

G4-F-256M

LOTS 27 & 28, CON. V & VI

WANANOSH TWP

*El. Elevation - Soil Report El. 120.0 * 5000 El. 300.0*

MR. B. M. ROSS
CONSULTING ENGINEER
GODERICH ONTARIO

B.H. 1894

STRUCTURE SITE No. 12-46

Report on
SOIL INVESTIGATION
for
HURON COUNTY BRIDGE
LOTS 27 & 28, CONCESSIONS V & VI
TOWNSHIP OF WAWANOSH
NEAR AUBURN

64-F 256 M

by
DOMINION SOIL INVESTIGATION LIMITED
363 Queens Avenue
LONDON ONTARIO
Reference No. 4-1-L8
March 10, 1964

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SUMMARY

Sound limestone bedrock was encountered relatively close to the surface at both extremities of the proposed structure. The level of the rock is 13 feet higher at the south end than at the north. Either footings or piled foundations are practicable at these locations, although it seems probable that a footing would be more suitable at the south end and a piled support at the north end.

A bearing pressure of 40,000 p.s.f. is recommended in the sound rock. Any convenient form of steel pile will be satisfactory.

No unusual construction problems are anticipated.

I INTRODUCTION

Verbal authorization was received from the office of Mr. B. M. Ross to carry out a soil investigation at a site in Huron County where it is proposed to replace an existing road bridge with a new structure. The site is located approximately 4 miles north of the village of Auburn where the county road crosses the Maitland River.

The requirements of the project were discussed with Mr. K. G. Dunn who provided the following information. The existing bridge is a 2-span steel truss with a total length of 243 feet. The new bridge will probably be of continuous steel or reinforced concrete beam design, having a 4-span arrangement of $72 + 98 + 98 + 72 = 340$ feet. It is intended that the south abutment of the new bridge will be 51 feet to the south of the south abutment of the existing bridge, so that the lateral centre lines of the 2 structures will be more or less coincident. Soundings made by the client during the summer located the level of the river bed below the centres of the existing north and south spans at Els. 114.5 and 113.4 respectively. It was assumed from visual observation that the river bottom is in the bedrock. It is proposed that the piers of the new structure should bear on the bedrock, and that the abutments should be supported on H-piles driven to rock, thus avoiding deep excavations through the existing road embankment.

Initially, authorization was received to drill 2 boreholes, one near each of the north and south abutments. In view of the difference in the elevations of the bedrock at these locations, an additional hole was drilled at the south abutment after consultation with Mr. Ross.

II FIELD WORK

Field work was carried out on the 15th to 17th of January and 2nd of March 1964, and consisted of 3 boreholes at the locations shown on enclosure 2. The holes were advanced to the bedrock by washboring and lined with Bx (3-inch) casing. Core was extracted from the rock using Axt and Bxt diamond bits. Standard Penetration tests were performed at frequent intervals of depth to determine the relative density or consistency of the soil above the rock, and to recover disturbed samples. Dynamic cone penetration tests were performed adjacent to boreholes 1 and 2.

The results of the field tests are recorded on geotechnical data sheets comprising enclosures 3 and 4.

III SUBSURFACE CONDITIONS

Details of the stratification at each borehole are shown on the data sheets and a general picture of the subsurface conditions is given by the profile on enclosure 2. The principal strata are as follows:

