineer, Geotechnical Office.

We believe a claim could result if the contractor is delayed in any way, especially with the poor soil conditions at this site and the necessary work required to build the granular pads as outlined in his proposed scheme.

C. S. Grebski,  
Structural Design Engineer.

CSG/cf

c.c. H. Martens  
A. Radkowski  
J. McDougall  
J. McAllister  
K. Selby

(a) ENGLEHART RIVER BRIDGE

Notes on Site Visit

May 28, 1975

On Wednesday, May 28, 1975, I visited the site of the Englehart River Bridge with Mr. Leo Lahti (GA) and Mr. Bill Schafer (Dineen, Contractor).

We walked over the whole site. We had a gang of men from the MTC local district office with a shovel and hand auger. We dug six augerholes and took samples from five of them. See separate notes by Lahti.

The condition of the site is very different now from that described in the MTC report dated November 8, 1972. The piers for the bridge are completed. They were constructed by excavating in steel sheet piled cofferdams, driving steel H piles to rock, casting the pile cap and then pulling the sheet piles. This work was done in the winter when the ground was frozen.

Access to the pier sites was by a construction road of sand and gravel tipped on the surface of the ground on the north side of the new bridge. This 'road' has been eroded considerably by surface water in the spring.

There has also been a considerable amount of soil tipped around the piers to fill the excavation. This material appears to be the local soil; it has not been compacted and in many places is completely saturated and in a 'soupy' condition.

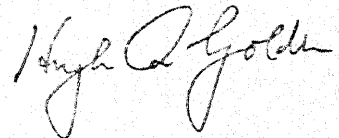
Some water is seeping out of the various fill areas including the 'road' which appears to be several feet thick in places.

Where the original ground surface is not covered by fill, the surface soil is a yellow soft to firm silty clay which changes gradually with increasing depth to a grey clayey silt which is dilatant, apparently saturated and probably has a high water content in relation to its liquid limit. We were unable to determine the position of the main water table. The seepage through the fill of sand and gravel probably constitutes a 'perched' water table. However, this material is draining except where it is trapped in pockets created by the pier excavations.

(a) Notes on Site Visit (continued)

It is clearly useless to consider putting in the drains suggested in our report dated March 1975 since, to be effective in this silt, they must be below the groundwater level; otherwise, the void water will be held by capillary tensions. Another approach must now be adopted.

HQG:cmm  
751019



Hugh Q. Golder.



(b) ENGLEHART RIVER BRIDGE

Notes on Hand Augerholes

May 28, 1975

Augerholes put down during site visit May 28, 1975; all augerholes on EAST bank.

Augerhole 1

Location: Through roadway approximately midway between east abutment and pier No. 4

Depth: 3.5 feet

Material: Silty sand and gravel fill (no sample)

Water: Seepage occurring from granular fill.  
Augerhole full after about 1 hour.  
Hole caving below about 1 foot during augering.

Augerhole 2

Location: In natural (?) bank on southeast side of pier No. 4

Depth: About 2.5 feet (angle hole)

Material: Clayey silt becoming silt, some clay with depth (brown)  
Took sample of "silt"

Water: Very light seepage noted after 1 hour

Augerhole 3

Location: Adjacent to water pipeline opposite pier No. 3

Depth: 3.5 feet

Material: Brown clayey silt becoming silt, some clay, trace sand  
Took bag sample of "silt"

Water: Approximately 3 inches of water at bottom of hole in about 45 minutes

(b) Notes on Hand Augerholes (continued)

Augerhole 4

Location: At top of river channel bank near pier No. 3  
Depth: 3.5 feet  
Material: Brown clayey silt (moist) becoming grey silt,  
some clay (dry) at 3.5 feet  
Took sample of clayey silt  
Water: No seepage ( $\frac{1}{2}$  hour)

Augerhole 5

Location: Midway down river channel bank  
Depth: 4 feet  
Material: Grey silt, some fine sand, trace clay  
Took sample  
Water: No seepage (15 minutes)

Augerhole 6

Location: Within material excavated from pier pile caps,  
midway between piers No. 3 and 4  
Depth: about 1.5 feet  
Material: Saturated grey silt, some clay  
Took sample.  
Sides of hole flowing in as soon as auger  
pulled out.  
Water: No free water but material is saturated

Laboratory Testing

Scheduled water contents, Atterberg limits, sieve and hydrometers  
on all five samples.

