

67- F-259 M

BRIDGE

LOTS 15/16, CON. 5

Mc GILLIVRAY TWP.

13A 2762
Site 19-31

A. M. SPRIET & ASSOCIATES LTD
CONSULTING ENGINEERS
LONDON ONTARIO

Report on
SOIL INVESTIGATION
for
PROPOSED BRIDGE
LOTS 15 & 16, CONCESSION 5
TOWNSHIP OF MCGILLIVRAY.

by
DOMINION SOIL INVESTIGATION LIMITED
369 Queens Avenue
LONDON ONTARIO

Reference No. 7-5-L7
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SUMMARY

The two boreholes revealed lacustrine deposits consisting of organic clay and silt, extending to a depth of about 30 feet below the creek bed, overlying very stiff to hard silty clay till which is known to extend to a considerable depth.

It is recommended that the structure be supported on a friction pile foundation and working loads are presented in the report for different lengths and types of pile.

Consolidation settlement of a friction pile foundation is estimated to be 3/4 inch.

I INTRODUCTION

Verbal authorization was received from A. M. Spriet & Associates, Consulting Engineers, to carry out a soil investigation at a site in the Township of McGillivray where it is proposed to replace an existing road bridge with a new structure.

The existing steel-truss structure is located on Lots 15 and 16, Concession 5 of the Township where the road crosses a tributary of the Ausable River.

It is understood that the proposed structure is a concrete rigid frame with about a 60 foot span. Also, the centre line will be moved about 40 feet to the south of the centre line of the existing bridge. The requirements of the project were discussed with Mr. A. M. Spriet, P.Eng., who supplied the foregoing information.

The purpose of this investigation was to reveal the subsurface conditions at the site and to determine the relevant soil properties for the design and construction of the new foundations.

II THE GEOLOGY OF THE SITE

The site lies in the physiographic region known as the Huron Slope. This forms the eastern side of Lake Huron between the Algonquin shorecliff and the Wyoming moraine, and rises from 600 feet to 850 or 900 feet above sea level.

The soil profile usually consists of shallow lacustrine deposits overlying glacial clay till which extends to a considerable depth.

III FIELD WORK

The field work, consisting of 2 boreholes was carried out during the period June 5 to 7, 1967, at the locations shown on Enclosure 2. The holes were advanced to the sampling depths by washboring methods and were lined with Bx size casing.

Standard penetration tests were carried out at frequent intervals of depth, as detailed on Appendix 'A' and the results are recorded on the Geotechnical Data Sheets as 'N' values.

Insitu vane shear tests were performed in cohesive strata to determine the undrained shear strength of the soil. The techniques employed in the test are outlined on Appendix 'B'.

Dynamic cone penetration tests were performed adjacent to each borehole location to obtain an indication of soil density changes with depth. The same source of energy was used to drive the cone as was used for the standard penetration test.

Elevations were referred to the centre of the deck of the existing bridge which was given the arbitrary value, El. 100 feet.

IV SUBSURFACE CONDITIONS

Detailed descriptions of the strata encountered in each borehole are given on the Geotechnical Data Sheets, comprising Enclosures 3 and 4, and a general picture of the soil stratigraphy is given in the form of a Subsurface Profile on Enclosure 2.

The natural soil profile consists of alluvial clay and silt strata, extending down to depths of 31 and 42 feet in boreholes 1 and 2 respectively, overlying a silty clay till stratum in which the boreholes were terminated.

Alluvial Clay and Silt Strata

The alluvial clay material has a variable undrained shear strength as indicated by insitu vane shear test results ranging from 600 to 2400 p.s.f., and these results are confirmed by the standard penetration test results which range from 1 to 10 blows per foot.

The relative density of the silt stratum encountered in borehole 2 is described as 'loose' to 'compact' as estimated from standard penetration test results ranging from 7 to 15 blows per foot.

Silty Clay (Glacial Till)

This stratum was encountered at El. 55 in borehole 1 and at El. 53 in borehole 2. A thin layer of compact well-graded sand, 2 1/2 to 3 feet thick, was revealed overlying the clay till stratum at both borehole locations.

The consistency of the clay till material is described as 'very stiff' to 'hard' as indicated by standard penetration test results ranging from 15 to 30 blows per foot.

