

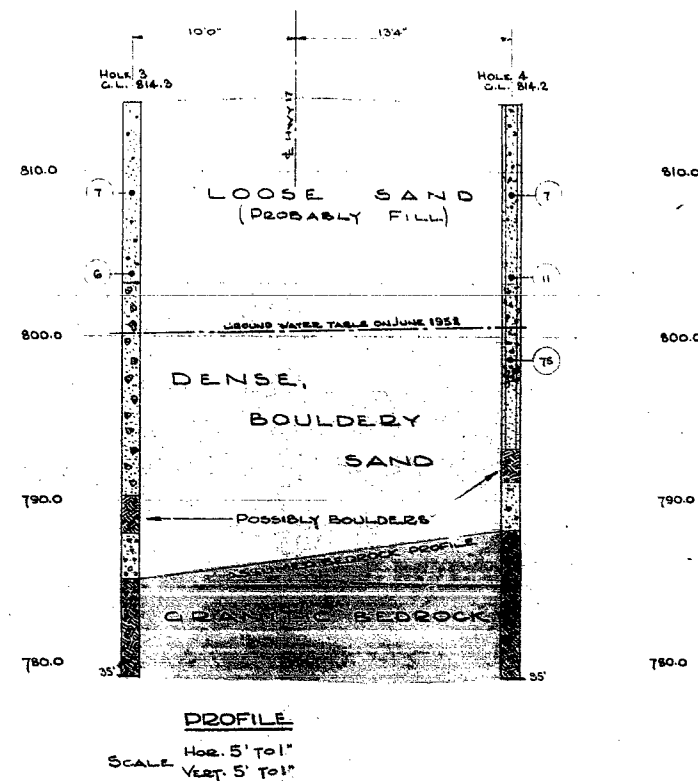
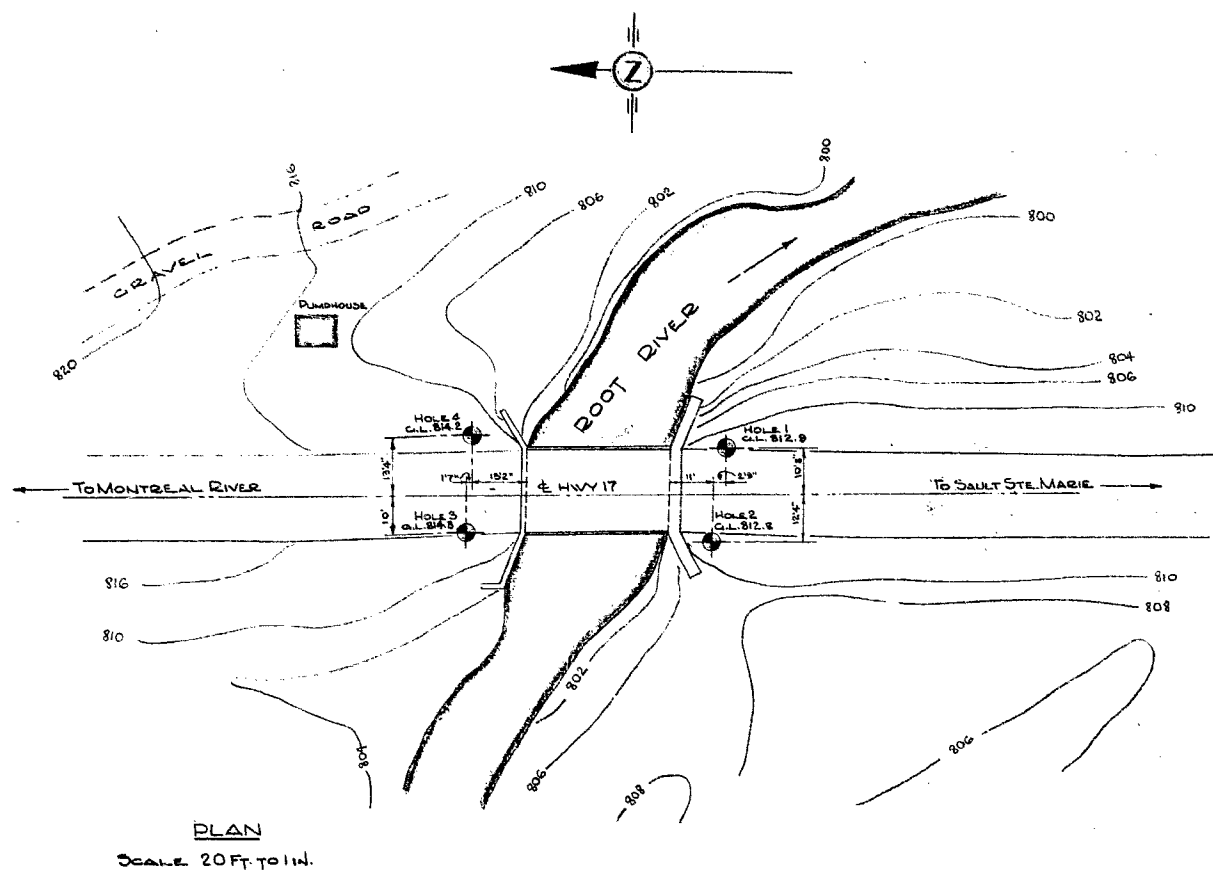
58-F-282C