LPL:emm  
751019

*L.R. Lahti*  
L.R. Lahti

(c) ENGLEHART RIVER BRIDGE

Notes on Access Roads and Crane Foundation Pads

May 29, 1975

Following the visit of Mr. Lahti and Dr. Golder to the site on May 28, 1975 (see notes on visit to site attached), it was evident that the conditions had changed to such an extent that the solution previously proposed was no longer possible. Accordingly, a statement of the problem as it now exists together with suggested solutions is given below.

There are two separate problems which should be treated separately. These are:

1. The access roads for plant and materials
2. The foundation pads for the cranes lifting the girders of the bridge into position

These are treated separately below.

1. The Access Roads

Several feet of granular fill already exists at the positions of the access roads on each bank. These roads were constructed for access of the piling and concreting plant for the construction of the foundations and piers. This work was done in winter conditions and stability was improved by the fact that the ground was frozen.

Since then quite large gullies have been eroded in this fill by runoff of surface water. Some water is still draining out of this fill.

If these roads are remade and possibly thickened by a few feet and compacted, they will carry normal construction equipment.

Mr. Schafer of Dineen is of the opinion that this road is capable of carrying the load of the stripped down cranes on timber pads provided that they are winched down the hill, and it is possible to pull them up again if there should be any tipping due to foundation soft spots. We are in agreement with his suggested approach. In view of the short time available, we think this risk must be accepted.

*Cannon does not accept this  
statement as written  
Spencer  
June 2nd, 1975.*

## 2. The Crane Foundation Pads

In this case, the risk factor is very different. There must be no tipping of the foundation pads when the cranes are lifting the bridge girders.

There are two types of weak soil below the areas of the pads:

- (a) the naturally occurring silt
- (b) remoulded silt and clay and loose sandy material backfilled into excavations above the pier footings

### (a) The Silt

The silt is fine grained, saturated, dilatant soil which is soft to very soft near the surface and increases in strength to stiff with depth. This silt is a frictional material and will consolidate under load to adequate strength. However, consolidation will be slow unless sand drains are used to shorten the drainage path.

### (b) The Uncompacted Backfill

In places, this material quakes under foot. This is because it is uncompacted and saturated and is not draining. Some drainage can be obtained immediately by cutting ditches downhill. However, some of this material may have to be dug out and replaced by good sand and gravel. These are on-site decisions.

## Construction of Pads

In general terms, we think that the foundation pads can be constructed by tipping sand and gravel in the pad areas after removing unsuitable material and improving drainage where possible to a thickness to permit a power auger to operate on the area.

Holes about 6 inches in diameter should then be drilled into the silt to a depth of about 15 to 20 feet and immediately backfilled with a suitably graded sand. A drainage layer should be provided at the surface unless the previously tipped material already functions as such. Piezometers should then be installed at appropriate places to measure the pore water pressure. Fill, several feet thick, should be placed over the pad area and then

(c) Notes on Access Roads and  
Crane Foundation Pads (continued)

increased gradually as drainage occurs until a surcharge equal to the crane load is in position. Details of this loading and monitoring remain to be worked out. When the silt is fully consolidated under the load, the surcharge will be removed and the crane can be moved onto the pad.



Hugh Q. Golder

HQG:cm  
751019



**Golder Associates**  
CONSULTING GEOTECHNICAL ENGINEERS

June 2, 1975.

Canron Ltd.,  
100 Disco Road,  
REXDALE, Ontario.  
M9W 1M1

ATTENTION: Mr. G.R. Sinclair, P.Eng.

RE: ACCESS AND FOUNDATION PADS FOR  
CRANES AT ENGLEHART RIVER, ONTARIO

Dear Sirs:

We enclose two copies of:

- (a) Notes on site visit, May 28, 1975
- (b) Notes on hand augerholes, May 28, 1975
- (c) Notes on access roads and crane foundation pads, which are self-explanatory

If you wish, will you please forward one copy of each of the notes to Dineen. From your telephone call to us, we gather that any further instructions may be from Dineen, but we would like to have this in writing please.

