

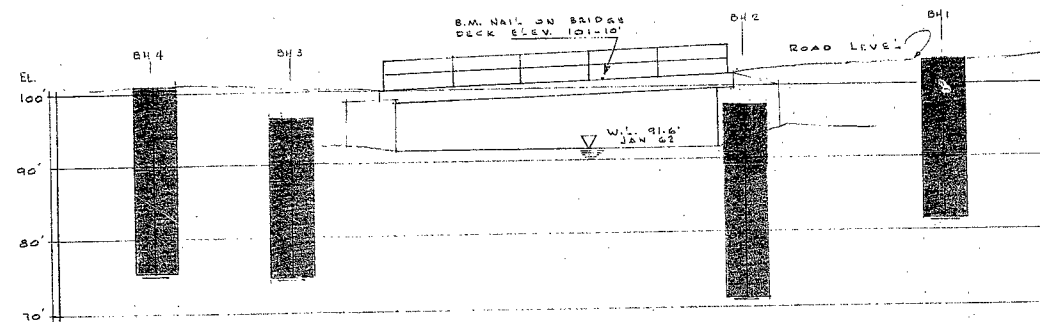
#62-F-308 M

ROAD BRIDGE

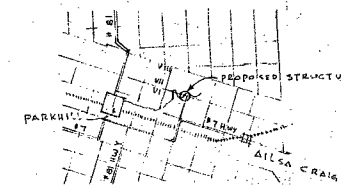
CON. VI/VII

WEST MCGILLIVRAY

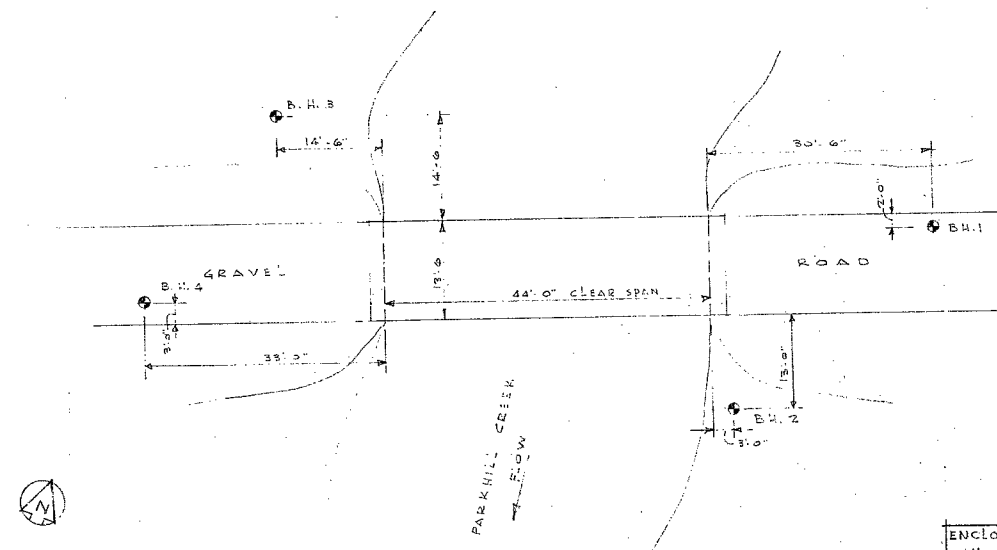
TOWNSHIP



SUBSURFACE PROFILE (LOOKING NORTH)
SCALE 1" TO 10'-0"



