

59-F-259C

Hwy. # 401

BRIDGE #14

RACEY, MACCALLUM AND ASSOCIATES
LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

Consulting Engineers
AND ASSOCIATED STAFF

MONTREAL



VANCOUVER

TORONTO

DONALD C. MACCALLUM, B.ENG., M.E.I.C., P.ENG.

H. JOHN RACEY, B.Sc., M.E.I.C., P.ENG.

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TORONTO DIVISION
27 CARLTON STREET

Reference: S-506/T-1571

Toronto 2, Ontario,
March 10, 1959.

59-F-259C

Department of Highways of Ontario,
Bridge Office,
280 Bavenport Road,
TORONTO, Ontario.

Attention: Mr. J. McAllister

RE: SOIL INVESTIGATION FOR PROPOSED BRIDGE #14,
HIGHWAY #401 - WATERLOO TOWNSHIP
DISTRICT #13, ONTARIO.

Dear Sirs:

The enclosed report presents the results of our soil investigation at the above location.

We hope the report is satisfactory to you; if you have any questions about it please do not hesitate to get in touch with us.

Thank you for this opportunity of being of service to you.

Yours very truly,

RACEY, MACCALLUM AND ASSOCIATES LIMITED


J. J. Schoustra, P.Eng.
Divisional Soil Engineer

JJS/KA

Department of Highways of Ontario
Bridge Office
280 Denison Street
Toronto

SOIL INVESTIGATION FOR PROPOSED BRIDGE #14,

HIGHWAY #401 - WATERLOO TOWNSHIP,

DISTRICT #13, ONTARIO.

Reference: S-500/T-1591

Racey, MacCallum and Associates Ltd.

March 10th, 1959.

SUPER IMPOSED DOCUMENT MAY
APPEAR AS MULTI-FEED ON FILM.

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TORONTO DIVISION
27 CARLTON STREET

Reference: A-500/T-1591

Toronto 2, Ontario,
March 10, 1959.

SOIL INVESTIGATION FOR PROPOSED BRIDGE #14, HIGHWAY #401 - WATERLOO TOWNSHIP DISTRICT #24, ONTARIO

3

Introduction

It is proposed to construct a bridge for the extension of Highway #401 over Blair County Road in the vicinity of Preston, Ontario.

This investigation was undertaken in order to obtain a comprehensive picture of the prevalent subsoil conditions and to advise on the bridge foundation design in the light of these conditions.

Fieldwork

Drilling was initially carried out at a site located in relation to the intersection of Blair Road, at chainage 15+00, and Highway #401 'A' as indicated on D.H.O. Drawing No. E-3500-1. This location was later found to be erroneous. The correct location of the site was indicated by Mr. Jordan, Waterloo County Engineer, at chainage 52+50 on Blair Road.

Accessibility to the site was extremely bad and much time was spent whilst hauling plant and equipment onto the site. Operations continued throughout in high winds and in temperatures well below zero. Much delay was incurred due to these adverse weather conditions.

borings were carried out, at the locations shown on Enclosure #1, by means of a truck mounted standard diamond drill rig which was specially equipped for soil sampling. Samples were recovered, by means of a 2" diameter split spoon sampler, for identification and classification purposes. The split spoon sampler was driven into the soil by means of a 140-lb. hammer falling freely

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Reference: E-100/T-1593

March 10, 1959

Fieldwork - Continued

a distance of 30 inches. The number of blows necessary to drive the sampler one foot is termed the standard penetration resistance. This value bears an established empirical relationship to the relative density of the subsoil, a measure of which is an aid in ascertaining allowable bearing pressures. A 60-degree point angle cone was driven into the soil to refusal at all four locations. This gave a record of the change in soil density with depth up to refusal. It was only deemed necessary to sink borholes and take soil samples at two diagonal locations. The extremely gravelly and bouldery nature of the soil rendered it impossible to advance the holes without diamond drilling. In such material the wear on diamond set bits is very high as compared to the amount of footage gained.