W.P. 908-58

Hwy. #17

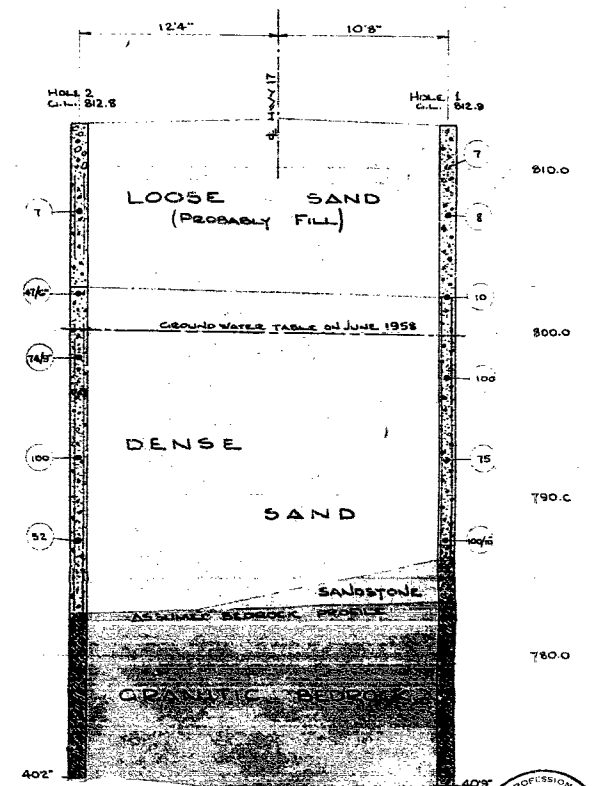
ROOT RIVER

#1 CROSSING



- LEGEND**
- TEST HOLE
 - ⊖ BLOWS/FOOT

NOTE:
THE SOIL BOUNDARIES HAVE BEEN ESTABLISHED AT THE BOREHOLE LOCATIONS ONLY. BETWEEN BOREHOLES THE SOIL STRATIGRAPHY HAS BEEN INFERRED.



e.m. peto & associates ltd.	
SOIL SITE INVESTIGATION AT HWY-17 ROOT RIVER BRIDGE #1 FOR DEPARTMENT OF HIGHWAYS OF ONTARIO	
OUR JOB No. 5867	DATE June 24-58
CLIENTS PLAN No.	PER. G.T.

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:-

5807

**850 roselawn avenue,
TORONTO, ONTARIO.
RUssell 1 - 4955.**

26th June, 1958.

58-F-282C

41 K/9 W

**Department of Highways of Ontario,
Bridge Office,
280, Davenport Road,
TORONTO,
Ontario.**

For the attention of Mr. J. C. McAllister.

Gentlemen,

**Soil Site Investigation
Proposed Highway 17 - Root River # 1 Crossing
W. P. 308 - 58**

We refer to your letter dated 27th May, 1958, and have pleasure in forwarding herewith four (4) copies of our report on the soil investigation at this site.