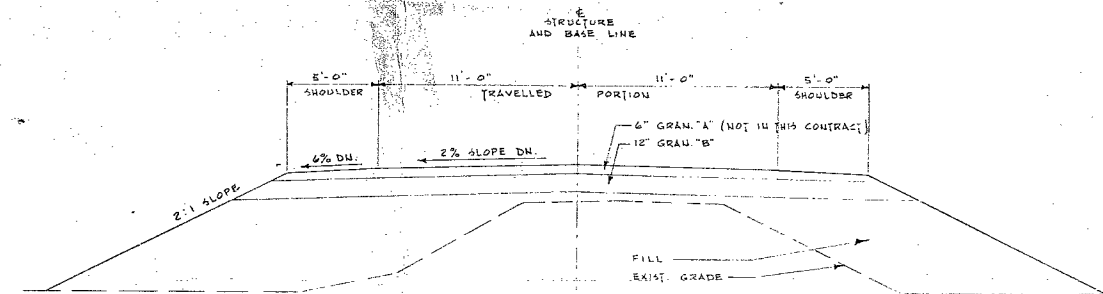
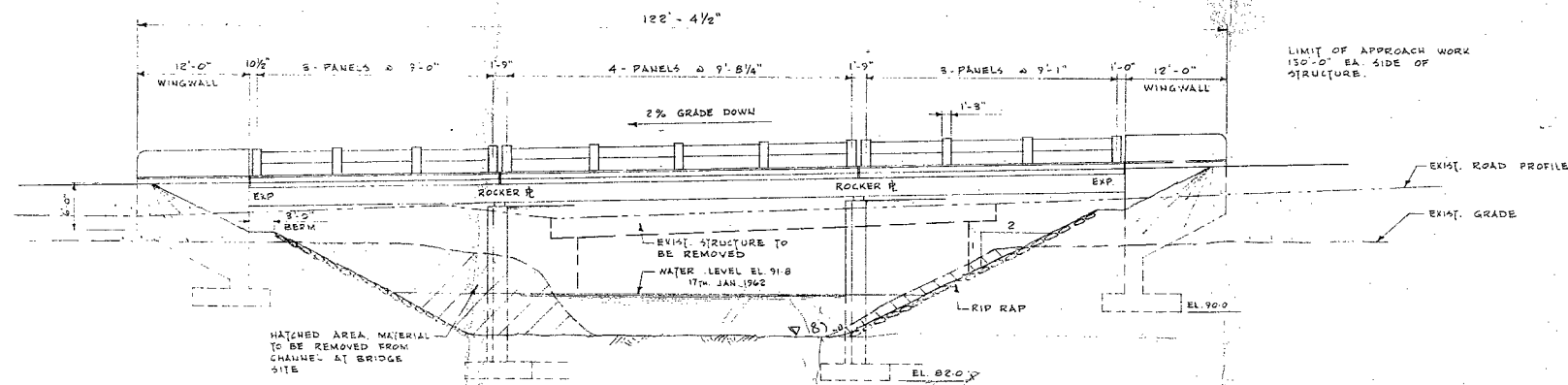
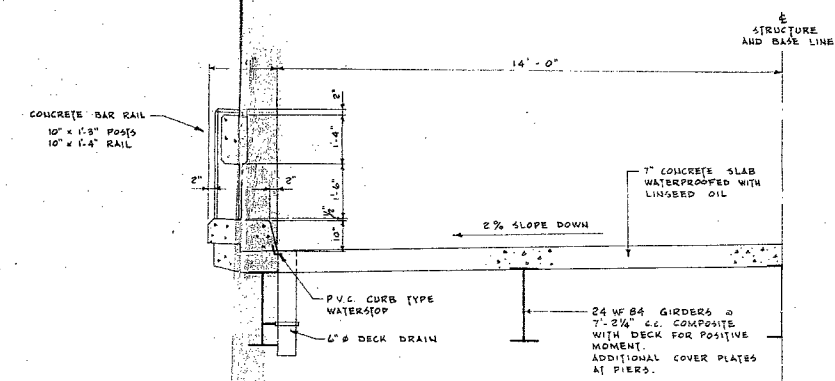
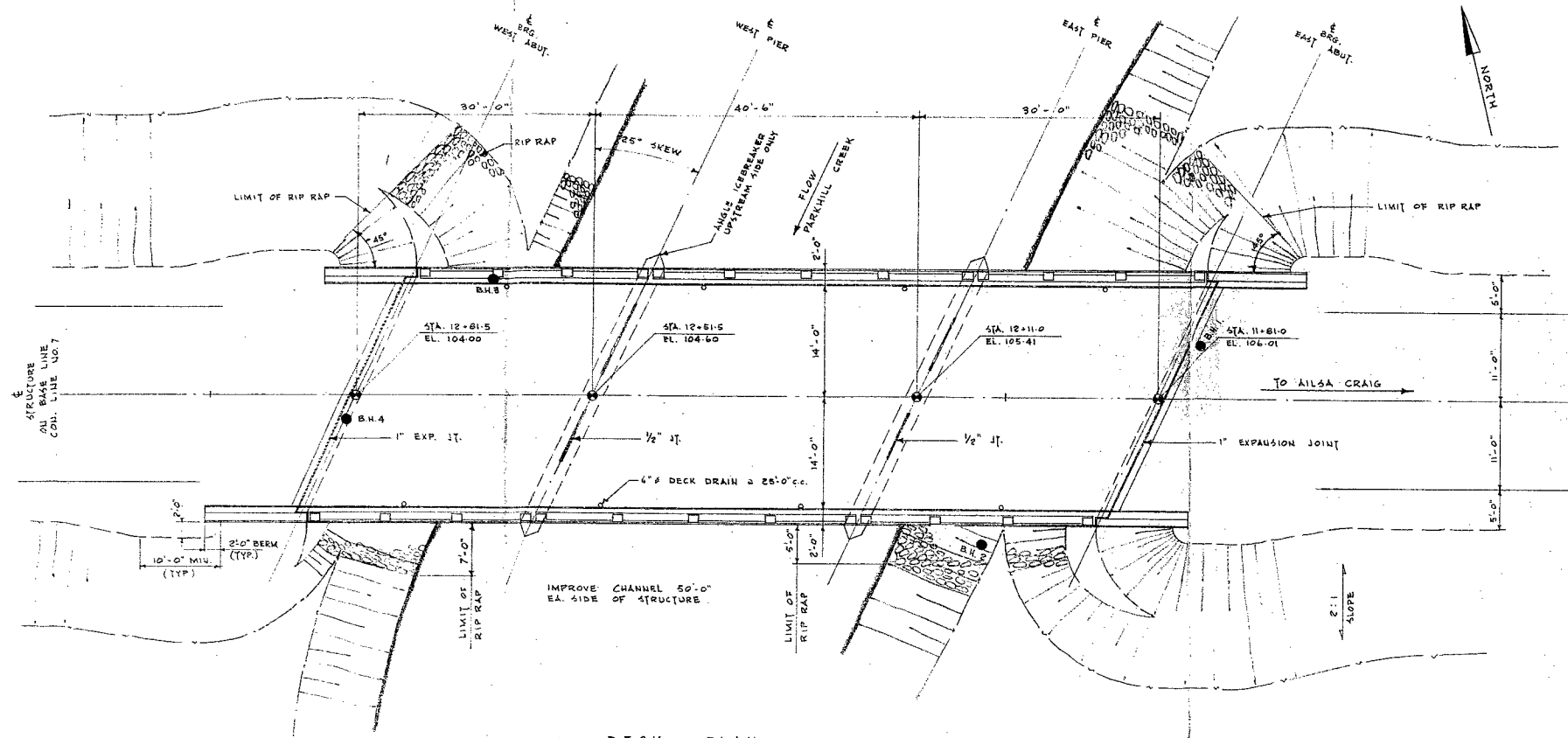
KEY PLAN
SCALE 1" TO 4 MILES



