Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Division,

(Foundation Section)

Attention: Mr. K. L. Kleinsteiber,
Municipal Bridge Liaison Engr.

August 29, 1962.

JOHN D. PATERSON, SOILS CONSULTANT, OTTAWA.
(Letter dated July 31, 1962)

Re: Payne River Bridge - United Counties of Stormont, Dundas & Glengarry.

We have received a letter dated July 31, 1962. from the Soil Consultant, John D. Paterson of Ottawa, with the attached log of the additional borehole that was carried out at the above-mentioned site. Also attached, was a test result summary sheet and a plan showing the location of the boring. Copies of these are attached to this memo.

We had reviewed the mentioned data but were unable to make our final comments because the question of elevations was still not clarified. Because the Bridge Consultant, Mr. Dave Cramm from C. C. Parker & Associates was on holiday, the work had to be postponed until his return.

The information concerning the different elevations we obtained from the Bridge Consultant, is now as follows:

Present road grade elev.	102.0 ft.
Proposed road grade elev.	108.3 ft.
Possible and probable final creek bed elevation after scour has taken place.	80.0 ft.
The maximum embankment height	28.3 ft.

cont'd. /2 ...

Mr. A. M. Toye
Attn: Mr. K. L. Kleinsteiber
August 29/62.

For the stability calculation, using Taylor's curves, we have taken the following values:

Undrained shear strength	550 p.s.f.
Unit weight of soil	125 p.c.f.
Slope height	28 ft.
Thickness of soft layer	14 ft.
Depth factor $D = \frac{28}{14}$	1.5
Stability number for 2:1 slope	0.16
Stability number for 1.5:1 slope	0.166

The calculation of the factor of safety, using the above values, gave the following results:

2:1 S.F. = 0.983 1.5:1 S.F. = 0.946

The above figures differ from the figure given in the Consultant's letter, because different stability numbers were used. Because there is at least 14 feet of soft clay under the 28 ft. high embankment, a depth factor of 1.5 has to be used. This assumption, of course, provides different, but we think, correct values of stability numbers.

From the above explanation, it becomes evident that the embankment would be unstable unless there are some other factors or influences that would contribute beneficially to its stability. Such factors are the consolidation and subsequent shear strength increase of the subsoil under the existing fill and the upper clay crust having a higher shear strength than the lower layer, but they have either not been established or not incorporated in the calculation.

It is, therefore, our opinion that unless additional and reliable evidence is provided that would prove the new embankment to be stable, the project has to be rejected as unstable and therefore unsafe.

AGStermac
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF
Attach.
cc: Foundations Office
Gen. Files.

JOHN D. PATERSON, B.SC., P.ENG.
CONSULTING ENGINEERS & GEOLOGISTS
OTTAWA, CANADA
MEMBERS:
ASSOC. OF PROFESSIONAL ENGINEERS OF ONTARIO
AMERICAN CONCRETE INSTITUTE

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TEL. PA 9-3722

July 31, 1962.

Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer,
Materials and Research Division,
Ontario Department of Highways,
Parliament Buildings,
Toronto, 5, Ontario.

Payne River Bridge – United Counties of Stormont, Dundas & Glengarry.

Dear Mr. Stermac,

This is further to my letter of July 20th in which we advised you that additional tests on the clay would be carried out. One additional test hole was put down at the location shown on the attached Test Boring Plan. It was not possible to get any closer to the present bridge site without considerable expense. The clay soil was sampled every two feet and the results are shown on the Soil Profile Sheet included. Also attached you will find the results of laboratory tests in graphical and table form.

From these results we have arrived at an average shear strength for the clay layer of 550 pounds per square foot. We have not included the result from TW 20 because it is obviously from the dried crust.

Using the value of 550 for shear strength, a stability number of .14 and a safety factory of 1.5, the safe height becomes 21 feet.

Assuming excavation to Elevation 80, H = 24 and the factor of safety for this height is 1.3.

I trust that this clarifies the situation for you and if you have any further questions please do not hesitate to get in touch with me.

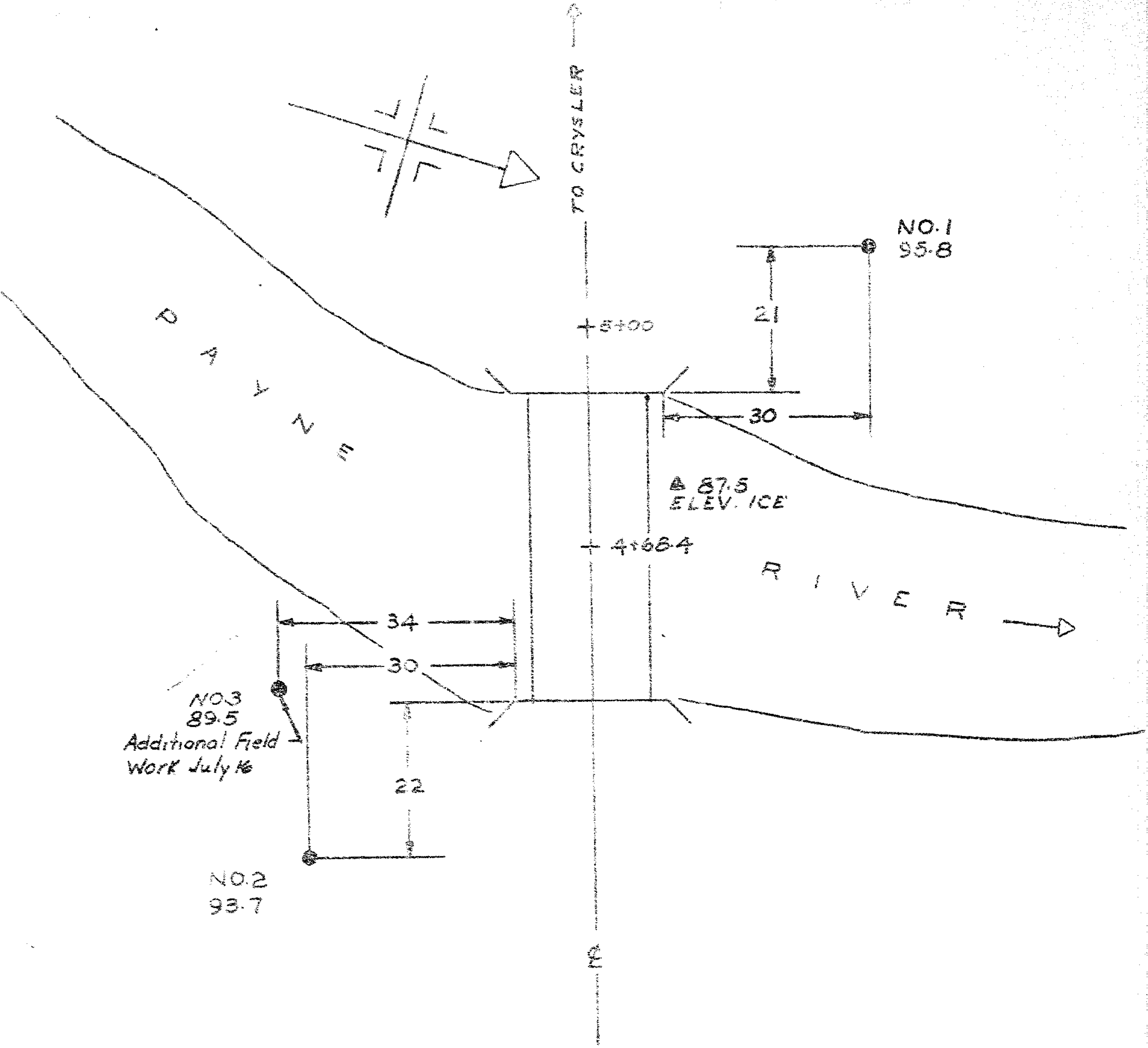
Yours truly,

J. D. Paterson, P. Eng.

Encls.