The soil conditions and engineering considerations are discussed in some detail in the soils report. Here for your convenience we are summarizing our conclusions and recommendations for your consideration.

1. The soil conditions on this site are fairly uniform. Three main classes of material were encountered as follows:-

- (a) Loose sand and silty sand fill**
- (b) Dense bouldery till**
- (c) Granitic bedrock.**

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APPEAR AS MULTI-FEED ON FILM.**

2. The observed free water level corresponded closely to the river level in all four holes. River level at the date of the investigation was approximately 800.2.

3. The bouldery till stratum and the bedrock are suitable foundation mediums. However, in view of the composition and dense condition of the till material, excavation in this stratum will be increasingly difficult with depth. Furthermore, these properties will virtually prohibit the use of piles without resorting to excavation in order that they may be pitched directly in position. For these reasons we are of the opinion that the bouldery till stratum is the most favourable material for the bridge foundations.

4. Practical considerations of scour and frost damage dictate the elevation at which the foundations should be placed. In our opinion this elevation is approximately 6 feet below river bed level.

5. The bearing capacity of the till stratum is amply sufficient for either continuous or isolated spread footings. However, the latter type possesses the merit of being more economical in material and requires less bulk excavation. Accordingly, we recommend the adoption of isolated spread footings as being most suited for these site and soil conditions.

6. In order that the structural merits of these two types of footings may be considered we give below the safe bearing capacity for each type of footing. These are:-

(a) Isolated Spread Footings

5 feet minimum dimension - 4 tons/sq. ft.

6 feet minimum dimension - 5 tons/sq. ft.

(b) Continuous Footings

5 feet minimum dimension - 5 tons /sq. ft.

We would stress the condition that load intensities in excess of 5 tons/sq. ft. should not be used on this site.

7. Settlement should be within tolerable limits with these loadings, furthermore the major part of any settlement will occur very shortly after the application of the load, and will be due to the reconsolidation of the sand loosened by excavation and the flow of seepage water.

8. The flow of water into the excavation below river level will be considerable. Accordingly it may be convenient, subject to structural conditions, to consider the possibility of placing a weak mass concrete pad, poured under water, to make good the material removed from the area between the water table and the bottom of the excavation.

APPROACH FILLS

1. Despite the increased height of the proposed embankment, an additional fill composed of a well compacted granular material will not introduce any stability problem.

2. The thrust on a closed abutment by such a fill would be in the order of 8 tons per foot run of wall.

3. The maximum vertical shear stress, induced in the soil by the proposed embankment will be approximately 1,180 lbs/sq.ft. The assumed dimensions and compacted soil density used in calculating this stress are:-

- (a) Top width of bank 44 feet
- (b) Base width of bank 156 feet
- (c) Bank height 35 feet
- (d) Compacted density 128 lbs/cu.ft.

We believe we have considered all the points on which you require information. However, should you wish to discuss any matter in

connection /

connection with this report, we shall be pleased to attend at your convenience.

Yours very truly,

E. M. PETO ASSOCIATES LTD.



mm:pf

E. M. Peto, P. Eng.

DEPARTMENT OF HIGHWAYS OF ONTARIO

HIGHWAY 17 - ROOT RIVER # 1 BRIDGE

W. P. 900 - 80, SAULT STE. MARIE

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e. m. peto associates ltd., 850 roselawn avenue, Toronto 10, Ontario

Job No. 5887

Client's Ref. No.

Date 28th June, 1958.

Report on

SOIL SITE INVESTIGATION

THREEDRUMS

at

**PROPOSED HIGHWAY 17 - ROOT RIVER #1 CROSSING
W.P. 908 - 58 3-1/2 MILES NORTH OF SAULT STE. MARIE**

for

DEPARTMENT OF HIGHWAYS OF ONTARIO.

INTRODUCTION

We were authorized to proceed with the soil investigation at the above site by Mr. J. C. McAllister under cover of his letter dated May 27th, 1958. At the same time we were also supplied with location plan and profile 1-B-327 and site plan and profile E-5370-1. Four suggested borehole locations were shown on the site plan.