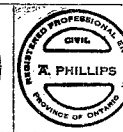
LOCATION PLAN
SCALE 1" TO 10'-0"

- LEGEND**
- BROWN CLAY SAND & GRAVEL
 - GREY CLAYEY SILT
 - BROWN SILT
 - GREY CLAYEY SILT TRACE OF ORGANICS

ENCLOSURE No 1	M. M. DILLON & CO. LTD.
	LONDON ONTARIO
REF. 2-1-14	SOIL INVESTIGATION
JAN. 1952	FOR
DRAWN BY J.T.	ROAD BRIDGES BETWEEN CONS. VI & VII
CHECKED BY J.P.	TOWNSHIP OF WEST MCGILLIVRAY
	DOMINION SOIL INVESTIGATION LIMITED
	363 QUEENS AVE. LONDON ONTARIO



NO.	DATE	DESCRIPTION	BY



SPECIFICATIONS
H20-44

M. M. DILLON & COMPANY
LIMITED
CONSULTING ENGINEERS
TORONTO

SCALE AS NOTED
DATE 6th FEB. 1962
DES. J.A.C. CHK. A.P.
DWG. L.L. CHK. A.P.
ENGR. IN CHARGE: A.P.
APPROVED:

GENERAL ARRANGEMENT
LINE 7 BRIDGE
(OVER PARKHILL CREEK)
LOT 10
TOWNSHIP OF WEST MCGILLIVRAY

JOB No. 5759-2
SHEET P-2
OF

Mr. A. M. Teye,
Bridge Engineer
Materials & Research Division,
(Foundation Section)

April 6, 1962.

REVIEW OF SOIL INVESTIGATION
REPORT BY DOMINION SOIL
INVESTIGATION, LTD.

(Bridge Office Ref. BA. 1367)

Attention: Mr. K. L. Kleinsteinber,
Municipal Bridge Liaison Engr.

Re: Road Bridge, Concessions VI & VII
Township of West McGillivray,
District No. 2

We have reviewed the report on the soil investigation for the above-mentioned structure, submitted by the Consultant, Dominion Soil Investigation, Ltd., and below, present our comments for your consideration:-

On Page 4 of the report, the Consultant presents a table of maximum allowable gross pressures at the different borehole locations. The extrapolation of the Standard Penetration results leading to the very pronounced variation of soil pressures at B.H. 1 does not seem to be warranted and we would suggest that the same allowable pressure of 5,500 p.s.f. be taken for both abutment footings for elevations 90 or below. This would not be a gross, but a net allowable pressure.

The same pressure can also be taken for the West pier, although it could be slightly increased, namely to 6,000 p.s.f. for elevation 84 or below. Here again, the value would represent a net pressure.

The investigation has, undoubtedly, shown that the soil in the area of B.H. 2 (East pier) is looser and a value of only 3,000 p.s.f. is suggested by the Consultant. This value again, has to be considered as a net pressure. On the basis of the presented factual information, no increase of this value is advised.

cont'd. /2 ...

Mr. A. M. Toye, Bridge Engr.
Attn: Mr. K. L. Kleinsteinber

April 6, 1962.

The bridge consultant, M. M. Dillon & Associates, has chosen elev. 90 for the abutment footings and elev. 82 for the pier footings. If there are no objections from the hydrological point of view, these elevations could be maintained.

AGS/MdeF
Attach.

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

cc: Foundations Office
Gen. Files.

P.S. -- Attached, we are sending your
copy of Soil Report.

MESSRS. M.M. DILLON & CO. LTD.
CONSULTING ENGINEERS
141 MAPLE STREET
LONDON ONTARIO

Report on
SOIL INVESTIGATION
for
ROAD BRIDGE
CONCESSIONS VI & VII
TOWNSHIP OF WEST MCGILLIVRAY

by
DOMINION SOIL INVESTIGATION LIMITED
363 Queens Avenue
LONDON ONTARIO

Reference No. 2-1-L4

January, 1962

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ENCLOSURES

	<u>No.</u>
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ENGINEERING DATA SHEETS	2 to 5
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INTRODUCTION

In accordance with a letter of authorization dated 12th of January, 1962, a soil investigation has been carried out at a site in the Township of West McGillivray where it is proposed to replace an existing single-span road bridge with a new 3-span structure. The road runs between Concessions VI and VII, and the site is approximately 4 miles east of Park Hill.

It was proposed that the piers for the new bridge would be carried on footings 5 or 6 feet below the stream bed, and that the abutments would be supported at a higher elevation if sufficient bearing capacity was found. The existing bridge has a clear span of 44 feet, and it was indicated that the new bridge would have spans of approximately 30, 40 and 30 feet.

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

I DESCRIPTION OF SITE AND GEOLOGY

The site lies in a depression on the west slope of the Wyoming Moraine which is one of a series of broad, high ridges comprised of glacial debris, running approximately parallel to the east shore of Lake Huron. The existing bridge spans the Park Hill Creek which flows southwards at this point along a slow meandering path.

II FIELD AND LABORATORY WORK

Field work was carried out during the period, 17th to 19th of January, 1962 and consisted of 4 boreholes at the locations shown on enclosure 1. The holes were placed as close as possible to the proposed positions of the piers and abutments of the new structure and were staggered so as to represent as much of the construction area as possible. The holes were advanced by driving and cleaning Bx casing without the use of wash water.

Dynamic cone penetration tests were made adjacent to each borehole and Standard Penetration Tests were made at frequent intervals, using a 2" O.D. split spoon. A constant driving energy was employed in both penetration tests using a 140 pound hammer dropping 30 inches. The former test provided a continuous record of the soil density and the latter, disturbed samples of the strata. The results of the field tests are recorded on engineering data sheets comprising enclosures 2 to 5.