Copy to:
Mr. D. C. Cramm, P. Eng.,
C. C. Parker & Associates, Ltd.,
795 Main St., W.,
Hamilton, Ontario.

JDP/MMC.



BM ELEV 100.0
NAIL IN T.P.
37 LT of STAT 2+94

TEST BORING PLAN
PROPOSED BRIDGE
COUNTY ROAD 12
LOT 16 CON 9 & 10
TOWNSHIP FINCH

SCALE 1" = 20'   MAR. 1962

JOHN D. PATERSON
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND LABORATORY TESTS

Location: County Road No. 12, Township of Finch.

Elevation (Zero Depth): 89.5.
Remarks: Test Boring only.

Borings by: F.E.Johnston Drilling Co.,Ltd. Date: July 16,1962.

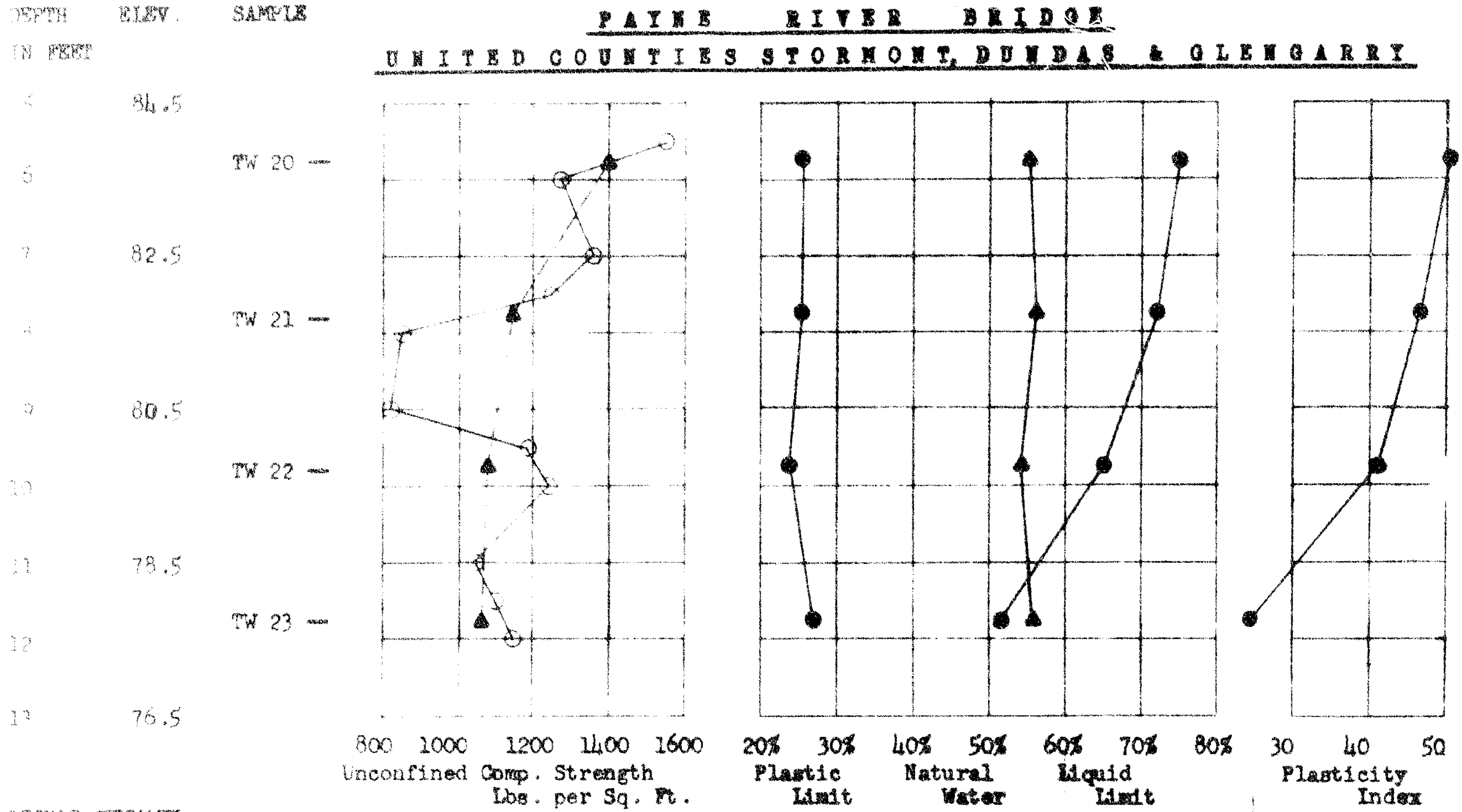
Sheet No: 1 of 1
Hole No: 3

Blows per Foot	Soil Description	Samples		U'c T/ft'	N	Depth in Feet	Elev.	Moisture Content Per Cent. 30 40 50 60 70
	Ground Surface					0	89.5	
	Stiff to medium stiff, silty grey clay with organic inclusions (old branches).					2		Ground Water Level - 1 Foot. Elev. Stream 87.1 - July 16/62.
	5.1					4		
	Medium stiff, silty, pinkish grey clay, with an odd white shell, a few isolated sand grains and intermittent fissuring. Thin lenses of silty, pink clay interbedded with the pinkish grey clay. Clayey silt at 13 feet.	TW	20	0.70		6		
		TW	21	0.57		8		
		TW	22	0.54		10	79.5	
		TW	23	0.53		12		
		TW	24	Too silty to test.	9 14.3' to 14.7'			
	14.3					14		
	Loose to medium dense silt. (Thickness of silt layer based on sample Hole 2).					16		
						18		
	20					20	69.5	
						22		
						24		

TEST HOLE NO. 3

PAYNE RIVER BRIDGE

UNITED COUNTIES STORMONT, DUNDAS & GLENGARRY



RESULT SUMMARY:

Sample No.	Plastic Limit	Liquid Limit	Plasticity Index	Natural Water Content	Average Unconfined Comp. Strength	Average Shear Strength
TW 20	25.4	75 ± 1	50.5 ± 1	55.2	1400	700
TW 21	25.3	72 ± 0.5	46.5 ± 0.5	56.0	1150	575
TW 22	23.8	65 ± 1	41 ± 1	54.1	1080	540
TW 23	26.9	51.5 ±0.5	24.5 ± 0.5	55.5	1060	530

JOHN D. PATERSON, B.SC., P.ENG.
CONSULTING ENGINEERS & GEOLOGISTS
OTTAWA, CANADA
MEMBERS:
ASSOC. OF PROFESSIONAL ENGINEERS OF ONTARIO
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Laboratory:
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TEL. PA 9-3722

July 20th, 1962.

Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer,
Materials & Research Division,
Department of Highways,
Parliament Buildings,
Toronto, 5, Ontario.

Payne River Bridge
United Counties of Stormont, Dundas & Glengarry

Dear Mr. Stermac,

After receiving your letter of July 10th it was decided that additional samples of the clay at the abovementioned bridge site should be taken in order to evaluate conditions more accurately. This decision was made with the concurrence of the consulting engineers, C. C. Parker & Associates.

