

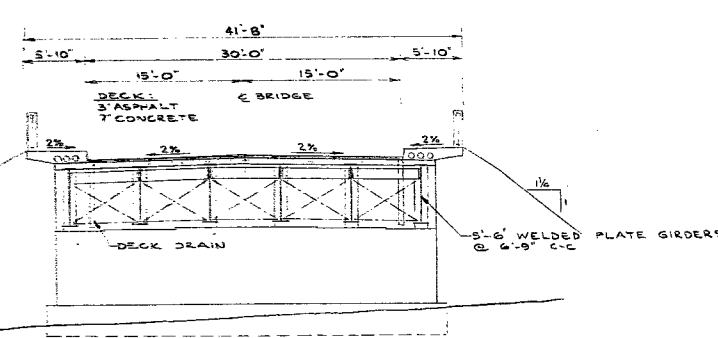
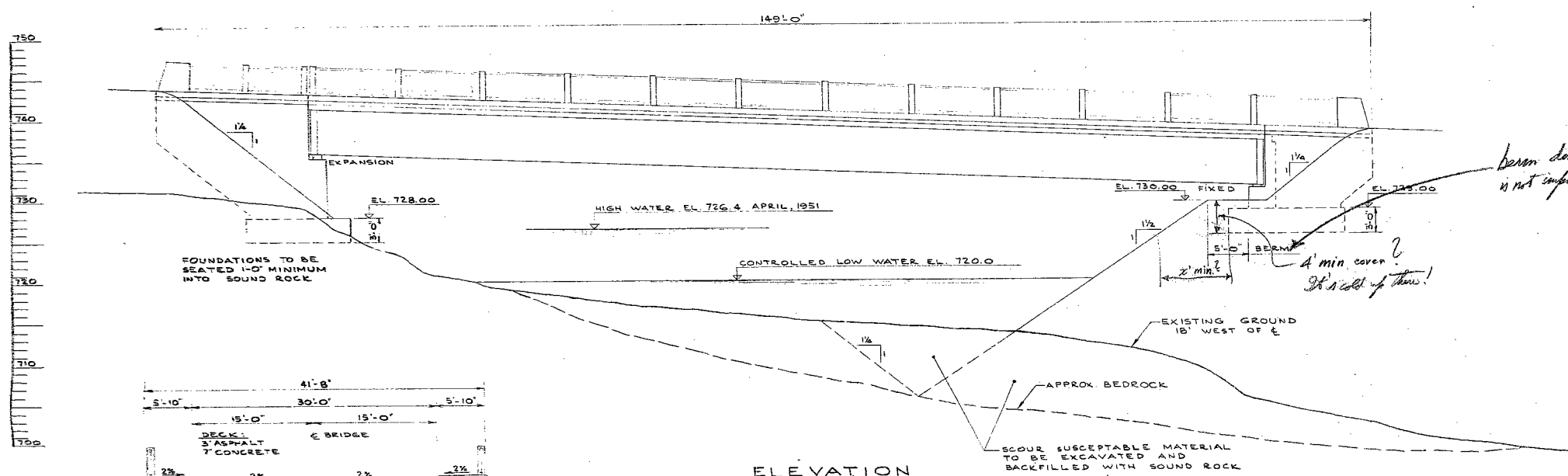
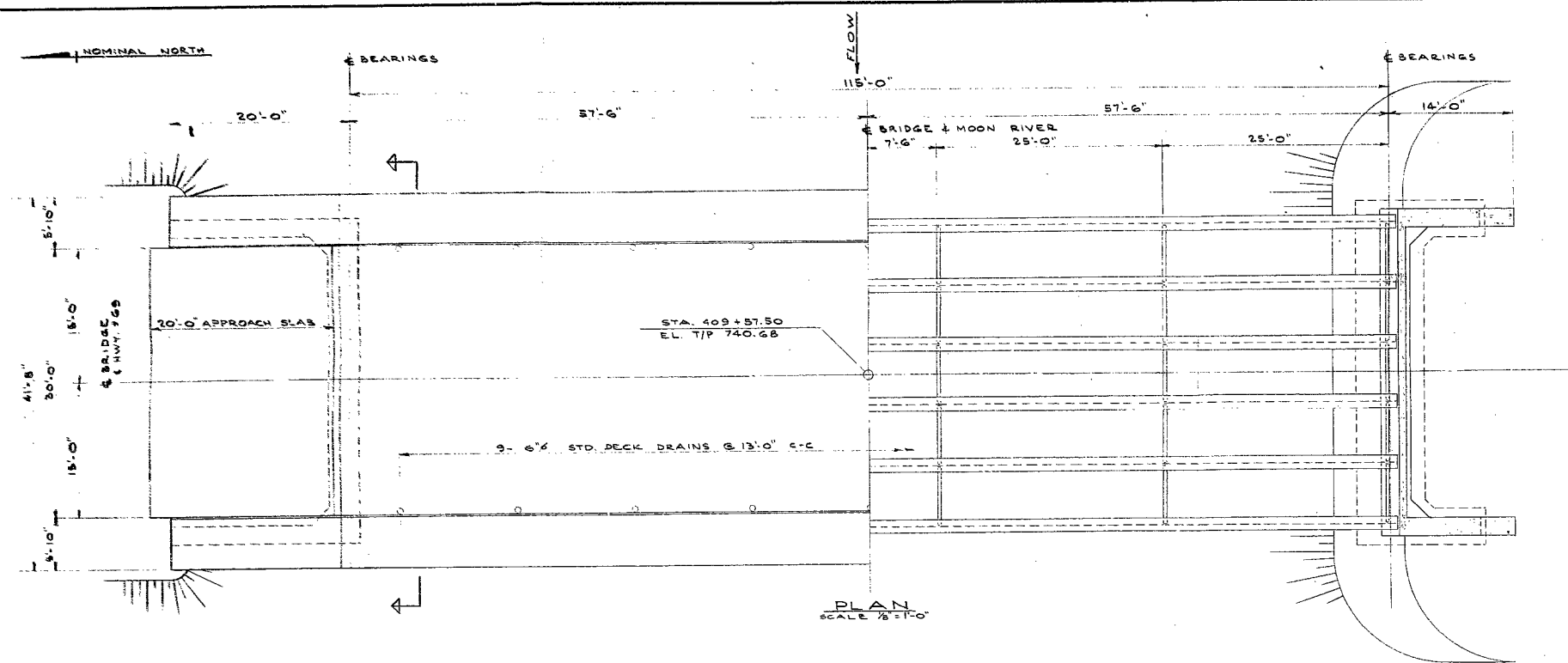
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W.P. 59-60 3

W.P. 187-61

MOON RIVER 3

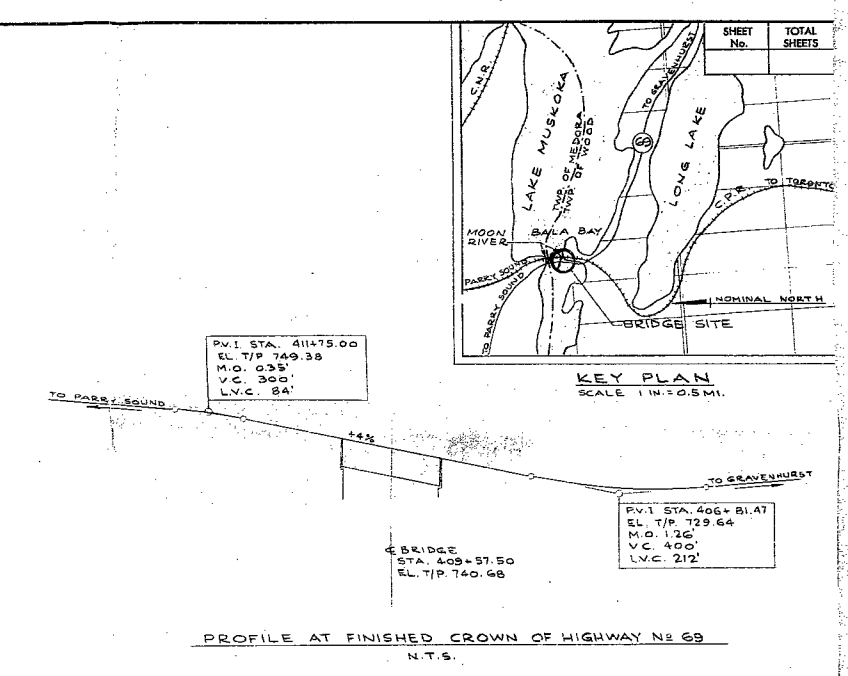
C.P.R. SUBWAY



RECOMMENDATIONS BY FOUNDATION SECTION

1. DISTANCE OF LOWER EDGE OF FOOTING FROM SLOPE SHOULD BE 10 (TEN) FEET. (THIS VALUE IS BASED ON SLOPEHENT)
2. COVER ABOVE FOOTING BASE SHOULD BE FIVE FEET.
3. CARE SHOULD BE TAKEN WHEN EXCAVATION CARRIED OUT

TED HEWSON FEB 4, 1963 *cy storming*



REVISIONS			
	1.28.63	A.P.	REDRAWN
DATE	BY	DESCRIPTION	

MORRISON, HERSHFELD, MILLMAN & HUGGINS, LTD.
CONSULTING ENGINEERS

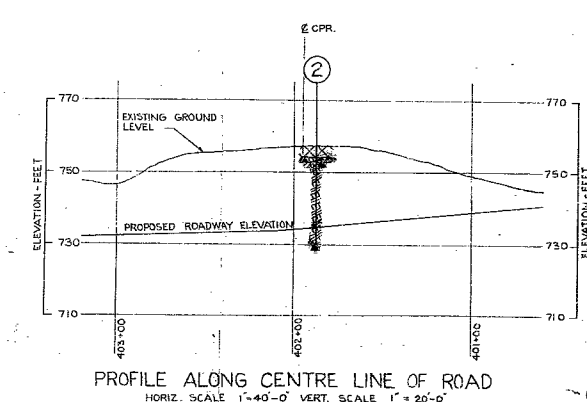
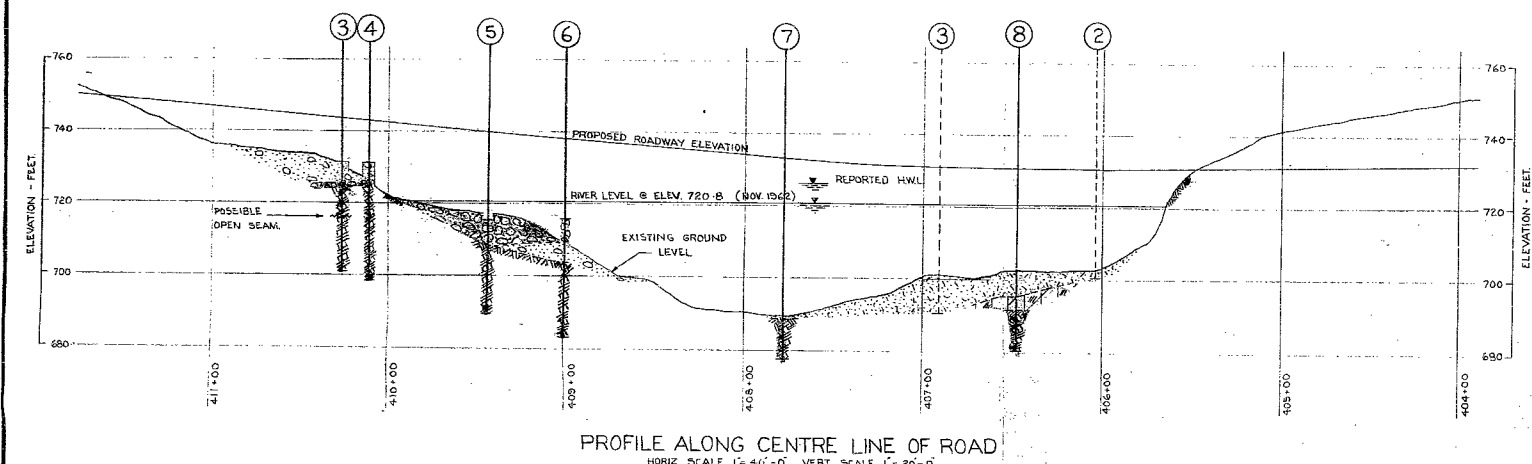
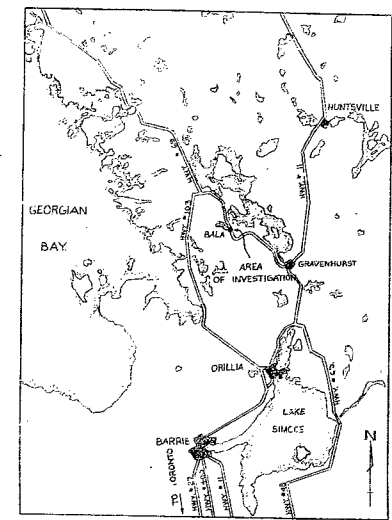
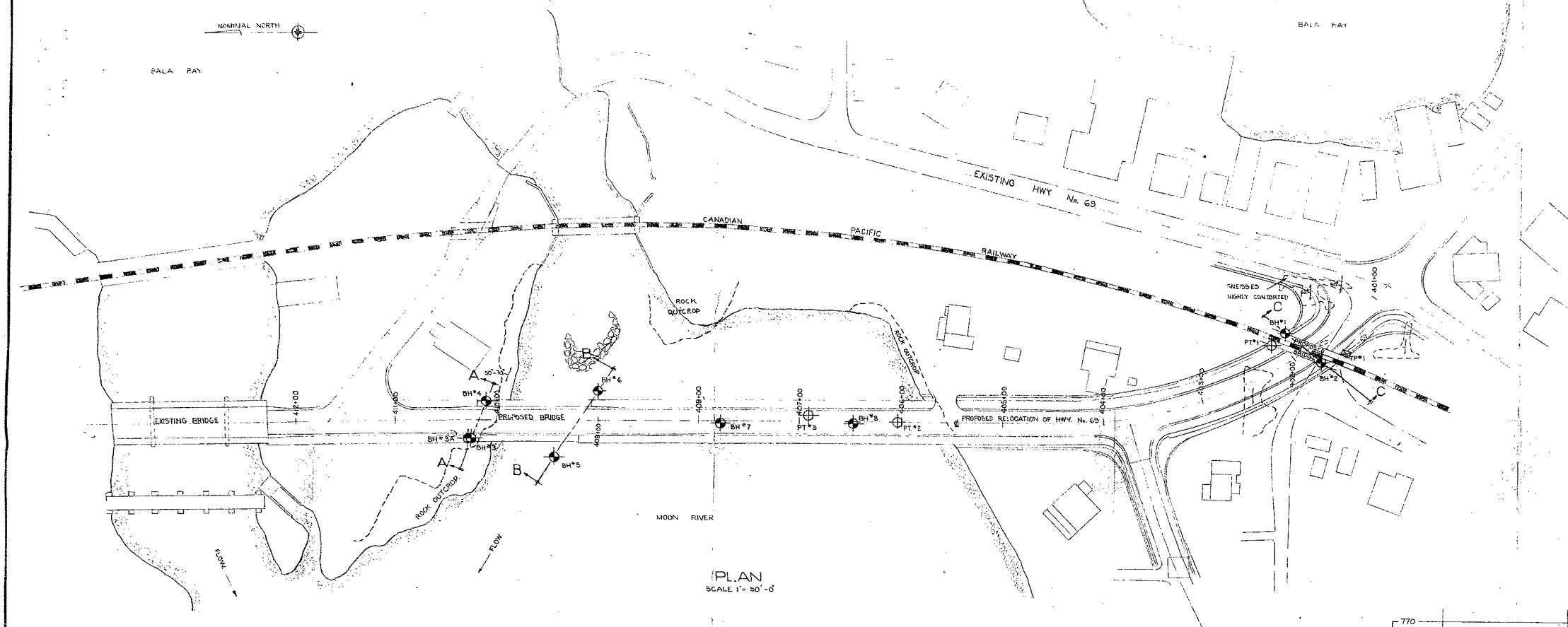
DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

MOON RIVER BRIDGE
AT BALA

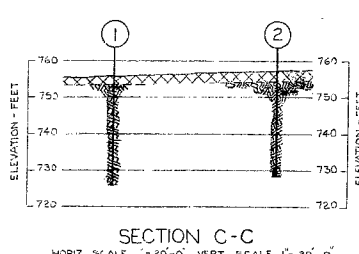
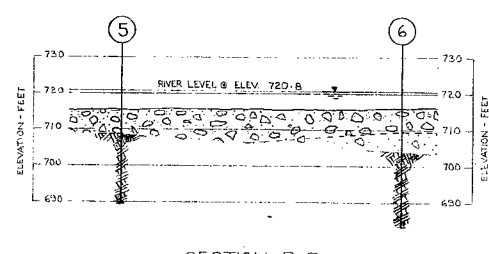
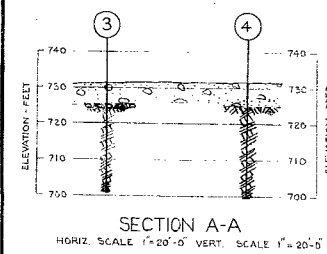
KING'S HIGHWAY No. 69 DIST. No. 11
CO. DISTRICT OF MUSKOKA
TWP. WOOD LOT 33 CON. 7

PROPOSED GENERAL ARRANGEMENT

APPROVED				SITE No.	W.P. No.
BRIDGE ENGINEER					187-61
DESIGN	R.C.A.	CHECK		CONTRACT No.	
DRAWING	A.P.	CHECK	R.C.A.		
DATE	JAN. 1963	LOADING	4-20 5-16	DRAWING No.	D4933-P1