Elevations have been referred to a benchmark on the deck of the existing bridge whose level was provided by the client's representative at the site.

Atterberg limits were determined for sample 4 from borehole 2 for the purpose of classification, and the following results were obtained:

Liquid limit	24.4%
Plastic limit	16.2%
Natural moisture content	23.0%
Plasticity index	8.2%

A grain-size analysis was performed on sample 5 from borehole 2 to provide a guide to the most suitable dewatering procedure. The results of this test are shown on enclosure 6.

III SUBSURFACE CONDITIONS

A subsurface profile is shown on enclosure 1. Apart from slight variations which are described in detail on the engineering data sheets, two principle strata were encountered:

- (i) a layer of brown, clayey sand and gravel of variable consistency extending from ground surface to approximately El. 90.0 feet. Much of this material in boreholes 1 and 4 is fill.
- (ii) below El. 90 (El. 87.7 at borehole 3) a bed of grey clayey silt. It will be seen from the results of the penetration tests that the consistency of this deposit is quite variable. It is stiffer at boreholes 1 and 3 than at 2 and 4, and there is no constant trend of penetration resistance with depth.

Water levels were recorded after completion of each hole and an average ground water level was determined corresponding approximately to the level of ice in the creek at the time of drilling viz., El. 91.6 feet.

IV BEARING CAPACITY AND SETTLEMENT

Footings should be located at or below the surface of the clayey silt layer, and in view of the variation in the properties of this stratum shown by the field penetration tests, the conditions at each hole will be discussed separately.

Abutment Footings - Boreholes 1 and 4

At borehole 1 the soil becomes progressively stiffer below El. 90 feet. There is sufficient bearing capacity at this level so that to limit the amount of excavation required, and to minimize dewatering problems, it is proposed that footings should be placed close to this elevation. The recommended soil pressures are:

El. (feet)	p.s.f.
90	4700
89	7000
88	8500

At borehole 4 there is a decrease in stiffness below El. 87 where the dynamic cone penetration test met refusal. It is proposed here that the footings should again be kept close to El. 90 to take advantage of the stiff upper layers. Recommended soil pressures are:

El. (feet)	p.s.f.
90	5500
89	5500
88	5500

The softer soil at lower elevations has sufficient bearing capacity to resist the vertical stresses caused by the above pressure on footings up to 10 feet in width, with no reduction in the factor of safety.

Pier Footings - Boreholes 2 and 3

The pier footings will be located at or below El. 84.0 feet.

At borehole 2 lower values were obtained in the Standard Penetration tests than at any other hole, and this is the only hole at which the dynamic cone penetration test did not encounter refusal. The Standard Penetration Test results in the clayey silt layer are approximately constant with depth, indicating a gradual decrease in stiffness or density. (At a constant stiffness or density the effect of increasing overburden should increase the penetration resistance.) This decrease is approximately compensated by an increase in bearing capacity with depth provided by the overburden. By the foregoing reasoning it is deduced that the allowable soil pressure does not vary significantly with depth, and a value of 3000 p.s.f. is recommended.

At borehole 3 the stiffness of the soil increases down to El. 81 feet, and thereafter decreases. A soil pressure of 6000 p.s.f. is proposed in the range El. 81 to El. 85 feet. As in the case of borehole 4 the bearing capacity of the soil at lower elevations has been checked against the vertical stresses created by a footing 10 feet wide at El. 81 or higher.

The foregoing recommended soil pressures are based on the correlation of Terzaghi and Peck between Standard Penetration resistance and bearing capacity. They are gross pressures, i.e. they include the weight of overburden. For convenience the results are summarised in the following table:

Maximum allowable Soil Pressures (p.s.f.)

Elevation (feet)	BH 1	BH 2	BH 3	BH 4
90	4700			5500
89	7000			5500
88	8500			5500
84		3000	6000	
83		3000	6000	
82		3000	6000	
81		3000	6000	

The total settlement associated with the above figures is not expected to exceed one inch, and most of this will occur during construction.

V

CONSTRUCTION

The principle problems associated with construction will be the dewatering of excavations and the avoidance of disturbance to the footing grade. Surface water can be diverted through a culvert or temporary channel, but constant seepage of ground water into the excavation should be expected. The grain-size analysis made on sample 5 from borehole 2 suggests that the material is probably too fine to be effectively dewatered by vacuum well points. It is proposed, therefore, that the excavations for the piers should be isolated using sheet piles driven deep enough to ensure hydraulic stability of the grade, i.e., the weight of submerged soil between the grade level and the tip of the piles should be sufficient to offset the unbalanced hydraulic pressure. Thus, for example, if the water level is 91.0 feet and the grade level 82.0 feet, the unbalanced pressure will be $62.5 \times 9 = 563$ p.s.f. The submerged weight of the soil may be taken as 60 p.c.f. so that the piles should be driven $563/60 = 9.4$ feet below the grade level. The excavation may be kept dry by pumping from a sump dug below the level of the grade, and to assist in guiding seepage water into this sump while avoiding disturbance to the grade, a thin layer of fine gravel should be spread on the bottom of the excavation.

At the location of the abutments dewatering may not be necessary if the footings are kept as high as possible (El. 90.0 feet). This will depend on the groundwater elevation at the time of construction, which will probably be influenced (i.e. depressed) by the excavation for the piers. If the water level should persist at an elevation above the footing grade, the procedure recommended for the pier footings should be followed.

Care should be exercised at all times to avoid disturbance of the silty soil which is inherently sensitive to the effects of vibration or free water on the exposed grade. If, for any reason, the grade should become disturbed or is found to include unusually soft material, the soil in such a case should be cut out and backfilled with lean concrete. Any form of compaction should be avoided. The consequences of disturbance to the footing grade for any reason may be settlements of a much higher order than those predicted.