- (a) Fill. The material consists mainly of sand and silt with 10 to 15% of fine gravel. It is in a compact to dense condition and possesses no cohesion except at borehole 3. Above the river level its moisture condition is damp.
- (b) Brown sandy silt till. This deposit appears only in borehole 2 and 3, and belongs to the kame moraine which comprises the high ground to the south. The till is compact to dense and, in its upper layers, contains approximately 5% of angular gravel not exceeding 1/2 inch in diameter. Below a depth of 10 feet the gravel content increases to about 25%. The deposit is almost entirely cohesionless, and is only slightly cemented.
- (c) Fractured limestone. In borehole 1 between depths of 19 and 23.5 feet there is a layer of shattered rock which apparently belongs to the same formation as the underlying bedrock. The core recovery in this layer was 25%, and it was possible to drive the Bx (3-inch) casing through it after running the Axt (1-13/16") core barrel.
- (d) Limestone bedrock. This is a hard brownish crystalline limestone, containing occasional seams of porous fossilized material. The stratification dips at about 10° to the horizontal, probably in a south-west direction (this being most common in Southern Ontario). The deposit may belong to either the Norfolk formation which covers most of the surrounding area, or to the Salina formation which outcrops near the Huron coastline.

The inferred rock profile is quite consistent with the physiography of the area. The valley is classified by Chapman and Putnam as a glacial spillway, passing between the ridges of kame moraines to the north and south, i.e. the site has been a water course for a long period of time, and has accommodated a glacial river very much larger than the present one. It is therefore quite conceivable that the sedimentary rock has been eroded to the shape of the inferred profile.

IV FOUNDATIONS

It has been shown that the bedrock is close to the surface, and is therefore the logical choice of bearing stratum.

At the south abutment, sound rock was encountered at El. 128.8 ± 0.6 feet which is 13.5 feet below the existing road surface. The simplest type of foundation will probably be a

footing bearing on the rock. A bearing pressure of 40,000 p.s.f. is recommended. The coefficient of friction between the rock and concrete may be taken as 0.5 and a factor of safety against sliding of 1.5 should be used. Additional sliding resistance may be readily generated by means of a key into the rock. It is most important that the rock surface should be thoroughly clean and roughened when the concrete is poured, all loose, fractured or weathered material being removed. If for any reason it is desirable to use piles at this abutment, any standard form of pile may be expected to reach refusal within the upper 12-inches of the limestone stratum.

The pier foundations should also bear on sound rock, in accordance with the foregoing recommendations. On the basis of the field data presented here, there is no reason to suppose that the rock exposed on the river bed is not sound.

At the north abutment a footing foundation would be quite deep, and the use of piles might be more economic. The depth to the surface of the shattered rock is 19 feet, and to sound rock 23.5 feet. The recommended respective bearing pressures for a footing are 10,000 p.s.f. and 40,000 p.s.f. Steel piles may be expected to penetrate the fractured rock, and meet refusal in the sound rock at or near El.114. Any convenient form of steel pile will be suitable, the only limitation on load being the fibre stress in the pile. This is generally limited to 12,000 p.s.i. under the maximum loading so that e.g. a BP10, 57 lb. per foot pile could be used for a working load of 100 tons.

The piles should be firmly seated on the rock but should not be overdriven. A variation of several feet in the rock surface should be expected over the area of the foundation.

The settlement of the supports under load will consist almost entirely of elastic deflection in the structure and the supporting rock. The magnitude of this deflection is unlikely to exceed 0.25 inch at any support.

V

REFERENCES

1. The Physiography of Southern Ontario by L. J. Chapman and D. F. Putnam of the Ontario Research Foundation - University of Toronto Press 1951.
2. Procedures for Testing Soils, ASTM, April 1958. pp.186 to 198. (Unified Soil Classification System - by A. A. Wagner).

3. Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering (Research on Determining the Density of Sands by Spoon Penetration Testing - by H. J. Gibbs and W. G. Holtz of the United States Bureau of Reclamation). London, 1957.
4. Soil Mechanics, Foundation and Earth Structures - by Gregory P. Tschebotarioff, McGraw-Hill Book Co., 1951.



DOMINION SOIL INVESTIGATION LIMITED

James Park
James Park, M.Sc., P.Eng.

LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE.

SOIL COMPONENTS AND GROUND WATER CONDITIONS.

BOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY	ORGANICS	BEDROCK	GROUND WATER LEVEL	DEPTH OF CAVE-IN
		COARSE	FINE	COARSE	MEDIUM	FINE						
Ø	> 8"	3"	¾"	4.76mm	2.0	0.42	0.074	0.002	>	NO SIZE LIMIT		
U.S. Standard Sieve Size :				No.4	No.10	No.40	No.200					

SAMPLE TYPES.

AS Auger sample

CS Sample from casing

ChS Chunk sample

RC Rock core

% Recovery

SS Split spoon sample

TP Piston, thin walled tube sample

TW Open, thin walled tube sample

WS Wash sample

SAMPLER ADVANCED BY static weight : w

" pressure : p

" tapping : t

OBSERVATIONS
MADE WHILE
CORING

Steady pressure

No pressure

Intermittent
pressureWashwater
returnsWashwater
lost

PENETRATION RESISTANCES.

DYNAMIC PENETRATION RESISTANCE : to drive a 2" ϕ , 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot.

STANDARD PENETRATION RESISTANCE, -N- : to drive a 2" outside dia, split spoon sampler 1 foot into the ground, expressed in blows per foot.

EXTRAPOLATED -N- VALUE

The energy for the penetration resistances is supplied by a 140 lb. hammer falling 30 inches

SYMBOL :



322

SOIL PROPERTIES.

W % Water content

LL % Liquid limit

PL % Plastic limit

PI % Plasticity index

LI Liquidity index

 γ

Natural bulk density (unit weight)

e

Void ratio

RD

Relative density

C_v

Coeff. of consolidation

m_v

Coeff. of volume compressibility

k Coeff. of permeability

C Shear strength — in terms of total stress

 ϕ Angle of int. friction — in terms of effective stress

C' Cohesion — in terms of effective stress

 ϕ' Angle of int. friction — in terms of effective stress

UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —

TRIAXIAL

UNCONFINED

LABORATORY

FIELD

POCKET
PENETROMETER
TEST

COMPRESSION TEST

VANE TEST

Strain at failure is represented
by direction of stem

20%
15% + 5%
10%

St : sensitivity = $\frac{\text{shear strength in undisturbed state}}{\text{shear strength in remoulded state}}$

SOIL DESCRIPTION.

COHESIONLESS SOILS :

RD :

COHESIVE SOILS :

C lbs/sq.ft.