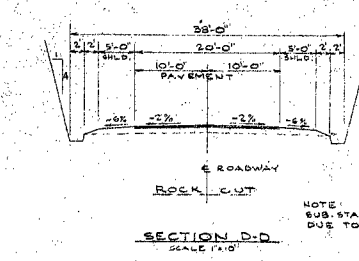
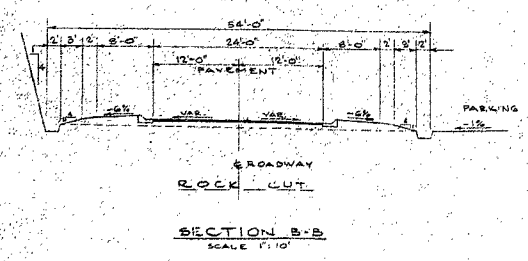
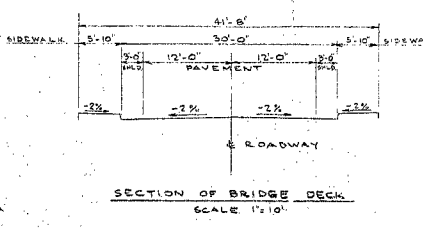
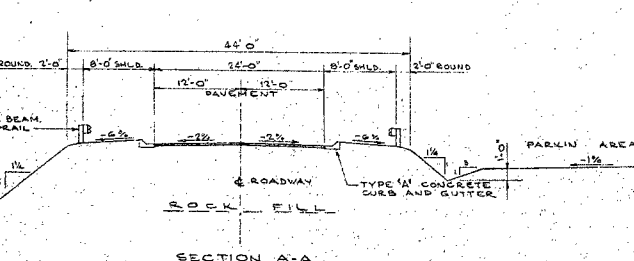
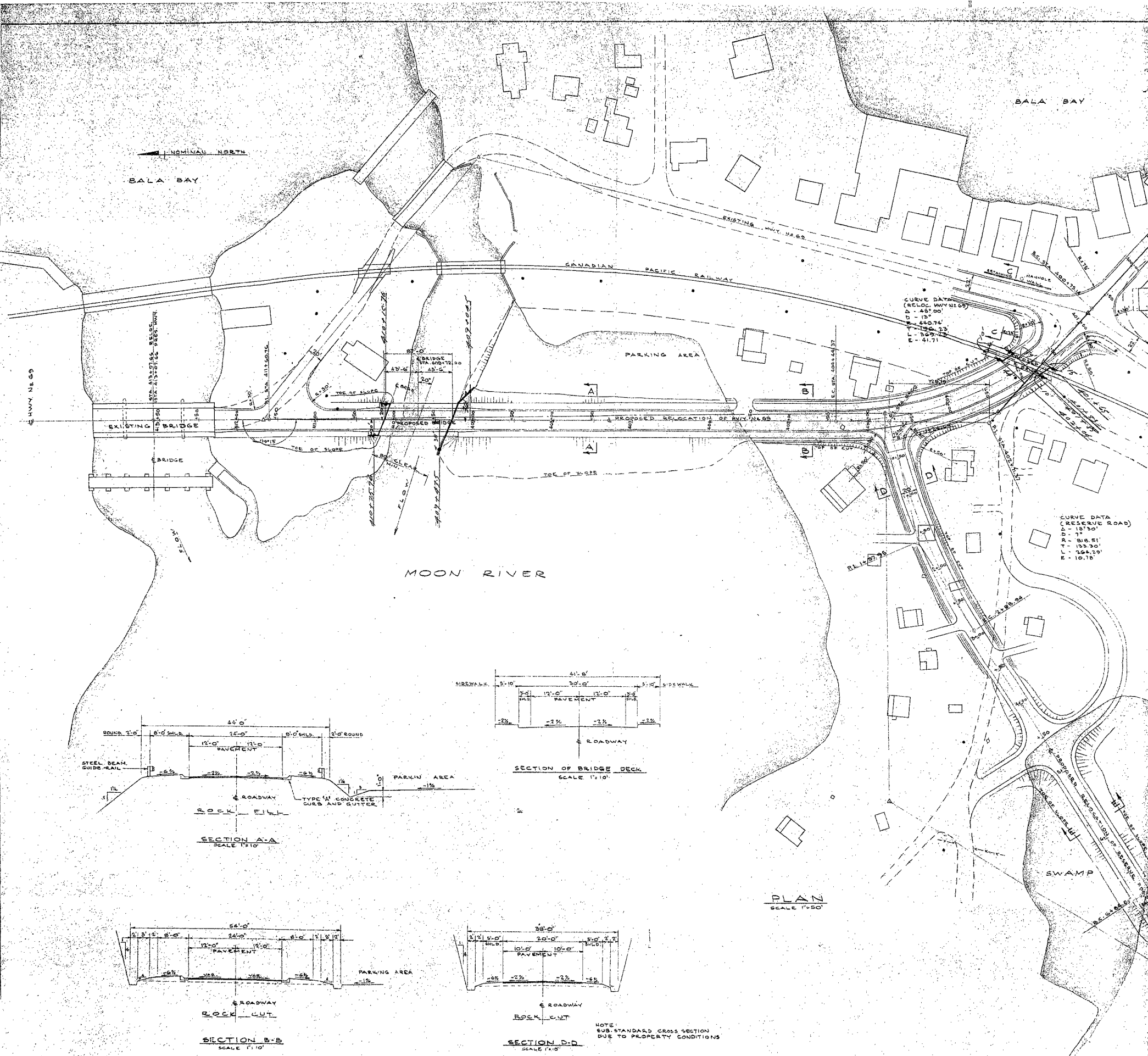


- LEGEND**
- BH \odot BOREHOLE IN PLAN
 - PT \oplus PENETRATION TEST IN PLAN
 - TP \otimes TEST PIT IN PLAN
 - ② BOREHOLE IN ELEVATION
 - ② PENETRATION TEST IN ELEVATION
 - ROCK OUTCROP IN PLAN
 - ROCK OUTCROP IN SECTION
 - 50' STRIKE AND DIP IN PLAN
 - WATER LEVEL IN ELEVATION
- STRATIGRAPHY**
- LOOSE GREY BROWN SAND AND GRAVEL FILL
 - VERY LOOSE WOOD CHIPS AND SAND
 - LOOSE, GREY AND BROWN FINE TO COARSE SAND WITH SOME GRAVEL AND OCCASIONAL BOULDERS
 - BOULDERS WITH FINE GREY SAND
 - VERY SOFT GREY VARVED SILTY CLAY
 - WEATHERED BEDROCK OR BOULDERS
 - SOUND MOTTLED LIGHT GREY AND BLACK Biotite GRANITE AND/OR PINK SYENITIC GNEISS BEDROCK

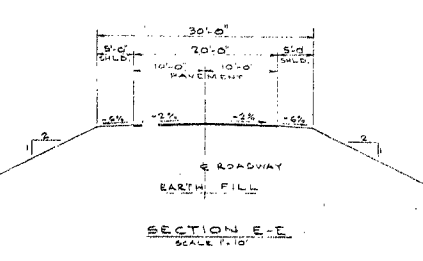
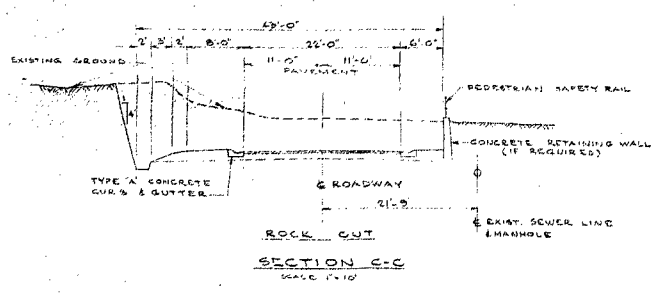


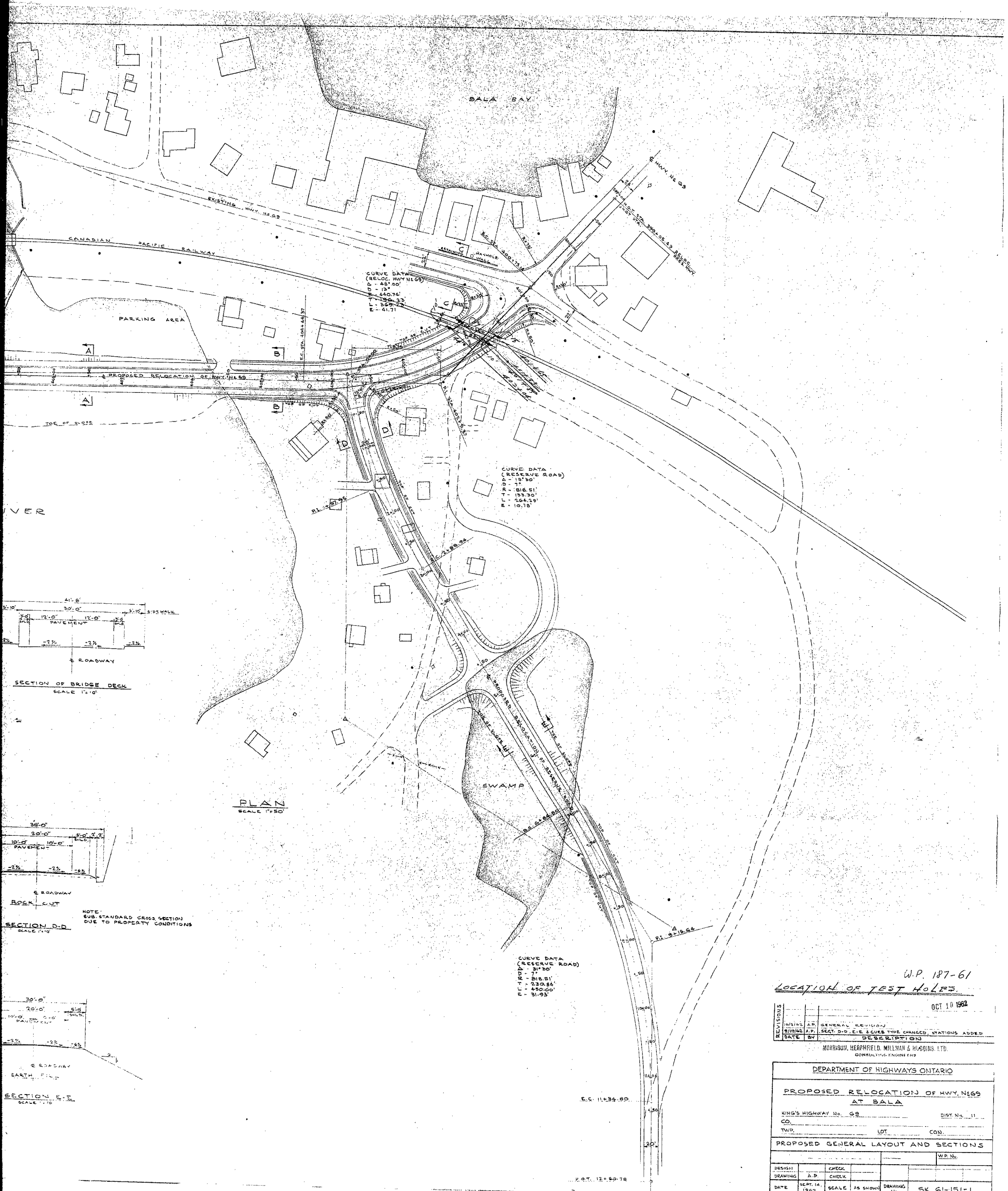
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DEPARTMENT OF HIGHWAYS, ONTARIO		GEOCON LTD	
TORONTO		DATE DEC 26, 1962 SCALE AS SHOWN	
PROPOSED MOON RIVER BRIDGE AND CPR SUBWAY		No. 5 7447	
BALA, ONTARIO			
BORING PLAN AND SOIL STRATIGRAPHY			



NOTE:
 SUB-STANDARD CROSS SECTION
 DUE TO PROPERTY CONDITIONS



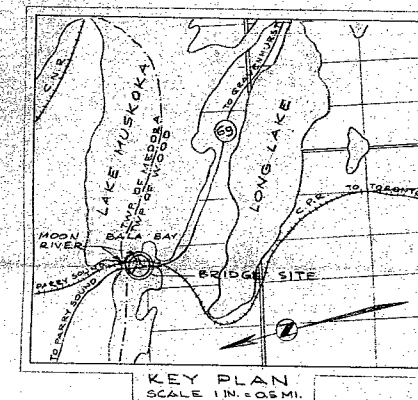
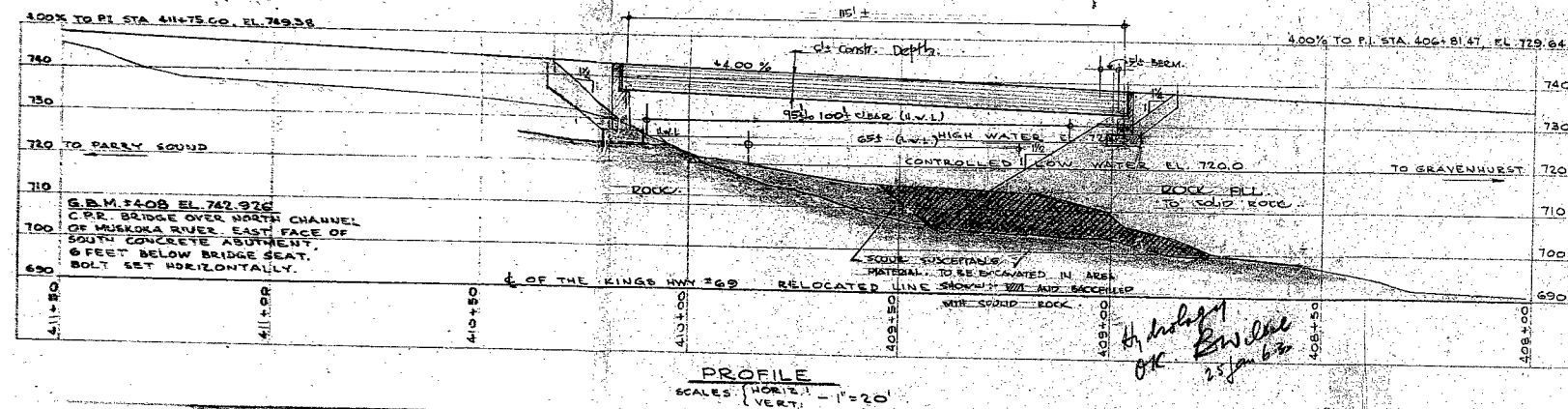
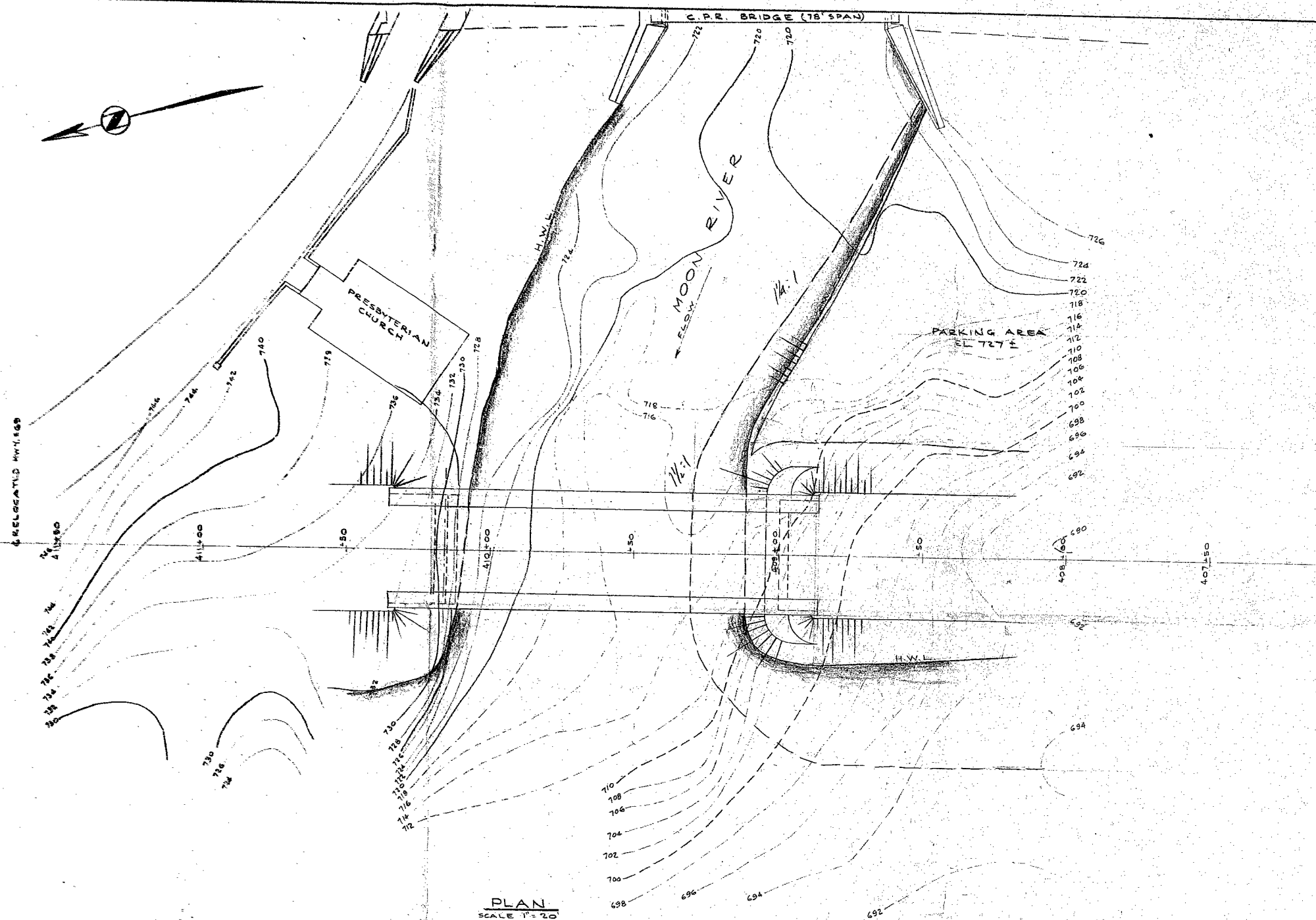


W.P. 187-61
LOCATION OF TEST HOLES

REVISIONS		OCT 10 1962	
DATE	BY	DESCRIPTION	
7/19/62	A.P.	GENERAL REVISION	
7/19/62	A.P.	SECT. D-D, E-E & CURVE TYPE CHANGED, STATIONS ADDED	

DEPARTMENT OF HIGHWAYS ONTARIO			
PROPOSED RELOCATION OF HWY. NO. 69 AT BALA			
KING'S HIGHWAY No. 69		DIST. No. 11	
CO.		CON.	
TWO		LOT	
PROPOSED GENERAL LAYOUT AND SECTIONS			
W.P. No.			
DESIGN	CHECK		
DRAWING	A.P.	CHECK	
DATE	SEPT. 14 1962	SCALE	AS SHOWN
		DRAWING No.	SK 61-151-1

SOME DEFECTS IN NEGATIVE DUE
TO CONDITION OF ORIGINAL DOCUMENTS



W.P. 187-61		W.P. 187-61	
ADDITIONS	DATE	REMARKS	BY
MORRISON, HERSHFIELD, MILLMAN & HUGGINS, LTD. CONSULTING ENGINEERS			
DEPARTMENT OF HIGHWAYS-ONTARIO- PLANNING & DESIGN BRANCH.			
DISTRICT No 11			
PROPOSED CROSSING AT MOON RIVER AND THE KING'S HIGHWAY No 69 - RELOCATION IN THE TOWN OF BALA TOWNSHIP OF WOOD DISTRICT OF MUSKOKA			
BRIDGE SITE			
SURVEY BY D. KIRBY		APPROVED DIRECTOR OF PLANNING & DESIGN	
DRAWN BY A. PETERSON		SCALE: AS SHOWN DATE OF SURVEY: APRIL, 1962 DATE OF PLAN: OCTOBER, 1962	
CHECKED BY R. C. AITKEN		W.O.N.S. X-ING NO.	
		PLAN-	

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Attention: Mr. S. McCombie.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.
January 7, 1963.