VI

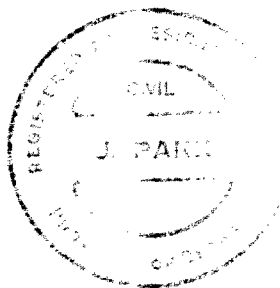
SUMMARY

1. The principal strata encountered were (1) a layer of brown, clayey sand and gravel extending to El. 90.0 feet approximately and (2) below this level a grey clayey silt.
2. Ground water was encountered at an average elevation corresponding to that of the ice level in the creek. (El. 91.6 feet)

3. A wide variation was found in the properties of the clayey silt layer in which the footings will be located. The bearing capacity of the soil at each borehole is discussed separately, and a summary of the recommended soil pressures is given on page 4.
4. Considerable care should be exercised during construction to avoid disturbance to the footing grade. A recommended dewatering procedure is outlined in Section V.

VII REFERENCES

1. The Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam of the Ontario Research Foundation - University of Toronto Press.
2. Procedures for Testing Soils, ASTM, April 1958. pp. 186 to 198. (Unified Soil Classification System - by A.A. Wagner).
3. Terzaghi and Peck: Soil Mechanics in Engineering Practice. John Wiley and Sons, New York 1948.



DOMINION SOIL INVESTIGATION LIMITED

James Park

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

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 1

Date 17 JAN 61

Project: ROAD BRIDGE

Location: WEST MCGILLIVRAY TWP.

Hole Locations: SEE ENCLOSURE 1

Hole Elevation and Datum:

Field Supervisor: J. P. Prep.: J. T.

Driller: G. G. Checked: J. P.

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S):

Penetration Resistance {P}

2" Split tube

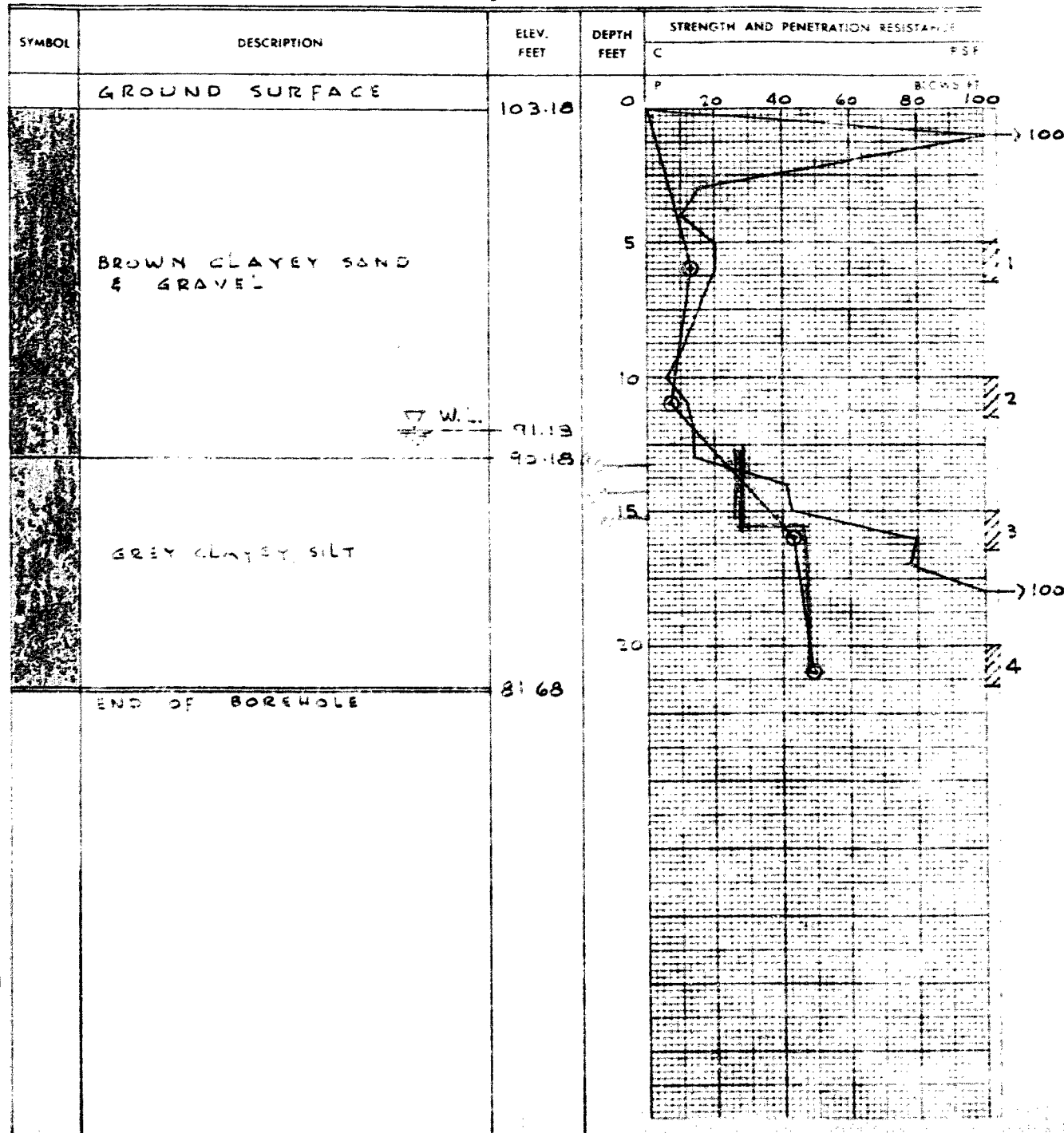
2" Dia. Cone

Casing

Sensing Method

2" Dia spiral tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 2

Date: 16/17 JAN. 62

Project: ROAD BRIDGE

Location: WEST MCGILLIVRAY TWP.

