

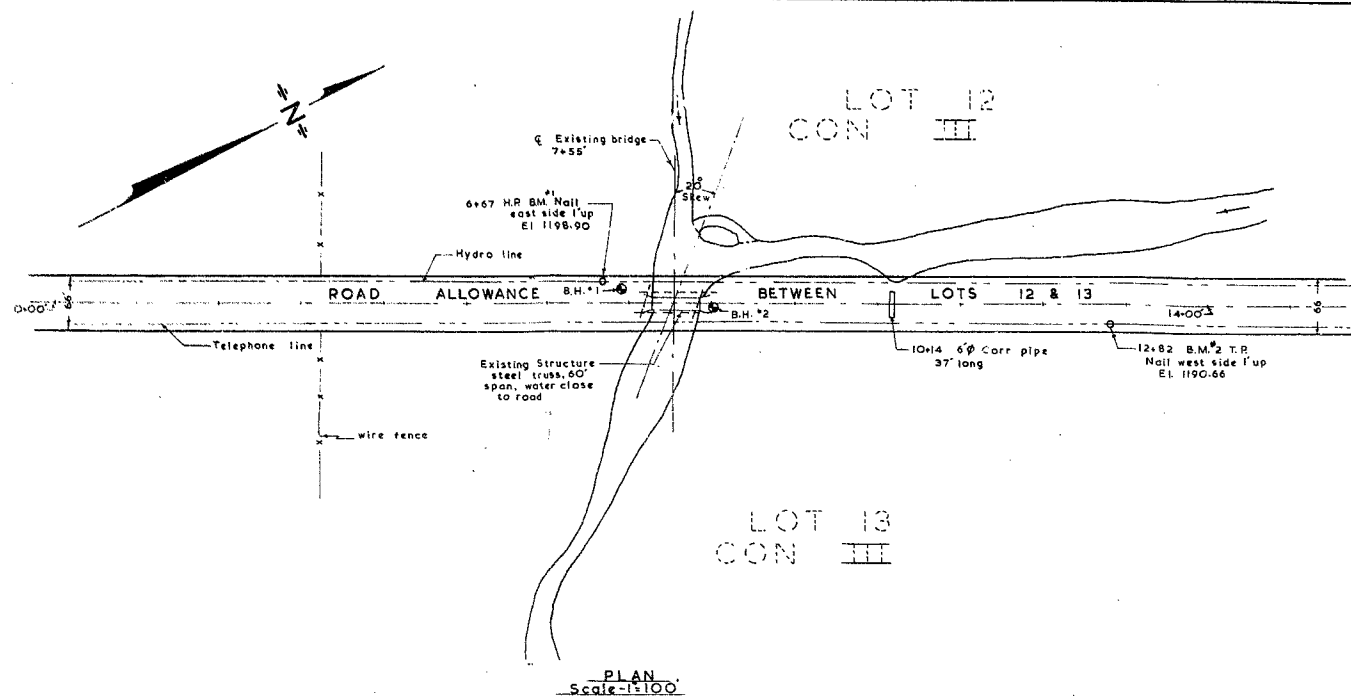
#63 - F - 261 M

BRIDGE OVER

SMITH CREEK

LOTS 12/13, CON. III

MORNINGTON TWP.