The ground surface elevation at all four locations was obtained and referred to the B.M. Elevation 963.83 feet Geodetic Datum, located in the west of the 1.0 foot elm, 235.0 feet left of Station 361+00 Highway /Hwy "A" as indicated on Plan E-3500-1.

Soil Conditions

Boring profiles and all pertinent data are plotted on Engineering Data Sheets for each borhole, and included as Enclosures Nos. 2, 3, 4, 5.

From these records it can be seen that the correlation of profiles and data from all four locations is good. These show low density to about 10 feet and then a gradual and steady increase with depth. Particularly close agreement between two pairs of penetration records was noted, i.e. Borholes Nos. 1, 4, and 2, 3. This agreement was indicative in deciding that only two borholes be required.

The site is located in an area known geologically as the Waterloo Sand Hills. In this region the dominant formation is that of kame consisting entirely of gravel and sand. Around the base of these kame hills exist areas of outwash sand.

The soil transected in the investigation consists of a mixture of outwash material overlying original kame material.

Four feet of surface organic silt material was transected in Borhole #2, but was not located or seen to be indicated at any of the other three locations. The profiles show a generalized soil section at the site to consist of loose to dense coarse sand and gravel with some silt down to Elevation 950 feet. From this elevation to Elevation 943 feet, a layer of dense to very dense brown fine sand, trace of silt with some coarse sand and gravel existed before the soil reverted to a very dense deposit of sand, gravel and boulders in a brown silt binder. Thus the whole of the soil transected was essentially granular and below Elevation 950 feet in a dense to very dense state.

Reference: S-2897-1531

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Soil Conditions - Continued

The water table was not contacted at either of the two locations. Owing to the large amount of surface melt water together with drilling wash water which was trapped around the drill site, it was impossible to locate the water table with any accuracy. However, taking into consideration the presence of small pools and ponds in the immediate vicinity, it was decided that the water table was at approximately Elevation 961 feet.

Foundations

From a study of the penetration records it was felt preferable to specify bearing pressures at different elevations for definite areas, i.e.: those pertinent to Boreholes #1 and 4 and Boreholes #2 and 3.

The penetration records indicate that the density of the first 10 feet of soil is variable over the site, after which all four records become in accord. Owing to the relatively high coarse gravel and boulder content of the soil, the penetration records are likely to err on the dense side. This together with the effect of submergence were taken into consideration whilst assessing bearing pressures allowable at various elevations.

In the region of Boreholes #2 and 3 footings could be installed at a higher elevation than for the other holes.

Allowable bearing pressures at different elevations for the two areas are specified in the table presented below.

Borehole	Average depth below ground	Elevation	Allowable Bearing Pressure
1 & 4	14 feet	950 feet	4,000 p.s.f.
	16 feet	948 feet	6,000 p.s.f.
	19 feet	945 feet	8,000 p.s.f.
2 & 3	6 - 7 feet	658 feet	3,000 p.s.f.
	14 - 15 feet	950 feet	5,000 p.s.f.
	16 - 17 feet	948 feet	6,000 p.s.f.
	19 - 20 feet	945 feet	8,000 p.s.f.

These pressures are recommended on the basis of a 1-inch maximum total and $1/4$ " maximum differential settlement, which have been found to be tolerable for conventional structures.

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Reference S-592/2-1591

As the ground water is apparently as high as around Elevation 961 feet, then the working area has to be dewatered so that excavation and placement of footings can proceed in the dry.

A single-stage wall point system would suffice and work efficiently in the granular material. Some difficulty might be encountered whilst installing the points owing to the presence of coarse gravel and boulders. A pump arrangement might be employed whilst installing footings at the more shallow depths.

The bridge could be installed on end bearing piles driven to refusal at around Elevation 935 feet. Such piles would be in the order of 30 feet long. Timber piles would be difficult to key into the refusing stratum. Thus, bearing in mind that batter piles would have to be installed to accommodate the side thrust from the abutment fill, precast concrete or steel piles would be preferable. In the case of construction point the use of steel piles would be preferable so that adjustments in length could be easily made as a result of differing refusal elevations.