Hole Location: SEE ENCLOSURE 1

Hole Elevation and Datum:

Field Supervisor: J. P.

Prep.: J. T.

Driller: G. G.

Checked: J. P.

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

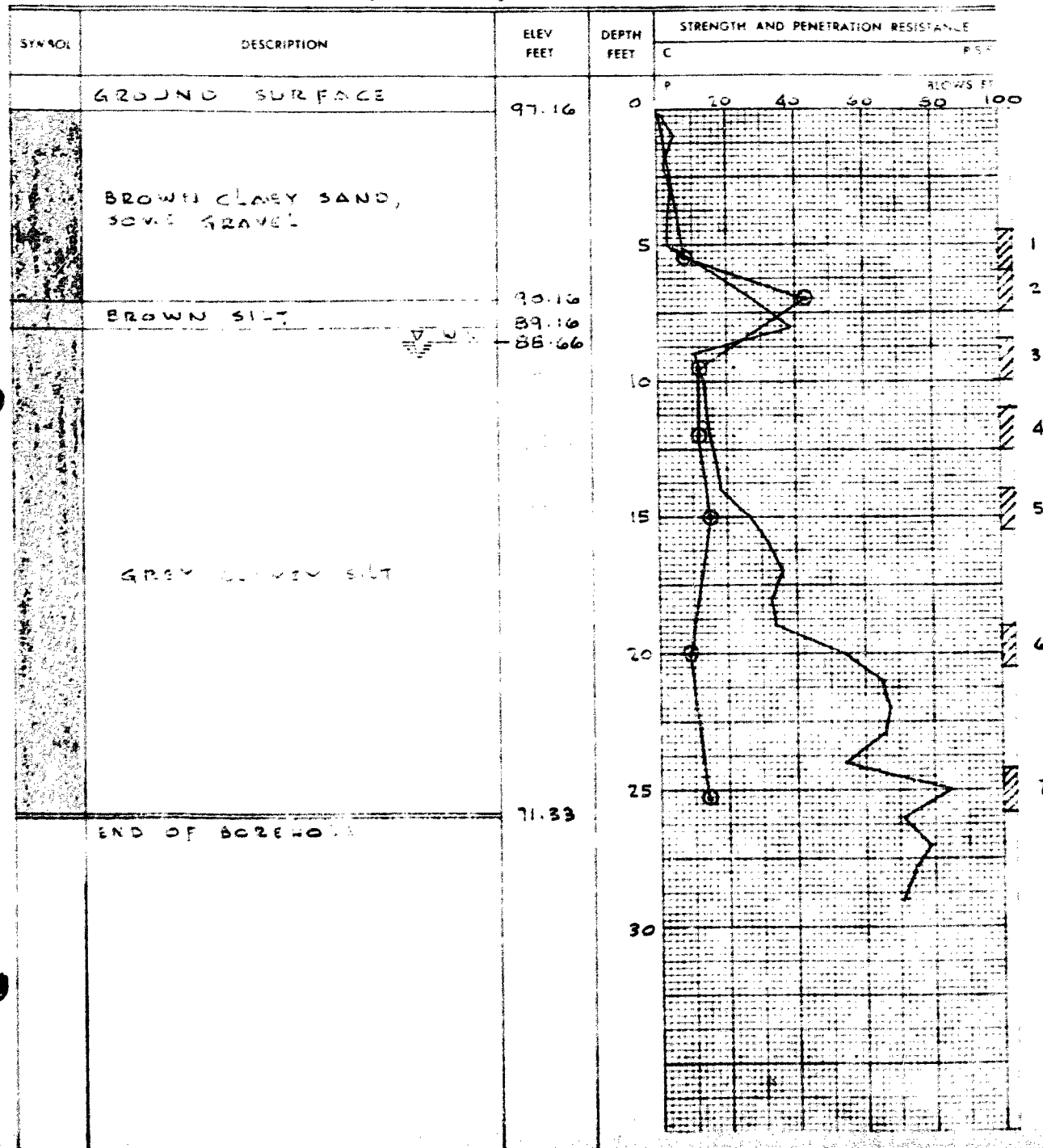
2" Dia. Cone

Casing

Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 3

Date: 17 JAN. 62

Project: ROAD BRIDGE
 Location: WEST MCGILLIVRAY TWP.
 Hole Location: SEE ENCLOSURE 1
 Hole Elevation and Datum:
 Field Supervisor: J.P. Prep.: J.T.
 Driller: G.G. Checked: J.P.

