

#

57-F-204C

HWY # 90

NOTTAWASAGA  
RIVER

BA 589

RACEY, MacCALLUM AND ASSOCIATES  
LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

Consulting Engineers  
AND ASSOCIATED STAFF

MONTREAL  VANCOUVER

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TORONTO

TORONTO DIVISION  
20 CARLTON STREET

REPORT NO: S500-669/T-627

Toronto, Ontario.  
March 29th, 1957.

Department of Highways of Ontario,  
c/o W. S. Atkins & Associates Ltd.,  
33 Price Street,  
TORONTO, Ontario.

Attention: Mr. Birkmeyer.

RE: FOUNDATION INVESTIGATION FOR  
THE PROPOSED BRIDGE REPLACEMENT  
ON HIGHWAY NO. 90 OVER THE  
NOTTAWASAGA RIVER

Dear Sirs:

Attached hereto is our report concerning the foundation conditions noted at the above mentioned bridge site. Reference to this report indicates that the subsoil profile is quite uniform across the site and consists of a stratum of loose to medium dense fine to medium sand which is underlain at elevation 575 feet, or a depth of  $36\frac{1}{2}$  feet by soft, sensitive, silty clay. This latter material appears to continue at least to a depth of 140 feet.

Various foundation proposals have been analysed and it would appear that a boxed caisson or a pile foundation carried to a depth of 30 feet into the upper sand stratum should provide the most economical method of bridge support. The estimated safe capacity of a pile has been taken as 11 tons per square foot, although load tests may well prove this estimate to be conservative. Comments regarding protection

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against river scour and the method of supporting the horizontal earth pressure on the abutments also have been given.

In view of generally weak nature of the subsoil at this bridge site, more attention has been given to the various soil problems than normally would be required. If there are any queries regarding the contents of this report, we shall be pleased to discuss them at your convenience.

We thank you for the opportunity to serve you in this regard.

Yours very truly,

RACEY, MacCALLUM AND ASSOCIATES LIMITED.

*W. A. Trow*

W. A. Trow. P. Eng.  
Divisional Soils Engineer.

WAT/AMcL.

Dept. of Highways of Ontario.  
c/o W.S. Atkins & Associates Ltd.,  
33 Price Street,  
Toronto, Ontario.

FOUNDATION INVESTIGATION FOR THE  
PROPOSED BRIDGE REPLACEMENT ON  
HIGHWAY NO. 90 OVER THE  
NOTTAWASAGA RIVER.

Reference: S-500-669/T-627

Racey, MacCallum & Associates Ltd.

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FOUNDATION INVESTIGATION FOR THE  
PROPOSED BRIDGE REPLACEMENT ON  
HIGHWAY NO. 90 OVER THE  
NOTTAWASAGA RIVER

PURPOSE OF THE INVESTIGATION AND SCOPE OF THE REPORT

The investigation was undertaken to determine the soil conditions at the above mentioned site and hence to assess the bearing capacity of the subsoil for the support of the proposed bridge structure. The subsoil was found to consist of two main strata; a loose to medium sand, and a soft to medium clay which underlay the sand at a depth of approximately 37 feet. Consideration is given to probable foundation support both for a two span continuous structure and for a single span free-supported structure by founding, at depth within the clay on long friction piles, within the upper sand stratum on short end bearing piles, or by means of boxed caissons.

LOCATION OF THE SITE AND BOREHOLES

The site is on highway 90 where the road crosses the Nottawasaga river approximately 2 miles east of Angus, Ontario. A sketch plan of the area indicating the location of the boreholes is shown on enclosure No. 1.

GEOLOGY AND HYDROLOGY OF THE AREA

The soil in this area is of recent deposition, and the clay encountered during the investigation, may in general, be considered as being normally loaded. The upper sand layer appears to have been river deposited in very recent times, as is indicated by the high fragmented timber content. This timber below a depth of about 10 feet is in a well preserved state.

The Nottawasaga river has cut itself a steep sided path through the loose upper sand, and it seems probable that any major change in flow pattern of the river could change the shape and course of its bed quite rapidly.

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GEOLOGY AND HYDROLOGY OF THE AREA (Cont'd)

Downstream of the existing bridge it is understood that the river is constricted, and that during flood periods the water "backs up" causing flooding over the road on both sides of the existing bridge. It is understood from conversations with local inhabitants that this constriction is to be removed within the next few years. The river is subject to rapid changes in water level, as is shown by a drop in level of 6 feet in two days during the field investigation. It is also reported, but not confirmed, that the low water level is very low.

FIELD INVESTIGATION AND DESCRIPTION OF THE SUBSOIL PROPERTIES

The field investigation was commenced on 27th February 1957, and completed on the 4th March. Borehole No.1 was taken to a depth of 75 feet with samples every 5 feet. From a depth of 75 feet to a depth of 140 feet, the hole was washed in an attempt to find bedrock. From ground surface to a depth of 36.5 feet, loose to medium dense, fine to medium sand containing broken timber was encountered. Below 36.5 feet the soil became soft to medium stiff silty clay with a measured sensitivity by means of the vane test ranging from 1.4 to 4. Similar results were obtained at borehole location No. 2, except for a thin clayey silt stratum observed at a depth of 12 - 16 feet. Samples were taken using a 2 inch I.D. standard split spoon or a 2 inch I.D. shelly tube, although most of the latter type of samples were lost and had to be recovered in the disturbed state using the split spoon. The loss of the shelly tube samples is considered to be due to the relatively soft condition of the clay and to its sensitivity. Vane tests were carried out to determine the natural undisturbed and remoulded strength of the clay, and the results are plotted, together with the penetration profiles on enclosures 3 and 4.