Very loose

0 - 15 %

Loose

15 - 35 %

Compact

35 - 65 %

Dense

65 - 85 %

Very dense

85 - 100 %

Very soft

less than 250

Soft

250 - 500

Firm

500 - 1000

Stiff

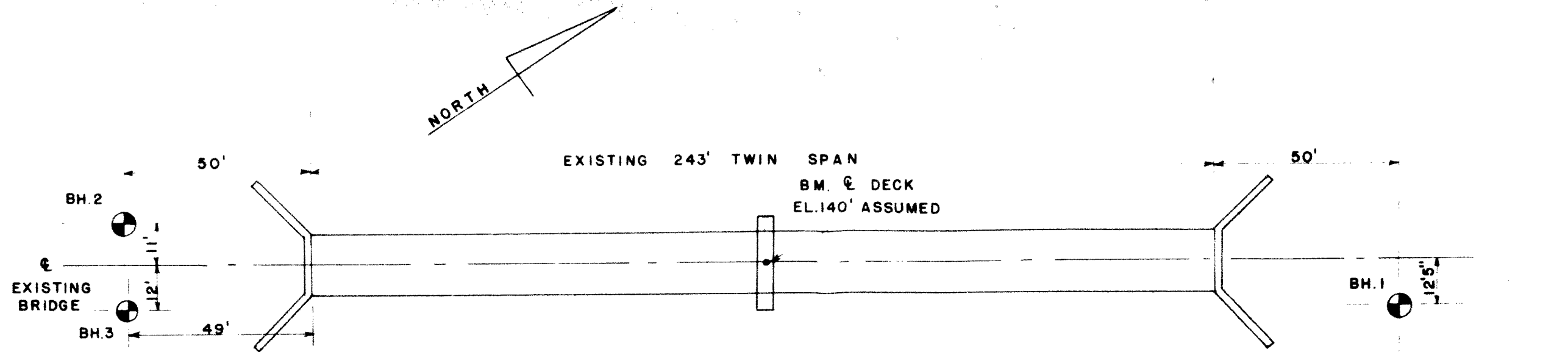
1000 - 2000

Very stiff

2000 - 4000

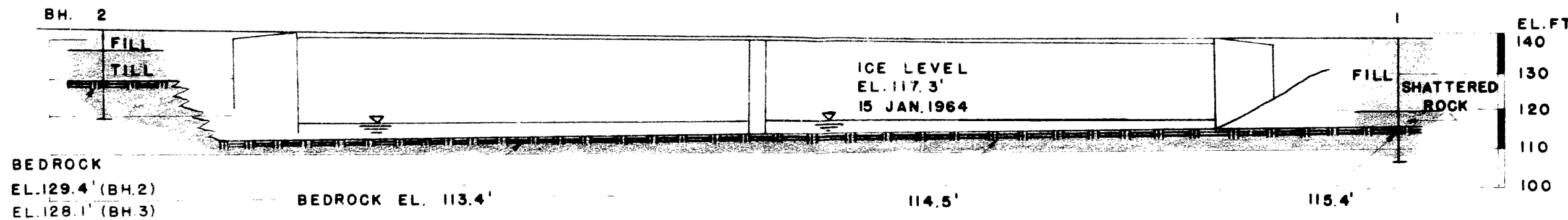
Hard

over 4000



LOCATION OF BOREHOLES

SCALE: 1 INCH TO 30 FEET



INFERRED SUBSURFACE PROFILE

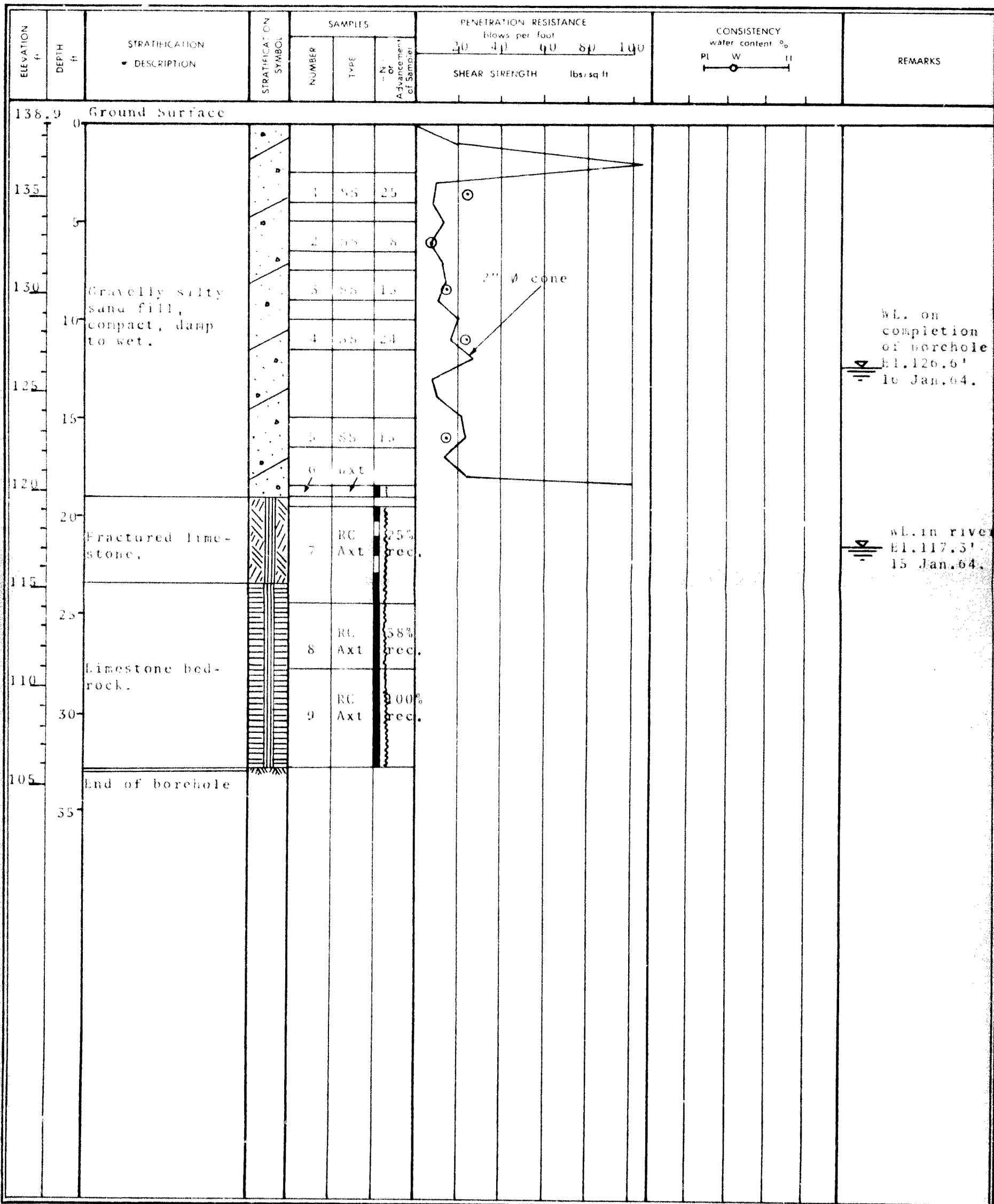
SCALE: 1 INCH TO 30 FEET

GEOTECHNICAL DATA SHEET FOR BOREHOLE

OUR REFERENCE NO. 4-1-12

CLIENT Mr. L. M. Ross
 PROJECT Huron County Bridge
 LOCATION Near Auburn
 DATUM ELEVATION 140.0' (deck of existing bridge at central pier)

METHOD OF BORING was boring,
 DIAMETER OF BOREHOLE 5x 1.5-inch
 DATE January 15-16, 1964.
 ENCLOSURE NO 5



GEOTECHNICAL DATA SHEET FOR BOREHOLES 2 and 3

OUR REFERENCE NO. 4-1-L8

CLIENT: Mr. W. H. Ross
PROJECT: Huron County bridge
LOCATION: Near Auburn
DATUM ELEVATION 140.0' (deck of existing bridge at central pier)

washboring,
METHOD OF BORING: Diamond drilling
DIAMETER OF BOREHOLE: 5-inch
DATE: January 16-17, 1964. (BH.2)
March 2, 1964 (BH.3)

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N or Advancement of Sampler	40	40	60	80	100	PL	W	LI	
142.4	0	Ground Surface													
140		Gravelly sandy silt fill, dense damp.		1	SS	22									
135	5	Brown sandy silt till, dense to compact, moist.		2	SS	34									
				3	SS	21									
130	10	gravelly		4	SS	12									
125	15			5	RC bxt	97% rec.									
		Limestone bed- rock.		6	RC bxt	100% rec.									
120	20			7	RC bxt	80% rec.									
	25	End of borehole													
142.3	0	Ground Surface													
140		Brown clayey silt fill, stiff.		1	SS	10									
	5	Compact, wet gravel and sand fill.		2	SS	30									
135				3	SS	20									
	10	Brown sandy silt till, dense, moist.		4	SS	20									
130		gravelly													
	15			5	RC Axt	87% rec.									
125		Limestone bed- rock.													
	20	End of borehole													

BOREHOLE 2

WL. on
completion
of borehole
El. 128.6'
17 Jan. 64.

WL. in river
El. 117.3'
15 Jan. 64.