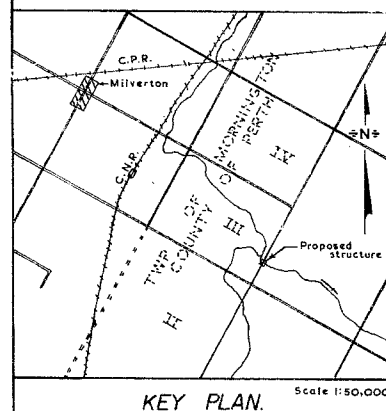
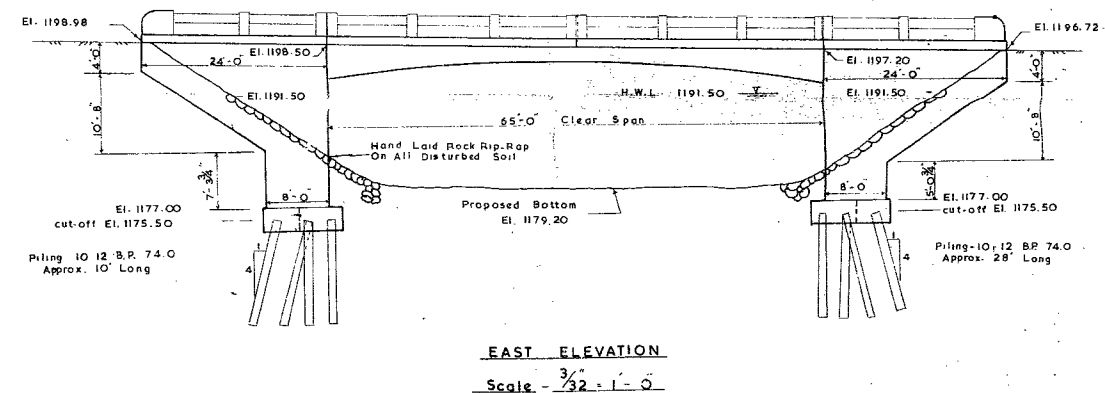
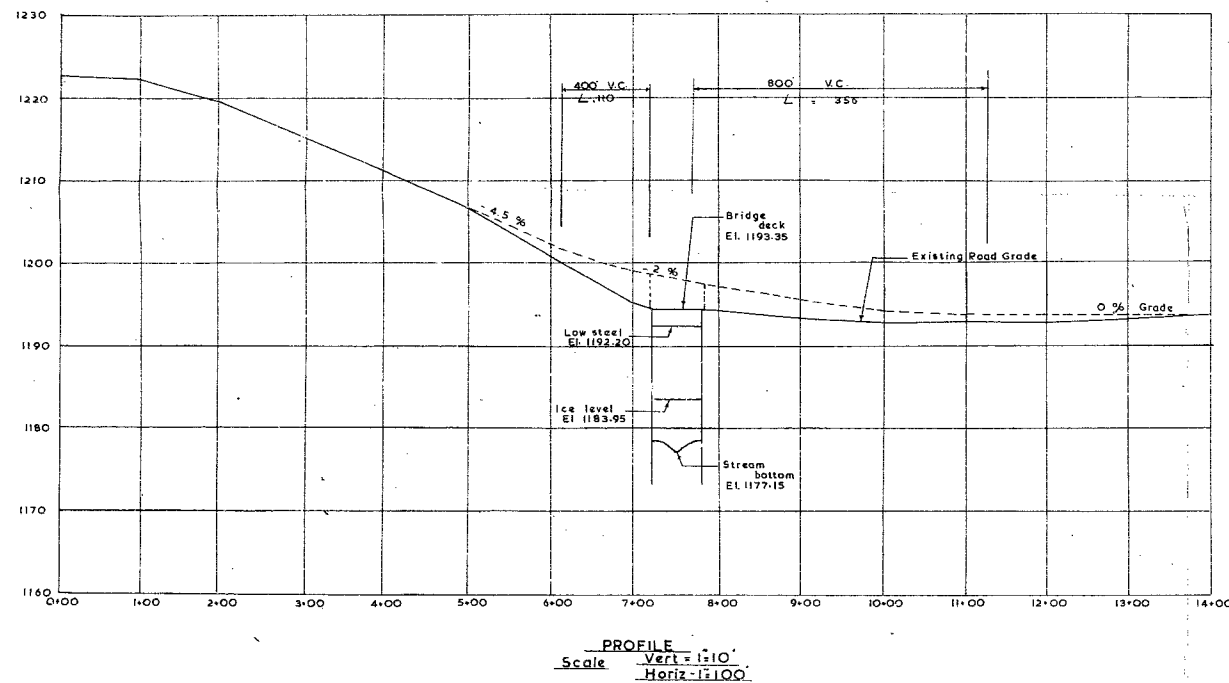


ELEV	DESCRIPTION	BLOWS PER FOOT
1193.70	Ground Surface	
1191.70	Organic Topsoil	
	Brown Gravelly Till, Very Dense, Damp.	32
1185.0	Hard Brown Silty Clay	44
		139
		103
	Very Dense Grey Sandy Silt Till, Numerous Cobbles Or Boulders, Slight Cohesion.	
1158.30	End Of Borehole	