The samples have now been taken and tests are underway to determine the unconfined compressive strength, Atterberg limits and moisture contents. The results will be forwarded to you along with our revised calculations for stability based on the new test results.

Yours truly,

J. D. Paterson, P. Eng.

JDP/MMC.

Materials and Research Division

July 10, 1962.

John D. Paterson, B.Sc., P.Eng.,
Consulting Engineers & Geologists,
250 Besserer Street,
Ottawa, Ontario.

Payne River Bridge
United Counties of Stormont, Dundas & Glengarry

Dear Mr. Paterson:

This is to reply to your letter of June 11th, 1962 addressed to Mr. K. Y. Lo, Supervising Foundation Engineer of this Section.

We are sorry to say, but even after the exchange of letters with you, we feel that the investigation has not procured the necessary information and we are therefore unable, at this stage, to ascertain whether problems actually exist or not.

We feel that, irrespective of the new grade elevation, the properties of the soft silty clay should have been determined in order to establish the stability of the banks during construction when excavation down to elevation 80.0 will be carried out.

As far as your interpretation of the information from the N.R.C. is concerned, we feel that either there is a typographical error in your letter, or there is some other mistake because the statement as it reads in your letter of June 11th, 1962, is incorrect and is a misinterpretation of whatever the N.R.C. has published on the local clays. The calculations based on your interpretation of the N.R.C. findings, are therefore erroneous and do not apply to the case in question - i.e., to the construction or end of construction stability problem.

Sincerely yours,

A G Stermac

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF

cc: Mr. K. L. Kleinsteiber
Foundations Office
Gen. Files.

JOHN D. PATERSON, B.Sc., P.Eng.
CONSULTING ENGINEERS & GEOLOGISTS
OTTAWA, CANADA
MEMBERS
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Laboratory:
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TEL. PA 9-3722

June 11th, 1962.

Mr. K. Y. Lo, P. Eng.,
Supervising Foundation Engineer,
Materials and Research Division,
Ontario Department of Highways,
Parliament Buildings,
Toronto, 5, Ontario.

Payne River Bridge
United Counties of Stormont, Dundas and Glengarry

Dear Mr. Lo,

This is in reply to your letter of June 7th requesting further clarification on the stability studies at the above-mentioned site.

In the original stability analysis mentioned in our original report we used an undrained shear strength of 600 lbs. per square foot and a stability number of .181.

After receiving the site plan and profile from C. C. Parker it was obvious that the present bridge approaches have stood up satisfactorily in spite of approximately 30 feet of low strength clay. Therefore, a second look was taken at the values obtained in our tests and it was decided to include the values obtained with the pocket penetrometer (average of 6 readings) in obtaining the average shear strength of the soil. The lowest value of .36 ton per square foot obtained in the unconfined compressive strength tests was excluded because of apparent disturbance. The average shear strength then worked out to 800 lbs. per square foot.

Without considering the depth factor the stability number was also revised. This revision was based on recent information obtained from the National Research Council which indicates that the local clays have an angle of shearing resistance and are not purely cohesive. Therefore, instead of using $\phi = 0$ a five-degree angle was used. From Taylor's graph the stability number became 0.12. The value of 0.181 was used in the original calculation only and hence the apparent disagreement in our figures.

If we consider ......

If we consider the depth factor and revert to $\phi = 0$, $D_F = \frac{34}{25} = 1.36$ and the stability number for a slope of 2 horizontal to 1 vertical is 0.15. H then becomes 28.5 feet for a factor of safety of 1.5. For H = 34 ft. (proposed grade of approaches at Elevation 108) the factor of safety is 1.25.

Referring back to our revised calculations in which C = 800, SN = 0.12, $\phi = 5°$ the factor of safety against slip failure is 1.57 for H = 34 ft.

We should not overlook the fact that the present approaches are approximately 10 feet high and no doubt the underlying clay has consolidated to some extent with subsequent increase in shear strength. The test holes were put down at the sides of these approach embankments where the soil is in its original state.

It is planned to increase the height of the approaches by only 4.3 feet but because of pile driving and vibrations from construction equipment the recommendation for open abutments was made.

Failure strains of three unconfined compression tests were as follows:

TW 14	13 - 13½ ft.	-	0.294 inch
TW 14	12½- 13 ft.	-	0.175 "
TW 15	15½- 16 ft.	-	0.151 "

Yours truly,

J. D. Paterson

J. D. Paterson, P. Eng.

JDP/MMC.

INSPECTION SERVICES
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SOIL INVESTIGATIONS

JOHN D. PATERSON, B.SC., P.ENG.
CONSULTING ENGINEERS & GEOLOGISTS
OTTAWA, CANADA
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Head Office:
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TEL. PA 9-3722

June 11th, 1962.

Mr. K. Y. Lo, P. Eng.,
Supervising Foundation Engineer,
Materials and Research Division,
Ontario Department of Highways,
Parliament Buildings,
Toronto, 5, Ontario.

Payne River Bridge
United Counties of Stormont, Dundas and Glengarry

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We should not overlook the fact that the present approaches are approximately 10 feet high and no doubt the underlying clay has consolidated to some extent with subsequent increase in shear strength. The test holes were put down at the sides of these approach embankments where the soil is in its original state.

It is planned to increase the height of the approaches by only 4.3 feet but because of pile driving and vibrations from construction equipment the recommendation for open abutments was made.

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TW 15	15½- 16 ft.	-	0.151 "

Yours truly,

J. D. Paterson

J. D. Paterson, P. Eng.

JDP/MMC.

Materials and Research Division

June 7, 1962.

John D. Paterson, B.Sc., P.Eng.,
Consulting Engineers & Geologists,
250 Besserer Street,
Ottawa, Ontario.

Payne River Bridge
United Counties of Stormont, Dundas & Glengarry

Dear Mr. Paterson:

Further to your letter of June 1, 1962, there are still a few questions unclarified. The points at issue are the following:

(a) According to your report and subsequent corrections stated in your letter in question, four unconfined compression tests were performed on samples TW 14 and TW 15. The average unconfined compression strength from sample TW 14 is 0.56 T/ft.^{2}, and from TW 15, 0.36 T/ft.^{2}. The overall average is then 0.46 T/ft.^{2}, giving an undrained shear strength of 460 p.s.f. However, in your analysis, a value of 800 p.s.f. has been used. We would also like to know the failure strains of each of your unconfined compression tests.

(b) Taking your value of undrained shear strength of 800 p.s.f., unit weight of soil 125 p.c.f., and using Taylor's stability number of 0.181, the critical height of the embankment is given by -

$$H = \frac{800}{.181 \times 125} \quad \text{for} \quad F = 1$$

$$= 35 \text{ feet.}$$

$$H = \frac{800}{.181 \times 125 \times 1.5} \quad \text{for} \quad F = 1.5$$

$$= 24 \text{ feet.}$$

cont'd. /2 ...

John D. Paterson,
Consulting Engineers, Ottawa. June 7, 1962.

These values do not agree with those stated in your report and your letter dated May 14, 1962, to Mr. D. C. Cramm.

Furthermore, for a slope of 2:1 and a depth factor D approximately 2, the stability number according to Taylor's chart, is approximately 0.17. The stability number of 0.181 corresponds to D = ∞ which we believe does not apply in this case.