BOREHOLE 3

DOMINION SOIL INVESTIGATION LIMITED
77 CROCKFORD BOULEVARD - SCARBOROUGH ONTARIO CANADA - TELEPHONE 21-2557

BRANCH
383 QUEENS AVENUE
LONDON, ONTARIO
TELEPHONE GE. 3-3881



FOUNDATION ENGINEERS

ASSOCIATED COMPANY
SOIL TESTING AND ENGINEERING LTD.
84 BRENTFORD ROAD,
KINGSTON 8, JAMAICA, WEST INDIES
TELEPHONE 68988

November 27th, 1964

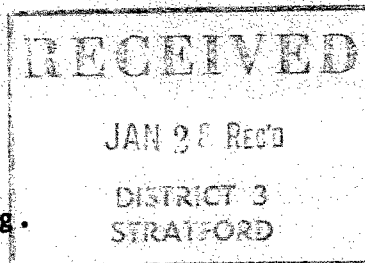
4-11-L10
Report

1894

Mr. B. M. Ross, P. Eng.,
Consulting Engineer,
41 West Street,
Goderich, Ontario.

Attention: Mr. K. G. Dunn, P. Eng.

Dear Mr. Dunn:



Additional Soundings to Determine
Bedrock Profile at Bridge BR-113

Further to our report dated March 10th, 1964 (Ref. 4-1-L8) we have completed this work in accordance with your letter of authorization dated November 10th, 1964.

The purpose of this additional investigation was to explore further the bedrock underlying the south abutment. For this purpose 6 cone penetration tests were made at the locations shown on enclosure 2; and one shallow borehole was put down adjacent to cone 5 to confirm that bedrock had in fact been encountered. It was also intended to evaluate the soundness of the rock at this point.

The results of the field tests are shown on enclosure 3, and the inferred rock profile is shown on enclosure 2. The rock was proved to a depth of 5 feet in borehole 4, and the average core recovery of 67.5% indicates that it is fairly sound. The bearing pressure of 40,000 p.s.f. recommended in our original report is, therefore, still applicable provided any loose or weathered material is removed from the bottom of the excavations.

We trust that this information will be sufficient to meet your requirements, but please do not hesitate to call us if we can be of service in any other way.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED

STRUCTURE SITE No. 12-46

E. J. W. Atkinson
E. J. W. Atkinson, M. Sc.,
Project Engineer.

CA/sg

LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE.

SOIL COMPONENTS AND GROUND WATER CONDITIONS.

BOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY	ORGANICS	BEDROCK	GROUND WATER LEVEL	DEPTH OF CAVE-IN
		COARSE	FINE	COARSE	MEDIUM	FINE						
Ø	> 8"	3"	¾"	4.76mm	2.0	0.42	0.074	0.002	>	NO SIZE LIMIT		
U.S. Standard Sieve Size :				No. 4	No. 10	No. 40	No. 200					

SAMPLE TYPES.

AS Auger sample

CS Sample from casing

ChS Chunk sample

RC Rock core

% Recovery

SS Split spoon sample

TP Piston, thin walled tube sample

TW Open, thin walled tube sample

WS Wash sample

SAMPLER ADVANCED BY static weight : w
 " pressure : p
 " tapping : t

OBSERVATIONS
 MADE WHILE
 CORING

Steady pressure
 No pressure
 Intermittent pressure

Washwater returns
 Washwater lost

PENETRATION RESISTANCES.

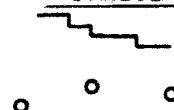
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LL % Liquid limit

PL % Plastic limit

PI % Plasticity index

LI Liquidity index

 γ

Natural bulk density (unit weight)

e

Void ratio

RD

Relative density

C_v

Coeff. of consolidation

m_v

Coeff. of volume compressibility

k Coeff. of permeability

C Shear strength in terms of total stress

 ϕ

Angle of int. friction

C'

Cohesion

 ϕ'

Angle of int. friction in terms of effective stress

UNDRAINED SHEAR STRENGTH.