BOREHOLE No 1

ELEV	DESCRIPTION	BLOWS PER FOOT
1193.20	Ground Surface	
1191.20	Gravel, Silt And Organics.	
	Dark Brown Cohesive Silt Fill, Trace Of Organics, Compact.	8
		11
		13
1181.80	Compact Silty Gravel And Sand Trace Of Organics.	25
1175.8		21
	Seams Of Grey Silt And Grey Clayey Silt, Loose Or Firm	5
		7
1164.30	Grey Sandy Silt Till, Very Dense, Contains Coarse Gravel Or Cobbles.	
1151.00	End Of Borehole	70

BOREHOLE No 2



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

#### DATA

- SPECIAL FEATURES WATERFALLS, DAMS, EXCEPTIONAL FLOODS, ICE, DRIFTWOOD, SLIDING BANKS, ETC. None Usual Amount Of Ice & Drift.
- (A) UPSTREAM & DOWNSTREAM BRIDGES (GIVE LOCATION, LENGTH, HEIGHT ABOVE N.H.W.L., AT CROSS-SECTIONAL AREA AT HIGH WATER & ESTIMATED AGE) 1 1/4 M. Upstream Span 57' Ht. Above H.W.L. 0.5' Area @ H.W.L. 6000', Age 40 Yrs; 1/2 M. Upstream Span 28.5' Ht. Above H.W.L. 1.8' Area @ H.W.L. 225', Age 30 Yrs; 1/8 M. Downstream Span 75' Ht. Above H.W.L. 2.5' Area @ H.W.L. 825', Age 8 Yrs.  
(B) REASONS WHY THESE BRIDGES ARE, OR ARE NOT, FAIR INDICATIONS OF SIZE OF PROPOSED BRIDGE. No Flooding Occurs At These Structures
- REASONS FOR CHANGES IN HEIGHT OR LENGTH FROM THAT OF OLD BRIDGE: Height Raised To Improve Road Grade.  
Span Increased To Allow For Skew

#### DATA (contd.)

- IS DITCH, STREAM, OR RIVER GRADIENT LIABLE TO BE LOWERED? No Just By Natural Scour
- NAVIGATION CLEARANCES REQUIRED, IF ANY. None
- RAILWAY CLEARANCE REQUIRED, IF ANY. None
- IF STRUCTURE IS OVER OR UNDER A RAILWAY HAS APPROVAL BEEN OBTAINED?  
(A) FROM RAILWAY CO. N/A  
(B) FROM BOARD OF TRANSPORT COMMISSIONERS. N/A
- HAS APPROVAL BEEN OBTAINED UNDER NAVIGABLE WATERS PROTECTION ACT? N/A
- IS A TEMPORARY DETOUR REQUIRED? Yes  
WHO WILL BUILD IT? Contractor  
WHO WILL MAINTAIN IT? Contractor
- INFORMATION AND EVIDENCE OF EXTREME FLOODING WAS OBTAINED FROM Road Superintendent AND REFLECTS HIGHEST WATER ELEVATION IN THE AREA OF THIS CONSTRUCTION TO BE 1191.5 AND THE LOWEST WATER ELEVATION TO BE 1182
- ROAD DESIGN INFORMATION:  
ESTIMATED A.D.T. 100  
DESIGN SPEED 40  
STOPPING SIGHT DISTANCE 275

#### STRUCTURE DATA

- NET SPAN LENGTH AND TYPE OF BRIDGE 65' Span Rigid Frame Reinforced Conc.
- ROADWAY WIDTH ON BRIDGE 24'
- NUMBER & WIDTH OF SIDEWALKS. None
- SKW ANGLE 20°
- TOTAL LENGTH & TYPE OF PILING 280' Lx. El. Of 12 B.P. 74.0 Steel H-Piles
- APPROX. VOLUME OF CONCRETE 500 CU YDS
- APPROX. WEIGHT OF STR. STEEL. None TONS
- APPROX. WEIGHT OF REINFORCEMENT 24 TONS
- APPROX. VOLUME OF APPROACH FILL 1800 CU YDS
- DRAINAGE AREA 54.5 SQ. MI.

FIELD INVESTIGATION MADE Dec. 13, 1963

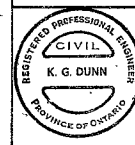
BY K.G. DUNN  
SURVEY ENGINEER

B.M. ROSS  
Consulting Engineer

Goderich Ontario  
OWNER TWP OF MORNINGTON MUNICIPAL DIST. No.  
Co. PERTH ROAD No.  
TWP MORNINGTON LOT 12 & 13 CON. III

#### SITE PLAN

December 20 1963  
DATE  
DESIGN ENGINEER  
BRIDGE NAME  
LOADING H 20S16  
BRIDGE No. BR-114  
DWS No. BR-114-1



DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,  
Principal Foundation Engineer,  
Materials & Research Section.