FOUNDATION INVESTIGATION REPORT - BY
Geocon, Limited, Consulting Engineers,
Proposed Moon River Bridge and C.P.R.
Subway, Bala, Ontario, District No. 11.

W.P. 59-60 and W.P. 187-61

Attached, we are forwarding to you the above-mentioned report submitted by the consultant, Geocon, of Toronto.

We have reviewed the report and find the factual information well presented and in general agreement with the recommendations contained in the report. With respect to the formation of the excavation slopes at the site of the subway, we would not, however, agree with the consultant's recommendation that both the north and the south slope be constructed in the same manner. According to the report, the rock strikes nearly perpendicular to the railroad tracks and dips from 30 to 50 degrees to the north (page 1). It is inferred that the inclination of the joints found in the cores is the same as the dip observed at the outcrops - in other words - downwards towards the north (page 3).

When a cut will be made at the crossing to lower the highway under the railroad, entirely different conditions will be created on the south and on the north excavation slopes. While the rock dip will create problems on the south slope, where the consultant's recommendation should be applied, no problems should be encountered on the north slope which can be vertical.

If the south slope is constructed as 1:1 and the footings are kept below a 35° line extending from the toe of the slope, as explained in the report, no anchors or bolts would, in our opinion, be necessary.

We would recommend that subexcavation, rather than blasting be used for the removal of the soft clay beneath the proposed causeway.

AGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2)

H. A. Tregaskes
H. D. McMillan
H. McArthur
E. H. Jones

F. Norman
Foundations Office

A. G. Stermac
A. G. Stermac,

PRINCIPAL FOUNDATION ENGINEER

T. J. Kovich
J. Roy
J. E. Gruspier
E. R. Saint
A. Watt
Gen. Files.

GEOCON LTD

HEAD OFFICE

150 ALLÉE ST., MONTREAL 18, QUEBEC

TELEPHONE UN. 6-7632

Rexdale, Ontario,
January 2nd, 1963.

DISTRICT OFFICES
14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. 244-6476

1425 WEST PENDER ST.
VANCOUVER 5, B.C.
TEL. MU. 1-8926

Department of Highways, Ontario,
Materials and Research Section,
Burlington, Ontario.

63-F-217C

Attention: Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer.

Re: Soil Conditions and Foundations,
Proposed Moon River Bridge
and C.P.R. Subway,
Bala, Ontario,
W.P. 59-60, W.P. 187-61.

Dear Sirs:

This letter accompanies our engineering report on the
above soil investigation.

We find that the soil conditions at the locations of
the three principal structures covered by this study consist
generally of a shallow thickness of granular overburden, then
gneiss bedrock.

The proposed grade separation structure and the Moon
River Bridge may be founded directly on bedrock. Allowable
bearing values and special provisions which are required, are
given in the report. The possible effect of a prominent set
of joints in the bedrock on the selection of side slopes of the
cut at the proposed grade separation is discussed. At the pro-
posed causeway location the foundation conditions are generally
suitable for dumped rock fill type construction, although in
one area some dredging or blasting of clay overlying bedrock
will be required as discussed.

We believe that our report details the information on
the soil and rock conditions required from this investigation.
Should you require further information, or if we can be of assis-
tance in the application of the information to design, we would
be very pleased if you would give us a call.

Yours very truly,

GEOCON LTD

M. A. J. Matich per J. J. H.
M. A. J. Matich, P. Eng.,
Vice-President and Chief Engineer.

MAJM/dw
57447

S7447
REPORT
TO
DEPARTMENT OF HIGHWAYS, ONTARIO
ON
SOIL CONDITIONS AND FOUNDATIONS
PROPOSED MOON RIVER BRIDGE AND C.P.R. SUBWAY
BALA ONTARIO
W.P. 59-60
W.P. 187-61

Distribution:

- 14 copies - Department of Highways, Ontario,
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Drawing in pocket at rear of report:	
S7447-1 Boring Plan and Soil Stratigraphy	

INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario by letter dated November 16th, 1962 to investigate the subsurface conditions at the sites of a proposed bridge and causeway across Moon River and a subway below the Canadian Pacific Railway tracks at Bala, Ontario. These structures are part of a proposed relocation of Highway #69 at this location.

The purpose of the investigation was to determine the depth of overburden and the engineering properties of the bedrock necessary for the design of the bridge abutments for the Moon River Bridge and the Canadian Pacific Railway Subway. The soil conditions along the causeway route and the resulting stability of the causeway fill was also to be investigated. This report contains all the factual information obtained, together with recommendations for design.

SUMMARIZED SOIL AND ROCK CONDITIONS

At the site of the proposed grade separation, there is a layer of sand and gravel fill about 2 to 3 feet in thickness. Underlying the fill to the depth investigated is biotite and/or syenite gneiss bedrock which is generally in sound condition. The rock strikes nearly perpendicular to the direction of the railroad tracks and dips from 30 to 60 degrees to the north. The river bottom at the south abutment of the proposed Moon River Bridge is underlain by about 6 feet of boulders and sand followed by from 1 to 5 feet of fine sand with some boulders. The rock at the north abutment location is overlain by 5 to 6 feet of sand with some gravel. Along the proposed causeway section, rock occurs directly at the river bottom for part of the length while part is underlain by up to 7 feet of very loose

wood chips and sand overlying about 4 feet of very soft varved silty clay, then bedrock. The bedrock at the causeway and bridge locations is also a relatively sound biotite granite and/or syenite gneiss which dips at about 50 to 70 degrees towards the north.

DISCUSSION

It is understood that the proposed relocation of Highway #69 at Bala, Ontario in the area covered by this investigation will involve three major structures. These are a grade separation structure at the Canadian Pacific Railway tracks, a rock fill causeway across the south arm of Moon River and a bridge connecting the northern extremity of the causeway to the island which divides the river at this point. The plan locations of the proposed structures are shown on your Drawing SK61-151-1 which has been used as a basis for our Boring Plan and Soil Stratigraphy Drawing S7447-1, given at the rear of this report. For convenience of presentation, the above three structures are discussed separately below:

1. Proposed Grade Separation Structure

It is proposed to carry the new Highway #69 under the existing Canadian Pacific Railway tracks at the site of an existing level crossing which is about 100 feet from the present Highway #69 as shown. This will involve carrying the railway tracks on a bridge and excavating about 25 feet in rock to the proposed profile shown on Drawing S7447-1. The main purpose of the investigation of the structure location was to examine the rock conditions as they affect the stability of the sides of the cut and the founding of the bridge abutments.

1. Proposed Grade Separation Structure (continued)

The bedrock at the site is generally a biotite granite gneiss. Outcrops occur in three locations within a radius of about 100 feet from the proposed bridge. An examination of these outcrops shows that the predominant jointing system is oriented in an east-west direction as shown on Drawing S7447-1. The dip measured in the outcrops is about 50 degrees to the horizontal and towards the north. In the cores obtained at boreholes 1 and 2, the jointing system was found to be inclined at angles varying from 35 to 60 degrees. It is inferred that the direction of the inclination of the joints is the same as the dip observed at the outcrops, in other words, downwards towards the north. The above jointing pattern in the bedrock is an important consideration in the design of both the cut slopes on the south side of the highway and the south abutment if the latter is carried on the bedrock above final highway elevation.

Since the predominant set of joints is at an average of about 50 degrees to the horizontal, it is recommended that in the vicinity of the abutments to the proposed overpass bridge, the sides of the rock cut be sloped back to at least 1 vertical to 1 horizontal. The estimated angle of friction, rock to rock, is 35 degrees. In actual fact, the effective overall angle of friction is greater than this value due to such factors as the interlocking effect of non-continuous joints, and end-effects. It is not possible to calculate what additional stability is provided by the above factors and it is recommended that these beneficial effects be considered as a factor of safety and that 35 degrees be used in design.

1. Proposed Grade Separation Structure (continued)

Foundations carried on the sound bedrock below final roadway elevation may be designed for an allowable bearing value of 20.0 tons per square foot. The same bearing value may be used for footings carried on the sound rock within the slopes providing that requirements with regard to slope stability be satisfied. In this case, the stability of a wedge of rock beneath the abutment or pier foundation should be checked for stability under the worst combination of vertical and horizontal forces as imposed by the bridge structure and the train and fill loadings. If only vertical loads were involved for example, this requirement would entail that the complete foundation carried on the slope be located below a line drawn at 35 degrees to the horizontal starting at the toe of the slope. The bedrock beneath the foundations carried in the slope could also be suitably reinforced with steel anchor rods or the horizontal loads could be resisted entirely by inclined tie rods anchored into the bedrock, instead of being resisted by rock to rock friction. It is recommended that the above requirements with regard to cut slopes and foundations be applied to both sides of the bridge.

The backfill behind the abutments should be composed of clean well compacted non-frost susceptible granular material. The horizontal force exerted by the fill on the abutment should be assumed to be the at-rest earth pressure, and a coefficient of earth pressure of 0.4 may be used. Adequate drainage should be provided for the backfill behind the abutments, to avoid the build-up of hydrostatic pressures.

Care should be exercised in the blasting of the rock at the proposed bridge location in order to avoid loosening of

1. Proposed Grade Separation Structure (continued)

the sides of the cut or shattering below proposed footing locations, thus reducing the carrying capacity of the rock. It is recommended that the final cut in the vicinity of the bridge structure, and footing excavations be examined prior to construction of the footings.

On either side away from the bridge structure, the sides of the rock cut could be made as steep as possible consistent with maintaining safety against slides or falling rocks.

2. Causeway and Moon River Bridge

a) Causeway

It is understood that it is proposed to use excavated rock obtained from adjacent road cuts, as the fill for the causeway. For the most part, the line of the causeway is underlain by either very loose wood chips and sand, or by bedrock directly. In these locations, it is believed that the rock fill will assume side slopes of about 1-1/4 horizontal to 1 vertical, and that they will be stable. In the vicinity of borehole 8, however, very soft varved sensitive clay underlies the wood chips and sand layer. Based on the shear strength values of 120 to 220 pounds per square foot, the proposed causeway with roadway elevation of 730 at this location, would be unstable at side slopes of 1-1/4 to 1 that would form naturally during dumping. Special measures will therefore be required in this locality. It is suggested that the clay, and overlying wood chips and sand therefore be either removed by dredging prior to dumping of the rock fill or that the clay be thoroughly remoulded by blasting prior to filling to ensure complete displacement of the clay under the weight of the rock

2. Causeway and Moon River Bridge (continued)

a) Causeway (continued)

fill. For this purpose, the nose of the rock fill should be pointed with an angle between the faces of about 90 degrees. Dumping should also be carried out as rapidly as possible after blasting to take advantage of the remoulding effect of the blasting. The necessary stability for the causeway cross-section at this location could also be achieved by the provision of stabilizing berms. However, this possibility has not been considered in detail further, because of the practical difficulty of constructing the required berms which would have to be placed underwater in advance of the main body of the causeway.

The granite gneiss is a hard and durable rock, and although no freeze-thaw or soundness tests have been carried out, it is believed that the rock would be suitable for use in causeway construction from the point of view of weathering.

Although the design of the causeway is beyond our present terms of reference, it is pointed out that selection of stone sizes may be necessary for purposes such as; protection of the sides against wave and ice action, protection of the nose against scour by high river currents should these occur during construction, and the forming of the junction between the causeway and the Moon River Bridge.

2. Causeway and Moon River Bridge (continued)

b) Moon River Bridge

From available information, this structure will be a single span skew bridge with an 80 foot minimum span. The north abutment will be founded on the island and the south abutment will join the bridge to the causeway.

At the location of the north abutment, bedrock was encountered at a depth of about 6 feet below present ground surface at boreholes 3 and 4, that is, at about elevation 725. At borehole 3A, put down about 3 feet from borehole 3, the bedrock surface was at about elevation 727 indicating that the bedrock surface is quite variable at this abutment location. In boreholes 3 and 3A, at about elevation 715, a 4 inch wide seam was encountered which appeared to be partially open and contain fines and weathered rock. The drill water was lost into the seam and during drilling borehole 3A, the wash water returned through the previously drilled borehole 3. Except in the immediate vicinity of the seam, the bedrock in the boreholes above and below the seam appeared to be sound.

The north abutment could be founded directly on the sound bedrock for which an allowable bearing value of 20.0 tons per square foot may be used. If the abutment is founded above elevation 715, it is recommended that the seam encountered in the rock be washed out over the plan area of the abutment and filled with cement grout prior to construction of the abutment.

2. Causeway and Moon River Bridge (continued)

b) Moon River Bridge

For the roadway elevation as proposed, the abutment, if of the solid type, would retain about 12 feet of fill above present ground level. It is recommended that this backfill be clean non-frost susceptible granular material as in the case of the abutments for the grade separation structure at the Canadian Pacific Railway. As before, a lateral earth pressure coefficient of 0.4, and adequate drainage of the backfill is recommended.

The soil conditions at the south abutment to the bridge, as represented by those encountered at boreholes 5 and 6, consist of an irregular thickness of overburden of boulders and sand. There is a five foot difference in the elevation of sound bedrock between boreholes 5 and 6, and it is expected that the bedrock surface at this abutment location would also be irregular between the boreholes, as at the north abutment.

Because of irregular and probably loose nature of the boulders and sand stratum and the susceptibility of the overburden as a whole to scour, particularly the sand, the overburden is not considered a suitable foundation stratum for the south abutment. It is therefore recommended that the abutment be founded on the sound bedrock for which an allowable bearing value of 20.0 tons per square foot may be used. As an alternative, the abutment could be carried on drilled caissons socketed into the bedrock. The choice of foundations is dependent on economics and other factors beyond the scope of this report.

2. Causeway and Moon River Bridge (continued)

b) Moon River Bridge

The details of the junction between the causeway and the Moon River Bridge will depend to a certain extent on the scheduling of the construction of the end of the causeway, and the bridge. Assuming that the abutment is built in advance of completion of the end of the causeway and that it is solid and therefore acts as a retaining wall, special measures would be advisable with regard to the backfilling procedures. These would particularly avoid the possible impact of large rocks tumbling down the front face of the causeway fill as this approaches the abutment at closure. A suggested method of providing protection for the abutment would be to incorporate a buffer zone of selected granular fill of limited size, immediately behind the abutment. This could be dumped adjacent to the abutment before completion of the causeway. A lateral earth pressure coefficient from the fill of 0.4 is recommended for design.

No consideration has been given in this report to such factors as ice pressure on the structure, and the effects of scour through the bridge opening once the river has been constricted by the causeway, since these are beyond our terms of reference. These factors would of course have to be taken into account in design.

CONCLUSIONS AND RECOMMENDATIONS

1. In general, at the three locations investigated there is a shallow thickness of granular overburden, then bedrock. The detailed soil and rock conditions are discussed in the report.

2. Because of the observed dip in the bedrock in the vicinity of the grade separation structure, special provisions are required in the design of the rock cut and the foundations for the structure as discussed in the report.

3. For stability of the causeway, special measures such as dredging or blasting will be required in a localized clay area, as discussed.

4. The abutments for the Moon River Bridge may be carried on bedrock at a recommended allowable bearing value of 20.0 tons per square foot. Special provisions such as grouting and backfilling procedures are recommended as given in the report.

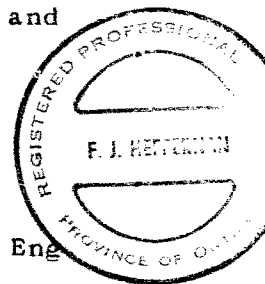
PERSONNEL

The field work for this investigation was carried out under the field supervision of Mr. H. W. Green, P. Eng. This report was written by Messrs. Green and Heffernan and reviewed by Mr. M.A.J. Matich.

FJH/dw
57447

F. J. Heffernan

F. J. Heffernan, P. Eng.



APPENDIX I

Procedure

Site and Geology

Soil and Rock Conditions

Water Conditions

Office Reports on Soil Exploration

PROCEDURE

The field work for this investigation was commenced on November 20th, 1962 and completed on December 13th, 1962. Nine boreholes were put down in BX and AX size using a skid-mounted machine drill rig. Two additional dynamic penetration tests were also put down along the route of the proposed embankment and one test pit and one penetration test were put down at the grade separation. Bedrock was cored in AXT size in all of the boreholes for depths ranging from 11 to 28 feet. Detailed logs of the borings are given on the Office Reports on Soil Exploration in this Appendix. The location of the boreholes, penetration tests and the test pit are shown on Drawing S7447-1 in the pocket at the rear of this report.

The laboratory testing of the soil samples was carried out in the Soil Mechanics Laboratory of Geocon Ltd in Toronto. The results are plotted on the Office Reports in this Appendix. The soil samples remaining after testing will be stored until June 1st, 1963, at which time you will be contacted for instructions regarding their disposal.

All elevations given in this report are referred to Geodetic Datum. Two temporary bench marks were established in the area, one near the river bridge site and another at the railroad-road intersection at the proposed grade separation. At the river bridge site the bench mark consists of a spike driven into a large exposed root of a tree of which the elevation was given to us as 728.34. At the proposed grade separation, the bench mark consists of a spike driven into the planking between the rails and it has an elevation of 757.65. Each bench mark and its elevation was established by a Department of Highways, Ontario survey party.

SITE AND GEOLOGY

II.

The site is located within the Town of Bala, Ontario, as shown on Drawing S7447-1 and consists of the relocating of the existing road to a location west of the C.P.R. tracks. The proposed road is composed, in fact, of a grade separation beneath the C.P.R. railway, and a causeway embankment in the Moon River adjacent to the railway, connecting with a proposed river bridge.

Much rock is exposed in the site area. It is composed of biotite granitic or syenitic gneiss of generally good quality as viewed in the exposures. It is overlain by granular soils composed of fine to coarse grey-brown sand and medium gravel with boulders to about 1 foot in diameter.

Geological information regarding the bedrock and overburden in the Bala area is non-existent.

SOIL AND ROCK CONDITIONS

The principal soil strata encountered at the sites are as follows:

a) Proposed Grade Separation

Sand and Gravel Fill

A layer of grey-brown sand and gravel fill is contained within the railway and road embankments at the existing level crossing. From visual examination, the material appears to be a well graded sand and gravel with a maximum grain size of about 1-1/2 inches. The thickness as encountered in the boreholes, penetration test and test pit ranged from about 2 to 3 feet.

a) Proposed Grade Separation (continued)

Sound Biotite Granitic and/or Syenitic Bedrock

At the proposed grade separation, bedrock is exposed or is covered with a thin mantle of granular soil. The rock is composed of biotite granite and/or syenite gneiss which strikes nearly perpendicular to the direction of the railroad tracks as shown on Drawing S7447-1. The rock dips to the north where exposed at about 50 degrees but is locally contorted both in strike and dip along the northerly corner of the exposure adjacent to the existing Highway 69.

Particular attention was given to the dip of the rock at depth. Holes were drilled to below grade to about elevation 726 or about 30 foot depth. The dip in this vertical length varied from about 30 to 60 degrees.