- DERIVED FROM -

TRIAXIAL

UNCONFINED

LABORATORY

FIELD

COMPRESSION TEST

VANE TEST

POCKET PENETROMETER TEST

Strain at failure is represented by direction of stem

20%
 15% + 5%
 10%

St : sensitivity = $\frac{\text{shear strength in undisturbed state}}{\text{shear strength in remoulded state}}$

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COHESIONLESS SOILS :

RD :

COHESIVE SOILS :

C lbs./sq.ft.

Very loose

0 - 15 %

Loose

15 - 35 %

Compact

35 - 65 %

Dense

65 - 85 %

Very dense

85 - 100 %

Very soft

less than 250

Soft

250 - 500

Firm

500 - 1000

Stiff

1000 - 2000

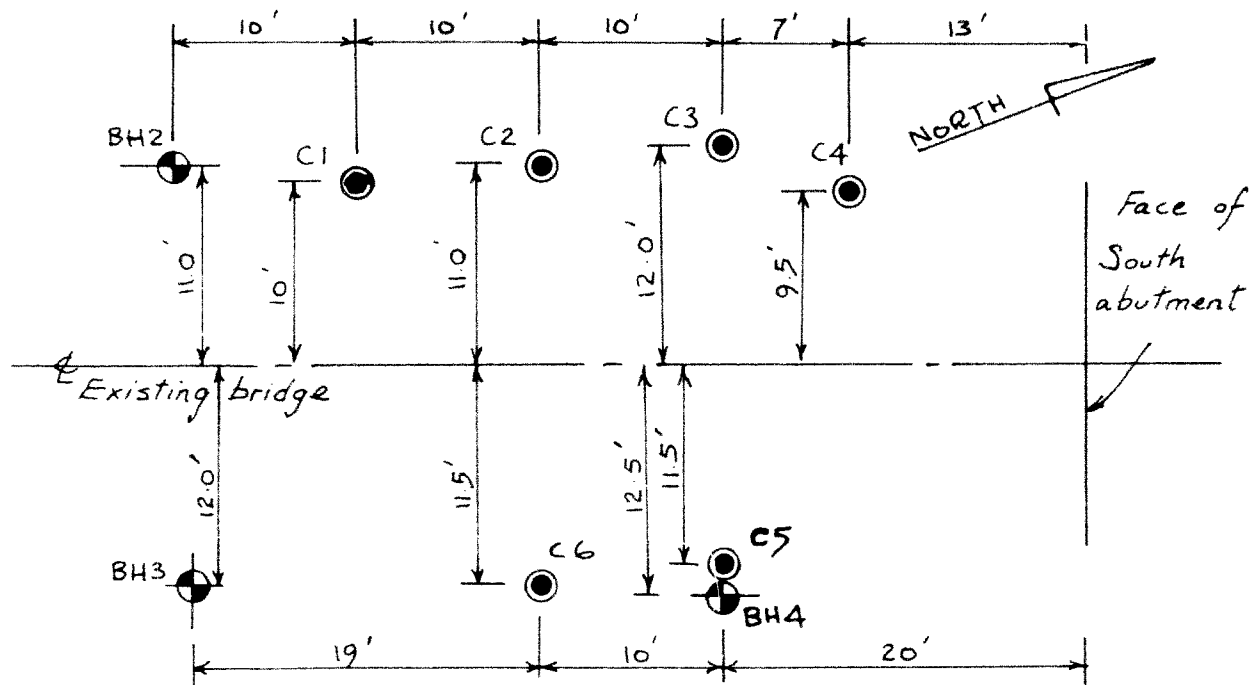
Very stiff

2000 - 4000

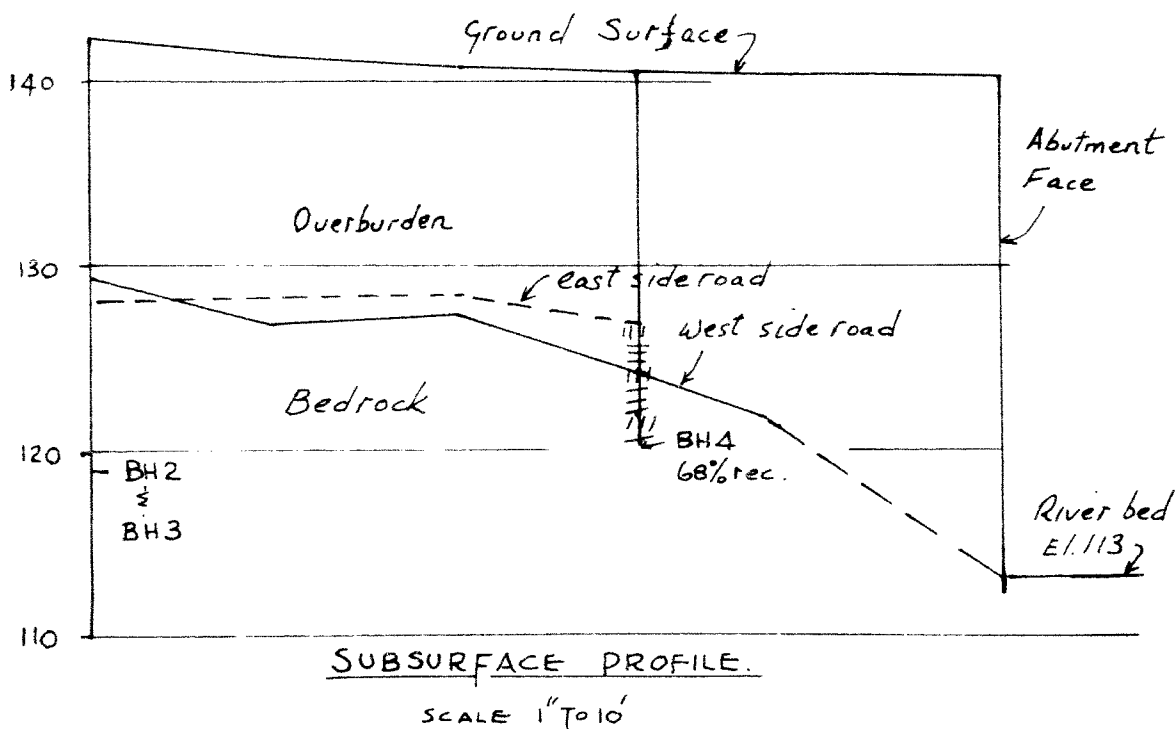
Hard

over 4000

Prep. By



LOCATION OF BOREHOLES (BH) AND
CONE PENETRATION TESTS (C)
 SCALE 1" TO 10'



OUR REFERENCE NO. 4-11-110

CLIENT: Mr. B. M. Ross
PROJECT: Bridge BR 113
LOCATION: Near Auburn
DATUM ELEVATION 140.0 feet

METHOD OF BORING: Washboring
DIAMETER OF BOREHOLE: Bx (3 inch)
DATE: November 19th, 1964

ENCLOSURE NO. 3

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE		CONSISTENCY water content % PL W LI	REMARKS
				NUMBER	TYPE	N or Advancement of Sampler	blows per foot	SHEAR STRENGTH lbs/sq ft		
141.4	0.0									
	10.0									
	20.0									
140.3	0.0	Ground Surface								
	5.0	Brown damp sandy gravelly clayey silt fill, very stiff		1	SS	18				
	10.0	Clayey silt with gravel and sand		2	SS	37				
	15.0	Limestone bedrock		3	RC Axt	72 %				
				4	Rc Axt	63 %				
		End of Borehole								

Borehole 4
Adjacent to
Cone C5

Cone	Elev.
1	141.37
2	140.81
3	140.49
4	140.41
5	140.27
6	140.91
BH4	140.27