FROM: G.C.E. Burkhardt

DATE: March 12, 1964.

OUR FILE REF.


IN REPLY TO

SUBJECT: Township of Mornington,  
Bridge over Smith Creek,  
Lot 12, 13, Con. III,  
County of Perth,  
Structure Site No. 26-88,  
Our File No. BA 1763.

Attached please find one copy of the Foundation Report, by Dominion Soil Investigation Limited, and one copy of the Preliminary Plan for your comments.

We would appreciate it very much, if we could have your comments on or before March 20th, 1964.

GCEB/kd

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

*Basically no comment*  
*March 18, 1964*

*A. Stermac*

BA. 1763

MR. B. M. ROSS  
CONSULTING ENGINEER  
GODERICH ONTARIO

Report on  
SOIL INVESTIGATION  
for  
ROAD BRIDGE  
LOTS 12 & 13, CONCESSION III  
TOWNSHIP OF MORNINGTON

by  
DOMINION SOIL INVESTIGATION LIMITED  
363 Queens Avenue  
LONDON ONTARIO  
Reference No. 3-12-L6  
December 1963

### SUMMARY

On the south side of the existing structure, very dense granular till deposits were encountered from a depth of 2 feet below the existing road surface. On the north side, from the same elevation, the borehole encountered fill to a depth of 11.5 feet, compact gravel and sand to 17.5 feet, soft or loose silt to 29.0 feet, and finally the same very dense granular till stratum.

On the north side, the considerable depth of excavation which would be required to reach a good bearing stratum appears to preclude the use of footing foundations. The use of steel piles with a working load of 60 tons per pile is recommended.

On the south side the structure could be supported either on a footing or on piles, but assuming that pile-driving equipment will be used for the north abutment, this will probably be the most economic solution for the south side also. At the south abutment it is believed that only steel H-piles will be capable of achieving the necessary penetration for protection against scour, a circumstance which appears to dictate the use of this type of pile throughout the project.

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 the Township of Mornington, where it is proposed to replace an existing road bridge with a new structure.

A plan of the site was supplied showing the outline of the existing bridge, and the requirements of the project were discussed with Mr. K. G. Dunn. It is understood that the new bridge will be of similar span and in approximately the same position as the existing 60-foot steel truss.

The purpose of this investigation has been to reveal the subsurface conditions and to determine the necessary soil properties for the design and construction of foundations.

## II PHYSIOGRAPHY

The site lies 2 miles to the southeast of the Town of Milverton, within the physiographic region known as the Stratford Till Plain. The bridge spans a headwater of the River Nith whose course is controlled for some distance to the north and south by the ridge of the Milverton Moraine. This ridge constitutes the high ground immediately to the south of the site, while to the north is the relatively flat terrain of the till plain, consisting of dense ground moraine. The course occupied by the river is not officially classified as a spillway, but the gravelly deposits found near the level of the bed suggest the former existence of a much larger stream, probably of glacial origin.

## III FIELD WORK

Field work was carried out on the 13th and 14th of December 1963, and the 7th and 8th of January 1964. The break in the field programme was caused by extreme cold and blizzard conditions which made ordinary wash-boring procedures impracticable. The work consisted of 2 boreholes at the locations shown on enclosure 2, and dynamic cone penetration tests were performed adjacent to each borehole. The holes were advanced by wash boring and lined with Bx (3-inch) casing. In both boreholes it was also necessary to employ diamond drilling techniques in very dense granular strata encountered at lower depths.

Standard Penetration tests were performed at frequent intervals of depth to determine the relative density or consistency of the soil, and to recover disturbed samples. The dynamic cone penetration tests provide a

continuous record of penetration resistance and reveal abrupt changes in stratification.

The results of the field tests are recorded on geotechnical data sheets comprising enclosures 3 and 4. Elevations have been referred to the level of the deck of the existing 60-foot truss at its centre point, taken as El. 1193.35 feet.

#### IV 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 following notes are intended only to amplify these data. The stratification in the two boreholes is quite different and they will be considered separately.