PROGRAMME OF WORK

June 2nd, 3rd, 1958:	Crew and equipment moved from Toronto to Sault Ste. Marie. Engineer travelled to Sault Ste. Marie.
June 4th, 1958:	Equipment moved to site, holes staked out, field work commenced.
June 21st, 1958:	Field work completed. Equipment moved to Root River # 2 site.

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GENERAL INFORMATION

- (a) Standard soil sampling procedures were followed. These are described in Appendix II.
- (b) All four test holes were diamond drilled (AX size) to prove reliability and continuity of bedrock.
- (c) Progress was slow and difficult in most of the test holes below the 11 feet depth, because of the many boulders, which had to be diamond drilled.
- (d) Detailed individual borehole logs are attached, and in addition, Appendix I gives the details of the rock core examination.
- (e) A site plan is attached, showing the test hole locations. Two soil cross-sections approximately parallel to the proposed new bridge abutments are also shown on the site plan. Elevations are to Geodetic datum, and are referred to a D. H. O. bench mark in the form of a nail and washer in the South-East root of a 0.8 feet white birch, 27 feet left of station 181 + 48, line "A". The elevation of the bench mark was taken to be 863.28.

SITE AND GEOLOGY

The topography at the site is gently rolling to hilly. The Root River, which is shallow and medium to fast flowing at the crossing site, has a relatively wide valley, but the valley is not well-incised. It appears that at this site, there is only a 6 to 7 foot difference in stage of the river between high water level and normal water level.

There are some signs of scour at the foot of the existing South Bridge abutment, and it is the South end of the crossing which is subject to most of the scour arising from the configuration of the river at that point.

SITE AND GEOLOGY (contd)

The general area around Lake Superior was glaciated in the Wisconsin era. The mantle of unconsolidated rock materials is commonly shallow in the vicinity of Sault Ste. Marie, and is underlain by hard Precambrian bedrock. Gneiss, granite and other acidic rocks are most common. Whether igneous or metamorphosed in nature, these rocks are highly resistant to weathering. Their hardness prevented the glaciers from removing any great quantities of material so that only a shallow deposit of till was left when the ice melted.

SOIL CONDITIONS

Soil conditions on this site are fairly uniform. Three main classes of material were encountered on this site, viz. loose sand and silty sand fill, dense bouldery till, and granitic bedrock.

(a) Loose Sand and Silty Sand Fill

At the borehole locations, a minimum of 10 feet of sandy fill has been placed to form the approaches to the present bridge. This fill consists of generally loose brown sand of assorted sizes, in a slightly moist to moist condition. This material has some silt content at various points. Standard penetration test results in this layer ranged from 6 to 11 blows per foot, with 7 blows being a good average value.

For design purposes the "in situ" density of this sand may be taken to be 105 p.c.f.

(b) Dense Bouldery Till

Immediately beneath the surface material, and directly overlying the bedrock at this site, is a stratum, some 15 to 20 feet thick, of silty fine to medium sand with gravel, and with boulders at depth, particularly on the North side of the river. The colour varies from brown to grey, becoming reddish-brown just above bedrock.

(b) Dense Bouldery Till (contd.)

This stratum is generally wet to saturated, and is water-bearing, since it was specifically noted at one of the test holes that the wash water was being lost while sampling in this material.

The bouldery till is dense to very dense. Standard penetration test results in this stratum ranged from 52 to over 100 blows per foot. In many cases the high density of the soil or the rocks and boulders made it impossible to drive the standard split spoon sampler, and it was necessary to use diamond drilling equipment in order to achieve any penetration.

(c) Bedrock

The underlying bedrock arises below elevations 788 to 783 at this site. It consists of mainly a hard granite of varying colours, from very pale grey to pink to mauve. Included with the granite in the top few feet of rock are seams of relatively soft, dull brownish-red sandstone. The bedrock is of excellent quality, and a compression test on a typical sample of granite indicated a crushing strength in excess of 10,000 p.s.i.

WATER CONDITIONS

The observed free water levels in all four test holes corresponded closely, and in addition were approximately equal to the river water level. Due to the rainy weather during the performance of our field work, fluctuations of up to a foot in the river level occurred and this was reflected by a similar change in the ground water level. This indicates the high permeability of the sandy boulder till soil.

The ground water table on this site at the time of our investigation was approximately at elevation 800.2, the same as the river water level.