Yours truly,

H.Q. GOLDER & ASSOCIATES LTD.,

L. R. Lahti, P.Eng.

HQG:LRL:cmm  
751019  
encs

Rec'd June 2/75  
SPW. 2633



## Memorandum

To: Mr. K. Selby,  
Supervising Engineer,  
Soils Mechanics Section,  
Geotechnical Office.

From: Construction Office,  
Third Floor, Central Bldg.

Attention:

Date: June 4, 1975.

Our File Ref.

In Reply to

Subject: Contract 74-105, Englehart River Bridge,  
Site No. 47-32, District 14.

---

Enclosed will be found two (2) copies of  
Dineen Roads and Bridges revised soil report  
concerning the structural steel erection  
procedure for the above noted structure.  
Please review this report and submit your  
comments to this office at your earliest con-  
venience.

D. Laframboise,  
Structural Steel Technologist.

DL/JC  
Encl.

c.c. C. Grebski  
J. I. McDougall.



June 12th, 1975

Project No. 75086

Mr. G. Dineen,  
Dineen Construction Limited,  
70 Disco Road,  
REXDALE, Ontario. M9W 1L9

Re: Englehart Bridge Pads.

Dear Mr. Dineen:

Further to our two meetings and our discussions with Messrs. Stan Weightman, George Sinclair, Stewart Eccles and Gord Armstrong of Canron Limited, concerning the lifting pads for the above project, we wish to advise you as follows:-

- 1) We have studied various letters and reports dated May 28th, May 29th, and June 2nd, 1975 from Golder Associates addressed to Canron Limited.
- 2) We have Canron Limited's drawing E4 showing the arrangement of the bridge piers and the positions of the cranes when they will be lifting the bridge girders.
- 3) We have briefly reviewed the basic contract drawings of the bridge.
- 4) We have not visited the site as you know, and are hence relying on reported information related to the present condition of same.
- 5) We have established that the Link-Belt LS-418A crawler mounted cranes to be used exert a pressure of between 1500 and 2000 pounds per square foot under their tracks, when the cranes are fully dressed for operation.
- 6) This pressure can be more than doubled when the crane is lifting its maximum load particularly if the load is diagonal to the axis of the machine.
- 7) We understand that it is your considered opinion as a contractor that the construction road can be stiffened by the addition of rock fill and crusher run stone such that it will safely carry the load of the crane at all times other than when it is in operation.

..2

June 12th, 1975

Mr. G. Dineen,  
Dineen Construction Ltd.

- 8) On the assumption that there would be at least 2'6" of stone above the existing material and that 12" deep timber mats are placed at right angles to the tracks when the machine is lifting in line with the tracks, we have calculated that a grid of piles at 5'0" on centre in each direction around the perimeter of the pad will carry the additional pressure from the lifting procedure.
- 9) We suggest therefore that two rows of piles should be installed round the edges of the lowest lifting point. These piles should be 18" diameter at the top, 8" diameter at the tip and a minimum of 35 feet long.
- 10) We would recommend that the edge of the crushed stone mat extend at least 13'0" from the centre line of the machine in each direction, and that the rows of piles be placed 5'0" and 10'0" from the centre line of the machine in its outermost position for operation. This arrangement involves a crushed stone mat which will be 34'0" wide on the west side of the river and 29'0" wide on the east side of the river.
- 11) The rows of piles should be staggered in plan and each row should be tied around the perimeter with a 3/4" diameter steel cable. This cable shall be set about 6" below the top of each pile and anchored thereto with suitable fasteners.
- 12) The arrangement devised will hence produce the effect of a silt mass reinforced at its perimeter with two rows of circumferentially tied wood piles.
- 13) The theory of this application is similar to that of a tied reinforced concrete column, and we have discussed its behaviour with Mr. Al Millard of Frankl Canada Limited. Mr. Millard feels that the installation should work satisfactorily in the materials existing at the site.
- 14) As you will appreciate, the zone in the silt in which the piles are driven will become additionally compacted and will hence become more dense. It will therefore be able to carry more vertical load itself and this more dense material will serve to retain the silt within the pad section.
- 15) This process of piling may be used elsewhere at the upper crane positions or for the road in specific areas where very soft material exists. These locations may be established in the field, but the process may be expected to increase the carrying capacity of the specific area by the pile capacity plus the additional density of the compacted material around the piles.

June 12th, 1975

Mr. G. Dineen,  
Dineen Construction Ltd.