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FIELD INVESTIGATION AND DESCRIPTION  
OF THE SUBSOIL PROPERTIES (Cont'd)

The average undisturbed shear strength of the clay as determined both by the vane and by an unconfined compression test is approximately 600 lbs. per foot, and the average sensitivity obtained by means of the vane test is approximately 3. The natural moisture content of the clay was found to be close to the liquid limit, as would be expected for the relatively low strength and penetration resistances noted.

A third borehole was originally planned for the centre of the river, but due to thin ice on the river this was not attempted.

DISCUSSION

At the time this investigation was undertaken a two span continuous design was suggested. On this basis the original soil investigation program required boreholes at each abutment and one for the centre pier. However the ice condition on the river precluded any possibility that the centre hole could be drilled at that time and it was temporarily abandoned. As a result of the information obtained from the two abutment boreholes, the two span design was considered undesirable due to the probable differential settlement to which such a structure would be subjected. Consequently design consideration was given to a single span structure and the centre borehole was completely abandoned.

To illustrate the line of reasoning resulting in the decision to change from a two span continuous structure to a single span structure, design calculations are included in the appendix. The design calculations are based upon design data supplied by Mr. Birkmeyer of Atkins and Associates, as follows.

Two Span Design

East abutment	90 Kips
West abutment	200 Kips
Centre Pier	400 Kips

Single Span Design

700 Kips per abutment



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DISCUSSION (Cont'd)

The value of  $C_c$ , the compression index, used in these calculations, has been taken as .25 which is somewhat higher than the value obtained by means of the Skempton equation  $C_c = .009 (L.L. - 10)$  where  $LL$  = Liquid Limit expressed as a percentage. However since the average sensitivity of the soil is approximately 3, this value of compression index is considered reasonable for the calculations of the approximate probable settlement.

Having determined, that for reasons of settlement, a single span simply supported structure is desirable, the problem resolves itself to determining the most economic method of support for such a structure. Calculations are shown giving three alternative proposals; by means of long friction piles in the underlying clay, by means of short piles within the upper sand, or by means of boxed caissons.

Briefly the results may be outlined as follows. Irrespective of the method of abutment support the centre pier of a two span bridge would have to be carried by friction piles. These piles would have to be 60 - 70 feet in length and the probable settlement of the pier would be of the order of 4 inches. Since the reaction loads vary, relative settlement could be expected between the abutments and the pier, and it is considered that the design problem of accommodating the probable stresses induced by differential settlement would present difficulties, and the design calculations would at best be based upon intelligent guesses.

Based upon this line of reasoning, a single span simply supported bridge which can accommodate fairly large settlements seems the best solution. As has been previously mentioned, a single span structure will have abutment loads of the order of 700 Kips. Discussion with Mr. Birkmeyer of Atkins and Associates, the design consultants, indicates that a through bridge was under consideration and that split abutments would fit the design considerations. Split abutments would give loads of 350 Kips per loading area. It is understood

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DISCUSSION (Cont'd)

that some differential settlement of a split abutment would not be critical from a structural viewpoint.

Based upon the calculations for a centre pier with a load of 400 Kips, it can be seen that pile lengths into the clay would be of the order of 40 - 50 feet and that settlements of 3 - 4 inches could be expected.

Calculations for support by means of piles or caissons within the upper sand layer are recorded in the appendix and indicate a safe capacity of the order of 11 tons per 12 inch diameter pile. These calculations are based upon the latest published work of G.G. Meyerhof \*. The calculations have, however, been treated somewhat conservatively since the effect of the well preserved timber content of the sand cannot be forecast. It seems, therefore, that a load test to determine the actual competency of this sand stratum might well prove profitable. The probable settlement of piles or box caissons would be about 3 inches.

Depth of Piles and cut off

The depth to which piles are driven is determined by either the necessity for obtaining sufficient length and hence friction in the case of friction piles or the elevation at which adequate end bearing may be obtained in the case of end bearing piles. Some indication of the lengths of friction piles which would be required to obtain adequate support have been given in the design calculation sheets. At this time it is not considered necessary to calculate the exact lengths required since the overall design has not been established.

In the case of piles within the sand stratum three factors are involved in determining the best driven length. They are:-

\* Penetration Tests and Bearing Capacity of Cohesionless soils  
G.G. Meyerhof - ASCE January 1956.

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DISCUSSION (Cont'd)

1. The elevation or range of elevations at which end bearing is greatest.
2. This elevation should be below the existing river bottom from scour considerations.
3. An adequate sand pad should be left between the pile tip and the clay stratum.

In the design calculations a sand pad 6 feet thick has been assumed. Based on an upper clay elevation of 575 feet, this gives a pile tip elevation of 581 feet. In hole No. 1, this represents a depth of 30.5 feet below existing ground surface. From the penetration profile it can be seen that this is the elevation at which the penetration resistance of the standard split spoon was greatest. Checking the penetration resistance at elevation 581 in borehole No. 2 or a depth of 32 feet, it can be seen that at this elevation the resistance is at its maximum value.