RECOMMENDATIONS AND CONCLUSIONS

1. The sand overlying the dense boulder till is in a loose condition and for this reason is not suitable as a foundation medium. However the till is dense and has a considerable boulder content, which although satisfactory from a foundation aspect, makes excavation in this material a difficult operation. Furthermore, excavation below elevation 799.0 introduces an additional problem arising from the free

RECOMMENDATIONS AND CONCLUSIONS (contd.)

water table which is evident at this depth. Having in mind the nature of the till stratum and the general site conditions it would appear that water will be readily abundant below elevation 799.0.

2. Bedrock is within measureable depth and although a suitable foundation, it is not readily accessible because of the difficulty that can be anticipated in attempting to penetrate the overlying boulder till, even with steel H piles.

3. The nature of the till material will ensure that settlements will be small and will normally take place during construction. For these reasons we recommend the placing of the bridge foundations in this stratum.

4. A suitable elevation for placing a foundation is 799.0, approximately equivalent to the existing stream water level, but some 4 feet above bed level. However, such an elevation is impractical having in mind scour effect and frost damage. Accordingly, we feel that despite the increasing difficulty of excavation that can be anticipated with depth, the foundation should be placed at approximately 6 feet below stream bed level.

5. The bearing capacity of this material is amply sufficient for either continuous or isolated spread footings.

In considering the merits of both types of foundations we are guided by such factors as

- (a) The very little difference at the depth recommended between the allowable minimum width for either type of footing, namely about 5 feet.
- (b) The type of footing requiring the lesser quantity of excavation and giving the greater economy in the use of material.

Accordingly, subject to structural requirements, we recommend the adoption of isolated spread footings as the most suitable and economical type of foundation for such site and soil conditions.

RECOMMENDATIONS AND CONCLUSIONS (contd.)

However, for your information we give below the permissible safe bearing capacity for both types of footing. These are:-

(i) **Isolated Spread Footings**

5 feet minimum dimension - 4 tons/sq.ft.

8 feet minimum dimension - 5 tons/sq.ft.

(ii) **Continuous Footings**

5 feet minimum dimension - 5 tons/sq.ft.

In this connection we would stress that load intensities in excess of 5 tons/sq.ft. should not be used on this site.

6. If the above recommended loadings are used, settlements should be within tolerable limits. In this regard it should be noted that the major part of any settlement would occur almost as soon as the loading was applied, and would be due to re-consolidation of the sand loosened by excavation and seepage water.

7. It may be of practical convenience, subject to structural requirements, to consider the possibility of placing a weak mass concrete pad, poured under water, to make good the material removed between the water table elevation and the bottom of the excavation. Such an expedient would be of value in overcoming the water problem, and would enable further work to be carried out clear of water at an earlier stage.

APPROACH FILLS

1. It is our understanding that the final highway grade at the new bridge will be 15 feet higher than the present grade, some 28 feet in all above present river water level. In addition, the North approach to the new bridge will be on approximately a 6% grade.

2. Provided that the new fill is granular material, well-compacted, there will be no stability problem with the new approaches.

APPROACH FILLS (contd.)

3. The thrust on a closed abutment by such a fill would be in the order of 8 tons per foot run of wall.

4. The maximum vertical shear stress induced in the soil by the proposed embankment will be approximately 1,100 lbs/sq.ft.

The assumed dimensions made in calculating this stress are:-

- (a) Top width of bank 44 feet
- (b) Base width of bank 156 feet
- (c) Height of bank 28 feet.

Bank constructed of granular fill compacted to 120 lbs/cu.ft..