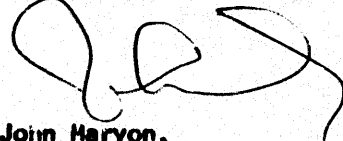
It is requested that you prepare sketches of the arrangements of the piles as they are installed and braced for our subsequent review prior to the operation of the cranes.

A rough sketch of the arrangement noted in this letter is enclosed for your information.

We trust that this report is of value to you.

Yours very truly,

JOHN MARYON AND PARTNERS LIMITED,

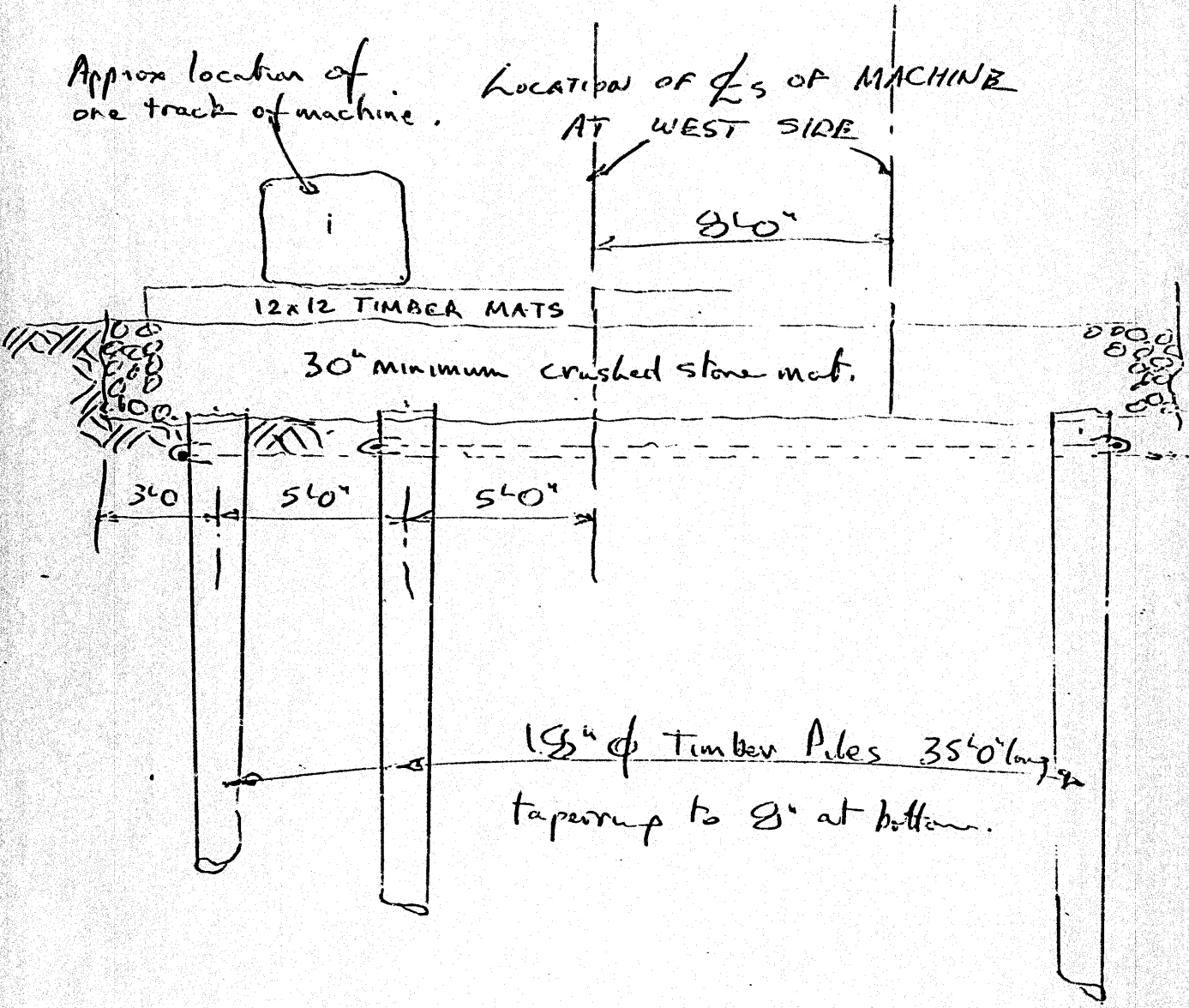


John Maryon,  
President.