The lowest elevation of the existing river bed is 584 and thus piles to elevation 581 will be 3 feet below the bottom of the river.

Therefore, piles driven to elevation 581 satisfy the three requirements - adequate bearing capacity, adequate sand pad and adequate depth below the existing river level.

The point at which the piles should be cut off is a matter of design convenience except that their  $\frac{\text{length}}{\text{Breadth}}$  ratio must be greater than 10, and also that timber piles must be cut off below low water level.

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DISCUSSION (Cont'd)Scour Considerations

From a conversation with Mr. B. Wilkey, hydrology engineer for the D.H.O., it is not thought that the river is undergoing active scour at the present time. However, it is known that downstream of the bridge site there is a constriction in the river which causes flooding at peak periods. It is understood from local residents that a proposal exists for removing this constriction, and this may alter the flow pattern of the river. The extent of the flow change cannot be forecast without a detailed study. Consideration therefore, should be given to the possibility of future scour.

The suggested driving depth of piles within the sand stratum is such that they should be end bearing at a depth of 3 - 4 feet, below the lowest point of the existing river bed. It is not thought, therefore, that scour is critical so far as end bearing is concerned. However, lateral scour could occur exposing the pile groups, and causing considerable lateral thrust to be taken by the piles. It may therefore prove advisable to protect the abutments by a well-graded granular filter with rip-rap cover. In order to allow for some segregation during deposition this filter should be approximately 4 feet thick.

Lateral Thrust in the Pile Groups.

Provision must be made for the lateral thrust of the soil behind the abutments and breast wall, tending to move the abutments towards the river. With reference to enclosure No. 2, it can be seen that the height of sand exerting active pressure is approximately 20 feet, made up of approximately 10 feet where there is no passive counter part, and approximately 10 feet where the passive force may be considered to be small. Further, local information suggests that the river flow at low water is slight and that at least an appreciable amount of this 20 feet will at times be above the water table. For the lack of exact information, the whole 20 feet will be considered as being possibly above the water table at some time, and will therefore exert the full natural unit weight active thrust.

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DISCUSSION (Cont'd)Lateral Thrust in the Pile Groups (cont'd)

Assuming a coefficient of active earth pressure  $K_a$  of .2 the total active earth pressure  $P$  per foot width of the bridge will be

$$P = K_a \frac{1}{2} \gamma h^2 = 4440 \text{ p.l.f.}$$

where  $\gamma = 110 \text{ lbs. per square foot}$

$h = 20 \text{ feet}$

If bridge abutments and breast wall cover a total width of 30 feet, the total active pressure against one end of the bridge will be  $4440 \times 30 = 133.2 \text{ Kips}$

This force of 133 Kips cannot be accommodated by battering the outside piles of the pile group. For practical purposes with a force of this magnitude the lateral resistance of the individual piles should be neglected, particularly in view of the very loose nature of the sand.

It follows therefore, that some form of anchorage must be used. Outline calculations as shown in the appendix which indicate that an anchor wall will be required which should be about 40 feet back from the abutment face. It should prove practicable to tighten the anchorage bolts such that any compressibility of the soil is taken up during construction and thus ensuring the development of full passive pressures.

Due to the known rapid rate of fall of the river level, weep holes should be provided through the breast wall to ensure drainage from behind the wall and prevent the creation of a saturated unit weight condition. A coarse granular filter should be inserted between the natural sand and the weep holes.

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### CONCLUSIONS

On the basis of the computations made in the appendix of this report, the following conclusions would appear to apply.

1. A two span continuous bridge does not seem practicable due to the probable large differential settlements between the points of load application.
2. A two span structure may be supported either by friction piles into the underlaying clay or by means of support within the upper sand region.
3. Support by means of friction piles will require piles 60 - 70 feet long and settlements of 3 - 4 inches may be expected.
4. The upper sand stratum may be used for support either by driving end bearing piles to elevation 581 or by setting a box type caisson at the same elevation.

The calculated end bearing values for the short piles in the sand stratum are thought to be conservative and it may prove economic to carry out a load test to ascertain the actual bearing capacity of this stratum at the suggested driven depth.
5. Settlements beneath either end bearing piles or caissons at elevation 581 may be expected to be about 3".
6. Due to a possible change in flow pattern caused by suggested downstream channel alterations, protection against scour at the abutments should be provided by means of a well graded granular filter with rip-rap cover. This protection should be carried about 25 feet upstream and downstream of the bridge.

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CONCLUSIONS (Cont'd)

7. Provision will have to be made to accommodate the lateral thrusts on the abutments and breast walls by some form of anchor wall. Outline calculations shown in the appendix indicate that the wall should be 40 feet from the face of the abutment.

8. Due to the rapid changes in river elevation adequate weep holes must be provided to ensure drainage of water from behind the breast wall after a sudden drop in water level.

*Peter E. Morton Monk.*

Peter E. Morton Monk.