Yours very truly,

KYL/MdeF

K. Y. Lo,
Supervising Foundation Engineer
For:
A. G. Stermac,
Principal Foundation Engineer

cc: Mr. G.C.E. Burkhardt
Foundations Office
Gen. Files

JOHN D. PATERSON, B.SC., P.ENG.
CONSULTING ENGINEERS & GEOLOGISTS
OTTAWA, CANADA
MEMBERS:
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June 1, 1962.

Mr. K. Y. Lo, P. Eng.,
Supervising Foundation Engineer,
Materials and Research Division,
Ontario Department of Highways,
Parliament Buildings,
Toronto, Ontario.

Payne River Bridge
United Counties of Stormont, Dundas & Glengarry

Dear Mr. Lo,

This is in reply to your letter of May 23rd in which you have asked for additional information regarding the stability problem at the above-mentioned proposed new bridge site.

Regarding your first query, (a), actually four unconfined compressive strength tests were made in Hole No. 2. There is a typographical error and the sample shown as SS 14 should be TW 14. The values shown for TW 14 and TW 15 are averages of two tests each taken from the same sample tube.

At the time this investigation was done no information was supplied by C. C. Parker regarding new grades, etc., so that when it was made obvious during the investigation that the bridge itself would have to be founded on piles careful sampling of the clay down to a depth of approximately 20 feet ~~had~~ WAS not been carried out.

After receiving a letter from Mr. Cramm dated April 27th which was accompanied by a site plan and profile it was realized that a stability problem might exist. Our letter of May 14th followed a study of this plan. Atterberg Limits and water content of the clay were not determined.

With regard ....

With regard to your second query, (b), the stability computations were made using D. W. Taylor's mathematical solution taken from a publication known as "Soil Mechanics for Road Engineers" published by the Department of Scientific and Industrial Research, Road Research Laboratory, London, England.

A shear strength of 800 p.s.f. and a stability number of .181 were used. The unit weight of the clay was taken as 125 pounds per cubic foot.

With respect to artesian pressure it is considered most likely that this water came from the bedrock. In a letter to Mr. Cramm dated April 6th we advised that Hole No. 1 was flowing at the rate of approximately two gallons per minute and that Hole No. 2 had stopped flowing. At this time approximately three weeks had elapsed since the holes were put down.

In view of our findings to date you may feel that it is desirable to investigate the clay layer in more detail and in this we would concur.

Yours truly,

J. D. Paterson

J. D. Paterson, P. Eng.

JDP/MMC.

Materials and Research Division

May 23, 1962.

John D. Paterson, B.Sc., P.Eng.
Consulting Engineers & Geologists,
250 Sesserer Street,
Ottawa, Ontario.

Payne River Bridge
United Counties of Stormont, Dundas & Glengarry

Dear Mr. Paterson:

Your letter of May 14, 1962, addressed to Mr. D.C. Cramm of C.C. Parker & Associates, Ltd. has been referred to us.

In order to evaluate the stabili y problem of the approach embankment at the river bank, as mentioned in your letter, we would appreciate if you can furnish us with the following information:

(a) According to the report, only one unconfined compression test was carried out, and the rest of the shear strength data were determined by pocket penetrometers which, at best, cannot be taken more than a rough guide. There is no information of the index properties or water contents of the clay. We would like to know what value of shear strength you used in your analysis and the data on the Atterberg limits and water contents of the soil, if available.

(b) It is not clear from the letter what method has been used in the stability computations.

(c) With respect to the artesian pressure, could you indicate at what depth is this encountered and the magnitude of the excess head?

KYL/MdeF

Yours very truly,

K. Y. Lo,
Supervising Foundation Engineer
For:

A. G. Stermac,
Principal Foundation Engineer

cc: Mr. G.C.E. Burkhardt
Foundations Office
Gen. Files.

JOHN D. PATERSON, B.Sc. P.ENG.

May 14, 1962.

Mr. D. C. Gramm, P. Eng.,
C. C. Parker & Associates, Ltd.,
795 Main Street, West,
Hamilton, Ontario.

Payne River Bridge
United Counties of Stormont, Dundas & Glengarry

Dear Mr. Gramm,

This is in reply to your recent letter requesting comments on the proposed road profile at the above-mentioned bridge site.

After looking over our preliminary site plan and profile and reviewing our soil report (No. S 252-62) we are concerned with the stability of the river banks when an additional four feet of fill is placed for the new grade. In our original stability analysis we stated that the safe maximum height was considered to be 18 feet. However, now that we have your profile and have plotted the soil profile on it, we find that low shear strength of the soil occurs to a depth of approximately 30 feet below the existing grade, when an additional four feet of fill is placed the figure becomes 34 feet.

In recalculating the stability, considering a plane parallel to the road the maximum safe height is 36 feet. This figure incorporates a safety factor of 1.5.

While the proposed new grade at Elevation 108 indicates a total thickness of 34 feet of critical soil we now think that you should consider an open abutment type of bridge with and embankment slopes not exceeding 2 horizontal to 1 vertical. The underlying clay is sensitive and construction operations during the placing of the new approach fills may result in slip failure in the clay at the river banks.

It is our understand that during the spring runoff the South Nation river backs up into the Payne river and, therefore, there should not be any great problem of erosion of the banks at this bridge site. Side slopes of the approach embankments can be safely held to 2 horizontal to 1 vertical, as shown on your site plan.

Yours truly,

J. D. Paterson, P. Eng.

JDP/MMC.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

ONTARIO

DEPARTMENT OF HIGHWAYS

Bridge Division

Memo to Mr. A. Stermac
Principal Foundation Eng.
Lab Bldg. Downsview

Date May 4, 1962

Subject United Counties of Stormont, Dundas & Glengarry. Br. over Payne Rv. Twp. of Finch, County of Stormont, Lot 15 Conc. IX/X, Our File BA 1400

From G.C.E. Burkhardt

Attached please find a copy of the Foundation Report, by John D. Patterson, and a copy of the Preliminary plan for your comments.

We intend to approve the preliminary design within the next two weeks. We would appreciate it very much if we could have your comments before May 18, 1962

G. C. E. Burkhardt,
for K.L.Kleinsteiber
Municipal Bridge Liaison Engineer

GCEB/m

No comment!
[illegible] May [illegible] 1962
[illegible]

BA 1400

JOHN D. PATERSON, B.SC., P.ENG.
CONSULTING ENGINEERS & GEOLOGISTS
OTTAWA, CANADA
MEMBERS
ASSOC. OF PROFESSIONAL ENGINEERS OF ONTARIO
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REPORT OF SOIL INVESTIGATION

SITE OF PROPOSED NEW BRIDGE

OVER PAYNE RIVER

COUNTY ROAD NO. 12

FINCH TOWNSHIP

FOR

UNITED COUNTIES OF STORMONT, DUNDAS & GLENGARRY

* * * * * * * *

C. C. PARKER AND ASSOCIATES

DESIGN CONSULTANTS

REPORT NO. S 252 - 62

OTTAWA, APRIL 2,1962

J. D. PATERSON

Introduction:

At the request of Mr. D. C. Cramm, P.Eng., C. C. Parker & Associates, Ltd., on behalf of the United Counties of Stormont, Dundas & Glengarry, a soil investigation was conducted at the site of a proposed new bridge across the Payne River on County Road No. 12, Lot 16, Concessions 9 and 10, Township of Finch. The site is approximately two miles east of Crysler, Ontario.

The concrete on the existing abutments is in poor repair and the bridge inadequate for present traffic.

The purpose of the investigation was to obtain sufficient information for foundation design of the new bridge as well as for construction procedure for the road embankment.