JM:t

Encl:

JOHN MARYON and PARTNERS LIMITED	Designed by	Date	Checked by
PROJECT <i>ENGLE HART BRIDGE</i>	Architect	Job No.	Portion of Building <i>FOUNDATION PADS</i>



TYPICAL DETAIL OF  
FOUNDATION PAD.



## DINEEN ROADS & BRIDGES LIMITED

70 DISCO ROAD · REXDALE, ONTARIO M9W 1L9 · 677-7220

June 12, 1975.

Ministry of Transportation & Communications,  
Construction Office,  
1201 Wilson Avenue,  
Downsview, Ontario  
M3M 1J8

Attention: Mr. D. Laframboise,  
Structural Steel Technologist

Gentlemen:

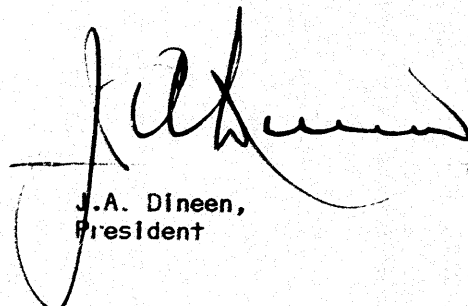
Re: Contract 74-105  
Englehart Bridge

With further reference to our discussions in connection with the erection procedure and access for equipment to the above, we attach hereto, for your information and comment, proposals prepared by ourselves and our Engineers, John Maryon & Partners.

These suggestions have, as you will note from the attached copy of letter being forwarded to Canron, and if there are no objections, this is the method by which we intend to proceed.

Yours truly,

DINEEN ROADS & BRIDGES LIMITED



J.A. Dineen,  
President

JAD:ns  
c.c. Mr. T. Sharpe



# DINEEN ROADS & BRIDGES LIMITED

70 DISCO ROAD · REXDALE, ONTARIO M9W 1L9 · 677-7220

June 12, 1975.

Canron Limited,  
100 Disco Road,  
Rexdale, Ontario.

Gentlemen:

## Re: Englehart Bridge

As promised during our meeting yesterday, we are enclosing herewith our reiteration of the procedures we propose to adapt in giving you access for the purpose of steel erection at the above project.

Attached for your perusal, is a copy of a letter of today's date from John Maryon & Partners, Consulting Engineers, in which they detail procedures we have suggested to you, Items 1 to 15 inclusive. Also enclosed, for your information, is a suggested cross section of the landing pads in the case of the two piers closest to the river. At the present time, it is not possible to determine the north-south dimension of these pads, but it will be arrived at more exactly in the field. At Mr. Maryon's request, we will furnish him with an exact overall length of these pads as they are constructed.

In regard to other operations of the roads, it is our intention, as we discussed with you at the meeting, to provide a minimum thickness of 4'-0" of granular material underlaying where necessary with coarser material, to ensure drainage from one side of the road to the other. We anticipate it will be necessary to dress the road and other areas where you propose to site your cranes, as conditions develop on the site, and we have also agreed to consider contingency plans as necessary, which might even mean driving some piles along the roadways in a similar pattern to that suggested by John Maryon & Partners. *Jack*

The provision of mats as an additional support for your equipment is your responsibility.

Continued...../

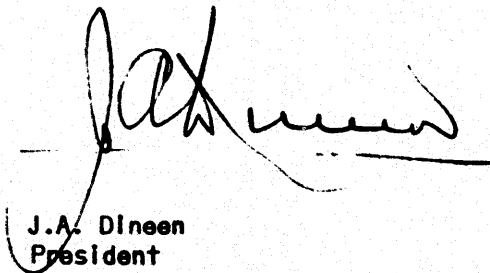
We are putting this work in hand commencing Monday, June 16th, 1975, and anticipate giving you the east side in total in approximately one week.

These are the procedures outlined to you in the meeting and the purpose of this letter is to provide you with written confirmation.

If you have any comments, please let us know immediately.

Yours truly,

DINEEN ROADS & BRIDGES LIMITED



J.A. Dineen  
President

JAD:ns  
Encis.