## A P P E N D I X

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1. Design calculations for friction piles into the clay stratum.
  - (a) Two Span Continuous bridge abutment Angus end.
  - (b) Two Span Continuous bridge Centre pier.
2. Design Calculations for foundations in the upper sand stratum.
  - (a) Single Span simply supported structure on piles.
  - (b) Single Span simply supported structure supported by a hollow box type caisson.
3. Outline calculations for the accommodation of the lateral thrust on the abutments.



DESIGN SHEET No. 1DESIGN CALCULATIONS FOR FRICTION PILES INTO CLAY STRATUM2 Span ContinuousAbutment Angus End

Load = 200 Kips    Average Undisturbed Cohesion  $c = 600$  lbs. per square foot

Average Sensitivity = 3

- ∴ Reasonable to use Factor of Safety  $F.S = 3$  on Undisturbed value.

Say piles 67 feet long cut off at elevation 604 feet

Elevation of top of clay strata 575 feet

∴ Length of Pile in sand = 29 feet

∴ Length of Pile in Clay = 31 feet

Take Pile diameter  $\phi = 12$  inches

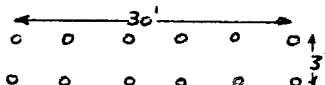
Consider Individual Pile Action

Surface Perimeter =  $31 \times 3.14 \times 1 = 97.5$  square feet per pile

∴ Safe Pile Load =  $97.5 \times \frac{c}{3} = 97.5 \times 200 = 19.5$  Kips per pile

Load = 200 Kips    ∴ Piles required  $\frac{200}{19.5} = 10.15$  say 12 for symmetry

Assume Pile grouping as shown



Group perimeter 66 feet

Group area 90 square feet

Consider Group Action

Group Surface Perimeter =  $66 \times 31 = 2046$  square feet

Total Group Shearing  
resistance around

Perimeter =  $2046 \times 200 = 409.2$  Kips

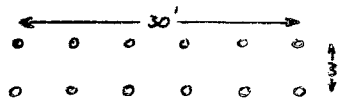
∴ Pile group action is safe.

Settlement

Load = 200 Kips

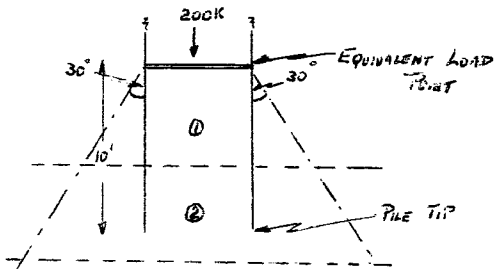
Length of pile in clay = 31 feet

Pile group thus



Assume equivalent load distribution at 10 feet above bottom of pile i.e. at elevation 554 feet or 57 feet below existing ground surface.

Pile group plan area =  $30 \times 3$  square feet



Assumed load spread with depth

Calculate Settlement in 6 feet strips.

Consider first 6 foot strip ① and work on loading intensity at mid point of strips.

$$\text{Settlement } S = \frac{D}{1+e} C_c \log \frac{P_o + P}{P_o}$$

$P_o$  = Overburden Pressure

$P$  = Applied Pressure

where  $D$  = Thickness of Strip under settlement

$e$  = Voids Ratio

$C_c$  = Compression Index

Assume  $C_c$  = .25

$e$  = .65

Calculate for  $P_o$  at mid point strip i.e. depth 60 feet

Weight of sand above the water table  $\gamma = 110 = 110 \times 10 = 1100$   
lbs. per sq. ft.

Weight of sand below the water table  $\gamma' = 60 = 60 \times 27 = 1620$

Weight of clay  $\gamma' = 60 = 60 \times 23 = 1380$   
lbs. per sq. ft.

Total 4100 lbs. per square foot =  $P_o$

Equivalent footing size at equivalent load point = 30 x 3 square feet

With pressure distribution as shown equivalent footing size at centre of first strip

$$= (30 + 3.5)(3 + 3.5) = 33.5 \times 6.5 = 217 \text{ square feet}$$

i.e. equivalent loading at mid strip  $\frac{200K}{217} = 925$  lbs. per square foot =  $P$

$$\therefore S = \frac{6 \times 12}{1.65} \times .25 \log \frac{4100 + 925}{4100} = 1.04 \text{ inches}$$

For next 6 foot Strip  $P_o = 4460$  lbs. per sq. foot  $P = 540$  lbs. per sq. foot

$$\therefore S' = \frac{72 \times .25}{1.65} \log \frac{4460 + 540}{4400} = .55 \text{ inches}$$

$\therefore$  Total Settlement of order of 2 inches.