Atterberg limit tests, which were carried out on two samples of the clay till, gave values of Liquid Limit of 32% and 34%; Plastic Limit of 13% and 14%; and Plasticity Index of 19 and 20,

indicating that the soil is a clay of low plasticity and compressibility. The Liquidity Indices which relate the natural moisture content to the Atterberg Limits were 0.3 and 0.4 confirming the 'stiff' consistency.

V GROUNDWATER CONDITIONS

The groundwater reached equilibrium at an average El. 87.6 in both boreholes, which was the same elevation as the water level in the creek at the time the field work was carried out.

VI DISCUSSION AND RECOMMENDATIONS

The soil profile consists of weak and compressible alluvial deposits extending to a depth of about 30 feet below the creek bed, therefore the most economical type of support for the bridge footings will be a piled foundation.

It will be appropriate for the piles to penetrate into the very stiff clay till stratum, with the resulting working load being mobilized partly by end-bearing and partly by friction along the side of the pile.

Due to the length of pile required, timber piles have not been considered practical, and in the following paragraphs suitable types of pile are discussed:-

(a) Steel Tube Piles (12-inch diameter)

On the basis of the soil profile obtained on this site it is estimated that the skin-friction developed per foot of penetra-

tion of the pile into the clay till stratum will be 2.5 tons, and the end-bearing component will be 14 tons.

A factor of safety of 2 may be applied to piles relying on the clay till alone for support, therefore the allowable working load may be calculated on the basis of 1.25 tons per foot of penetration into the clay till, plus a 7 ton end-bearing component. On this basis a 40 tons capacity pile will require a penetration of 26 feet into the clay till stratum.

(b) Precast Concrete Piles

As in the case of the steel tube piles, the ultimate bearing capacity of a precast concrete pile will consist of an end-bearing and a skin-friction component. The following ultimate load values have been calculated for 9-inch, 12-inch and 15-inch square sections:-

<u>Section</u>	<u>Ultimate Bearing Capacity</u>	
	End-Bearing	Skin-Friction /foot of penetration into clay till
9-inch square	11	2.5
12-inch square	18	3.0
15-inch square	28	3.5

A factor of safety of 2 should be applied to the foregoing loading figures in calculating the working load.

Settlement

It is estimated that consolidation settlement of a structure supported on a friction pile foundation will be about 3/4 inch.

General

The foregoing estimates of bearing capacity of piles are only theoretical predictions, therefore in practice, the piles should be driven to a satisfactory set in accordance with an accepted dynamic pile driving formula.

From previous experience in this area the use of a steel tube pile foundation will probably be the least expensive, although choice of pile may depend on other factors such as availability of material, speed of construction etc.

It is anticipated that when the creek has been diverted away from the pile cap excavations, seepage of water into the excavations will be very small.

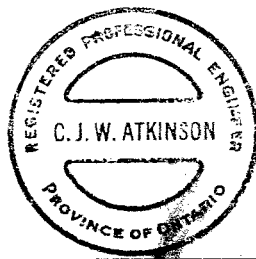
Approach Embankments

The minimum value of the undrained shear strength of the alluvial silty clay stratum, which will support the approach fills, was found to be 600 p.s.f. Therefore using this value, the maximum height of approach fills should be limited to 25 feet to maintain a factor of safety against shear failure of 1.3.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED

C.J.W. Atkinson
C.J.W. Atkinson, M.Sc., P.Eng.,
Branch Manager



APPENDIX A

STANDARD PENETRATION TESTS

In order to determine the relative density of non-cohesive soils, such as sands and gravels, the standard penetration test has been adopted. The test also gives an indication of the consistency of cohesive soils.

A two-inch external diameter thick-walled sample tube is driven into the ground at the bottom of the borehole by means of a 140 lb. hammer falling freely through 30 in. The tube is first driven an initial 6 in. to allow for the presence of disturbed material at the bottom of the borehole. The number of standard blows (N) required to drive the sampler a further 12 in. is recorded. The sample tube used is one originally developed by the Raymond Concrete Pile Company in the United States, where a sufficient number of tests have been made in conjunction with field investigations to show that the results, although essentially empirical, may be applied to foundation design.