Fieldwork Procedure:

Two test holes were put down on diagonally opposite sides of the existing structure as shown on the Test Boring Plan. The holes were put down opposite to the suggested locations because of greater ease in setting up the equipment.

At each hole a cone probe was driven to refusal, casing put down, the soils sampled, and bedrock located.

The cone probes were driven to check the uniformity of the soils.

All drilling operations were conducted by the firm of F. E. Johnston Drilling Co., Ltd., under the constant direction and supervision of a member of our staff. A standard drilling rig fully equipped for soil testing and mounted on a trailer was used for all drilling operations.

Sampling and Testing:

Samples of the soils encountered were taken at each hole by means of Shelby thin-walled tubes (for cohesive soils) and by means of the split spoon sampler (for granular soils).

Samples of boulders and bedrock were recovered by diamond drilling, classified and stored in core boxes.

Shelby tube samples were taken to the laboratory, extruded and tested for unconfined compressive strength where possible. Extruded samples which were unsuitable for unconfined tests were tested by means of the pocket penetrometer.

The standard penetration test was conducted on all split spoon samples and the results are recorded as "N" values. Each of these samples was retained in a plastic bag.

Observations: ....

Observations:

(a) Soil Types.

The soil above the loose silt and silty glacial till layer is a silty clay varying in silt content and containing various inclusions, (sand, organic material, shells). Below this layer is a 10- to 11-foot band of dense to very dense glacial till overlying bedrock.

Details of the borings are shown on the Soil Profile and Laboratory Test Sheets which form part of this report.

(b) Groundwater.

At the completion of the investigation water flowed from both test holes indicati·· low pressure artesion flow.

(c) Test Results.

The unconfined compressive strength and pocket penetrometer test results are reported on the Soil Profile sheets. Both the laboratory tests and the standard penetration tests indicate low strength soil.

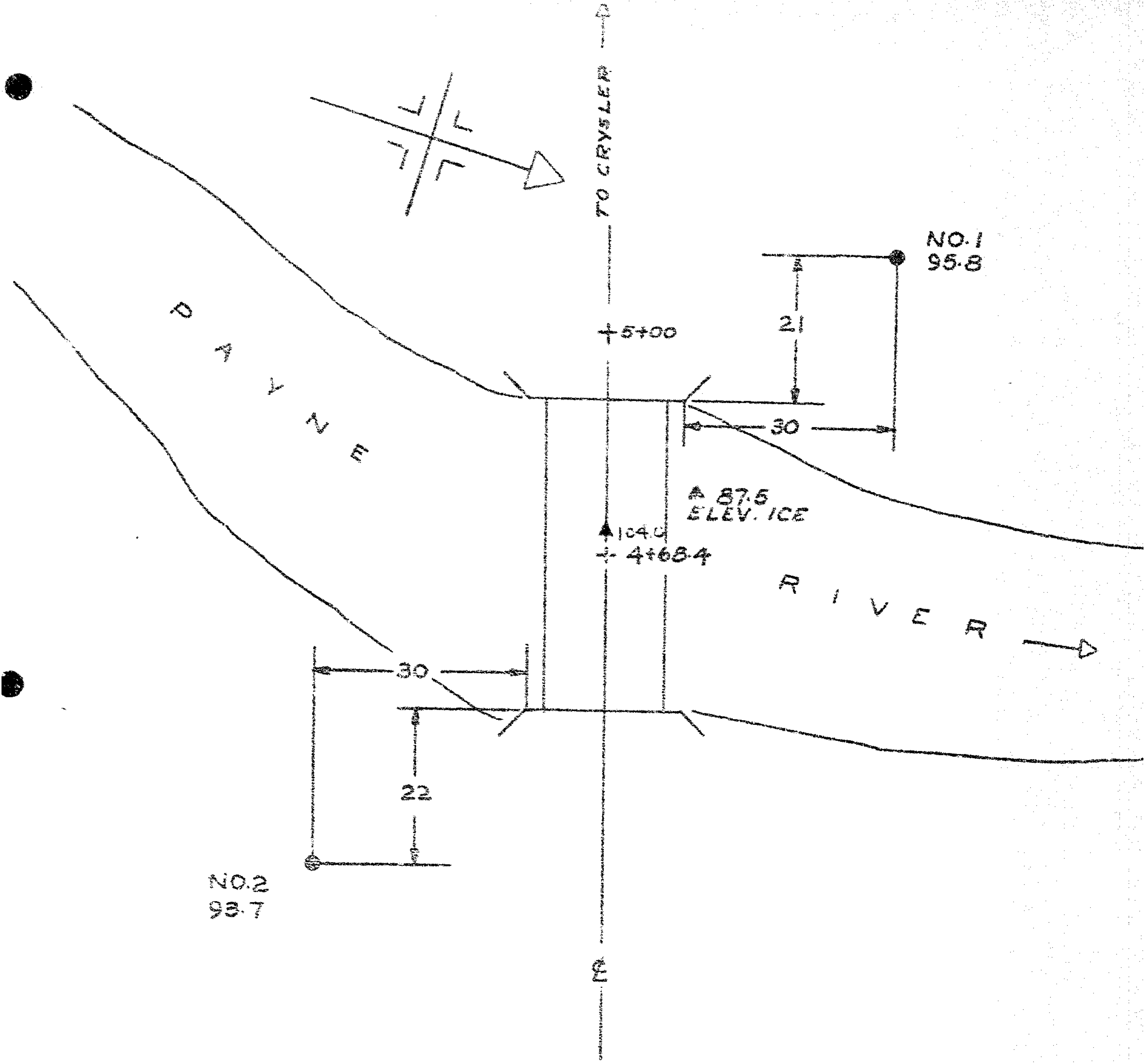
Conclusions & Recommendations:

None of the soil overlying the dense glacial till is considered suitable on which to place footings for bridge abutments. It is, therefore, recommended that piles be driven into the dense till which is located from 24 to 28 feet below surface. Based on the assumption that the pile caps will be at Elevation 83, the expected length of the piles will be from 15 to 20 feet. Under the conditions prevailing the piles will be situated below the river level and, therefore, pressure-creosoted timber piles can be considered for the foundation. If steel "H" piles are used the penetration into the till will be greater and the possibility of deflection of the pile by boulders exists. The use of timber piles is considered to be more suitable for this site because of the existence of some artesian water pressure.

A study of the existing bridge elevation and the embankment approaches indicates that the new bridge will be only slightly higher and that three feet would be a maximum increase in elevation. Using a safety factor of 1.5 the stability analysis for the approach embankments indicates that the safe maximum height is 18 feet and, therefore, no problem with embankment stability is expected. Side slopes should not exceed 1.5 horizontal to 1 vertical and 3 feet of granular material should overly existing native soil if it is used for the new wider embankments.

J. D. Paterson, P. Eng.

Ottawa, April 2nd, 1962.
JDP/MMC.



BM. ELEV. 100.0
NAIL IN T.P.
37 LT of STAT 2+94

TEST BORING PLAN
PROPOSED BRIDGE
COUNTY ROAD 12
LOT 16 CON 9 & 10
TOWNSHIP FINCH

SCALE 1" = 20'   MAR. 1962

JOHN D. PATERSON
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND LABORATORY TESTS

Location: County Road No. 12, Township of Finch.

Elevation (Zero Depth): 95.8.
Remarks: Cone Probe and Test Boring.

Sheet No: 1 of 2

Borings by: F.E.Johnston Drilling Co., Ltd. **Date:** March 8 & 9, 1962.