## 2 Span Continuous

### Centre Pier

Load = 400 Kips    Average Undisturbed Cohesion  $c$  assumed the same  
as for abutment calculation = 600 lbs. per sq.foot

Factor of Safety = 3

Assume length of pile in clay = 50 feet

Take pile diameter  $\phi$  = 12 inches

### Consider Individual Pile Action

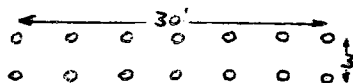
Surface Perimeter of Single Pile  
=  $50 \times 3.14 \times 1 = 157$  square feet

∴ Safe Pile load =  $157 \times \frac{600}{3} = 31.4$  Kips per pile

Load = 400 Kips    Piles required  $\frac{400}{31.4} = 12.7$  for symmetry use 14

### Consider Group Action

Assume group thus



Group surface perimeter =  $66 \times 50 = 3300$  square feet

Total Group shearing resistance around perimeter  $3300 \times 200 = 660$  Kips

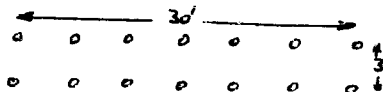
∴ Group action is safe

2 Span ContinuousCentre Pier

## Settlement

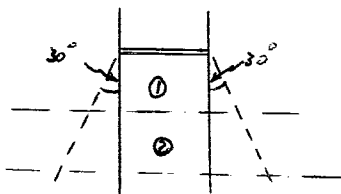
Load = 400 Kips    Length of pile in clay 50 feet

Pile Group thus



Assume equivalent load point 17 feet above bottom of pile  
i.e. at elevation 541 feet or 43 feet below river bottom.

Pile group plus area = 30 x 3 square feet



Assumed load spread with depth

Calculate settlement in 6 foot strips

Consider first 6 foot strip ① and work on loading intensity at mid point of strip.

Calculate for  $P_o$  at mid point of strip i.e. depth of 46 feet below river bottom

Weight of saturated sand  $\gamma' = 60 = 10 \times 60 = 600$  lbs.per sq.foot

Weight of clay  $\gamma' = 60 = 36 \times 60 = 2160$

2660 lbs.per sq.foot  
=  $P_o$

Equivalent footing size at equivalent load point 30 x 3 sq. feet

With pressure distribution as shown equivalent footing size at centre of first strip =  $(30 + 3.5)(3 + 3.5) = 217$  square feet

i.e. Equivalent loading at mid strip =  $\frac{400K}{217} = 1.84$  Kips per sq.foot  
= P

$$\therefore S = \frac{6 \times 12}{1.65} \times .25 \log \frac{2660 + 1840}{2660} = 2.5 \text{ inches}$$

For next 6 feet strip  $P_0 = 2660 + (6 \times 60) = 3020$  lbs.

New equivalent area =  $(3 + 10.4)(30 + 10.4) = 13.4 \times 40.4 = 540$  sq. feet

$\therefore$  Equivalent loading 740 lbs. per square foot.

$$\text{Hence } S' = \frac{72}{1.65} \times .25 \log \frac{3020 + 740}{3020}$$

$$= 1.04 \text{ inches.}$$

This indicates a total settlement of the order of 4 inches. This calculated value is possibly somewhat too high since the clay is probably slightly over consolidated to the extent that the original overburden probably reached the elevation of the existing river banks. However, it is felt that the above calculation illustrates the magnitude of the settlement problem.

Single Span simply supported structure with bearing in the sand stratum.

Assuming split abutments the load per half of split abutment = 350 Kips. From the penetration profiles shown on enclosures 3 and 4 the penetration resistance between elevations 585 and 576 varies between 15 and 23 blows on the standard split spoon.

Meyerhof states that for pile lengths where

$\frac{\text{length in sand}}{\text{diameter}}$  ratio is greater than 10

$$Q_p = \frac{4N A_p}{F.S.} + \text{Skin Friction} \quad \text{expressed in tons}$$

where  $Q_p$  is the allowable load per pile

$N$  is the number of blows on the standard split spoon

$A_p$  is the cross sectional area of the pile tip

$F.S.$  is the factor of safety

Ignoring skin friction which should be less than 10% of the total

$$\text{we have } Q_p = \frac{4N A_p}{F.S.}$$

Around elevation 582 the penetration resistance is 20 blows per foot, and it is thought that pile tests might demonstrate that  $N = 20$  is a reasonable value to use in the Meyerhof equation. However, without the corroborating evidence of pile tests some consideration must be given to the effect of the timber fragments in the sand. Since their effect is not known it will be assumed that they reduce the bearing capacity and  $N = 10$  will be used in the calculation of allowable pile loads. This reduction in the effective  $N$  should also account for any slight variation in sand density at locations other than were tested.

Assuming  $N = 10$  Diameter of piles = 12 inches and  $F.S. = 3$

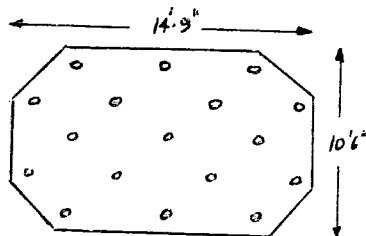
$$Q_p = \frac{4 \times 10 \times .785}{3} = 10.5 \text{ Tons per pile}$$

or 21 Kips per pile

For a split abutment load of 350 Kips  $\frac{350}{21} = 17$  piles are required

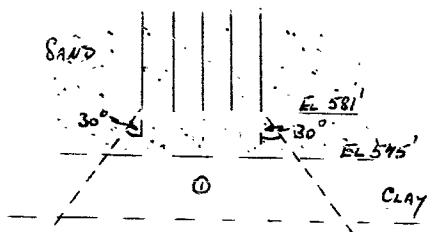
# Settlement for piles in upper sand stratum

Assume pile group



Pile group pla area 13 x 9 square feet

Say piles driven to 58l giving approximately 6 feet to clay layer.