For sands:

Values of N	Density
Less than 10	Loose
Between 10 and 30	Compact
Between 30 and 50	Dense
Greater than 50	Very dense

APPENDIX B

INSITU VANE SHEAR TEST

In soft to stiff clays, and particularly sensitive clay soils such as frequently occur in alluvial deposits, it is difficult to obtain reasonable undisturbed samples for the determination of the undrained shear strength. In order to overcome this difficulty, the vane test was developed as an in-situ method of measuring the shear strength.

The apparatus consists of a 4-inch long by 2-inch wide rectangular 4-bladed rotating vane attached to a thin rod, which is pushed into the undisturbed soil below the bottom of the borehole to the depth at which the test is to be made.

A torque is then applied to the vane and the maximum torque when failure occurs is recorded. The vane is then rotated 10 times to remould the soil and after one minute the torque test is repeated. The shear strength of the soil can then be calculated from the torque and the dimensions of the vane, and the sensitivity of the material estimated from the ratio of the original torque to the final torque after remoulding.

Enclosures

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"	3/4"	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	RC Rock core	TP Piston, thin walled tube sample
CS Sample from casing	% Recovery	TW Open, thin walled tube sample
ChS Chunk sample	SS Split spoon 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.

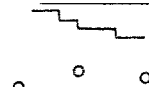
DYNAMIC PENETRATION RESISTANCE : to drive a 2"Ø, 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot.

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 :



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SOIL PROPERTIES.

W % Water content	γ Natural bulk density (unit weight)	k Coeff. of permeability
LL % Liquid limit	e void ratio	C Shear strength in terms of total stress
PL % Plastic limit	RD Relative density	φ Angle of int. friction in terms of effective stress
PI % Plasticity index	Cv Coeff. of consolidation	C Cohesion
LI Liquidity index	m _v Coeff. of volume compressibility	φ' Angle of int. friction

UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —

TRIAXIAL COMPRESSION TEST

UNCONFINED TEST

LABORATORY

VANE TEST

FIELD

POCKET PENETROMETER TEST

Strain at failure is represented by direction of stem

20%
15% + 5%
10%

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

SOIL DESCRIPTION.

COHESIONLESS SOILS :

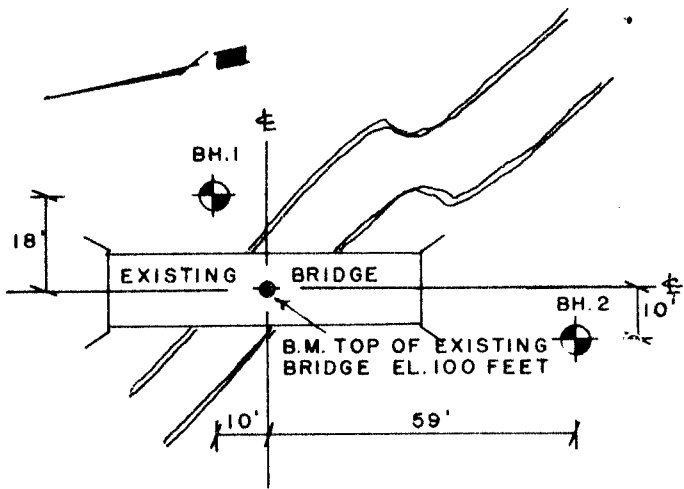
RD :

COHESIVE SOILS :

C lbs./sq ft







Very loose	0 - 15 %
Loose	15 - 35 %
Compact	35 - 65 %
Dense	65 - 85 %
Very dense	85 - 100 %

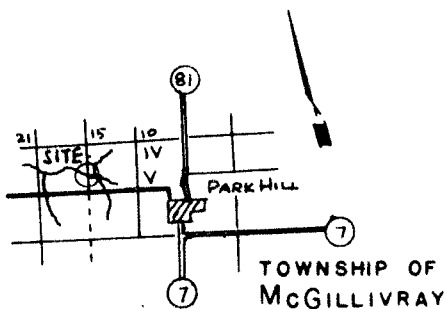
Very soft	less than 250
Soft	250 - 500
Firm	500 - 1000
Stiff	1000 - 2000
Very stiff	2000 - 4000
Hard	over 4000