At borehole 1 the upper 2 feet of material is a mixture of topsoil, the natural gravelly soil, and possibly some road fill. From 2'-0" to 8'-8" there is a very dense gravelly till consisting of 60% ( $\pm$ ) of subangular gravel and sand up to 1-1/2 inches in diameter, in a matrix of clayey silt. A thin seam or hard brown silty clay of intermediate plasticity intrudes between 8'-8" and 9'-6". Below 9'-6" the soil is a very dense sandy silt till. Boulders, cobbles and coarse gravel particles were frequently encountered. While drilling through this material, and a few pieces of core were recovered, consisting both of limestone and metamorphic rocks. The silt matrix is very dense and slightly cemented, giving the deposit a low permeability and some cohesion. All of the strata below a depth of 2 feet in this borehole appear to be glacial deposits.

In borehole 2, the road embankment was found to extend to a depth of 11'-6". It is a compact, damp to dry, mainly silty material, and is dark brown in colour owing to the presence of organics. Between 11'-6" and 17'-6" there is a compact deposit of silty gravel and sand containing traces of organics. This is probably a glacio-fluvial deposit. Between 17'-6" and 29'-0" a grey silt was encountered. Although no definite laminations were observed, seams of both cohesive and cohesionless material are present. Their consistencies can be described, respectively, as *firm* and *loose*. This material is clearly a post-glacial deposit, and its layered structure suggests there may have been some "ponding" of meltwaters in the area beside the moraine. It is also noted that this is the site of the confluence of streams from the north and south. A consequent reduction in velocity of the water could thus have lead to the deposition of fluvial sediments.

Very dense granular material was encountered at 29'-0", and it was only possible to advance the hole beyond this depth by diamond drilling. The recovery of samples was limited to a few small pieces of core and the sandy material brought up in the wash water. From the appearance of these materials, it is concluded that the deposit is similar to the sandy silt till encountered in borehole 1.

Groundwater was encountered in borehole 1 at a depth of 7 feet (El.1186.7) and in borehole 2 at the level of the river (El.1182.5).

## V FOUNDATIONS

The use of footing foundations will be considered first. The level of the bed of the river is El.1176 (+) indicating that footings should be located at El.1170 or 1171 to allow for scour. At borehole 2 on the north side of the structure this lies within the firm or loose silt stratum whose bearing capacity is quite inadequate. The 'N' value of this material is in the range 5 to 8, the maximum safe soil pressure would be about 2000 p.s.f., large settlements would be expected and construction conditions in this sensitive material would be extremely difficult if not impossible. The highest satisfactory elevation for the footings would be El.1164 at the top of the till stratum where a soil pressure of the order of 10,000 p.s.f. might be used. This level, however, is 12 feet below the river bed, 18 feet below the prevailing water level, 29 feet below the existing road grade and probably more than 30 feet below the new road grade. It is concluded that the use of piles for the north abutment at least would be much more economic than a footing design.

For the conditions at borehole 2 it is estimated that a 12-inch diameter steel pipe pile designed for a working load of 60 tons would reach a satisfactory set between Els.1163 and 1158. A steel H-pile such as a BP 12 (74 lb./foot) to achieve a similar working load would probably penetrate beyond the limit of the boreholes. Assuming that material with properties giving 'N'=100 blows is encountered at El.1150 ±, the theoretical ultimate bearing capacity of a pile driven to this level is 105 tons. In practice it is anticipated that such piles designed for a safe working load of 60 tons will reach a satisfactory set between Els.1150 and 1145.

At the south side of the structure near borehole 1 the following safe soil pressures are applicable to strip footing design:

Els. 1191 to 1184      7,000 p.s.f.

Els. 1184 to 1164      15,000 p.s.f.



The corresponding settlement under the proposed loading would be immediate upon the application of load and would not exceed *one inch*.

To carry the footing down to a safe level for scour protection, say El. 1170, would require a fairly deep excavation through very dense material. Also, it is not known how far to the south the loose silty condition encountered at borehole 2 extends, and it might be necessary to excavate well below El. 1170. Alternatively the footing could be constructed at any convenient higher elevation within a steel sheet pile enclosure driven deep enough to provide permanent scour protection. However, the driving conditions for such sheet piling in the dense till would be very hard, and since bearing piles will almost certainly be driven at the north abutment it will probably be more economic to use the same type of support on the south side. Because of the dense soil conditions on this side, it is unlikely that pipe piles could be driven sufficiently deep to provide scour protection. This leads to the choice of steel H-piles for the south abutment, and presumably the same type of pile should be used on the north side.