Assumed load spread with depth

Consider 10 feet clay strip

Effective area at centre of strip  $(13 \times \frac{22}{\sqrt{3}})(9 \times \frac{22}{\sqrt{3}}) = 550$  sq.feet

Intensity of loading  $\frac{350K}{550} = 630$  lbs. per sq.foot

Calculate for  $P_o$  at mid strip

Weight of sand above water table  $\gamma = 110$ .  $10 \times 110 = 1100$  lbs.  
per sq.foot

Weight of sand below water table  $\gamma' = 60$ .  $26 \times 60 = 1560$

Weight of clay  $\gamma' = 60$ .  $5 \times 60 = 300$

Total 2960 lbs. per sq. foot =  $P_o$



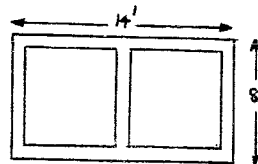
$$\begin{aligned}
 \text{Settlement } S &= \frac{D}{1+e} C_c \log \frac{P_o + P}{P_o} \\
 &= \frac{10 \times 12}{1 + .65} \times .25 \log \frac{2960 + 630}{2960} \\
 &= \frac{120}{1.65} \times .25 \times .08 = 1.46 \text{ inches}
 \end{aligned}$$

$$\begin{aligned}
 \text{For second 10 feet strip } S_1 &= \frac{120}{1.65} \times .25 \log \frac{3560 + 200}{3560} \\
 &= .46 \text{ inches}
 \end{aligned}$$

Total clay settlement is therefore 2 inches

To this clay settlement must be added the settlement to be expected in the sand. This sand settlement should occur immediately after the application of the load and may be expected to be  $1/2$  to  $3/4$  inches. Thus the probable total settlement is  $2\frac{1}{2}$  - 3 inches.

An alternative to the piles within the sand stratum is a box caisson with a plan as shown. Assuming the bottom of the caisson to be set 6 feet above the top of the clay and with a plan area approximately equal to that of the pile groups, the intensity of loading would also be approximately that of the pile group area. Consequently the probable settlement of a box type caisson could be expected to be  $2\frac{1}{2}$  - 3 inches.

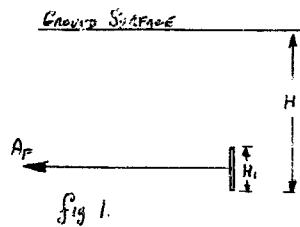


### Calculation of anchor blocks: size and the distance required from abutment.

As has been shown in the discussion section of this report the lateral thrusts cannot be accommodated successfully by means of batter piles and some form of anchor block is therefore required. Under worst conditions it has been shown that a force of 4400 lbs. per foot run of abutment could be developed. The whole of this force will be assumed acting on the anchor beams.

The force required to pull an anchor beam with a height  $H_1$ , see Figure 1, is approximately equal to the bearing capacity of a continuous footing with a width  $H_1$ , whose base is located  $H - \frac{H_1}{2}$  below the

ground surface. On the basis of the standard penetration measurements the safe bearing capacity of such a footing is approximately equal to 500 lbs. per square foot and therefore  $H_1 = 9$  feet. This illustrates that the anchor beam must take the form of an anchor wall.



Assuming an anchor wall form of construction and referring to Figure 2 we have

$$A_p = \frac{P_p - P_a}{F.S.} \quad \text{where F.S. = Factor of safety.}$$

$$\text{i.e. } 2 \times 4400 = P_p - P_a$$

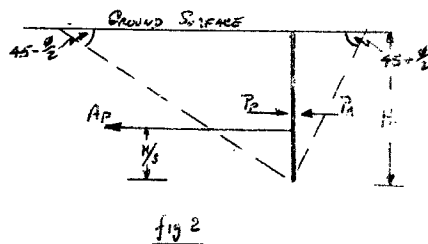
$$\text{Now } P_p = \frac{1}{2} \gamma H^2 \tan^2 (45 + \frac{\phi}{2})$$

$$\text{and } P_a = \frac{1}{2} \gamma H^2 \tan^2 (45 - \frac{\phi}{2})$$

$$\text{Assuming } \phi = 30^\circ$$

$$P_p = \frac{1}{2} \gamma H^2 \tan^2 60$$

$$P_a = \frac{1}{2} \gamma H^2 \tan^2 30$$



Now in worst flood conditions the ground water can reach grade level

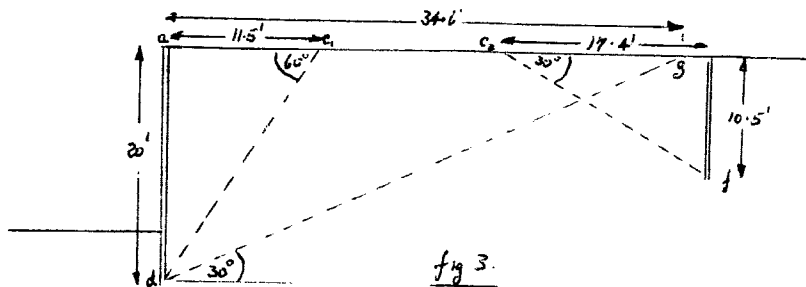
$$\therefore \gamma \text{ should read } \gamma' = 60 \text{ lbs. per sq. foot}$$

$$\therefore P_p - P_a = \frac{1}{2} \times 60 \times H^2 (3 - 1/3)$$

$$\therefore H^2 = \frac{2 \times 4400 \times 2}{60 \times 2.67} = 110$$

$$\therefore H = \underline{10.5 \text{ feet.}}$$

The minimum distance between the abutment and the anchor wall is determined by the condition that the base  $e_2$  of the passive wedge adjoining the anchor wall should not intersect the base  $d_2$  of the active wedge adjoining the abutment see Figure 3.