Hole No: 1

Blows per Foot	Soil Description	Samples		U'c T/ft²	N	Depth in Feet	Elev.	Moisture Content Per Cent (30, 40, 50, 60, 70)
Cone	Ground Surface					0	95.8	
50 7 5 3 2 2	Weathered clayey silt with minor organic inclusions (to 4)					3		Water flowing ~~from completed~~ bore hole.
1 3 4	Weathered clayey silt with sand. (to 8)	SS	2		3	6		
4 3 3	Soft, grey, silty clay. (to 12)	SS	3		3	9		
7 8 6	Grey, till-like, clayey sand with pieces of wood (roots). (to 15.5)	TW	4	Lost		12		
8 8 8 8 11 13 16	Soft to firm, pinkish-grey, silty clay with patches of fissuring and silt bands. (to 22)	SS	5		3	15	80.8	
		TW	6	0.75 pp		18		
		TW	7	0.80 pp		21		
24 31 27 20 13 61	Loose silt and sandy glacial till. (to 28)	SS	8		7	24		
120 for 0.5'	Dense, gravelly, glacial till. (to 38.8)	SS(	9		9	27		
		(	9		67	30	65.8	
		SS	10		44	33		
						36		
	BEDROCK Good quality limestone with minor thin shale partings. (to 45.8)	Core Recovery.		97%		39		
						42		
						45	50.8	

JOHN D. PATERSON CONSULTING ENGINEERS OTTAWA CANADA	SOIL PROFILE AND LABORATORY TESTS Location: County Road No. 12, Township of Finch.

Elevation (Zero Depth): 93.7.
Remarks: Cone Probe and Test Boring

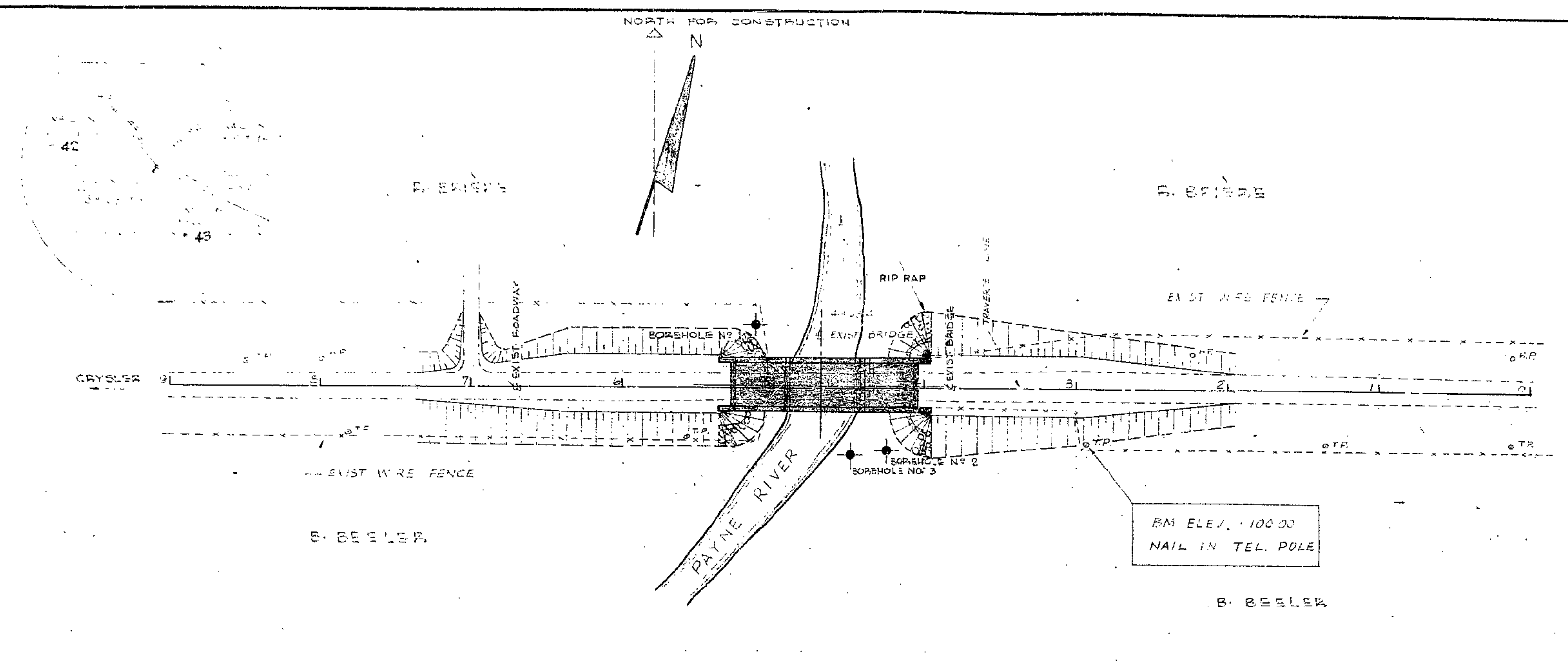
Sheet No: 2 of 2

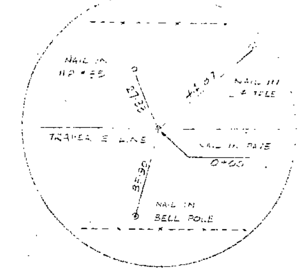
Borings by: F.S.Johnston Drilling Co., Ltd. Date: March 13 & 14,1962.

Hole No: 2

Blows per Foot	Soil Description	Samples		U'c T/ft'	N	Depth in Feet	Elev.	Moisture Content Per Cent. 30 40 50 60 70
Cone	Ground Surface					0	93.7	
10 4 3	Black clayey silt with organic inclusions. 2							
[illegible]	Loose, blackish-brown, clayey silt, with a musty odour.					3		Water flowing from completed bore hole.
[illegible]		SS	12		8	6		
12	8					9		
13 15	Soft, grey, silty clay with an odd sand grain and shell.	SS	13		6			
14 14 11		SS	14	(1.26 (0.56	on	12		
12 11		TW	15	0.36		15	78.7	
13	17							
22 28 25	Loose silt and silty glacial till.					18		
23 21		SS	16		10	21		
25 41	24	SS	17		3	24		
27 43 70	Very dense glacial till, with boulders.					27		
66 66		SS	18		106			
79 110 for 0.8'						30	63.7	
		SS	19		100 for 0.2'	33		
	35.5	Core						
	BEDROCK Good quality limestone with minor shale lenses and some calcite. 39.5	Core		92% Recovery		36		
						39		
						42		
						45	48.7	

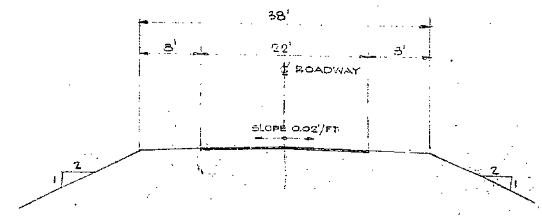
62-F-301 M
CTY. RD. # 12
NEW BRIDGE
PAYNE RIVER
FINCH TWP.