The rock both in surface exposure and at depth appeared to be generally in sound condition. In borehole 2, the uppermost 18 inches was inclined to be seamy and slightly weathered. This uppermost section may be a boulder or weathered bedrock and has been classified as such on the Office Report and on Drawing S7447-1.

b) Proposed River Bridge and Embankment Site

Fine to Coarse Sand with Some Gravel

Above the water level at the north abutment of the bridge, a layer of fine to coarse sand with some gravel and occasional boulders up to 1 foot in thickness exists. The thickness of the layer as encountered in the boreholes

b) Proposed River Bridge and Embankment Site (continued)

Fine to Coarse Sand with Some Gravel (continued)

ranged from 5.3 to 6.4 feet.

Boulders and Sand

At the south abutment, the river bottom is underlain by boulders and sand. The upper six feet of this stratum which was diamond drilled throughout, appears to consist of boulders with sand in the voids, while below this, the material consists of fine grey sand with some boulders. The thickness of the sand section appears to range from about 1.0 foot at borehole 5 to about 5.0 feet at borehole 6.

Very Loose Wood Chips and Sand

A very loose wood chip and sand deposit underlies the river bottom in the causeway area. This wood chip and sand layer offered practically no penetration resistance whatsoever during sampling and representative samples were difficult to recover. The thickness of the material ranges up to 7 feet as encountered in borehole 8 and as inferred in penetration test 3.

Very Soft Grey Varved Silty Clay

Underlying the wood chip and sand layer is a stratum of medium grey varved silty clay. The thickness of the clay is inferred to range from 0 to about 4 feet below the causeway area.

b) Proposed River Bridge and Embankment Site (continued)

Very Soft Grey Varved Silty Clay (continued)

Atterberg limits were carried out on one sample of the clay and gave a value of 53 for the liquid limit and 21 for the plastic limit. The corresponding natural moisture content ranged from 79 to 81 percent. This high moisture content is in keeping with the very soft consistency and sensitive nature of the clay.

During sampling, the sampling tube sank readily into this material under the weight of rods alone for about half of the tube length. The remainder of the tube was pressed easily into the clay.

Three undrained triaxial tests gave compressive strength values ranging from 0.12 to 0.22 tons per square foot with an average of 0.18 tons per square foot. Based on these values, and on the slight observed resistance to penetration, the consistency of the clay has been estimated to be very soft.

This material, where it exists overlies the bedrock surface directly in the causeway area.

Sound Biotite Granite and/or Syenite Gneiss

Much bedrock is exposed at the river bridge site as shown on Drawing S7447-1. The rock is composed of biotite granite and/or syenite gneiss. The strike of the exposed rock is east-west and the rock dips to the north at about 50 to 70 degrees in general. Along the shoreline to the

b) Proposed River Bridge and Embankment Site (continued)

Sound Biotite Granite and/or Syenite Gneiss (continued)

west of the proposed structure an area of flat shelving of the rock may be observed.

Both corner locations of the land abutment were drilled to depths of about 25 feet. In borehole 3, an open seam in the rock was encountered at about elevation 715 and was confirmed in the adjacent borehole 3A located 3 feet away. The seam appears to be from 2 to 4 inches in thickness and contains open portions as the drill rods were wrenched down by hand about 1-1/2 inches in borehole 3A. During drilling, the rock is easily ground in this seam. Pumping tests were conducted and muddy water was continuously pumped out at about 100 gallons per hour. Furthermore, observation on the drop of the water level in the borehole after filling, suggests that the seam is connected with the river. When drilling in borehole 3A, the return water also flowed out of borehole 3. In borehole 4, the seam as such was encountered at the same approximate elevation. However, it consisted only of weathered core surfaces over a thickness of about 5 feet between elevations 710 and 715. At the south abutment of the river bridge the bedrock is of sound condition and does not contain any open seams.

The rock underlying the causeway embankment is composed of biotite granite and/or syenite gneiss as before and is generally of sound condition. At least 10 feet of core were recovered from each hole and core recovery was high.

WATER CONDITIONS

VII.

At the time of the investigation the level of Moon River was at about elevation 721. The reported high water level of Moon River at the site is 726.3. No water level was encountered in the permeable overburden in the land holes.

EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"

The object of this form is to enable a comprehensive study of the soil to be made by combining on one sheet all of the information obtained from the boring. An explanation of the various columns of the report follows.

ELEVATION AND DEPTH

This column gives the elevation and depth of boundaries between the various soil strata. The elevation is referred to the datum shown in the general heading.

WATER CONDITIONS

In this column the water level in the casing at the time of boring or the water table in the ground, determined by a series of observations in a piezometer or standpipe, is indicated to scale by a horizontal line with the symbol W.L. or W.T. above the line. A notation of any complicated groundwater conditions will be made in this column.

DESCRIPTION

A description of the soil, using standard terminology, is contained in this column. The consistency of cohesive soils and the relative density of non-cohesive soils are described by the following terms:

Consistency	U-Strength Tons/sq. ft.	Relative Density	Standard Penetration Resistance, Blows/ft.
Very soft	0.03 to 0.25	Very loose	0 to 4
Soft	0.25 to 0.5	Loose	4 to 10
Firm	0.5 to 1.0	Compact	10 to 30
Stiff	1.0 to 2.0	Dense	30 to 50
Very stiff	2.0 to 4.0	Very dense	over 50
Hard	over 4.0		

STRATIGRAPHIC PLOT

The stratigraphic plot follows the standard symbols of the National Research Council, Canada.

ELEVATION SCALE

The information in all columns is plotted to a true elevation scale which is shown in this column.

GRAPHS

The main body of the report forms a graph which is used to plot to correct elevation the important soil properties which are obtained through field and laboratory tests. The scales and symbols for the plotting are shown at the head of the column.

OTHER TESTS

In this column are shown, by symbol, the other field or laboratory tests which have been performed on the soil and for which the results have not been plotted on the above graph.

SAMPLES

The first three columns describe the condition, type and number of each sample obtained from the boring. The location and extent of each sample is plotted to scale.

In the last column is shown the penetration resistance in blows of 4200 inch-pounds required to drive one foot of the sampler into the ground. When a 2 inch Drive Sampler is used the result obtained is termed the "Standard Penetration Resistance".

GEOCON

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OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 77147 BORING # 1 DATUM GEOGRAPHIC CASING AX
 BORING DATE 12/1/66 REPORT DATE DEC 2, 1966 COMPILED BY AEL CHECKED BY F.J.H.
 SAMPLER HAMMER WT 4 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

 DISTURBED
 FAIR
 GOOD
 LOST

SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - FLEEVE-OPEN
 S.F. - FLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

SOIL PROFILE										SAMPLES				
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W%					OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTING BLOWS/FT.
					O NAT. LW Δ Pw									
					DYNAMIC PENETRATION TEST BLOWS PER FOOT									
					54 1/2									
755.9 0.0		GROUND LEVEL		755										
753.6 2.3		LOOSE BROWN SAND AND GRAVEL FILL		750										
		57° DIP @ 4' DEPTH 51° DIP @ 2' DEPTH 55° DIP @ 10.5' DEPTH		745	2" CORE LENGTHS 6"-1" CORE LENGTHS					RC RECOV 100%	AKT RC	1		
				740	PARTIAL WATER LOSS									
				735	CORE ENDS WEATHERED 2" CORE LENGTHS									
		SOUND		730	WEATHERED FRACTURE									
		MOTTLED LIGHT GREY AND BLACK BIOTITE GRANITE GNEISS BEDROCK		725	6"-1" CORE LENGTHS									
		GARNETIFEROUS FROM 2.5' - 3.0'		720	2" CORE LENGTHS									
				715	6"-1" CORE LENGTHS									
		40° DIP @ 20' DEPTH 45° DIP @ 20-23' DEPTH 35° DIP @ 24' DEPTH 50° DIP @ 27-29' DEPTH		710										
725.9 30.0		END OF HOLE		705										

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57447 BORING # 2 DATUM GEODETIC CASING AX.
 BORING DATE NOV. 22, 1962 REPORT DATE DEC. 2, 1962 COMPILED BY ALL CHECKED BY F. J. H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
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 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

SOIL PROFILE									SAMPLES			
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W% O NAT. □ LW △ Pw		OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.	
					DYNAMIC PENETRATION TEST BLOWS PER FOOT							
		</										

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 37447 BORING # 3 DATUM GEODETIC CASING AX
 BORING DATE NOV 27 1962 REPORT DATE DEC 6 1962 COMPILED BY A.E.L. CHECKED BY F.J.H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

SOIL PROFILE							SAMPLES				
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W% O NAT. □ LW △ Pw		OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
					DYNAMIC PENETRATION TEST BLOWS PER FOOT						
731.4 0.0		GROUND LEVEL		730							
725.5 5.9 724.4 7.0		LOOSE BROWN AND GREY FINE TO MEDIUM SAND WITH SOME GRAVEL AND OCCASIONAL BOULDERS		725							
		WEATHERED BEDROCK OR BOULDERS		720							
		SOUND GARNETIFEROUS BIOTITE GRANITE GNEISS BEDROCK POSSIBLE OPEN SEAM AT 16.2' OF ABOUT 4" THICKNESS.		715							
		70° DIP @ 7-16' DEPTH 52° DIP @ 22.0' DEPTH 50° DIP @ 27.0' DEPTH		710							
701.1 30.3		END OF HOLE		700							

BH * 3

8' - 1' CORE LENGTHS

5' - 1' CORE LENGTHS
2' CORE LENGTHS @ 13.4'

RC RECOY 52%

RC RECOY 100%

AXT. RC.

1

2

3

4

5

6

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57447 BORING # 3 A DATUM GEODETIC CASING AX
 BORING DATE NOV. 28, 29 1962 REPORT DATE DEC. 6, 1962 COMPILED BY AEL CHECKED BY F. J. H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 Qc - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES

SOIL PROFILE										SAMPLES							
ELEV.N. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W% ○ NAT. □ LW ▲ Pw					OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.			
					DYNAMIC PENETRATION TEST BLOWS PER FOOT												
732.0 0.0		GROUND LEVEL			<div>SH #3A</div>												
726.8 5.2		LOOSE GREY BROWN FINE TO MEDIUM SAND WITH SOME GRAVEL AND OCCASIONAL BOULDERS.		730													
				725													
		SOUND BIOTITE GRANITE GNEISS BEDROCK PARTLY GARNET IFEROUS POSSIBLE OPEN SEAM AT 17.2' DEPTH OF ABOUT 4" THICKNESS.		720													
				715													
		40° DIP @ 6' DEPTH 30° DIP @ 13' DEPTH 42° DIP @ 16' DEPTH 58° DIP @ 24' DEPTH		710													
706.8 25.2		END OF HOLE		705													
					</												

SH-3A

RC RECOV 100%

RC RECOV 90%

RC RECOV 100%

AXT RC

1
2
3
4
5
6
7
8
9
10
11

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 7447 BORING # 4 DATUM GEODETIC CASING AX
 BORING DATE NOV. 30, 1962 REPORT DATE DEC. 8, 1962 COMPILED BY AEL. CHECKED BY F. J. H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES

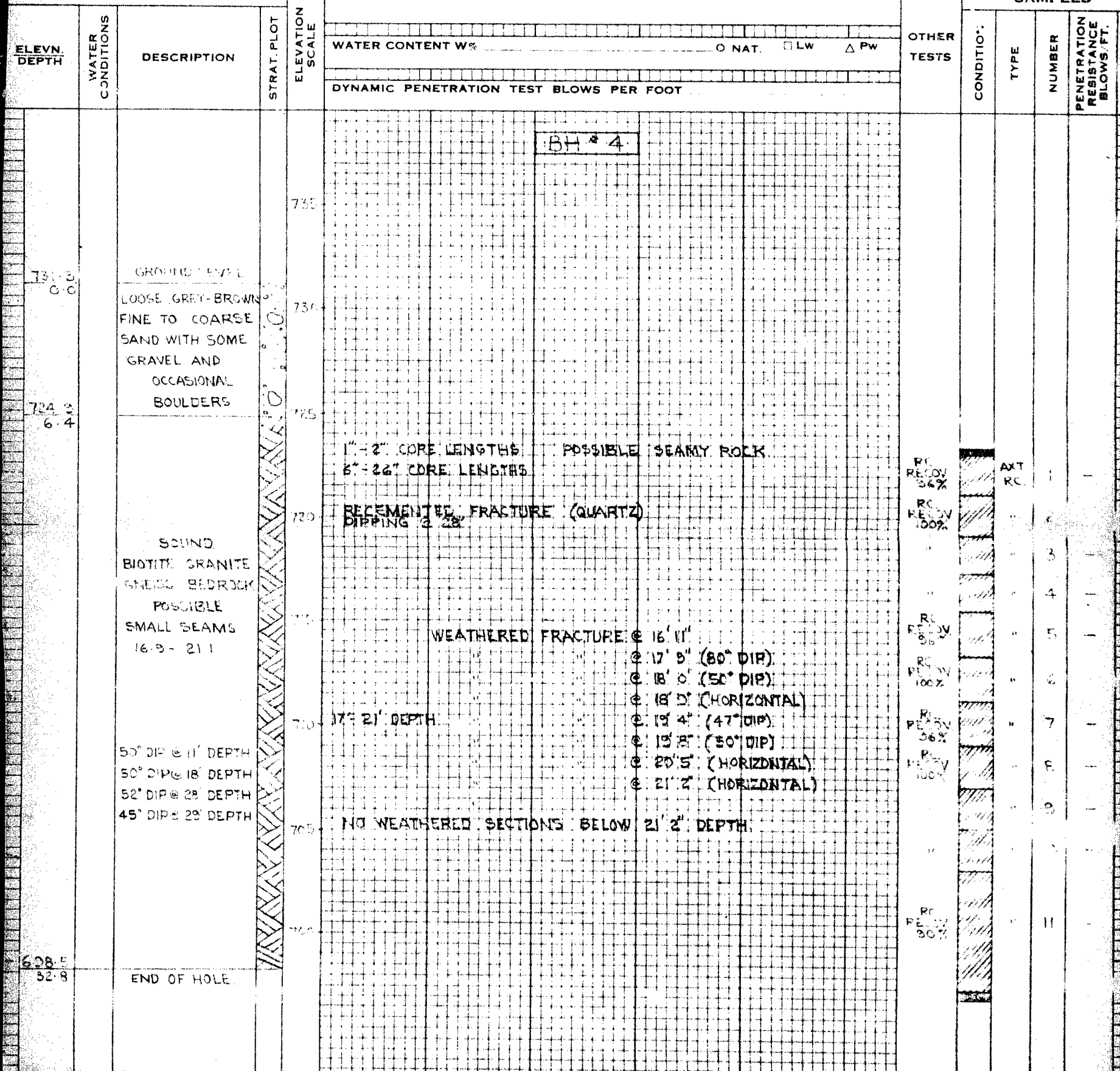
A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE
F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
QC - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57447 BORING # 5 DATUM GEODETIC CASING AX
 BORING DATE DEC. 17, 1962 REPORT DATE DEC. 4, 1962 COMPILED BY A.E.L. CHECKED BY F.J.H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



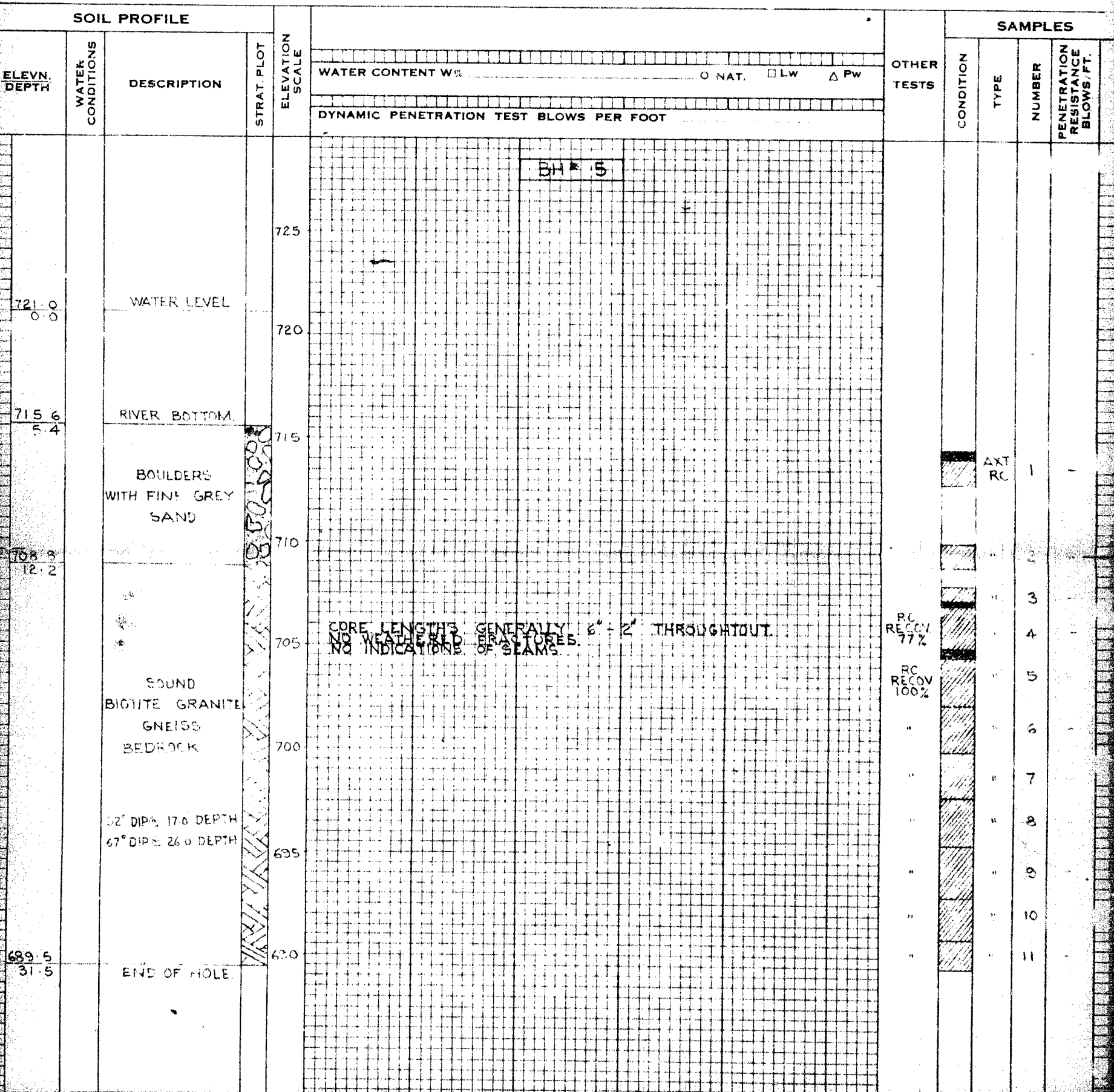
A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

SAMPLE TYPES

ABBREVIATIONS

DISTURBED
FAIR
GOOD
LOST

A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- QC - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW

Y - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION

WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE						OTHER TESTS		SAMPLES			
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W% O NAT. □ LW △ PW			CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
				DYNAMIC PENETRATION TEST BLOWS PER FOOT							
720.6 0.0		WATER LEVEL		720							
716.0 4.6		RIVER BOTTOM		715							
		BOULDERS AND FINE GREY SAND		710							
709.3 11.3		FINE GREY SAND WITH FEW POSSIBLE BOULDERS		705							
703.7 16.9		SOUND BIOTITE GRANITE GNEISS BEDROCK		700							
		70° DIP @ 15' 0" DEPTH		695							
		70° DIP @ 34' 0" DEPTH		690							
					BH # 6						
					PINK FIELDSPATHIC SECTION 18' 5" - 19' 0"						
					CORE LENGTHS GENERALLY 6" - 18"						
							RC RECOV 100%			3	
							"			4	
							"			5	
							"			6	
							"			7	
							RC RECOV 87%			8	
							RC RECOV 100%			9	

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S 7447 BORING # 7 DATUM GEDETIC CASING AX
BORING DATE DEC 6 1962 REPORT DATE DEC 17 1962 COMPILED BY AEL CHECKED BY FJH
SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

 DISTURBED
 FAIR
 GOOD
 LOST

SAMPLE TYPES

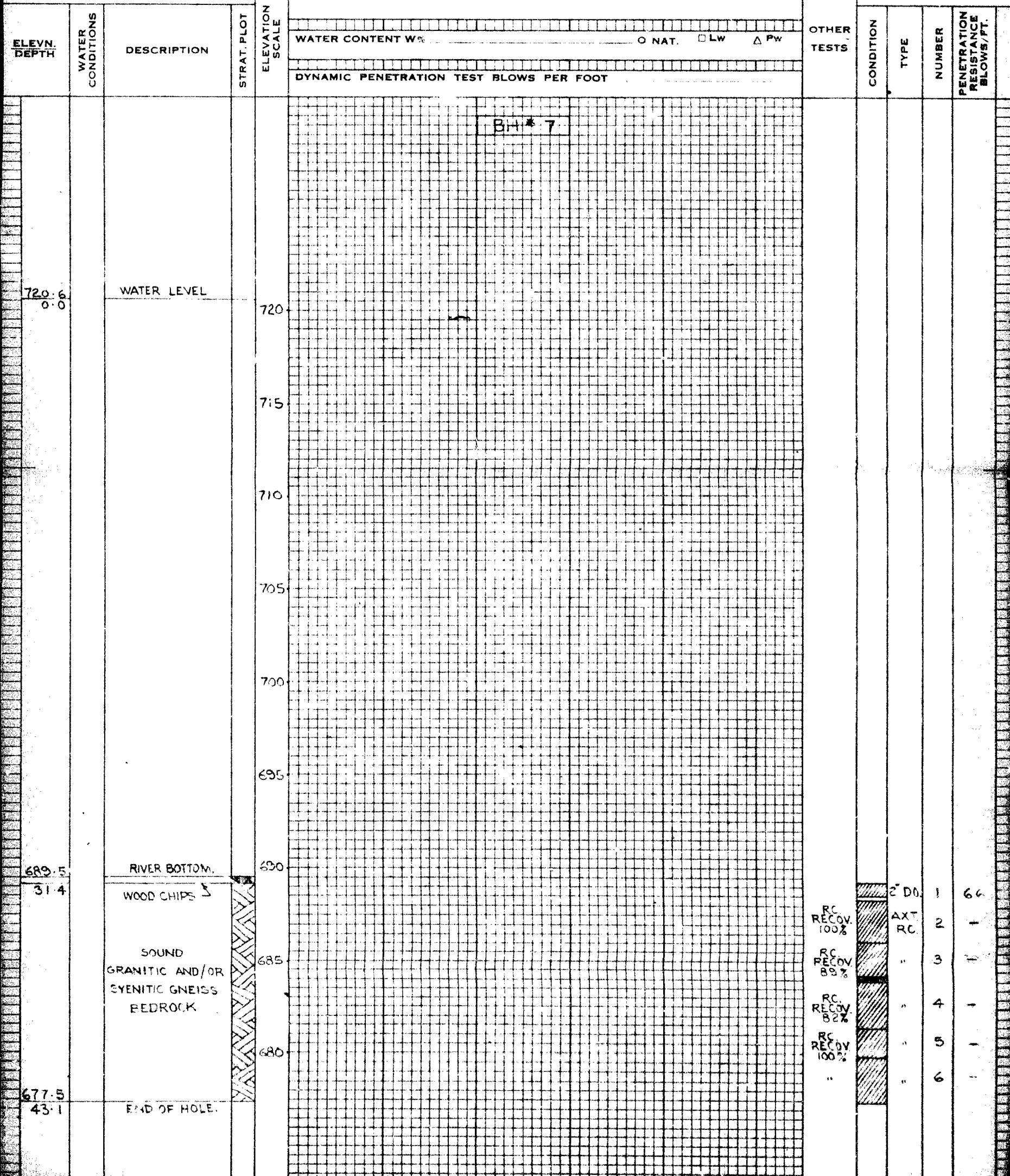
A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE
F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
QC - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT \$ 7447 BORING # 8 DATUM GEODETIC CASING AX.
 BORING DATE DEC. 7, 1962 REPORT DATE DEC. 17, 1962 COMPILED BY A.E.L. CHECKED BY F.J.H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

 DISTURBED
 FAIR
 GOOD
 LOST

SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

COMPRESSION STRENGTH TONS / SQ. FT.

UNCONFINED COMPRESSION TEST

0.1 0.2 0.3 0.4 0.5

WATER CONTENT W% 20 40 60 80 100

DYNAMIC PENETRATION TEST BLOWS PER FOOT

OTHER TESTS

SAMPLES

CONDITION

TYPE

NUMBER

PENETRATION RESISTANCE BLOWS/FT.

ELEV. DEPTH

WATER CONDITIONS

DESCRIPTION

STRAT. PLOT

ELEVATION SCALE

720.7
6.0

WATER LEVEL

720

715

710

705

700

695

690

685

680

702.5
18.2

RIVER BOTTOM

VERY LOOSE
 WOOL CHIPS
 AND SAND.

695.6
25.1

VERY SOFT GREY
 VARVED SILTY CLAY

691.9
28.8

SOUND GRANITIC
 AND / OR
 SYENITIC GNEISS
 BEDROCK

680.7
40.0

END OF HOLE.

BH # 8

γ = 101
 γ = 99
 γ = 83
 γ = 95

RC
 RECOV
 100%

RC
 RECOV
 96%

RC
 RECOV
 87%

2" SO

SO

TO

AXT RC

"

"

HYD. PUSH

1

2

3

4

5

6

7

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7447 PEN TEST # 1 AND TEST PIT # 1 DATUM GEODETIC CASING -
 BORING DATE NOV 23, 1962 REPORT DATE DEC. 20, 1962 COMPILED BY AEL CHECKED BY F. J. H.
 SAMPLER HAMMER WT. LBS. DROP INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES																
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W% O NAT. □ LW △ PW							OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
					DYNAMIC PENETRATION TEST BLOWS PER FOOT											
756.0 0.0		GROUND LEVEL		755	PT #1											
753.7 2.3		PROBABLY SAND AND GRAVEL														
		REFUSAL		750												
757.0 0.0		GROUND LEVEL		755	TP #1											
755.3 1.7		GREY BROWN SAND AND GRAVEL FILL														
		BEDROCK SURFACE END OF TEST PIT														

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7447 PEN. TEST *2 DATUM GEODETTIC CASING BX
 BORING DATE DEC 10 1962 REPORT DATE DEC 31 1962 COMPILED BY AEL CHECKED BY F. J. H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

SAMPLE TYPES

ABBREVIATIONS



DISTURBED
FAIR
GOOD
LOST

A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

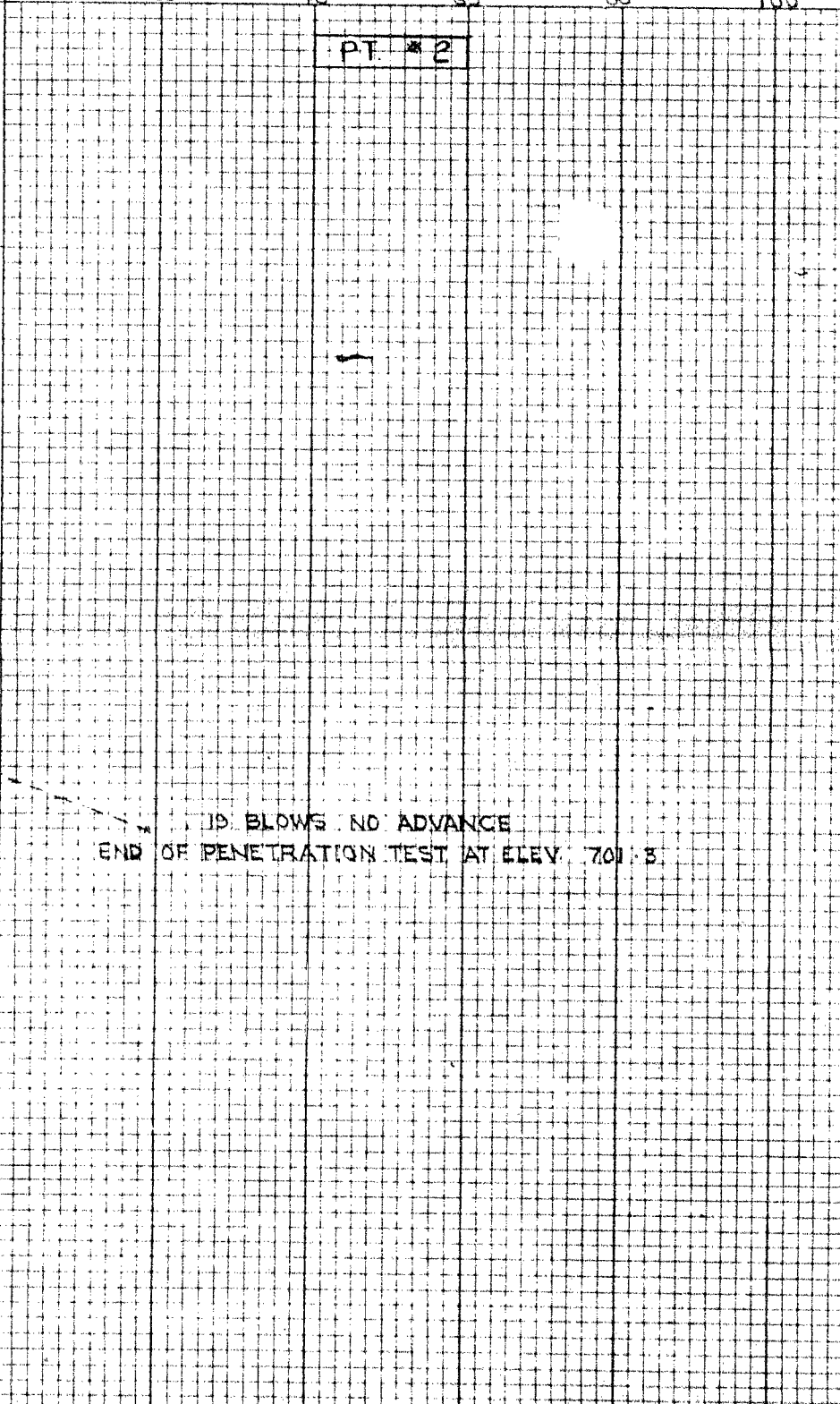
F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
QC - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW

γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W% O NAT. □ LW ▲ Pw					OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
					DYNAMIC PENETRATION TEST BLOWS PER FOOT 20 40 60 80 100									
721.1 0.0		WATER LEVEL		720	<div>PT. #2</div> 									
				715										
				710										
				705										
702.9 18.2		PROBABLY VERY LOOSE WOOD CHIPS AND SAND		700	19 BLOWS NO ADVANCE END OF PENETRATION TEST AT ELEV. 701.3									
701.3 19.8		RIVER BOTTOM		700										
		REFUSAL												

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S.7447 PEN. TEST * 3 DATUM GEODETIC CASING
 BORING DATE DEC. 10, 1962 REPORT DATE DEC. 20, 1962 COMPILED BY AEL CHECKED BY F. J. H.
 SAMPLER HAMMER WT. LBS. DROP INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

ELEV. DEPTH
 WATER CONDITIONS
 DESCRIPTION
 STRAT. PLOT
 ELEVATION SCALE

WATER CONTENT W% NAT. LW PW

DYNAMIC PENETRATION TEST BLOWS PER FOOT
 20 40 60 80 100

OTHER TESTS

SAMPLES

CONDITION
 TYPE
 NUMBER
 PENETRATION RESISTANCE BLOWS FT.

721.1
 0.0

WATER LEVEL

PT. # 3

701.2
 19.9

RIVER BOTTOM

PROBABLY
 VERY LOOSE WOOD
 CHIPS AND SAND
 CHANGING
 TO VERY SOFT
 VARVED SILTY CLAY

WEIGHT OF HAMMER

690.5
 30.6

REFUSAL

END OF PENETRATION TEST AT ELEV. 690.5

23-64-10

Mr. A. M. Teye,
Bridge Engineer,
Bridge Division.

Attention: Mr. A. McComb.

Mr. A. G. Starnac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

January 7, 1963.

63-F-217(C)

FOUNDATION INVESTIGATION REPORT - BY
Geocan, Limited, Consulting Engineers,
Proposed Moon River Bridge and C.P.R.
Subway, Sault, Ontario, District No. 11.
W.P. 59-60 and W.P. 137-61

Attached, we are forwarding to you the above-mentioned report submitted by the consultant, Geocan, of Toronto.

We have reviewed the report and find the factual information well presented and in general agreement with the recommendations contained in the report. With respect to the formation of the excavation slopes at the site of the subway, we would not, however, agree with the consultant's recommendation that both the north and the south slope be constructed in the same manner. According to the report, the rock strikes nearly perpendicular to the railroad tracks and dips from 30 to 50 degrees to the north (page 1). It is inferred that the inclination of the joints found in the core is the same as the dip observed at the outcrop - in other words - downwards towards the north (page 3).

When a cut will be made at the crossing to lower the highway under the railroad, entirely different conditions will be created on the south and on the north excavation slopes. While the rock dip will create problems on the south slope, where the consultant's recommendation should be applied, no problems should be encountered on the north slope which can be vertical.

If the south slope is constructed as 1:1 and the footings are kept below a 35° line extending from the toe of the slope, as explained in the report, no anchors or bolts would, in our opinion, be necessary.

We would recommend that subexcavation, rather than blasting be used for the removal of the soft clay beneath the proposed causeway.

AC/MSG

attach.

cc: Messrs. A. M. Teye (2)

H. A. Freganek

H. E. McMillan

H. McArthur

J. E. Jones

F. Bureau

Foundations office

A. G. Starnac

A. G. Starnac,
PRINCIPAL FOUNDATION ENGINEER

V. J. Kovach

J. Roy

J. A. Crispier

G. E. Saint

A. Watt

Gen. Files.

Materials and Research Division

November 16, 1962

Geecon, Limited,
Consulting Engineers,
14 Maas Road,
Bexdale, Ontario.

Attention: Mr. F. Heffernan

Re: W.F. 59-60, Hwy. 69, C.P.R. Subway at Bala; ✓
W.F. 187-61, Hwy. 69, Moon River at Bala. ✓
District No. 11.

Dear Sir:-

Please consider this your authority to carry out foundation investigations at the above sites. Plans and profiles were provided to your representative on November 14, 1962.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Fourteen copies of each completed foundation report, plus one additional copy of the subsoil profile, should be submitted to the Foundation Section as soon as possible. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Charges for the work performed will be in accordance with your Schedule of Rates, dated October, 1960, and invoices to be addressed to the attention of the undersigned.

Note:- As Toronto is the nearest recognized mobilization point, payment for mobilization will be from there, as discussed with your representative.

HMM/Miel

Yours very truly,

A. Rutha

cc: Messrs. S. McCombie
H. McArthur
H. C. Dernier
T. J. Kovich
W. D. Smith (2)
Mrs. T. Tate
Foundations Office
Gen. Files (2)

A. Rutha,
MATERIALS & RESEARCH ENGINEER

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Eng.
Room 107, Lab. Bldg.

FROM: J. C. McAllister

DATE: October 11, 1962.

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 59-60 C.P.R. Subway at Bala
187-61 W.P. ~~171-61~~ Moon River at Bala
Hwy. #69 District #11

Attached please find plans and profiles of the proposed relocation of Hwy. #69 at Bala. The location of the required test holes at the C.P.R. Subway and the Moon River Bridge are shown.

The borings should be carried out to give the information requested in a letter to Mr. T.J. Kovich, dated 19th, Sept., 1962, a copy of which is enclosed.

JCMcA/et

J. C. McAllister

J. C. McAllister,
for S. McCombie,
Bridge Planning Engineer.

September 19, 1962

Materials & Research Section,
Department of Highways, Ontario,
Parliament Buildings,
Toronto 2, Ontario.

Att'n: Mr. T. J. Kovich, P.Eng.
Senior Soils Engineer
Re: W.R. 187-61 and W.P. 59-60
Relocation of Highway #69 at Bala

Gentlemen:

Enclosed are two prints of our preliminary drawing #SK61-151-1 showing the "Proposed General Layout and Sections" for the relocation of Hwy. #69 at Bala.

We are anxious to get some preliminary soils information for both roads and bridges, particularly in the area of the swamp on the connecting road to Hwy. #103 and at the grade separation structure.

From what we can see on the site, rock outcrops or is close to the surface all over the site, with the exception of the area of the swamp.

We would like to get your thoughts on the suitability of the rock formation at the grade separation as a founding material for the bridge abutments, paying particular attention to the physical state of the rock close to the surface and at the depth of cut of about 20 to 25 feet. The foliation, dip and (strike) etc. of the rock will have a direct bearing on the design of perched abutments if these prove to be practical. *strike*

Your ideas on the granular base courses and the pavement design would be appreciated.

The fill sections in the Moon River will be rock, using the excavated material from the cuts. There does not seem to be any overburden on the river bed but this should be confirmed.

We do not anticipate any foundation problems at the river bridge site as firm rock is in evidence. Nevertheless, this should be confirmed as large boulders could be deceiving if present.

Preliminary prints of the proposed road profiles will be sent shortly, as soon as they are ready.

Yours very truly,
MORRISON, HERSHFIELD, MILLMAN & HUGGINS, SEP 20 1962

J. T. Gregg
J. T. Gregg, P.Eng.

JTG/ml - enc. - 61-151
cc. Mr. H. McArthur
Mr. F. I. Hewson ✓

D. H. O.
TORONTO
RECEIVED

OFFICE
BRIDGE

Please file with report

acp

Mr. H.D. McMillan,
Road Design Engineer.

Materials & Research Division.

February 6th, 1963.

A.P. #244-62, Hwy. #69, Relocation at Bala.