For the conditions at borehole 1, BP 12 (74 lb. per foot) H-piles designed for a safe working load of 60 tons per pile are expected to reach a satisfactory set between Els. 1170 and 1165.

It must be emphasized that the calculations from which pile lengths are predicted are inherently crude, especially in dense gravelly till materials such as are found on this site. The presence of boulders or cobbles may completely upset such predictions, so that they should be treated only as the most approximate guide. All piles should be driven to a satisfactory set in accordance with an accepted dynamic pile driving formula, irrespective of the elevation at which such a set is achieved.

For a structure supported on piles, or for a footing support at the south end, no unusual construction problems are anticipated.

## VI 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. Terzaghi and Peck: Soil Mechanics in Engineering Practice. John Wiley and Sons, New York 1948.
5. Standard Penetration Tests and Bearing Capacity of Cohesionless Soils, by G. G. Meyerhof, ASCE Paper 866, January 1956.



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
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  
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.

DYNAMIC PENETRATION RESISTANCE : to drive a 2"  $\phi$ , 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

$\gamma$  Natural bulk density (unit weight)  
e Void ratio  
RD Relative density  
Cv Coeff. of consolidation  
mv Coeff. of volume compressibility

k Coeff. of permeability  
C Shear strength — in terms of total stress  
 $\phi$  Angle of int. friction — in terms of total stress  
C' Cohesion — in terms of effective stress  
 $\phi'$  Angle of int. friction — in terms of effective stress