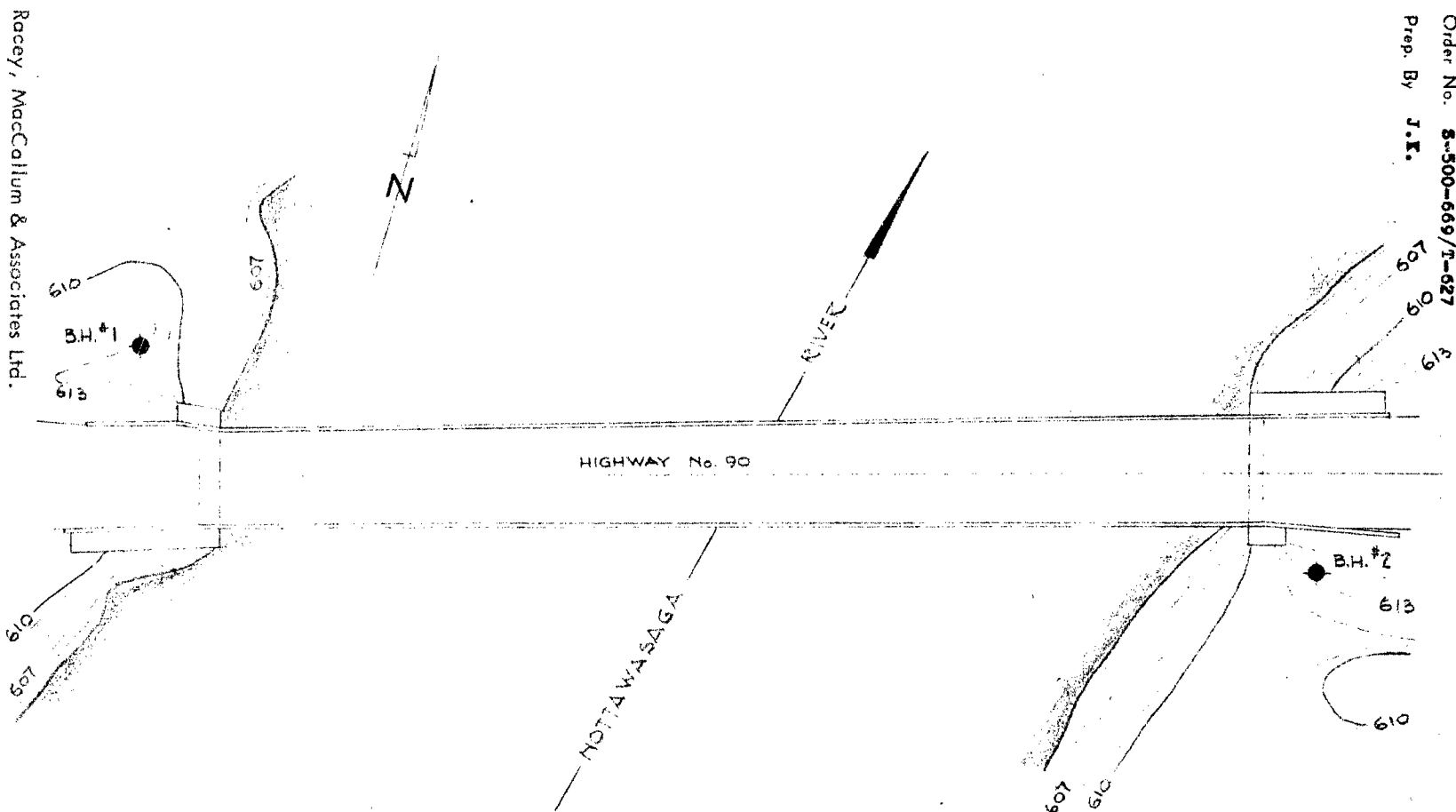


A further requirement is that the upper edge of the anchor wall should be beneath the slope line  $d g$  which rises from the lower edge of the abutment at an angle  $\phi$  to the horizontal.

Assuming the anchor wall has a depth of 10.5 feet and that the zone of active failure can extend to a depth of 20 feet at the abutment face, it can be shown (see Figure 3) that the anchor wall should be at least 35 feet from the abutment.