With such good bearing strata available, the use of spread footings would seem more economic and preferable.

Construction of the bridge will entail placing up to 30 feet of approach fill against both abutments. Computation of lateral pressures for footing and abutment design is usually based on the assumption that the granular fill is in the neutral or at rest state. Whence:

$$P = K \times \frac{1}{2} H^2 \cdot \gamma$$

where: H = height of fill in feet

γ = density of fill p.s.f.

P = lateral thrust in pounds

K = assumed as 0.4 for granular back-fill.

In this particular instance the lateral thrust will be in the order of 11 tons.

Considering a rigid frame structure then placing granular fill against the abutment up to finished grade will cause the logs to bow inwards. This bowing will produce an "arching" effect in the granular fill, which will in turn produce a non-linear pressure distribution. With a linear distribution the resultant acts at $H/3$ from the foot of the abutment, however with the transformed distribution this resultant could act at $H/2$ feet though the magnitude of the thrust will remain appreciably the same.

In the case of a non-rigid structure most of the yielding will take place at the top of the abutment at the point of contact with the deck structure. For this condition it has been determined that a deflection of $.001 \times H$ is sufficient to transform the back-fill from the at-rest to the active state whereby, in this case, a value of $K = 0.3$

Reference S-580/2-1971

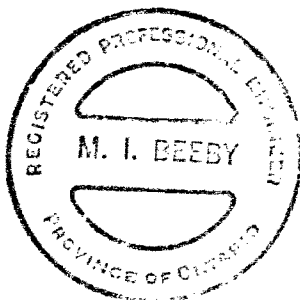
March 10, 1979

would be allowable. However, the tilting of the abutment will be constrained to a certain extent by the action of the deck. Therefore it is suggested that the magnitude of the side thrust still be computed on the basis of $K = 0.4$, and the thrust be considered to act at a height of $H/3$.