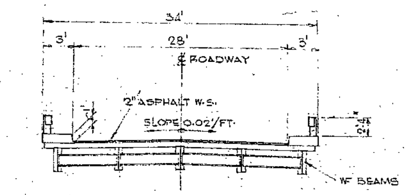
SITE PLAN

SCALE 1" = 50'-0"



TYPICAL SECTION THRU APPROACHES

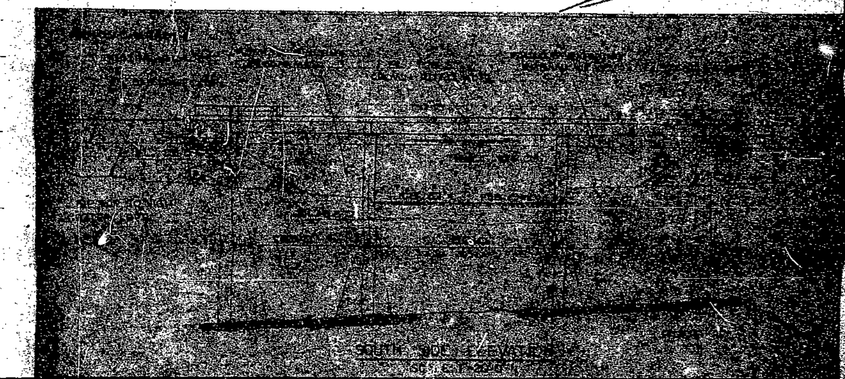
SCALE 1" = 10'-0"

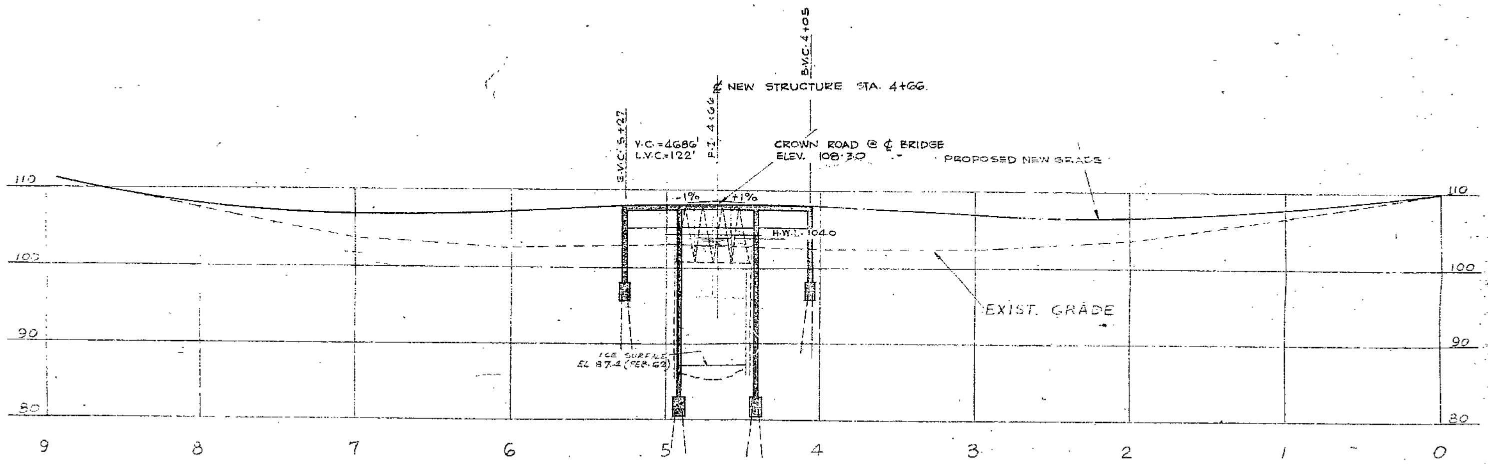


TYPICAL SECTION THRU BRIDGE

SCALE 1" = 10'-0"

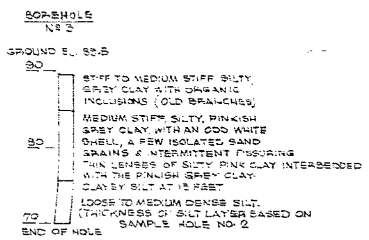
Revised 7/2/63

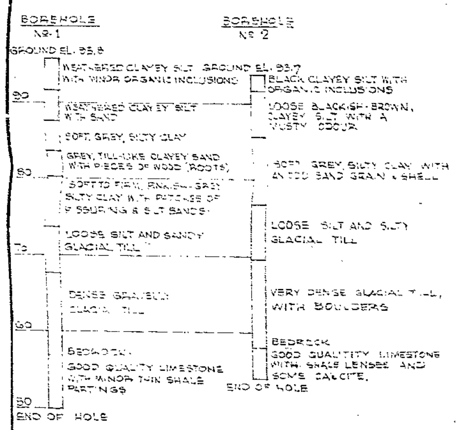




PROFILE ALONG ℄ OF EXIST. ROAD

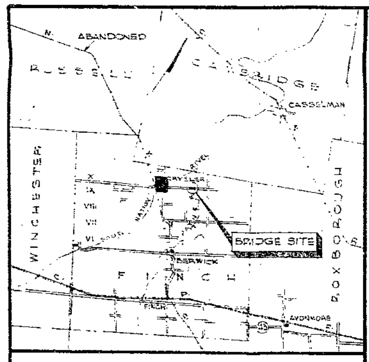
SCALE HORIZ. 1" = 50'-0"
VERT. 1" = 10'-0"





TEST BORING RESULTS

BORINGS ARE BY JOHN D. PATERSON, OTTAWA, ONTARIO.
BORINGS ARE FOR GENERAL INFORMATION ONLY AND ARE NOT GUARANTEED BY THE COUNTIES.



KEY PLAN

FOLLOW SEPARATE INSTRUCTIONS FOR PREPARATION OF BRIDGE SITE PLAN WHEN MAKING BRIDGE SURVEY.

DATA

1. SPECIAL FEATURES. WATERFALLS, DAMS, EXCEPTIONAL FLOODS, ICE, DRIFTWOOD, SLIDING BANKS ETC. BACKWATER FROM SOUTH NATION RIVER AFFECTS H.W.L. AT SITE.

2. (A) UPSTREAM & DOWNSTREAM BRIDGES (GIVE LOCATION, LENGTH, HEIGHT ABOVE N.H.W.L., NET CROSS-SECTIONAL AREA AT HIGH WATER & ESTIMATED AGE). 1ST UPSTREAM – LOT 17, CON. VIII & IX, TWP. FINCH, 45' CLEAR PONY TRUSS, DECK AT N.H.W.L. NET AREA 600 SQ.FT. 50 YEARS OLD.
2ND UPSTREAM – LOT 17, CON. VII & VIII, TWP. FINCH, 90' CLEAR T.TRUSS, 2' ABOVE N.H.W.L., NET AREA 1220 SQ.FT. 50 YEARS OLD. DOWNSTREAM – NONE.
(B) REASONS WHY THESE BRIDGES ARE, OR ARE NOT, FAIR INDICATIONS OF SIZE OF PROPOSED BRIDGE. 1ST UPSTREAM BRIDGE APPEARS TOO SMALL.

3. REASONS FOR CHANGES IN HEIGHT OR LENGTH FROM THAT OF OLD BRIDGE. DECK SHOULD BE RAISED ABOVE H.W.L., LARGER OPENING REQUIRED.

DATA (CONT'D.)