E. M. PETO ASSOCIATES LTD.



MM:pt

E. M. Peto, P. Eng.

BOREHOLE LOG

Checked By M. Mindess

SOIL DESCRIPTION		COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVEL, SOIL MOISTURE & REMARKS
GROUND SURFACE				0' 0"					
FILL: SMALL BOULDERS IN SAND & GRAVEL 0' TO 3'				812.8					
SILTY MEDIUM SAND, SOME PEBBLE GRAVEL				5' 0"		1	S.S.	7	SLIGHTLY MOIST
COARSE SAND & GRAVEL				10' 0"		2	S.S.	47/6"	VERY MOIST TO WET.
FINE GRAVEL IN MATRIX OF SILTY COARSE TO MEDIUM SAND				15' 0"		3	S.S.	74/9"	WET
COARSE SAND & VERY FINE GRAVEL						4	W.S.	-	
GRAVEL IN MATRIX OF SILTY VERY FINE SAND				20' 0"		5	S.S.	100	WET. M.C. = 11.0%
S ABOVE WITH CONSIDERABLE COARSE SAND, VERY COARSE SAND				25' 0"		6	S.S.	52	WET.
						7	W.S.	-	
				30' 0"					BEDROCK AT 30'0"
GRANITE				782.8			R.C.		
DARK BROWNISH-RED.									
4" SEAM SOFT SANDSTONE. COARSE-GRAINED GRANITE.				36' 3"			R.C.		100% CORE RECOVERY
DULL MAIVE VERY PALE GREY AND PINK.				40' 2"					
				772.6					
HOLE TERMINATED.									

BOREHOLE LOG

Borehole No. 4

Boring Date June 12-14, 1958.

Checked ByM. Mindess

ABBREVIATIONS

V. T. IN SITU VANE SHEAR TEST

Q/6 UNCONFINED COMPRESSIVE STRENGTH

W. L. WATER LEVEL IN CASING

W. T. GROUND WATER TABLE IN SOIL

R. C. ROCK CORE

R. C. ROCK CORE

SOIL DESCRIPTION	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. & Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
SILTY MEDIUM SAND, SOME PEBBLE GRAVEL	DK. BROWN	Loose	0' 0" 814.2		1 CASING	-	-	SLIGHTLY MOIST
SILTY FINE TO MEDIUM SAND, SOME PEBBLES	MIXED BROWN	Loose	5' 0"		2 SS	7	-	SLIGHTLY MOIST
SILTY FINE SAND	REDDISH- BROWN	Loose To Compact	10' 0" 803.2		3 SS	11	-	MOIST, GRAVELLY BELOW 11'
			15' 0"					W.L. 13'6" TO 14'0", JUNE 12-14, 1958
GRAVEL IN MATRIX OF SILT AND FINE TO COARSE SAND	MIXED BROWN	VERY DENSE			4 SS	75	-	
VERY COARSE SAND	GRAY & BROWN				5 WS	-	-	BOULDERS FROM 15 1/2 FT. TO 20 FT. DRILLED THROUGH BOULDER FROM 15 1/2 TO 16 1/2 FT.
SANDSTONE, HORIZONTALLY STRATIFIED.	DULL RED.		20' 0" 793.2			R.C.		POSSIBLY A 2 FT. BOULDER.
3' SEAM OF SAND AND GRAVEL			23' 0" 791.2					
GRANITE.	DULL BROWNISH- RED.		26' 0" 788.2				(AX SIZE CORE)	
COARSE-GRAINED GRANITE	PALE GREY AND PINK.		30' 0"			R.C.		
PLEASE SEE APPENDIX I FOR DETAILS OF ROCK CORE.			35' 0" 779.2					CRUSHING STRENGTH OF ROCK SPECIMEN FROM 34 FT. DEPTH GREATER THAN 10,000 P.S.I.
				HOLE TERMINATED.				

APPENDIX I
ROCK CORE EXAMINATION

All four test holes on this site were cored to prove soundness and reliability of the underlying bedrock. The pattern established showed a few feet of dull red, relatively soft metamorphic sandstone underlain by very hard, acidic, igneous granite. The bedrock is generally sound and in very good condition, except for a 3 feet thick sand seam immediately below the sandstone at borehole # 4.

The hardnesses are referred to the Mohs Hardness Scale, on which a well-tempered steel knife blade has a hardness of 5, and plate glass has a hardness of 5-1/2.

Hole # 1

Runs undistinguishable

Overall core recovery 79.8%

26'0" - 27' 5"

Dull mauve and greyish-green, hard granite.

27'0" - 28'6-1/2"

Dull red with grey flecks, sandstone, horizontally bedded. Hardness 3-1/2.

28'6-1/2" - 29'5"

Dull brownish - red, with grey and green flecks, sandstone.
Hardness - 4-1/2.