## 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 :

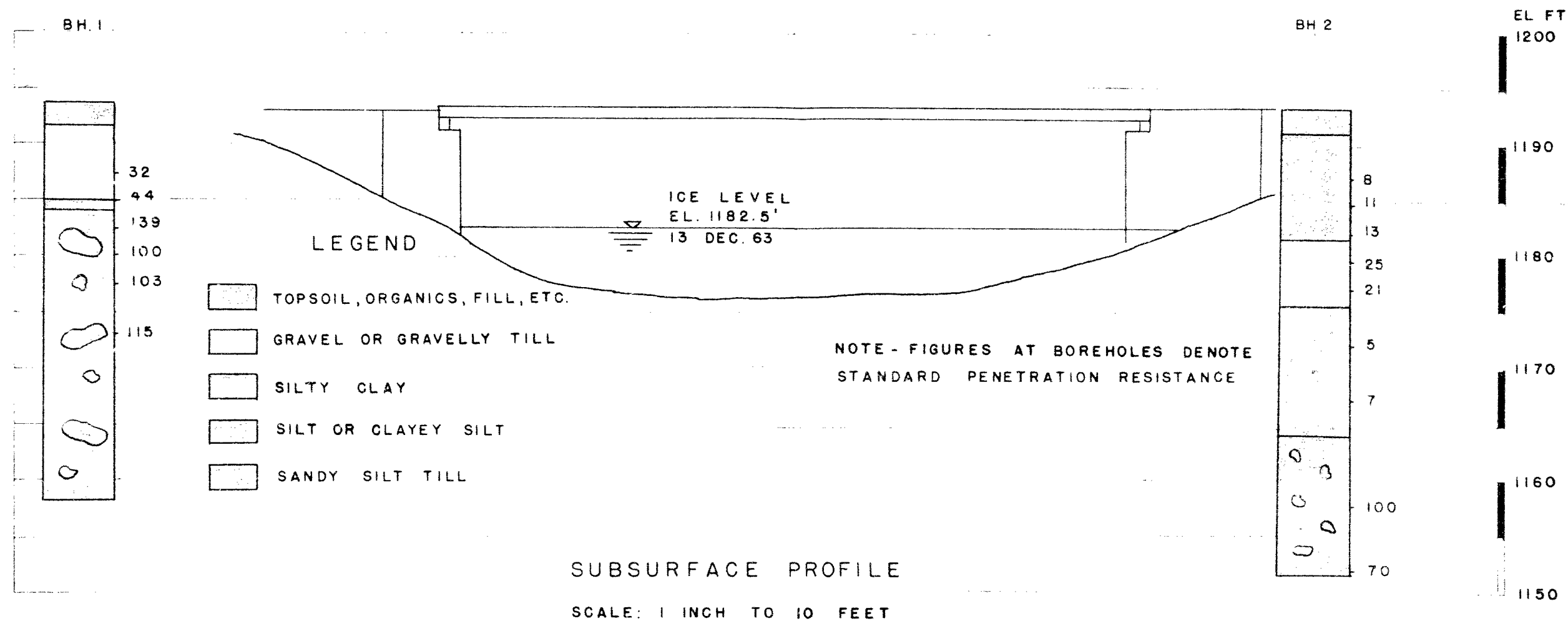
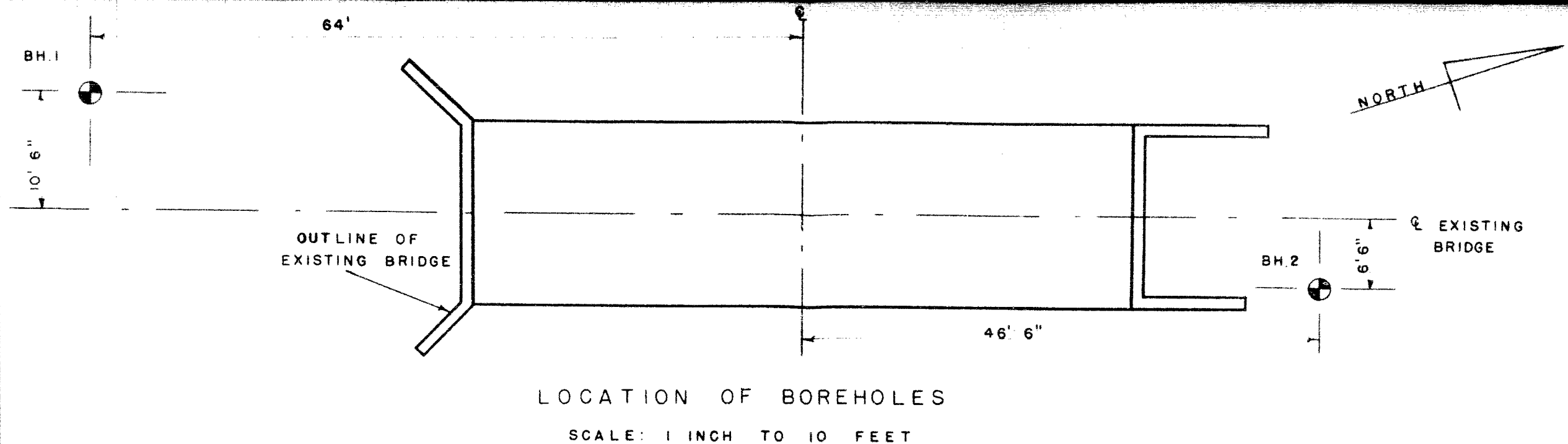
RD :

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

COHESIVE SOILS :

C lbs./sq.ft

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



# GEOTECHNICAL DATA SHEET FOR BOREHOLE 1.....

OUR REFERENCE NO. 3-12-10

CLIENT Mr. D. M. Ross  
PROJECT Road Bridge  
LOCATION Township of Mornington  
DATUM ELEVATION Geodetic

METHOD OF BORING Washboring  
DIAMETER OF BOREHOLE 6x (3-inch)  
DATE December 1963,  
January 1964.

ENCLOSURE NO. 2

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot		CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	N or Advance of Sampler	20	40	60	80	
1193.7		Ground Surface									
		Organic topsoil (gravelly).									
90	5	Brown gravelly till, very dense, damp.		1	SS	32					
85		Hard brown silty clay.		2	SS	44					
	10			3	SS	139					
80				4a	Rc Axt						
				4b	SS	100					
	15			5	SS	103					
75		Very dense grey sandy silt till, numerous cobbles or boulders, slight cohesion.		6	SS	115					
70	25			7	Rc Axt 10%						
				8	Rc Axt 2%						
65	30			9	Rc Axt 4%						
60	35	End of borehole									

2"  $\phi$  cond

Borehole was advanced by diamond drilling from 20'-5" to 35'-4". The bit encountered pressure throughout this depth, with increased pressure when cobbles or boulders were encountered.