Conclusions

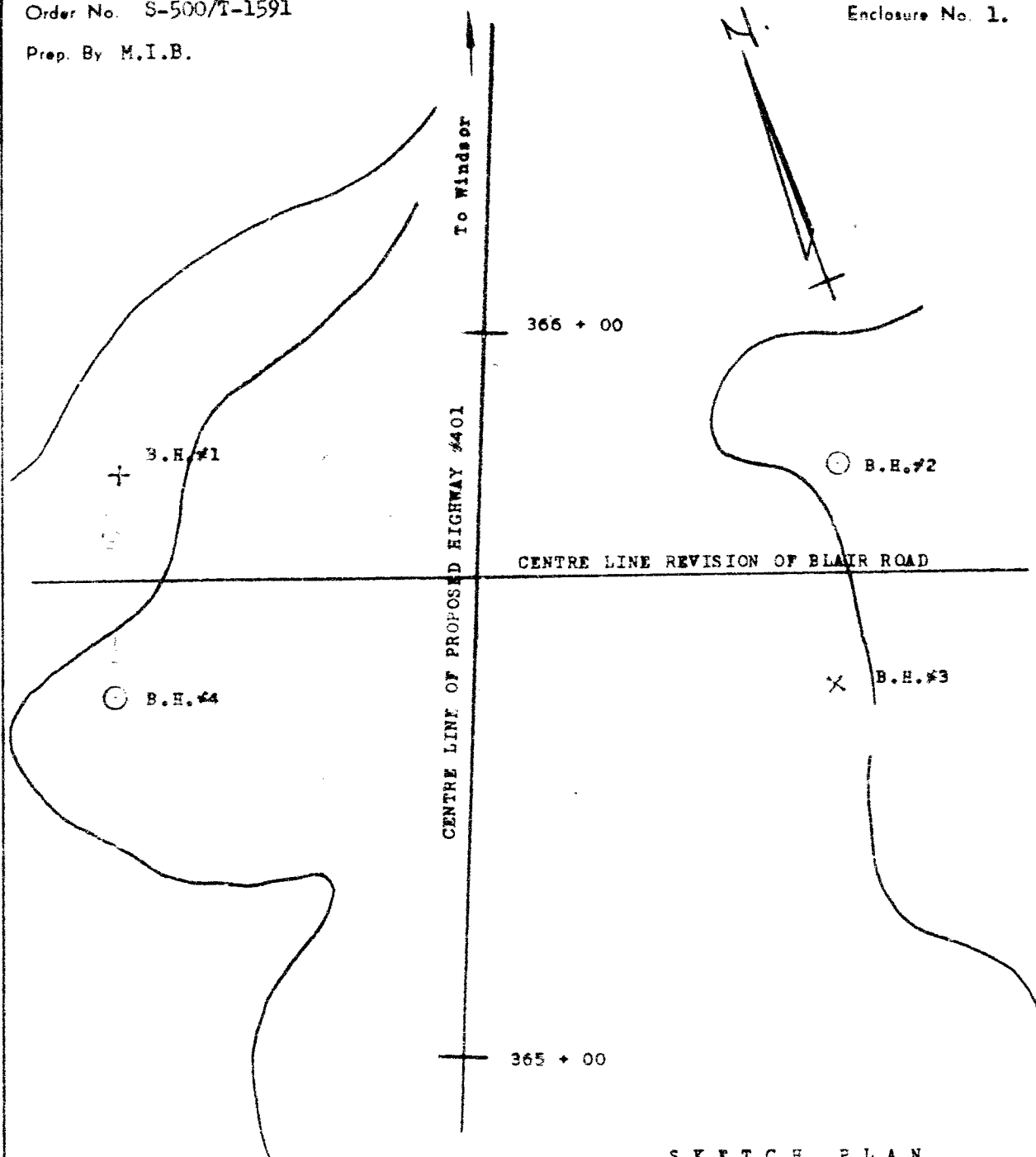
From the results of the investigation and their subsequent interpretation the following conclusions have been derived:

1. The subsoil is essentially granular consisting of coarse sand and gravel to fine sand with trace of silt.
2. Above elevation 950 feet the soil density is variable but below this elevation the soil is in a dense to very dense state.
3. The bridge structure may be founded on spread footings. Allowable bearing pressures at various elevations are specified within this report.
4. As an alternative, the structure may be founded on steel piles, approximately 30 feet long, driven to refusal. Better piles would be required to accommodate the lateral thrust from the abutment fill.
5. Ground water level appears to be at Elevation 661 feet.
6. A single-stage well point system should be installed to enable footings to be formed in the dry.
7. Computation of the lateral pressures on the abutments should be carried out assuming the fill to be in the at-rest state and due consideration should be given to the relevant points in the presented discussion.



M. I. Beedy, P.Eng.,
Project Engineer.

MIR/KA



SKETCH PLAN

PROPOSED BRIDGE SITE #14,
HIGHWAY #401, WATERLOO TOWNSHIP,
DISTRICT 12, ONTARIO.

LOCATION OF BORINGS

Scale: 1 inch to 20 feet.

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for ~~Standard~~ **Cone #1.**

Project: **Proposed Bridge Site #14,**
 Location: **Highway #401, Waterloo Township.**
 Hole Location: **See Enclosure No. 1.**
 Hole Elevation and Datum: **963.4 ft.**
 Field Supervisor: **H.E.** Prep.: **M.I.B.**
 Driller: _____ Checked: **J.S.** Date: _____