29'5" - 40'0"

Very pale grey and pink, and some dark mauve, fine-grained granite.
Hardest - 5.

Note: Possible lost 1 to 2 feet of soft sandstone at top of this hole.

ROCK CORE EXAMINATION

Hole # 2

First run from 30'0" - 35'0"

Core recovery 100.0%

30'0" - 35'0"

Dark brownish-red to mauve granite,
some very pale grey to pink seams.
Hardness - 5.

Second run from 35'0" - 40'2"

Core recovery 100.0%

35'0" - 36'3"

Granite as above.

36'3" - 38'7"

Dark mauve, soft mudstone and very
pale grey soft sandstone.

38'7" - 40'2"

Very pale grey and pink, coarse
grained granite. Hardness - 5.

ROCK CORE EXAMINATION

Hole # 3

First run from 24'0" - 26'0"

Core recovery 42.5%

24'0" - 24'6"

Dull red some grey blocks, fine grained sandstone. Hardness 2-1/2 to 4.

24'6" - 25'3"

Pale grey, some pink, seams of mauve, granite. Hard.

25'3" - 26'0"

Core lost, could have been badly fractured.

Second run from 29'0" - 35'0"

Core recovery 67.5%

29'0" - 29'10"

Very pale grey and pink, granite. Hardness - 5.

29'10" - 31'2"

Dull brownish-red to mauve sandstone, stratified. Hardness - 4-1/2.

31'2" - 31'5"

Pink sandstone. Hardness - 5.

31'5" - 33'11"

Very pale grey, coarse grained granite.

33'11" - 34'8"

Dull brownish-red sandstone. Hardness - 3-1/2.

34'8" - 35'0"

Pink and dark brownish-red granite. Hard.

ROCK CORE EXAMINATION

Hole # 4

First run from 21'0" - 23'0"

Core recovery 88.5%

21'0" - 23'0"

Dull red with grey flecks fine-grained sandstone. Horizontally stratified. Hardness - 4.

Note: This was possibly a boulder.

23'0" - 23'0"

Seam of sand.

Second run from 26'0" - 30'0"

Core recovery 41.7%

26'0" - 26'2-1/2"

Pale grey sandstone. Hardness - 5.

26'2-1/2" - 27'3"

Probably some loss here. Mottled dull red to mauve granite.

27'3" - 27'6"

Dull brownish-red sandstone, stratified.

27'6" - 28'6"

Some loss here. Dull brownish-red to mauve fine-grained granite. Hardness - 5.

28'6" - 30'0"

Dull brownish-red mixed with very pale grey fine-grained granite.

Third run from 30'0" - 35'0"

Core recovery 100.0%

30'0" - 35'0"

Pale pink and grey, coarse grained granite. Hardness - 5.

APPENDIX II

METHOD OF OPERATION

The field investigation work is carried out by means of a skid-mounted diamond drill rig.

Standard sampling procedures are followed. Casing is driven and cleaned, either by tubes or by wash water.

Samples are recovered ahead of the casing at frequent intervals, with either a 2 inch or 3 inch O.D. split barrel sampling tube, Shelby tube, or split barrel sampling tube fitted with brass liners and special sharp cutting nose.

The standard penetration test results are recorded when sampling with the regular 2 inch O.D. split barrel sampler, these being the number of blows of a 140 pound hammer falling 30 inches, required to drive the sampling tube a distance of one foot into disturbed soil.

The Dutch cone probe test is made by driving the drill rods into the ground with a 2-1/4" - 90° cone tip. The number of 4200 inch pound blows per foot of penetration are recorded, as in the standard penetration test.

Where required, "in situ" shear strength tests are made ahead of the casing, using modified Acker vane test equipment.

Disturbed samples are visually classified in the field, sealed in sample jars, and are re-examined, and tested as necessary, in the soils laboratory. Undisturbed samples are returned to the laboratory for later examination and testing, as required.

The test holes are bailed at the end of the day and on completion. Subsequent water level readings are taken for the duration of the field work. Water pressure readings are recorded when Artesian water conditions are encountered. Moisture content samples are recovered at frequent intervals to assist in the soil classification and the interpretation of water table results.