In the Foundation Report for the two structures at Bala (WP's 59-60 and 187-61), dated January 7, 1963, it was recommended that sub-excavation of the clay and wood chips in the Causeway portion be undertaken. This has now been reconsidered by this office and it is recommended that instead the following procedures be followed:

- (1) No removal of the clay and wood chips by means of excavation or blasting be done. It is expected that the method of placement and the height of the superimposed fill (30' to 40') will effectively displace these materials.
- (2) For estimating purposes assume that the fill will reach hard bottom as indicated on the profile submitted with the Foundation Report. The soft and loose materials are expected to partially displace sideways and partially move into the voids in the rock fill.
- (3) As mentioned at the top of Page 6 of the Foundation Report, a Special should be included in the contract directing the method of placement of the rockfill. This should cover the length of the fill between stations 406 and 408/50. The following is a suggested wording:
Rock Fill - Station 406 to 408/50.

In order to expedite the displacement of the underlying soft clay and wood chips the following placement procedure is to be followed --- The rock fill shall be advanced in a wedge-shape with the nose pointed so that the angle between the faces is always at about 90 degrees. The rock fill shall be advanced in one lift only, maintaining final rock grade at all times. Dumping once commenced, shall be as continuous as is practicable.

TJK/hl
c.c.H.A. Tregaskes,
T.C. Muir,
E.S. Jones,
H. McArthur,
Morrison, Hershfield, Millman, Huggins Ltd.,
S. McCombie,
A. Sternac,
T.J. Kovich,
Filed.

T.J. Kovich,
For: C.A. Wong,
Principal Soils Engineer.

MORRISON, HERSHFIELD, MILLMAN AND HUGGINS, LIMITED
CONSULTING ENGINEERS

January 28, 1963

Bridge Design Section,
Department of Highways, Ontario,
Parliament Buildings,
Toronto 2, Ontario.

Att'n: Mr. F. I. Hewson, P.Eng.
Consultant Liaison Engineer

Re: Revised May. #69 at Bala
W.P. 59-60; W.P. 187-61; W.P. 244-62

Gentlemen:

This letter will serve to place on record recent developments concerning the above job.

Following our study of the foundation investigation B41565, received in this office on January 11, 1963, we have come to the conclusion that our preliminary design for the Moon River Bridge is no longer the most economical and practical way of crossing this river. We have discussed the various ways of revising the structure to suit the actual conditions with your Messrs. Wilke, Grebaki & Davis (Jan. 13/63). It was agreed at a meeting in your office on January 25, 1963 that the structure shown on drawing D4933-P rev. Jan. 28/63, is the most desirable arrangement. The bridge, as now proposed, will be a simple composite steel span 115 ft. long, supported by a "U" type north abutment founded on bedrock, and by a perched south abutment resting on rock fill. This latter point should be discussed with soils and road design folk but we understand that a precedent has already been set on another project. It is our intention that all compressible or scourable material below the rock fill be removed prior to the construction of the embankment and abutment. Some blasting may be required to accelerate settlement of the rock fill.

Enclosed are three prints of our preliminary drawing D4933-P, rev. Jan. 28/63 for which we would appreciate your approval and/or comments. We are also enclosing the print of the Bridge Site drawing on which we have Mr. Wilke's signature approving the hydraulic aspects of the new arrangement.

It should also be noted that on January 23, 1963 we submitted two prints of our preliminary drawing D4934-P for your consideration. This drawing shows our proposed design for the C.P.R. subway.

Yours very truly,

MORRISON, HERSHFIELD, MILLMAN & HUGGINS, LIMITED


J. T. Gregg, P.Eng.

JTG/ml
enc.

61-151

cc. Mr. A. Stermac ✓
Mr. H. McArthur

Mr. T. J. Kovich,
Regional Soils Engr.

Mr. A. G. Stermac,
Principal Foundation Engr.

January 17, 1963

Your Memo - January 17/63

Foundation Report dated January 7/63, Hwy. #69,
Moon River Bridge and C.P.R. Subway at Bala,
W.P. 59-60 & W.P. 187-61. -- District No. 11.

In connection with your memo concerning the above report, we would like to make the following comment:

Subexcavation in lieu of blasting, was recommended in the covering letter to the report submitted by Geocon, Ltd., because of the layer of wood chips and sand overlying the soft clay. It is felt that this layer may act as a cushion for the dumped rock and may prevent the displacement of the soft clay, even after blasting.

Because no data or information on the behaviour of such a material under the described conditions is known to us, we felt that such a recommendation is warranted.

However, if such an operation is impractical or uneconomical, another method of construction should, of course, be given preference. Because only a relatively thin layer of soft clay was encountered, it is believed that even if difficulties are experienced, they would be of a minor nature and could readily be taken care of.

We welcome and appreciate your comments.

AGS/MdeF

cc: Foundations office
Gen. Files.

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

MEMORANDUM

TO: Mr. A. Stermac,
Principal Foundations Engr.

FROM: T.J. Kovich,
Regional Soils Engr.

DATE: January 17/63.

OUR FILE REF.

IN REPLY TO

SUBJECT: Foundation Report dated January 7/63, Hwy.#69,
Moon River Bridge and CPR Subway at Bala,
WP 59-60 & WP 187-61.

I have reviewed the above noted report and would like to make a few comments regarding the proposal to excavate the soft clay and wood chips in the causeway section. According to the one borehole and two penetration tests, the materials reached a maximum thickness of 11' under 20' of water and are located between stations 406 and 407/80.

I question (a) the need of excavation and (b) the feasibility of it considering the economics.

The report states that the L.L. for the clay is 53 and the natural moisture content 79 to 81 percent. In addition, it was reported that one half of the sampling tube sank into the clay very readily just by the weight of the rods alone. The wood chips were reported to offer no penetration resistance whatsoever. With this information I would feel that the dead weight of the fill would cause the chips and clay to readily either displace sideways or to fill the voids in the rock fill. This could be expected since (a) the fill will be rock and will be 30' high, (b) the fill will be end dumped maintaining the proposed grade elevation, (c) as the fill is dumped the momentum of the rock fragments, which will be quite large, would likely result in the rock digging deeply into the bottom of the soft layer, and (d) the rock fragments will be sharp which will aid in the penetration of the soft material.

You have stated in the covering letter of the report that subexcavation in lieu of blasting should be carried out. This would present a number of difficult problems. The cheapest way would be to dragline from the end of the advancing fill. This would present the difficulty of excavating or trying to excavate in an area into which large rock fragments would have rolled or bounced. The alternate would be to mount a dragline on a barge which of course, would be very expensive. Even considering that a machine on a barge or even a dredge were used

it would be difficult to keep the excavation open because of the fluid state of the material and because of the turbulence engendered in the water by the nearby flow from the control dam.

I am not trying to deprecate the report but I do feel that this one particular point requires reconsideration.

TJK/hl
c.c. T.J. Kovich,
Files.


T.J. Kovich,
Regional Soils Engineer.

Materials and Research Division

November 16, 1962

Geoson, Limited,
Consulting Engineers,
14 Haas Road,
Rexdale, Ontario.

Attention: Mr. F. McFarlane

Re: W.F. 59-60, Hwy. 69, C.P.R. Subway at Bala; ✓
W.F. 187-61, Hwy. 69, Moon River at Bala. ✓
District No. 11.

Dear Sir:-

Please consider this your authority to carry out foundation investigations at the above sites. Plans and profiles were provided to your representative on November 14, 1962.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Fourteen copies of each completed foundation report, plus one additional copy of the subsill profile, should be submitted to the Foundation Section as soon as possible. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Charges for the work performed will be in accordance with your schedule of Rates, dated October, 1960, and invoices to be addressed to the attention of the undersigned.

Note:- As Toronto is the nearest recognized mobilization point, payment for mobilization will be from there, as discussed with your representative.

RDJ/Adaf

* Yours very truly,

cc: Messrs. S. McCampie
H. McArthur
E. C. Bannier
T. J. Kovach
W. D. Smith (2)

Mrs. T. Tate
Foundations Office
Gen. Files (2)

J. Butts,
MATERIALS ENGINEER

GEOCON LTD

HEAD OFFICE

180 VALLÉE ST., MONTREAL 18, QUEBEC

TELEPHONE UN. 6-7632

Rexdale, Ontario,
January 2nd, 1963.

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. 244-6476

1425 WEST PENDER ST.
VANCOUVER 5, B.C.
TEL. MU. 1-8926

Department of Highways, Ontario,
Materials and Research Section,
Downsview, Ontario.

Attention: Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer.

Re: Soil Conditions and Foundations,
Proposed Moon River Bridge
and C.P.R. Subway,
Bala, Ontario,
W.P. 59-60, W.P. 187-61.

Dear Sirs:

This letter accompanies our engineering report on the above soil investigation.

We find that the soil conditions at the locations of the three principal structures covered by this study consist generally of a shallow thickness of granular overburden, then gneiss bedrock.

The proposed grade separation structure and the Moon River Bridge may be founded directly on bedrock. Allowable bearing values and special provisions which are required, are given in the report. The possible effect of a prominent set of joints in the bedrock on the selection of side slopes of the cut at the proposed grade separation is discussed. At the proposed causeway location the foundation conditions are generally suitable for dumped rock fill type construction, although in one area some dredging or blasting of clay overlying bedrock will be required as discussed.

We believe that our report details the information on the soil and rock conditions required from this investigation. Should you require further information, or if we can be of assistance in the application of the information to design, we would be very pleased if you would give us a call.

Yours very truly,

GEOCON LTD

M. A. J. Matich per F. J. H.
M. A. J. Matich, P. Eng.,
Vice-President and Chief Engineer.

MAJM/dw
57447

ST. JOHN'S

HALIFAX

MONTREAL

TORONTO

VANCOUVER

S7447
REPORT
TO
DEPARTMENT OF HIGHWAYS, ONTARIO
ON
SOIL CONDITIONS AND FOUNDATIONS
PROPOSED MOON RIVER BRIDGE AND C.P.R. SUBWAY
BALA ONTARIO

W.P. 59-60

W.P. 187-61

Distribution:

- 14 copies - Department of Highways, Ontario,
Downsview, Ontario.
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INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario by letter dated November 16th, 1962 to investigate the subsurface conditions at the sites of a proposed bridge and causeway across Moon River and a subway below the Canadian Pacific Railway tracks at Bala, Ontario. These structures are part of a proposed relocation of Highway #69 at this location.

The purpose of the investigation was to determine the depth of overburden and the engineering properties of the bedrock necessary for the design of the bridge abutments for the Moon River Bridge and the Canadian Pacific Railway Subway. The soil conditions along the causeway route and the resulting stability of the causeway fill was also to be investigated. This report contains all the factual information obtained, together with recommendations for design.

SUMMARIZED SOIL AND ROCK CONDITIONS

At the site of the proposed grade separation, there is a layer of sand and gravel fill about 2 to 3 feet in thickness. Underlying the fill to the depth investigated is biotite and/or syenite gneiss bedrock which is generally in sound condition. The rock strikes nearly perpendicular to the direction of the railroad tracks and dips from 30 to 60 degrees to the north. The river bottom at the south abutment of the proposed Moon River Bridge is underlain by about 6 feet of boulders and sand followed by from 1 to 5 feet of fine sand with some boulders. The rock at the north abutment location is overlain by 5 to 6 feet of sand with some gravel. Along the proposed causeway section, rock occurs directly at the river bottom for part of the length while part is underlain by up to 7 feet of very loose

wood chips and sand overlying about 4 feet of very soft varved silty clay, then bedrock. The bedrock at the causeway and bridge locations is also a relatively sound biotite granite and/or syenite gneiss which dips at about 50 to 70 degrees towards the north.

DISCUSSION

It is understood that the proposed relocation of Highway #69 at Bala, Ontario in the area covered by this investigation will involve three major structures. These are a grade separation structure at the Canadian Pacific Railway tracks, a rock fill causeway across the south arm of Moon River and a bridge connecting the northern extremity of the causeway to the island which divides the river at this point. The plan locations of the proposed structures are shown on your Drawing SK61-151-1 which has been used as a basis for our boring Plan and Soil Stratigraphy Drawing S7447-1, given at the rear of this report. For convenience of presentation, the above three structures are discussed separately below:

1. Proposed Grade Separation Structure

It is proposed to carry the new Highway #69 under the existing Canadian Pacific Railway tracks at the site of an existing level crossing which is about 100 feet from the present Highway #69 as shown. This will involve carrying the railway tracks on a bridge and excavating about 25 feet in rock to the proposed profile shown on Drawing S7447-1. The main purpose of the investigation of the structure location was to examine the rock conditions as they affect the stability of the sides of the cut and the founding of the bridge abutments.

1. Proposed Grade Separation Structure (continued)

The bedrock at the site is generally a biotite granite gneiss. Outcrops occur in three locations within a radius of about 100 feet from the proposed bridge. An examination of these outcrops shows that the predominant jointing system is oriented in an east-west direction as shown on Drawing S7447-1. The dip measured in the outcrops is about 50 degrees to the horizontal and towards the north. In the cores obtained at boreholes 1 and 2, the jointing system was found to be inclined at angles varying from 35 to 60 degrees. It is inferred that the direction of the inclination of the joints is the same as the dip observed at the outcrops, in other words, downwards towards the north. The above jointing pattern in the bedrock is an important consideration in the design of both the cut slopes on the south side of the highway and the south abutment if the latter is carried on the bedrock above final highway elevation.

Since the predominant set of joints is at an average of about 50 degrees to the horizontal, it is recommended that in the vicinity of the abutments to the proposed overpass bridge, the sides of the rock cut be sloped back to at least 1 vertical to 1 horizontal. The estimated angle of friction, rock to rock, is 35 degrees. In actual fact, the effective overall angle of friction is greater than this value due to such factors as the interlocking effect of non-continuous joints, and end-effects. It is not possible to calculate what additional stability is provided by the above factors and it is recommended that these beneficial effects be considered as a factor of safety and that 35 degrees be used in design.

1. Proposed Grade Separation Structure (continued)

Foundations carried on the sound bedrock below final roadway elevation may be designed for an allowable bearing value of 20.0 tons per square foot. The same bearing value may be used for footings carried on the sound rock within the slopes providing that requirements with regard to slope stability be satisfied. In this case, the stability of a wedge of rock beneath the abutment or pier foundation should be checked for stability under the worst combination of vertical and horizontal forces as imposed by the bridge structure and the train and fill loadings. If only vertical loads were involved for example, this requirement would entail that the complete foundation carried on the slope be located below a line drawn at 35 degrees to the horizontal starting at the toe of the slope. The bedrock beneath the foundations carried in the slope could also be suitably reinforced with steel anchor rods or the horizontal loads could be resisted entirely by inclined tie rods anchored into the bedrock, instead of being resisted by rock to rock friction. It is recommended that the above requirements with regard to cut slopes and foundations be applied to both sides of the bridge.

The backfill behind the abutments should be composed of clean well compacted non-frost susceptible granular material. The horizontal force exerted by the fill on the abutment should be assumed to be the at-rest earth pressure, and a coefficient of earth pressure of 0.4 may be used. Adequate drainage should be provided for the backfill behind the abutments, to avoid the build-up of hydrostatic pressures.

Care should be exercised in the blasting of the rock at the proposed bridge location in order to avoid loosening of

1. Proposed Grade Separation Structure (continued)

the sides of the cut or shattering below proposed footing locations, thus reducing the carrying capacity of the rock. It is recommended that the final cut in the vicinity of the bridge structure, and footing excavations be examined prior to construction of the footings.

On either side away from the bridge structure, the sides of the rock cut could be made as steep as possible consistent with maintaining safety against slides or falling rocks.

2. Causeway and Moon River Bridge

a) Causeway

It is understood that it is proposed to use excavated rock obtained from adjacent road cuts, as the fill for the causeway. For the most part, the line of the causeway is underlain by either very loose wood chips and sand, or by bedrock directly. In these locations, it is believed that the rock fill will assume side slopes of about 1-1/4 horizontal to 1 vertical, and that they will be stable. In the vicinity of borehole 8, however, very soft varved sensitive clay underlies the wood chips and sand layer. Based on the shear strength values of 120 to 220 pounds per square foot, the proposed causeway with roadway elevation of 730 at this location, would be unstable at side slopes of 1-1/4 to 1 that would form naturally during dumping. Special measures will therefore be required in this locality. It is suggested that the clay, and overlying wood chips and sand therefore be either removed by dredging prior to dumping of the rock fill or that the clay be thoroughly remoulded by blasting prior to filling to ensure complete displacement of the clay under the weight of the rock

2. Causeway and Moon River Bridge (continued)

a) Causeway (continued)

fill. For this purpose, the nose of the rock fill should be pointed with an angle between the faces of about 90 degrees. Dumping should also be carried out as rapidly as possible after blasting to take advantage of the remoulding effect of the blasting. The necessary stability for the causeway cross-section at this location could also be achieved by the provision of stabilizing berms. However, this possibility has not been considered in detail further, because of the practical difficulty of constructing the required berms which would have to be placed underwater in advance of the main body of the causeway.

The granite gneiss is a hard and durable rock, and although no freeze-thaw or soundness tests have been carried out, it is believed that the rock would be suitable for use in causeway construction from the point of view of weathering.

Although the design of the causeway is beyond our present terms of reference, it is pointed out that selection of stone sizes may be necessary for purposes such as; protection of the sides against wave and ice action, protection of the nose against scour by high river currents should these occur during construction, and the forming of the junction between the causeway and the Moon River Bridge.

2. Causeway and Moon River Bridge (continued)

b) Moon River Bridge

From available information, this structure will be a single span skew bridge with an 80 foot minimum span. The north abutment will be founded on the island and the south abutment will join the bridge to the causeway.

At the location of the north abutment, bedrock was encountered at a depth of about 6 feet below present ground surface at boreholes 3 and 4, that is, at about elevation 725. At borehole 3A, put down about 3 feet from borehole 3, the bedrock surface was at about elevation 727 indicating that the bedrock surface is quite variable at this abutment location. In boreholes 3 and 3A, at about elevation 715, a 4 inch wide seam was encountered which appeared to be partially open and contain fines and weathered rock. The drill water was lost into the seam and during drilling borehole 3A, the wash water returned through the previously drilled borehole 3. Except in the immediate vicinity of the seam, the bedrock in the boreholes above and below the seam appeared to be sound.

The north abutment could be founded directly on the sound bedrock for which an allowable bearing value of 20.0 tons per square foot may be used. If the abutment is founded above elevation 715, it is recommended that the seam encountered in the rock be washed out over the plan area of the abutment and filled with cement grout prior to construction of the abutment.

2. Causeway and Moon River Bridge (continued)

b) Moon River Bridge

For the roadway elevation as proposed, the abutment, if of the solid type, would retain about 12 feet of fill above present ground level. It is recommended that this backfill be clean non-frost susceptible granular material as in the case of the abutments for the grade separation structure at the Canadian Pacific Railway. As before, a lateral earth pressure coefficient of 0.4, and adequate drainage of the backfill is recommended.