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

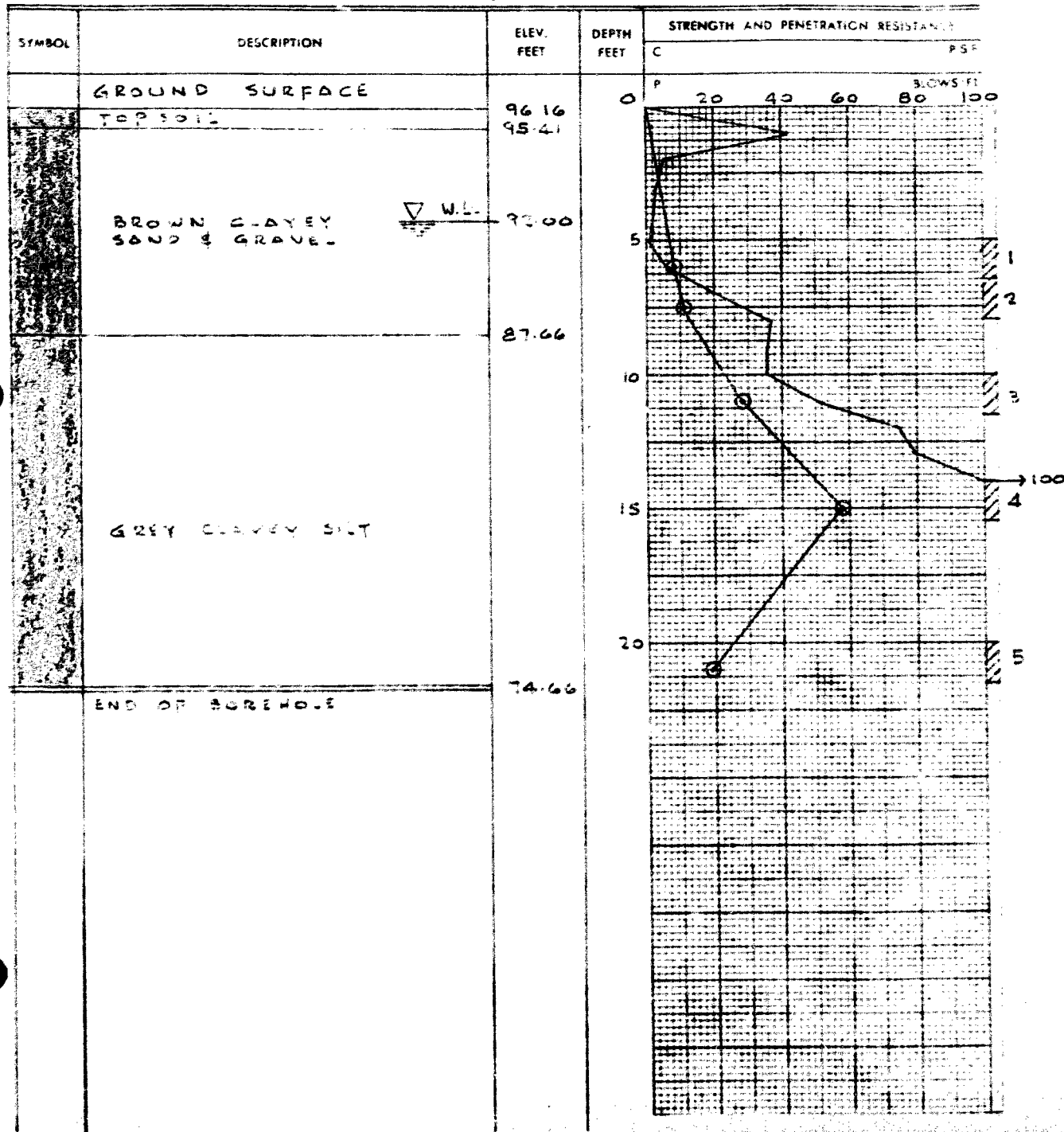
Casing

⊕
+S⊕
+S

Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 4

Date: 18.19 JAN. 62

Project: ROAD BRIDGE

Location: WEST MCGILLIVRAY TWP.

Hole Location: SEE ENCLOSURE #1

Hole Elevation and Datum:

Field Supervisor: J. P. Prep.: J. T.

Driller: G. G. Checked: J. P.

LEGEND

Shear Strength (C)