## Memorandum

To: Mr. D. Laframboise  
Structural Steel Technologist  
Construction Office  
Central Building

From: Soil Mechanics Section  
Geotechnical Office  
West Building, Downsview.

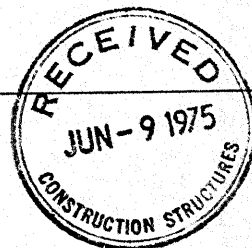
Attention:

Date: 5th June 1975

Our File Ref.

In Reply to

Subject:



ENGLEHARDT RIVER BRIDGE  
Structural Steel Erection  
Contract 74-105 Dist. #14

We have reviewed the revised soil report and Drawing # E4 (prepared by Camron Ltd.) both of which were submitted by Dinean Construction to your Office on 4th June 1975 for approval in connection with the scheme for structural steel erection on the above mentioned contract. Our comments are as follows:-

1. The access road and pads are more or less as proposed previously. The construction of pads however, will now require that some 'unsuitable' surface soil be replaced by granular material, that sand drains be installed, and that consolidation of soil by surcharging be effected. The soils consultant after visiting the site has decided that the problem of stabilising the upper subsoil cannot be solved by installing a drainage system as he originally proposed since the 'held' water in the soil will not drain unless consolidated under load. Based on the new factual data relating to surface soil conditions at the site we are in agreement that the problem is as stated in the soil report, and that the method of solution suggested in the report is a satisfactory one.
2. Our requirements for acceptability of the scheme should be as follows:
  - a) Drainage must be provided for granular fill which replaces native 'unsuitable' soil.
  - b) Pad installation work must proceed from the bottom of the slope up.
  - c) Excavated material must be removed from the site as it is dug out. It must not be stockpiled on the slopes.
  - d) An experienced Foundation Engineer should be employed by the Contractor to supervise the installation of pads and drains.
  - e) At completion of the steel erection work the slopes should be restored to the satisfaction of the Ministry.
3. Since the locations of pads are not exactly as in the previous scheme we should request that overall slopes and partial slopes be re-checked for stability.

cont'd...

4. We note that the pads adjacent to Pier 1 and Pier 4 impose some significant loading on the Piers. Mr. A.Radkowski of the Structural Office should be asked to comment on this.
5. We suggest that you approve the scheme as submitted subject to the requirements of 2. and 3. being met in order not to delay the Contractor. .

*K.G. Selby*

K.G. Selby  
Supervising Engineer

C.C. H. Martens  
A. Radkowski  
J. McDougall  
J. McAllister

Files:  
Record Services.

slopes be re-checked for stability.

cont'd...

## REVIEW OF 3RD SCHEME SLOPE STABILISATION

Dated June 12<sup>th</sup> 1975.

There are two aspects to the problem these being effect on slope stability and local stability of the pad foundations.

OVERALL STABILITY This can be affected by excavation by placing fill and by loading due to crane. Provided the amounts of fill are not greater than on original scheme overall slope stability should be O.K. No excavation is to be done and crane loads will be as before. Ministry should require that drainage outlets be provided for any granular placed in depression.

LOCAL STABILITY This is affected by the condition of the subsoil below the pads and may result in tipping of the crane if inadequate. At the moment the soil is in very poor condition in some areas so details are required as to what the Contractor intends to do about this and we should request that an assurance be given from their consultants that whatever they do will work. As far as the piling proposal is concerned it should be O.K. but the other locations require some clarification.

## SUMMARY OF PILE DRIVING RECORDS

W.O. 72-11086 W.P. 40-70 CONT. 74-105 DIST. 14  
SITE BLANCHE RIVER & HWY# 11

DATE DRIVEN 11 Nov. 74 - 17 FEB 75 WEIGHT OF ANVIL \_\_\_\_\_  
HAMMER TYPE LB-520 WEIGHT 2.535 T ENERGY 30,000 FT/LB.

[illegible]

Oversized Drawing

General Layout

Foundation Layout.

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31M-36  
DIST. 14 REGION NORTHERN  
W.P. No. 40-70-01  
CONT. No. 74-105  
W. O. No. 72-F-84  
STR. SITE No. 47-32  
HWY. No. 11  
LOCATION HWY. 11 & ENGLEHART  
RIVER.

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT 2

REMARKS: DOCUMENTS TO BE UNFOLDED  
BEFORE MICROFILMED.

6-11-79 SEPT 1979