4. IS DITCH, STREAM, OR RIVER GRADIENT LIABLE TO BE LOWERED: NO
5. NAVIGATION CLEARANCES REQUIRED, IF ANY: NONE
6. RAILWAY CLEARANCE REQUIRED, IF ANY: NONE
7. IF STRUCTURE IS OVER OR UNDER A RAILWAY, HAS APPROVAL BEEN OBTAINED
   (A) FROM RAILWAY CO.: NOT APPLICABLE
   (B) FROM BOARD OF TRANSPORT COMMISSIONERS: N.A.
8. HAS APPROVAL BEEN OBTAINED UNDER NAVIGABLE WATERS PROTECTION ACT? N.A.
9. IS A TEMPORARY DETOUR REQUIRED? YES
   WHO WILL BUILD IT? COUNTIES
   WHO WILL MAINTAIN IT? COUNTIES
10. INFORMATION AND EVIDENCE OF EXTREME FLOODING WAS OBTAINED FROM LOCAL RESIDENTS. AND REFLECTS HIGHEST WATER ELEVATION IN THE AREA OF THIS CONSTRUCTION TO BE 104.0 AND THE LOWEST WATER ELEVATION TO BE 87.0
11. ROAD DESIGN INFORMATION.
    ESTIMATED ADT: 190 (YEAR 1962), 470 (YEAR 1982)
    DESIGN SPEED: 50 M.P.H.
    STOPPING SIGHT DISTANCE: 350 FT.

STRUCTURE DATA

1. NET SPAN LENGTH AND TYPE OF BRIDGE: 3 SPANS, 35'-50'-35' = 120' SIMPLE STEEL GIRDERS.
2. ROADWAY WIDTH ON BRIDGE: 28'
3. NUMBER & WIDTH OF SIDEWALKS: NONE
4. SKEW ANGLE: 0°
5. TOTAL LENGTH & TYPE OF PILING: STEEL H PILES
6. APPROX. VOLUME OF CONCRETE ______ CU. YDS.
7. APPROX. WEIGHT OF STR. STEEL ______ TONS
8. APPROX. WEIGHT OF REINFORCEMENT ______ TONS
9. APPROX. VOLUME OF APPROACH FILL 100' EACH SIDE OF STRUCTURE ______ CU. YDS.
10. DRAINAGE AREA 57 SQ. MILES

FIELD INVESTIGATION MADE FEB. 27 1962

BY N. HANLAN
SURVEY ENGINEER

C.C. PARKER & ASSOCIATES LTD.
CONSULTING ENGINEERS
HAMILTON ONTARIO

STRUCTURE NO. 32-37

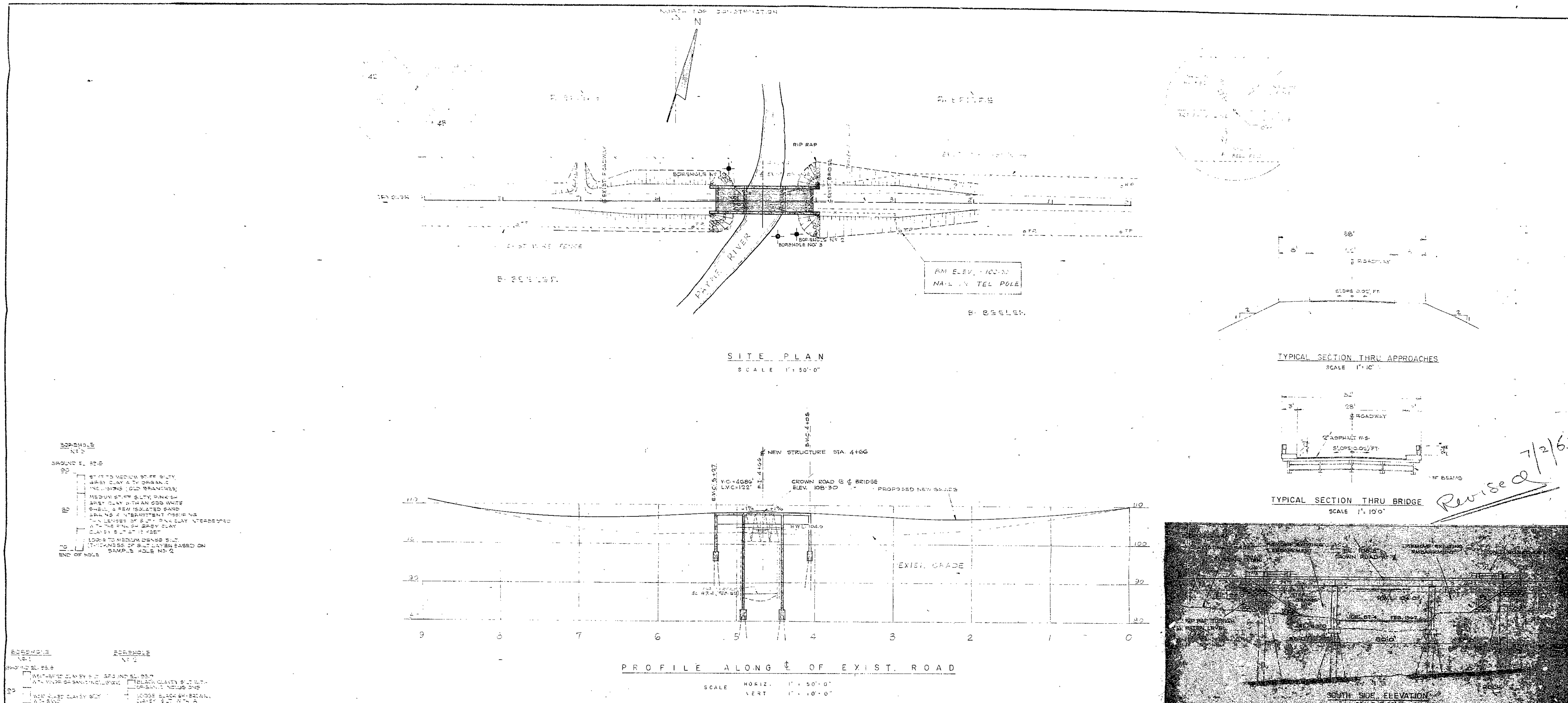
BRIDGE ~~OVER PAYNE RIVER~~

OWNER UNITED COUNTIES OF STORMONT DUNDAS & GLENGARRY	MUNICIPAL DIST. NO. 3
CO. OF STORMONT	ROAD NO. 12
TWP. OF FINCH	LOT 16 CON. IV

SITE PLAN AND PROFILE

REGISTERED PROFESSIONAL ENGINEER
CIVIL
D. C. CRAMM
PROVINCE OF ONTARIO

DEC. 15, 1962 DATE	D. C. Cramm DESIGN ENGINEER	
BRIDGE NAME: PAYNE RIVER BRIDGE		
LOADING H20-S16	COUNTY JOB NO. 6227	DWG. NO. 1006-1



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SCALE HORIZ. 1" = 50'-0"
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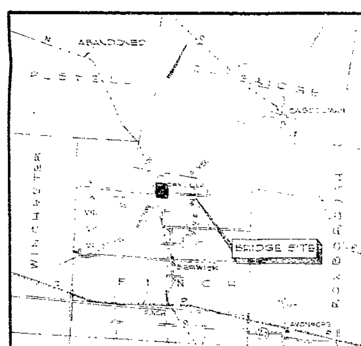

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STR. 32-37
BRIDGE ~~OVER PAYNE RIVER~~

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MUNICIPAL DIST. NO. 3
ROAD NO. 12
LOT 16 CON. IX & X

SITE PLAN AND PROFILE

REGISTERED PROFESSIONAL ENGINEER
CIVIL
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LOADING	COUNTY JOB NO.	DWG. NO.
H20-S16	6227	1006-1