The soil conditions at the south abutment to the bridge, as represented by those encountered at boreholes 5 and 6, consist of an irregular thickness of overburden of boulders and sand. There is a five foot difference in the elevation of sound bedrock between boreholes 5 and 6, and it is expected that the bedrock surface at this abutment location would also be irregular between the boreholes, as at the north abutment.

Because of irregular and probably loose nature of the boulders and sand stratum and the susceptibility of the overburden as a whole to scour, particularly the sand, the overburden is not considered a suitable foundation stratum for the south abutment. It is therefore recommended that the abutment be founded on the sound bedrock for which an allowable bearing value of 20.0 tons per square foot may be used. As an alternative, the abutment could be carried on drilled caissons socketed into the bedrock. The choice of foundations is dependent on economic and other factors beyond the scope of this report.

2. Causeway and Moon River Bridge (continued)

b) Moon River Bridge

The details of the junction between the causeway and the Moon River Bridge will depend to a certain extent on the scheduling of the construction of the end of the causeway, and the bridge. Assuming that the abutment is built in advance of completion of the end of the causeway and that it is solid and therefore acts as a retaining wall, special measures would be advisable with regard to the backfilling procedures. These would particularly avoid the possible impact of large rocks tumbling down the front face of the causeway fill as this approaches the abutment at closure. A suggested method of providing protection for the abutment would be to incorporate a buffer zone of selected granular fill of limited size, immediately behind the abutment. This could be dumped adjacent to the abutment before completion of the causeway. A lateral earth pressure coefficient from the fill of 0.4 is recommended for design.

No consideration has been given in this report to such factors as ice pressure on the structure, and the effects of scour through the bridge opening once the river has been constricted by the causeway, since these are beyond our terms of reference. These factors would of course have to be taken into account in design.

CONCLUSIONS AND RECOMMENDATIONS

1. In general, at the three locations investigated there is a shallow thickness of granular overburden, then bedrock. The detailed soil and rock conditions are discussed in the report.

2. Because of the observed dip in the bedrock in the vicinity of the grade separation structure, special provisions are required in the design of the rock cut and the foundations for the structure as discussed in the report.

3. For stability of the causeway, special measures such as dredging or blasting will be required in a localized clay area, as discussed.

4. The abutments for the Moon River Bridge may be carried on bedrock at a recommended allowable bearing value of 20.0 tons per square foot. Special provisions such as grouting and backfilling procedures are recommended as given in the report.

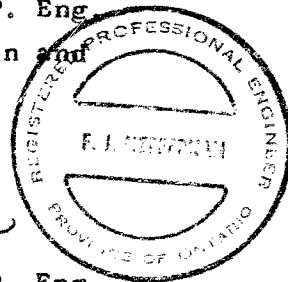
PERSONNEL

The field work for this investigation was carried out under the field supervision of Mr. H. W. Green, P. Eng. This report was written by Messrs. Green and Heffernan and reviewed by Mr. M.A.J. Matich.

FJH/dw
57447

F. J. Heffernan

F. J. Heffernan, P. Eng.



APPENDIX I

Procedure

Site and Geology

Soil and Rock Conditions

Water Conditions

Office Reports on Soil Exploration

PROCEDURE

The field work for this investigation was commenced on November 20th, 1962 and completed on December 13th, 1962. Nine boreholes were put down in BX and AX size using a skid-mounted machine drill rig. Two additional dynamic penetration tests were also put down along the route of the proposed embankment and one test pit and one penetration test were put down at the grade separation. Bedrock was cored in AXT size in all of the boreholes for depths ranging from 11 to 28 feet. Detailed logs of the borings are given on the Office Reports on Soil Exploration in this Appendix. The location of the boreholes, penetration tests and the test pit are shown on Drawing S7447-1 in the pocket at the rear of this report.

The laboratory testing of the soil samples was carried out in the Soil Mechanics Laboratory of Geocon Ltd in Toronto. The results are plotted on the Office Reports in this Appendix. The soil samples remaining after testing will be stored until June 1st, 1963, at which time you will be contacted for instructions regarding their disposal.

All elevations given in this report are referred to Geodetic Datum. Two temporary bench marks were established in the area, one near the river bridge site and another at the railroad-road intersection at the proposed grade separation. At the river bridge site the bench mark consists of a spike driven into a large exposed root of a tree of which the elevation was given to us as 728.34. At the proposed grade separation, the bench mark consists of a spike driven into the planking between the rails and it has an elevation of 757.65. Each bench mark and its elevation was established by a Department of Highways, Ontario survey party.

SITE AND GEOLOGY

II.

The site is located within the Town of Bala, Ontario, as shown on Drawing S7447-1 and consists of the relocating of the existing road to a location west of the C.P.R. tracks. The proposed road is composed, in fact, of a grade separation beneath the C.P.R. railway, and a causeway embankment in the Moon River adjacent to the railway, connecting with a proposed river bridge.

Much rock is exposed in the site area. It is composed of biotite granitic or syenitic gneiss of generally good quality as viewed in the exposures. It is overlain by granular soils composed of fine to coarse grey-brown sand and medium gravel with boulders to about 1 foot in diameter.

Geological information regarding the bedrock and overburden in the Bala area is non-existent.

SOIL AND ROCK CONDITIONS

The principal soil strata encountered at the sites are as follows:

a) Proposed Grade Separation

Sand and Gravel Fill

A layer of grey-brown sand and gravel fill is contained within the railway and road embankments at the existing level crossing. From visual examination, the material appears to be a well graded sand and gravel with a maximum grain size of about 1-1/2 inches. The thickness as encountered in the boreholes, penetration test and test pit ranged from about 2 to 3 feet.

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a) Proposed Grade Separation (continued)

Sound Biotite Granitic and/or Syenitic Bedrock

At the proposed grade separation, bedrock is exposed or is covered with a thin mantle of granular soil. The rock is composed of biotite granite and/or syenite gneiss which strikes nearly perpendicular to the direction of the railroad tracks as shown on Drawing S7447-1. The rock dips to the north where exposed at about 50 degrees but is locally contorted both in strike and dip along the northerly corner of the exposure adjacent to the existing Highway 69.

Particular attention was given to the dip of the rock at depth. Holes were drilled to below grade to about elevation 726 or about 30 foot depth. The dip in this vertical length varied from about 30 to 60 degrees.

The rock both in surface exposure and at depth appeared to be generally in sound condition. In borehole 2, the uppermost 18 inches was inclined to be seamy and slightly weathered. This uppermost section may be a boulder or weathered bedrock and has been classified as such on the Office Report and on Drawing S7447-1.

b) Proposed River Bridge and Embankment Site

Fine to Coarse Sand with Some Gravel

Above the water level at the north abutment of the bridge, a layer of fine to coarse sand with some gravel and occasional boulders up to 1 foot in thickness exists. The thickness of the layer as encountered in the boreholes

b) Proposed River Bridge and Embankment Site (continued)

Fine to Coarse Sand with Some Gravel (continued)

ranged from 5.3 to 6.4 feet.

Boulders and Sand

At the south abutment, the river bottom is underlain by boulders and sand. The upper six feet of this stratum which was diamond drilled throughout, appears to consist of boulders with sand in the voids, while below this, the material consists of fine grey sand with some boulders. The thickness of the sand section appears to range from about 1.0 foot at borehole 5 to about 5.0 feet at borehole 6.

Very Loose Wood Chips and Sand

A very loose wood chip and sand deposit underlies the river bottom in the causeway area. This wood chip and sand layer offered practically no penetration resistance whatsoever during sampling and representative samples were difficult to recover. The thickness of the material ranges up to 7 feet as encountered in borehole 8 and as inferred in penetration test 3.

Very Soft Grey Varved Silty Clay

Underlying the wood chip and sand layer is a stratum of medium grey varved silty clay. The thickness of the clay is inferred to range from 0 to about 4 feet below the causeway area.

b) Proposed River Bridge and Embankment Site (continued)

Very Soft Grey Varved Silty Clay (continued)

Atterberg limits were carried out on one sample of the clay and gave a value of 53 for the liquid limit and 21 for the plastic limit. The corresponding natural moisture content ranged from 79 to 81 percent. This high moisture content is in keeping with the very soft consistency and sensitive nature of the clay.

During sampling, the sampling tube sank readily into this material under the weight of rods alone for about half of the tube length. The remainder of the tube was pressed easily into the clay.

Three undrained triaxial tests gave compressive strength values ranging from 0.12 to 0.22 tons per square foot with an average of 0.18 tons per square foot. Based on these values, and on the slight observed resistance to penetration, the consistency of the clay has been estimated to be very soft.

This material, where it exists overlies the bedrock surface directly in the causeway area.

Sound Biotite Granite and/or Syenite Gneiss

Much bedrock is exposed at the river bridge site as shown on Drawing S7447-1. The rock is composed of biotite granite and/or syenite gneiss. The strike of the exposed rock is east-west and the rock dips to the north at about 50 to 70 degrees in general. Along the shoreline to the

b) Proposed River Bridge and Embankment Site (continued)

Sound Biotite Granite and/or Syenite Gneiss (continued)

west of the proposed structure an area of flat shelving of the rock may be observed.

Both corner locations of the land abutment were drilled to depths of about 25 feet. In borehole 3, an open seam in the rock was encountered at about elevation 715 and was confirmed in the adjacent borehole 3A located 3 feet away. The seam appears to be from 2 to 4 inches in thickness and contains open portions as the drill rods were wrenched down by hand about 1-1/2 inches in borehole 3A. During drilling, the rock is easily ground in this seam. Pumping tests were conducted and muddy water was continuously pumped out at about 100 gallons per hour. Furthermore, observation on the drop of the water level in the borehole after filling, suggests that the seam is connected with the river. When drilling in borehole 3A, the return water also flowed out of borehole 3. In borehole 4, the seam as such was encountered at the same approximate elevation. However, it consisted only of weathered core surfaces over a thickness of about 5 feet between elevations 710 and 715. At the south abutment of the river bridge the bedrock is of sound condition and does not contain any open seams.

The rock underlying the causeway embankment is composed of biotite granite and/or syenite gneiss as before and is generally of sound condition. At least 10 feet of core were recovered from each hole and core recovery was high.

WATER CONDITIONS

VII.

At the time of the investigation the level of Moon River was at about elevation 721. The reported high water level of Moon River at the site is 726.3. No water level was encountered in the permeable overburden in the land holes.

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57447 BORING # 1 DATUM GEODETIC CASING AX
 BORING DATE NOV 20, 1962 REPORT DATE DEC 6, 1962 COMPILED BY A.E.L. CHECKED BY F.J.H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W% ○ NAT. □ LW △ PW		OTHER TESTS	SAMPLES			
								CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
755.9 8.0		GROUND LEVEL									
753.6 2.3		LOOSE BROWN SAND AND GRAVEL FILL		755							
		57° DIP @ 4.5' DEPTH									
		50° DIP @ 9.5' DEPTH									
		55° DIP @ 10.5' DEPTH		750							
		SOUND									
		MOTTLED LIGHT GREY AND BLACK BIOTITE GRANITE GNEISS BEDROCK		745							
		GARNETIFEROUS FROM 2.5' - 9.0		740							
		40° DIP @ 20.5' DEPTH									
		45° DIP @ 20-23' DEPTH									
		35° DIP @ 24.0' DEPTH		730							
		50° DIP @ 27-28' DEPTH									
725.9 30.0		END OF HOLE		725							

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57447 BORING # 2 DATUM GEODETIC CASING
 BORING DATE NOV. 22, 1964 REPORT DATE DEC. 16, 1964 COMPILED BY AEL CHECKED BY F. J. H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

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 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

SOIL PROFILE										SAMPLES				
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE						OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
					WATER CONTENT W% O NAT. <input type="checkbox"/> LW <input type="checkbox"/> PW <input type="checkbox"/>									
					DYNAMIC PENETRATION TEST BLOWS PER FOOT									
757.4 0.0		GROUND LEVEL LOOSE		760										
754.2 3.2		GREY BROWN SAND AND GRAVEL FILL		755										
752.7 4.7		WEATHERED BEDROCK OR BOULDERS		750	BROKEN SEAMY BEDROCK WEATHERED CORE ENDS 2'-4" CORE LENGTHS					RC REC'D 100%		AXT RC	1	
				740	BUTTE RICH BANDS SPACED 1/2"-1" APART 6"-1' CORE LENGTHS								3	
		SOUND PINK GRANITIC AND / OR GNEISS BEDROCK		745									4	
		40° DIP 8'-12' DEPTH 45° DIP 12'-14' DEPTH 60° DIP 16'-18' DEPTH 50° DIP 18'-22' DEPTH		740	6"-1' CORE LENGTHS									
				730									5	
728.6 48.5		END OF HOLE		730										

OFFICE REPORT ON SOIL EXPLORATION

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 62407 BORING # 3 DATUM GEODETIC CASING AX
BORING DATE NOV 27, 1964 REPORT DATE DEC 10, 1964 COMPILED BY AEL CHECKED BY F. J. A
SAMPLER HAMMER WT. 140 LBS. DROP 40 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. - LBS. ENERGY)

SAMPLE CONDITION



SAMPLE TYPES

A.S. - AUGER SAMPLE	F.S. - FOIL SAMPLE
S.T. - SLOTTED TUBE	S.O. - SLEEVE-OPEN
W.S. - WASHED SAMPLE	S.F. - SLEEVE-FOOT VALVE
D.O. - DRIVE-OPEN	T.O. - THIN WALLED OPEN
D.F. - DRIVE-FOOT VALVE	R.C. - ROCK CORE
C.S. - CHUNK SAMPLE	

ABBREVIATIONS

V	- IN-SITU VANE TEST	γ	- WET UNIT WEIGHT
M	- MECHANICAL ANALYSIS	K	- PERMEABILITY
U	- UNCONFINED COMPRESSION	C	- CONSOLIDATION
QC	- TRIAXIAL CONSOLIDATED QUICK		
Q	- TRIAXIAL QUICK	WL	- WATER LEVEL IN CASIN
S	- TRIAXIAL SLOW	WT	- WATER TABLE IN SOIL

SOIL PROFILE

SOIL PROFILE															SAMPLES				
ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W% O NAT. □ LW △ Pw										OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
					DYNAMIC PENETRATION TEST BLOWS PER FOOT														
731.4 6.0		GROUND LEVEL			BT * 3														
725.5 5.9 724.4 7.0		LOOSE BROWN AND GREY FINE TO MEDIUM SAND WITH SOME GRAVEL AND OCCASIONAL BOULDERS		730															
		WEATHERED BEDROCK OR BOULDERS		725															
				720	8' CORE LENGTHS														
		SOUND GARNETIFEROUS BIOTITE GRANITE GNEISS BEDROCK POSSIBLE OPEN SEAM AT 16'-2" OF ABOUT 4" THICKNESS.		715															
				710	3' CORE LENGTHS 2' CORE LENGTHS 0'3" 4"														
		70° DIP ± 7-15° DEPTH 52° DIP ± 22-30° DEPTH 50° DIP ± 37-50° DEPTH		705															
701.1 30.3		END OF HOLE		700															

OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

SAMPLE TYPES

ABBREVIATIONS



DISTURBED
FAIR
GOOD
LOST

A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
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QC - TRIAXIAL CONSOLIDATED QUICK
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γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION

WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE								SAMPLES				
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W%			OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
					O NAT. □ LW △ Pw							
					DYNAMIC PENETRATION TEST BLOWS PER FOOT							
732.0 0.0		GROUND LEVEL			BH*3A							
		LOOSE GREY BROWN FINE TO MEDIUM SAND WITH SOME GRAVEL AND OCCASIONAL BOULDERS.		730								
726.8 5.2				725								
		SOUND BIOTITE GRANITE GNEISS BEDROCK		720								
		PARTLY GRAPHITIFEROUS POSSIBLE OPEN SEAM AT 17.2' DEPTH OF ABOUT 4" THICKNESS		715								
		40° DIP @ 6' DEPTH										
		33° DIP @ 13' DEPTH										
		42° DIP @ 16' DEPTH		710								
		58° DIP @ 24' DEPTH										
706.8 25.2		END OF HOLE		705								

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57447 BORING # 4 DATUM GEODETIC CASING A X
 BORING DATE NOV. 30, 1962 REPORT DATE DEC. 6, 1962 COMPILED BY A.E.L. CHECKED BY F.J.H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
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 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES

LOG FROM HOLE					SAMPLES							
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W%			OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
					DYNAMIC PENETRATION TEST BLOWS PER FOOT							
					BH # 4							
				735								
731.3 0.0		GROUND LEVEL										
		LOOSE GREY-BROWN FINE TO COARSE SAND WITH SOME GRAVEL AND OCCASIONAL BOULDERS		730								
724.3 6.4				725								
				720	1'-2" CORE LENGTHS POSSIBLE SEAMY ROCK 6'-26" CORE LENGTHS							
				715	RECEMENTED FRACTURE (QUARTZ) DIPPING @ 28°							
		SOUND BIOTITE GRANITE GNEISS BEDROCK POSSIBLE SMALL SEAMS 16.9 - 21.1		710								
				705	WEATHERED FRACTURE @ 16' 11" @ 17' 9" (80° DIP) @ 18' 0" (50° DIP) @ 18' 5" (HORIZONTAL) @ 19' 4" (47° DIP) @ 19' 8" (50° DIP) @ 20' 5" (HORIZONTAL) @ 21' 2" (HORIZONTAL)							
		57° DIP @ 11' DEPTH 50° DIP @ 18' DEPTH 52° DIP @ 28' DEPTH 45° DIP @ 29' DEPTH		700	17° 21' DEPTH							
				695	NO WEATHERED SECTIONS BELOW 21' 2" DEPTH							
				690								
				685								
				680								
				675								
				670								
				665								
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				35								
				30								
				25								
				20								
				15								
				10								
				5								
				0								

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57447 BORING # 5 DATUM GEODETIC CASING AX
 BORING DATE DEC 7, 1962 REPORT DATE DEC 4, 1962 COMPILED BY AEL CHECKED BY F.J.H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W%			OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS, F.T.
					O NAT. □ LW ▲ Pw							
					DYNAMIC PENETRATION TEST ELOWS PER FOOT							
					BH # 5							
721.0 0.0		WATER LEVEL		725								
				720								
715.6 5.4		RIVER BOTTOM		715								
		BOULDERS WITH FINE GREY SAND		710					AXT RC		1	
708.8 12.2				705							2	
				700							3	
				705							4	
		SOUND BIOTITE GRANITE GNEISS BEDROCK		700							5	
		12° DIP - 17.6 DEPTH 67° DIP - 26.0 DEPTH		695							6	
				690							7	
				685							8	
				680							9	
				675							10	
689.5 31.5		END OF HOLE		670							11	

CORE LENGTHS GENERALLY 6" - 2' THROUGHOUT.
 NO WEATHERED FRACTURES.
 NO INDICATIONS OF SLAMS.