Unconfined compression
Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

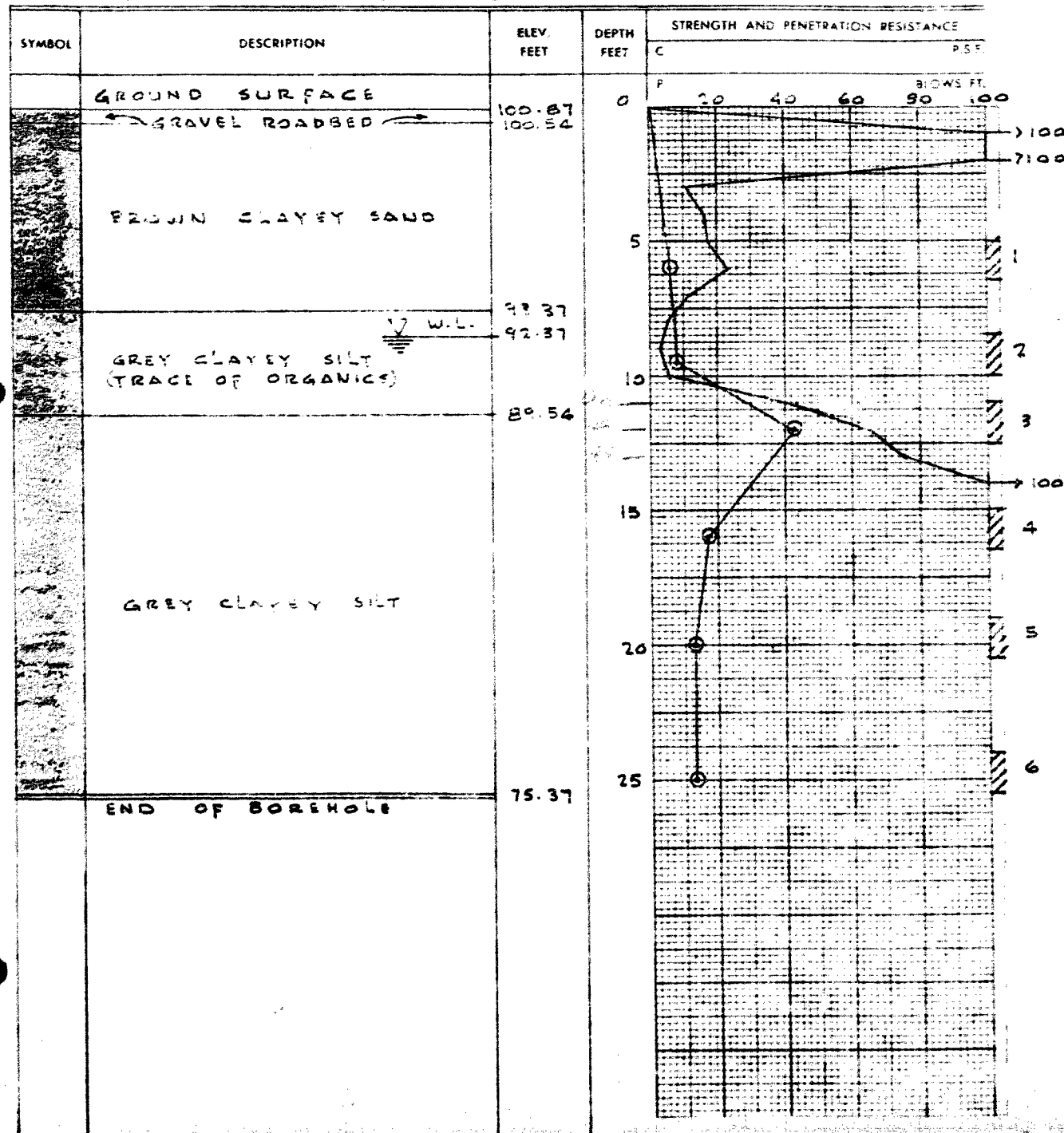
2" Dia. Cone

Casing

Sampling Method

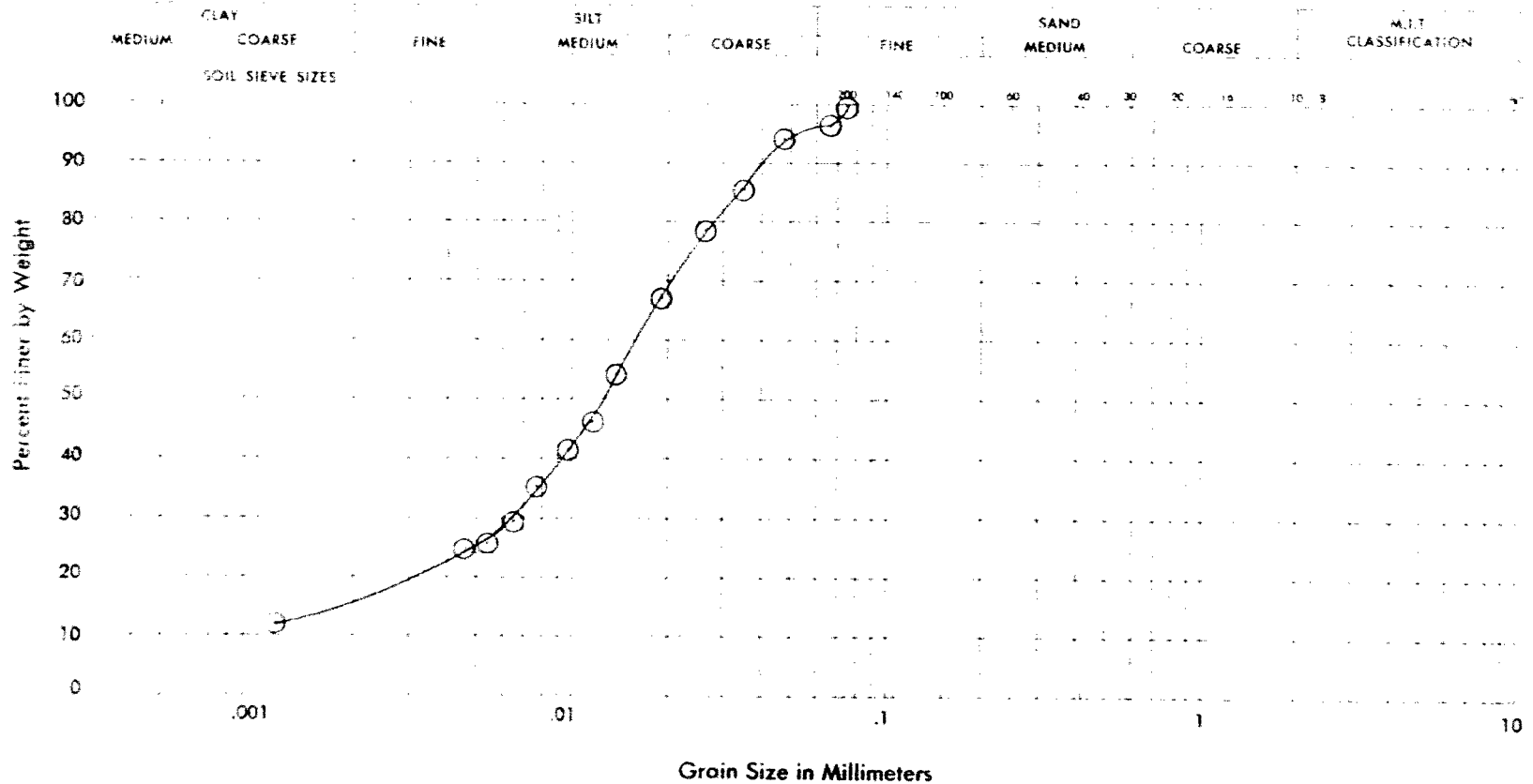
2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

GRAIN SIZE DISTRIBUTION



Project Road Bridge, Township of
West McGillivray

Grain Size in Millimeters

Borehole 2
Sample 5

Order No. 2 1 L-1