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: SB

CH'D: JP

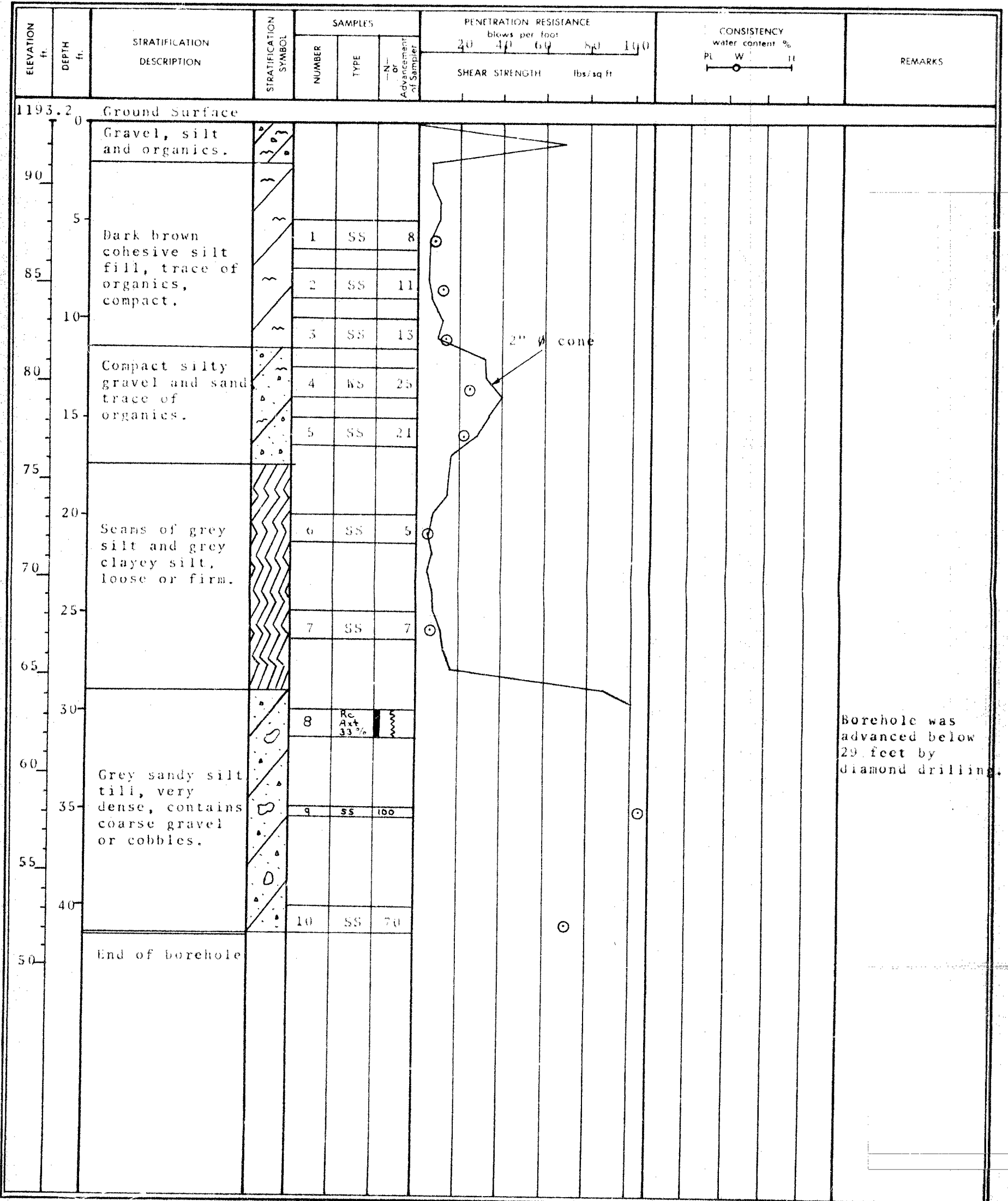
# GEOTECHNICAL DATA SHEET FOR BOREHOLE 3.....

OUR REFERENCE NO. 3-12-L0

CLIENT: Mr. B. M. Ross  
 PROJECT: Road Bridge  
 LOCATION: Township of Mornington  
 DATUM ELEVATION: Geodetic

METHOD OF BORING: Washboring  
 DIAMETER OF BOREHOLE: 5x (3-inch)  
 DATE: December 1963.

ENCLOSURE NO. 4



VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: SB

CH'D: JP