RC RECV 77%

RC RECV 100%

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57447 BORING # 6 DATUM GEODETIC CASING AX
 BORING DATE DEC. 5, 1962 REPORT DATE DEC. 17, 1962 COMPILED BY A.E.L. CHECKED BY F.J.H.
 SAMPLER HAMMER WT 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. - LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

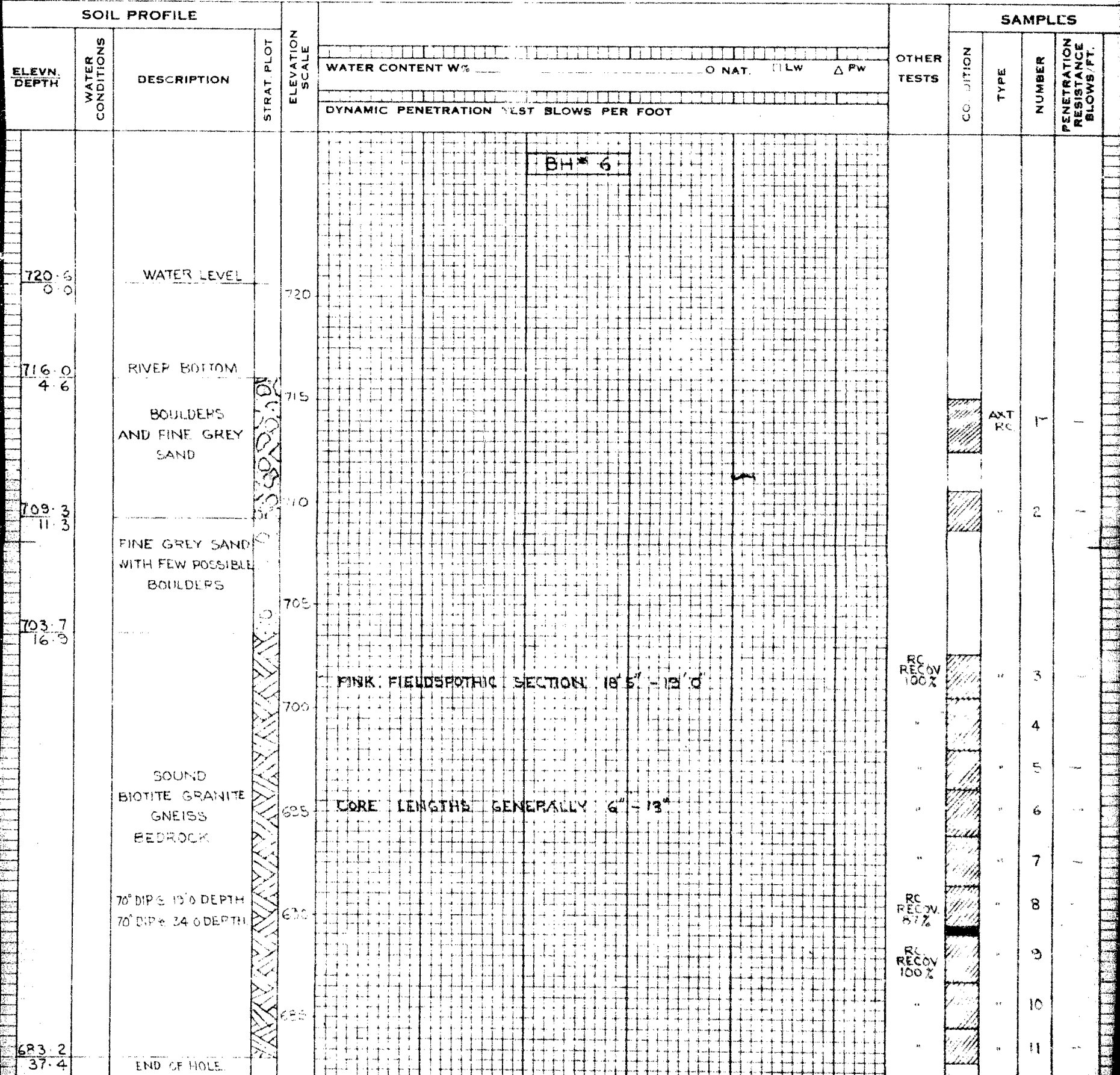
SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 Qc - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE



OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION


 DISTURBED
 FAIR
 GOOD
 LOST

SAMPLE TYPES

A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
QC - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW

γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

[illegible]

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT \$ 7447 BORING # 8 DATUM GEODETIC CASING AX.
BORING DATE DEC. 7, 1962 REPORT DATE DEC. 17, 1962 COMPILED BY A.E.L. CHECKED BY F.J.H.
SAMPLER HAMMER WT 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
QC - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE

COMPRESSION STRENGTH TONS / SQ. FT.

UNCONFINED COMPRESSION TEST

0.1 0.2 0.3 0.4 0.5

WATER CONTENT W% 0 NAT. □ LW Δ PW

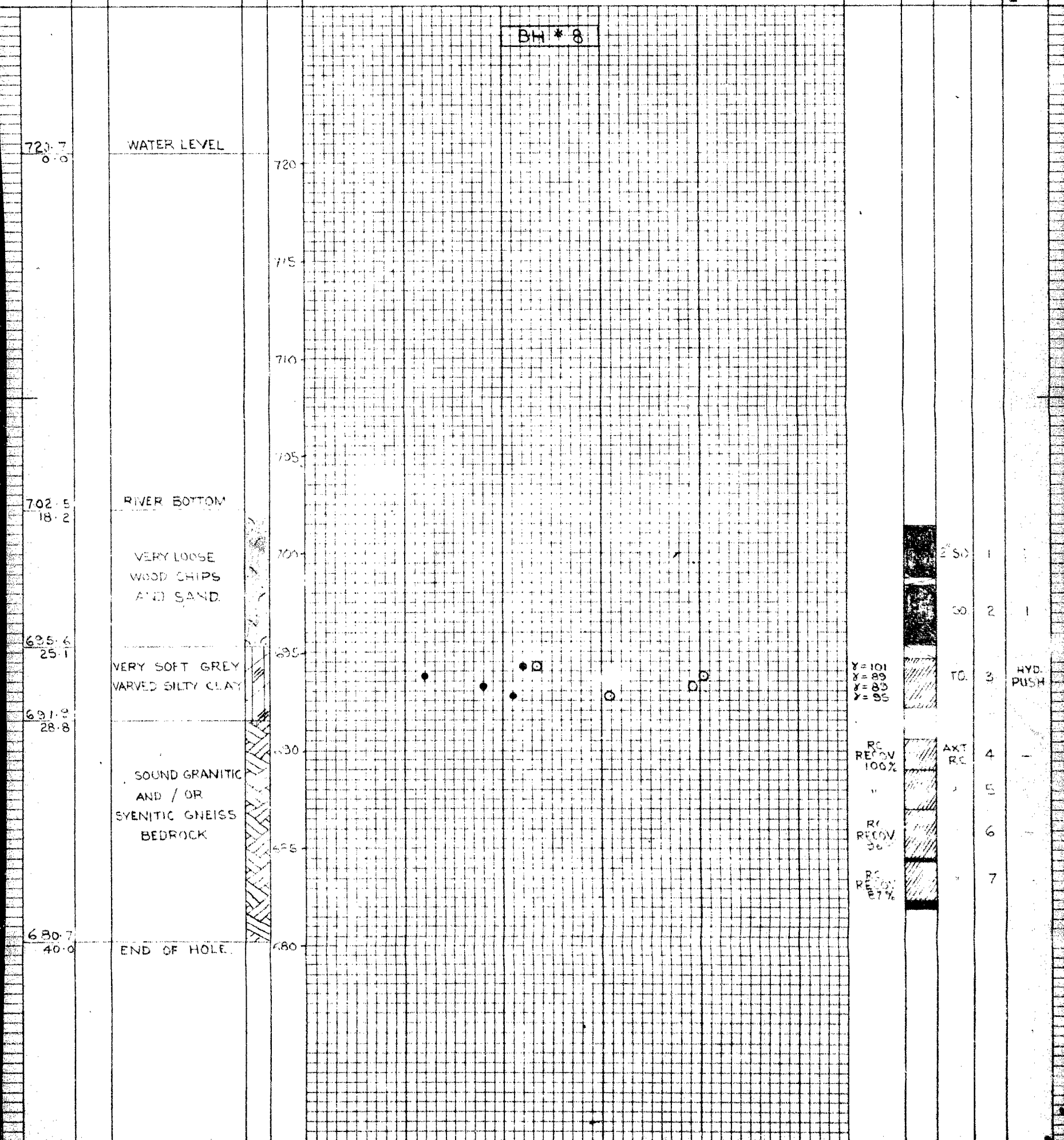
20 40 60 80 100

DYNAMIC PENETRATION TEST BLOWS PER FOOT

SAMPLES

CONDITION
TYPE
NUMBER
PENETRATION
RESISTANCE
BLOWS/FT.

OTHER
TESTS



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7447 PEN TEST # 1 AND TEST PIT # 1 DATUM GEODETIC CASING
 BORING DATE NOV 23, 1962 REPORT DATE DEC. 20, 1962 COMPILED BY AEL CHECKED BY F. J. H.
 SAMPLER HAMMER WT. LBS. DROP INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST γ - WET UNIT WEIGHT
 M - MECHANICAL ANALYSIS K - PERMEABILITY
 U - UNCONFINED COMPRESSION C - CONSOLIDATION
 Qc - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE				ELEVATION SCALE	OTHER TESTS				SAMPLES			
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT						CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
					WATER CONTENT W% O NAT. <input type="checkbox"/> LW <input type="checkbox"/> Pw <input type="checkbox"/>							
					DYNAMIC PENETRATION TEST BLOWS PER FOOT							
756.0 0.0		GROUND LEVEL		755	PT #1							
753.7 2.3		PROBABLY SAND AND GRAVEL										
		REFUSAL										
				750								
					TP #1							
757.0 0.0		GROUND LEVEL		760								
755.3 1.7		GREY BROWN SAND AND GRAVEL FILL		755								
		BEDROCK SURFACE END OF TEST PIT										

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7447 PEN. TEST #2 DATUM GEODETIC CASING BX.
 BORING DATE DEC. 10, 1962 REPORT DATE DEC. 31, 1962 COMPILED BY AEL. CHECKED BY F. J. H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



SAMPLE TYPES

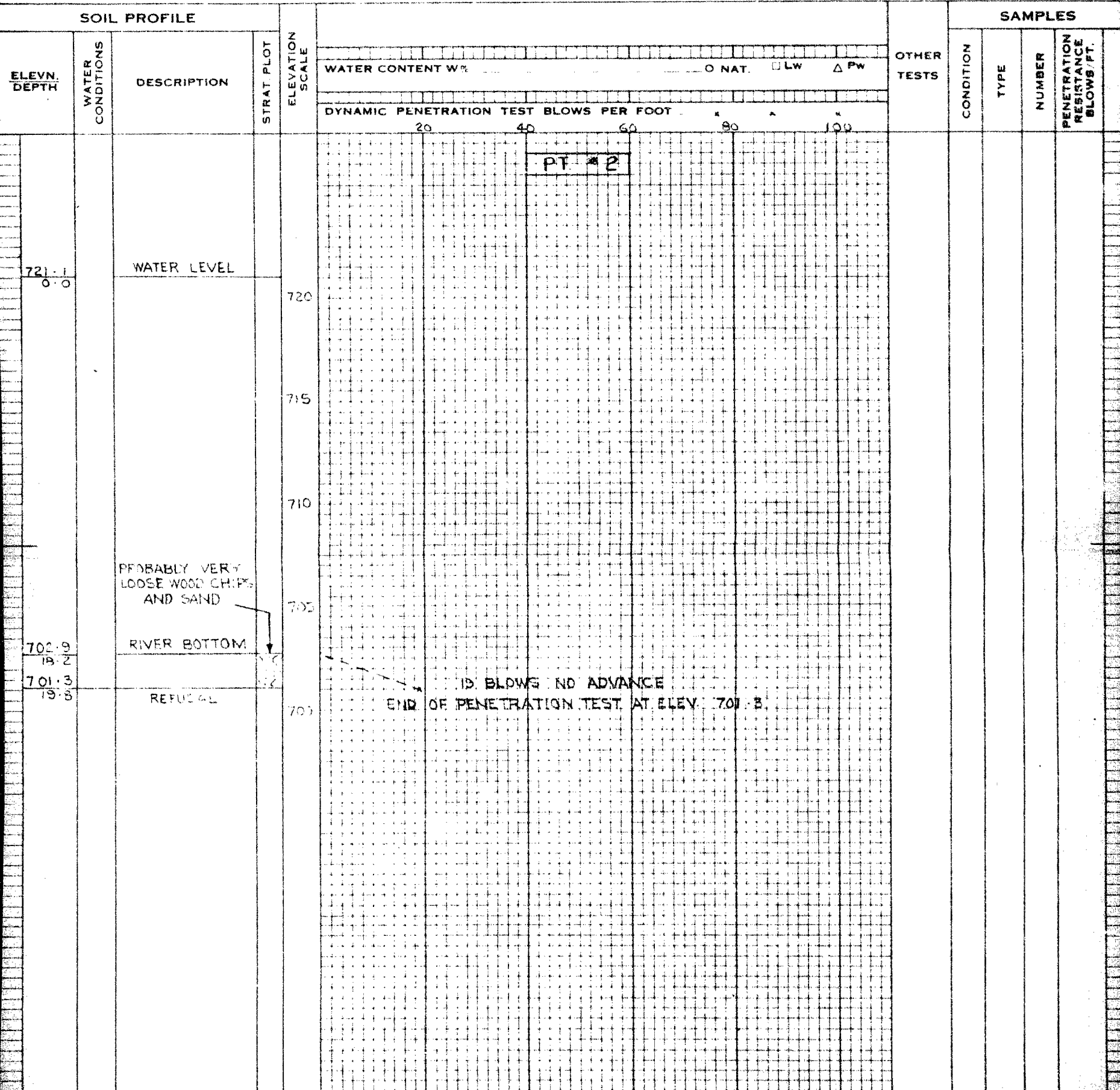
A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

SAMPLES



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57447 PEN. TEST * 3 DATUM GEODETIC CASING -
 BORING DATE DEC. 10, 1962 REPORT DATE DEC. 20, 1962 COMPILED BY AEL CHECKED BY F. J. H.
 SAMPLER HAMMER WT. LBS DROP INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

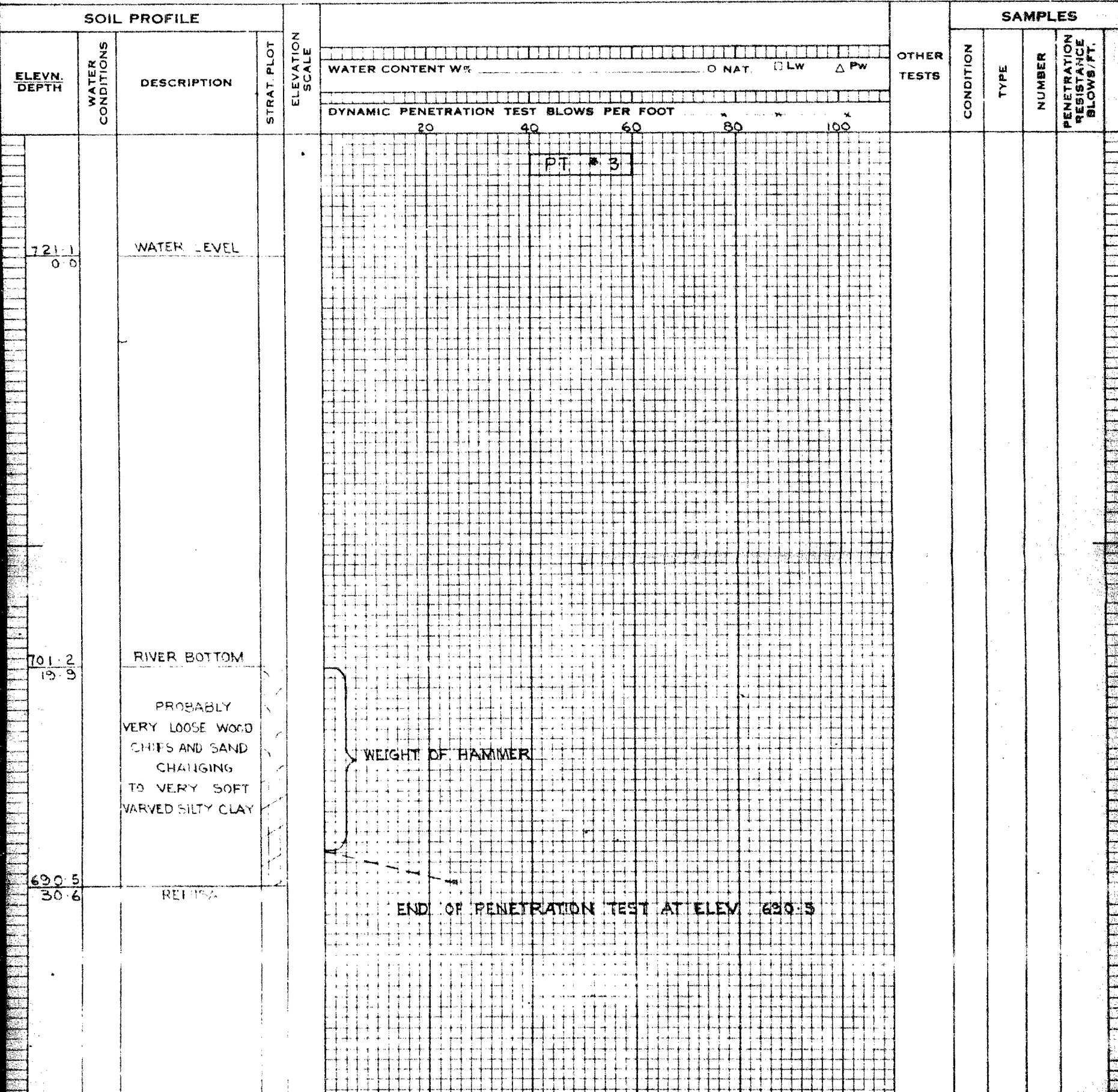
SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE



Department of Highways Ontario

Copy for the information of

Mr. A. Stermac, Principal Foundation Engineer,
Room 107, Lab. Bldg.

Mr. E. H. Jones,
District Engineer,
HUNTSVILLE, Ontario.

Bridge Division,
Downsview, Ontario.

Attention: **Mr. M. J. Bernhardt,** **October 23, 1964.**
District Construction Engineer.

Contract 64-210
Moon River Bridge at Bala
Hwy. 69 - District 11

WP 117-21

This letter will confirm our telephone conversation this morning.

We feel there is no need to grout the seams encountered in the rock during the foundation investigation. This seam is from 2 to 4 inches in thickness and is 10 feet below the bottom of the footing. The expense of properly grouting it can not be justified since the structure can readily withstand the settlement expected if a failure in the rock did occur.

We would like to inspect the footing excavation before any concrete is placed, and will be visiting the site next Tuesday.

AEMck/lm
cc. B. R. Davis
A. Stermac

A. E. McKim
A. E. McKim,
Bridge Control Engineer.