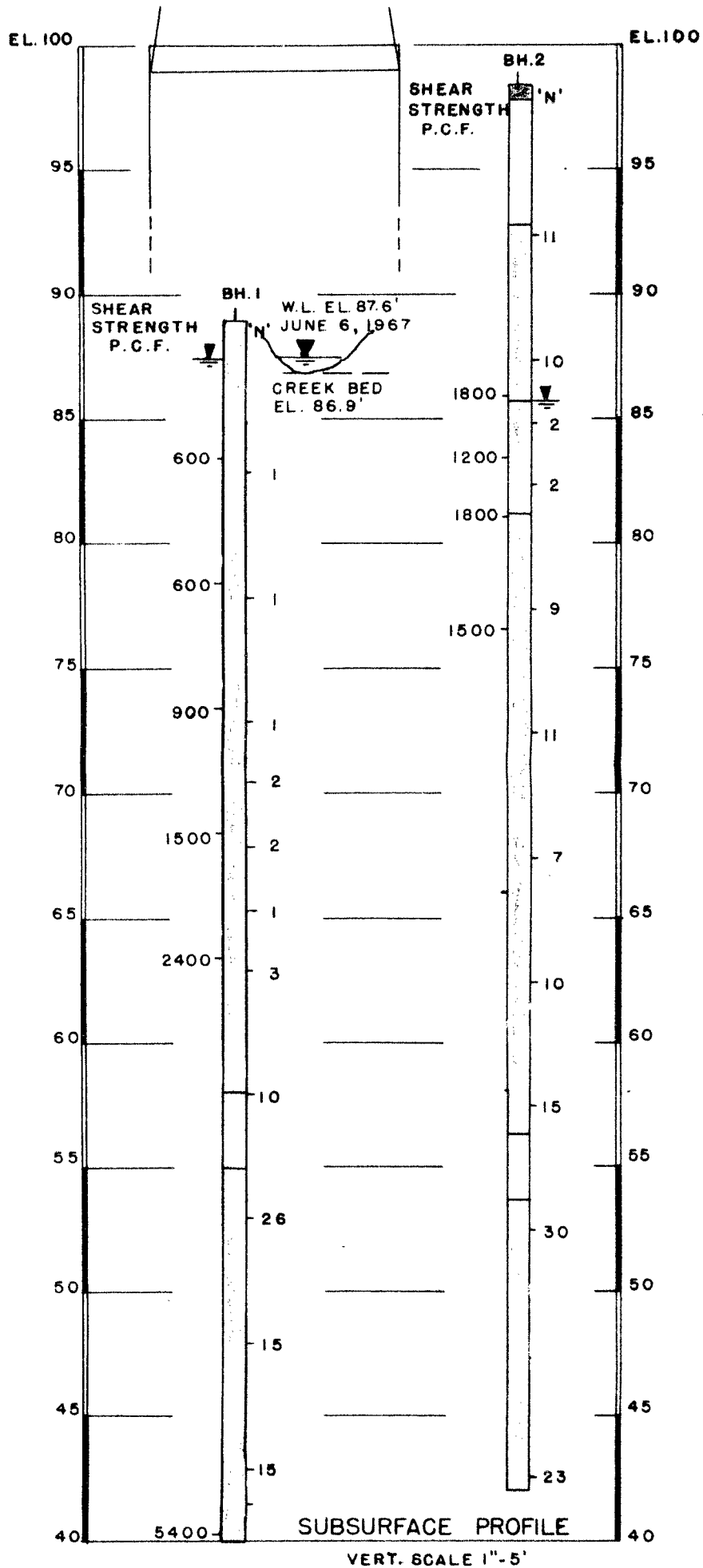
LOCATION OF BOREHOLES
SCALE 1" = 30'

LEGEND

-  FIRM TO STIFF ORGANIC SILTY CLAY
-  GRAVEL FILL
-  CLAYEY SILT FILL
-  LOOSE TO COMPACT SILT
-  COMPACT WELL-GRADED SAND
-  VERY STIFF TO HARD SILTY CLAY, TILL



KEYPLAN



SUBSURFACE PROFILE

VERT. SCALE 1" = 5'