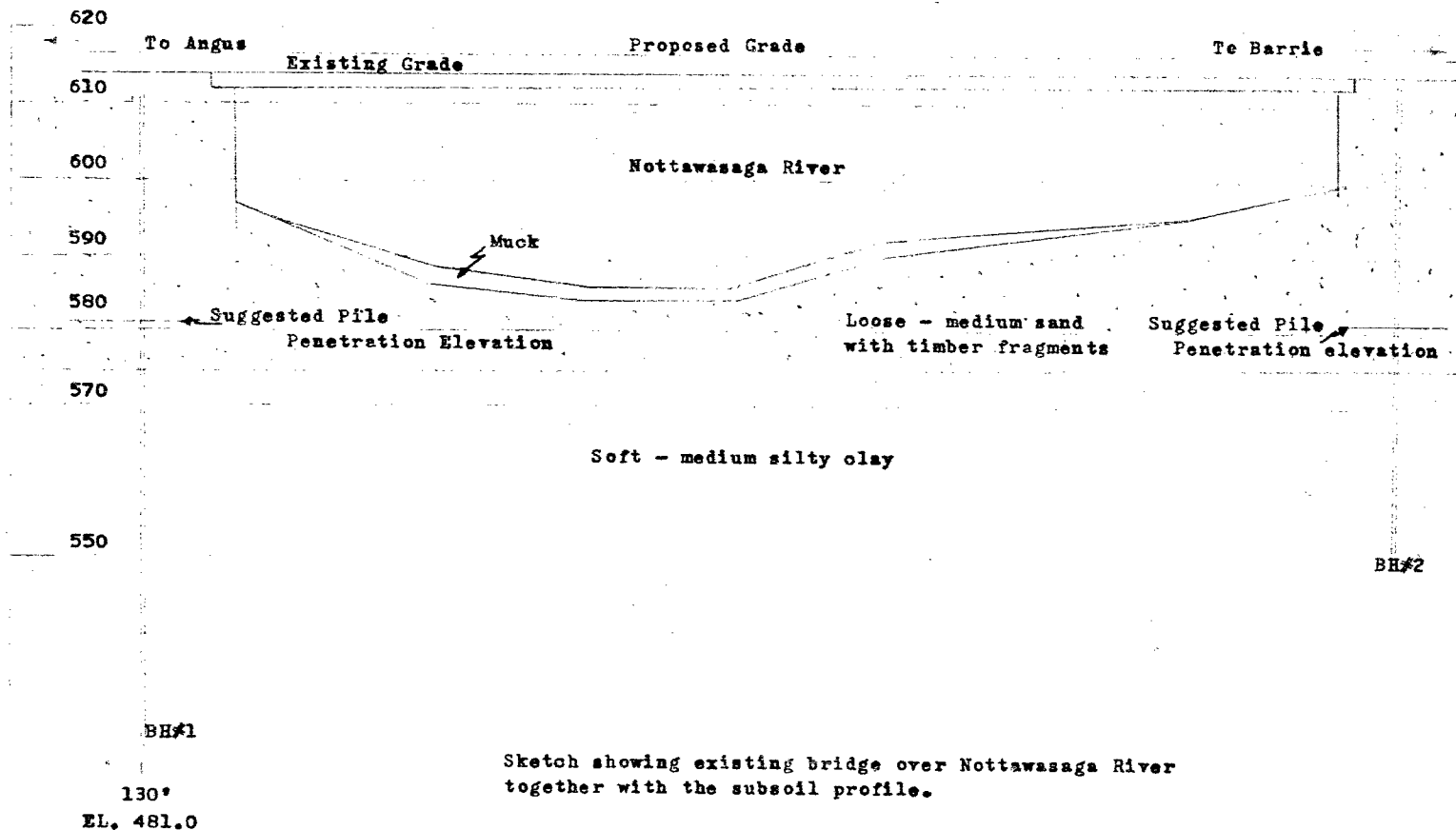
Order No. S-500-669/T-627  
Prep. By J.E.

Enclosure No. 1



PLAN SHOWING FOUNDATION INVESTIGATION BOREHOLES  
AT  
EXISTING BRIDGE OVER NOTTAWASAGA RIVER

Racey, MacCallum & Associates Ltd.



**RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole: 1

Project: Foundation investigation for proposed bridge re- Field Supervision: C.W.  
Location: placement on H.W. 90 over Nottawasagen River near Driller: M.C.  
Hole Location: See Enclosure No. 1. Angus, Ontario. Prep.: P.F.  
Hole Elevation and Datum: 611.5 Checked:  
Field Work Begun ..... Ended ..... Date:

Field Supervision: C W.

Driller: M.C.

Prep.: P.H.

Checked:

Date:

### LEGEND

### Sampling Method

2" Dia. split tube

2" Shelby tube

### Penetration Resistance

2" Split tube

2" Dia. Cone

### Casing

### Strength

- Unconfined compression
- Vane test and sensitivity

### Consistency

### Natural moisture

Liquid limit

Plastic limit

### Natural Unit Weight

[illegible]

**RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole: 2

Project: Foundation investigation for proposed bridge re- Field Supervision: C.W.  
 Location: placement on H.W. 90 over Nottawasagen River near Driller: M.L.  
 Hole Location See Enclosure No. 1. Angus, Ontario. Prep.: P.M.  
 Hole Elevation and Datum: 613 Checked:  
 Field Work Begun ..... Ended ..... Date:

**LEGEND**

Sampling Method  
 2" Dia. split tube  
 2" Shelby tube

Penetration Resistance  
 2" Split tube  
 2" Dia. Cone  
 Casing

**Strength**

Unconfined compression  
 Vane test and sensitivity

**Consistency**

Natural moisture  
 Liquid limit  
 Plastic limit

Natural Unit Weight \*