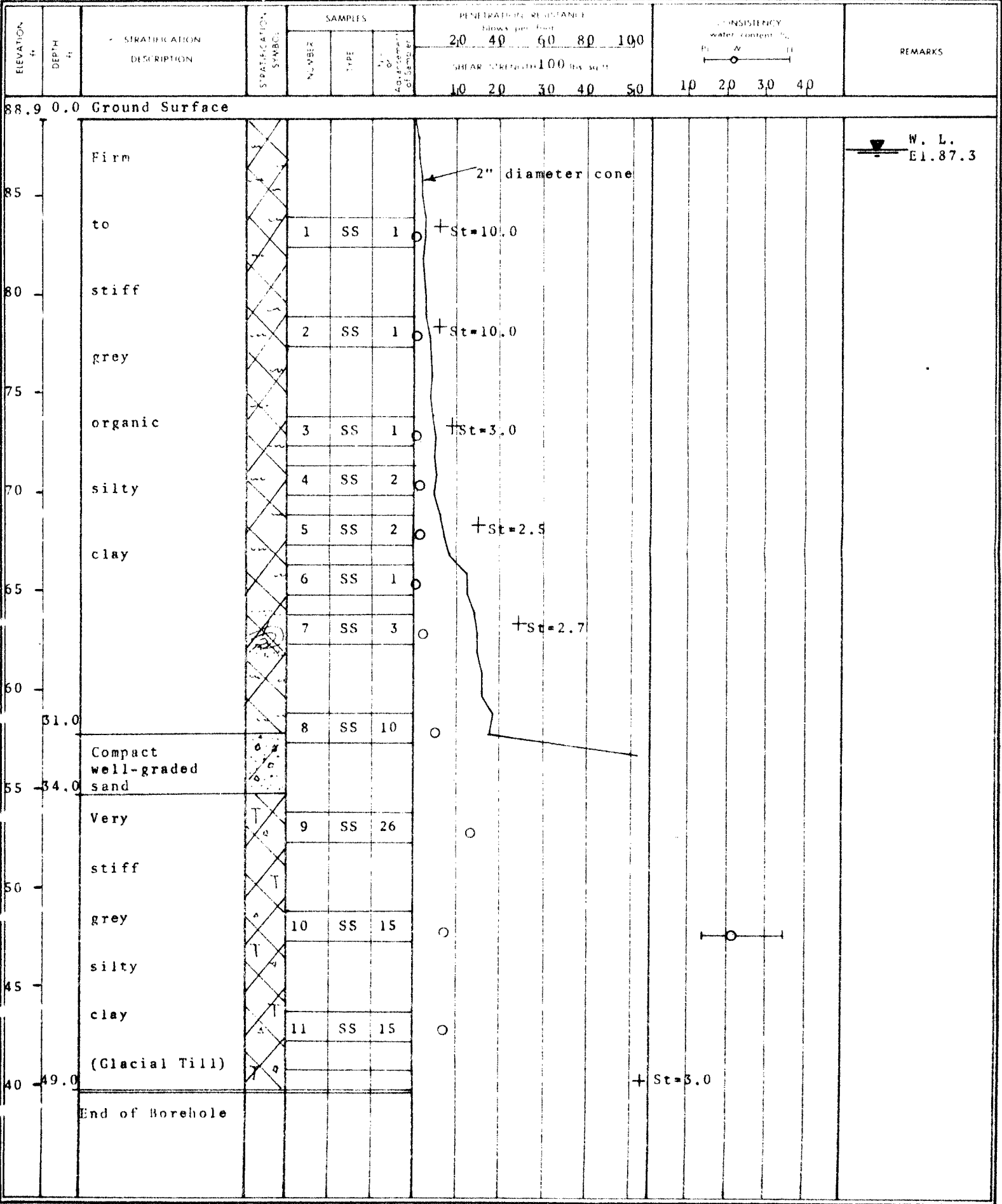
GEOTECHNICAL DATA SHEET FOR BOREHOLE L

OUR REFERENCE NO 7-5-L7

CLIENT A. M. Spriet & Associates
PROJECT Proposed Bridge
LOCATION Township of McGillivray
DATUM ELEVATION 100 feet

METHOD OF BORING Washboring
DIAMETER OF BOREHOLE Bx (3-inch)
DATE June 5 & 6, 1967

ENCLOSURE NO 3



GEOTECHNICAL DATA SHEET FOR BOREHOLE 2

OUR REFERENCE NO. 7-5-17

CLIENT A. M. Spriet & Associates
PROJECT Proposed Bridge
LOCATION Township of McGillivray
DATUM ELEVATION 100 feet

METHOD OF BORING Washboring
DIAMETER OF BORING 8x (3-inch)
DATE June 6 & 7, 1967

ENCLOSURE NO. 4