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

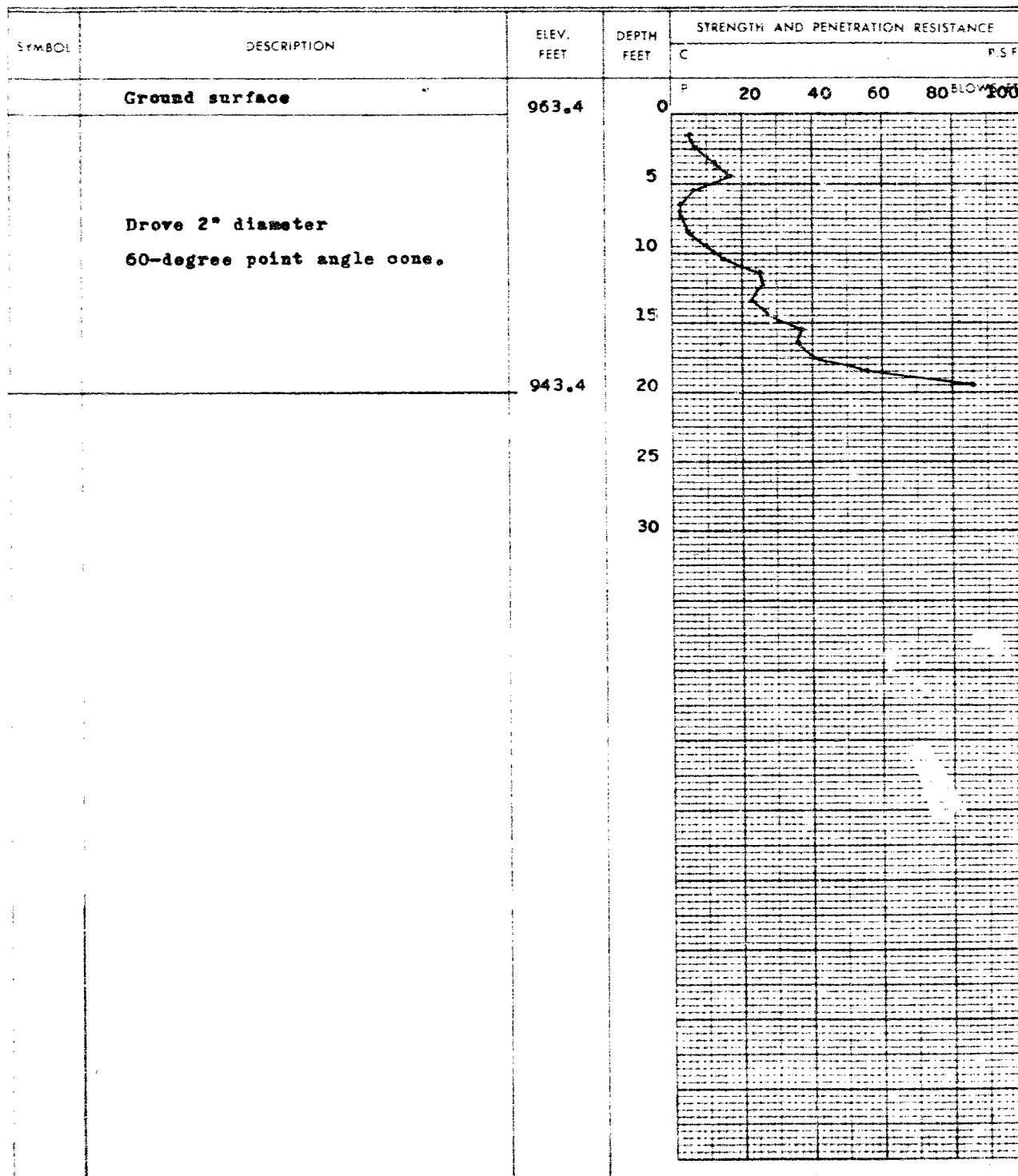
2" Split tube

3" Dia. Cone

Casing

⊕
+5

⊕ ⊕



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Foundation Engineering Division

Engineering Data Sheet for Borehole: 2.

Project: Proposed Bridge Site #14,
 Location: Highway #401, Waterloo Township.
 Hole Location: See Enclosure No. 1.
 Hole Elevation and Datum: 964.5 ft.
 Field Supervisor: R.H. Prep.: M.I.B.
 Driller: Checked: J.S. Date:

LEGEND

Shear Strength: C

Unconfined compression

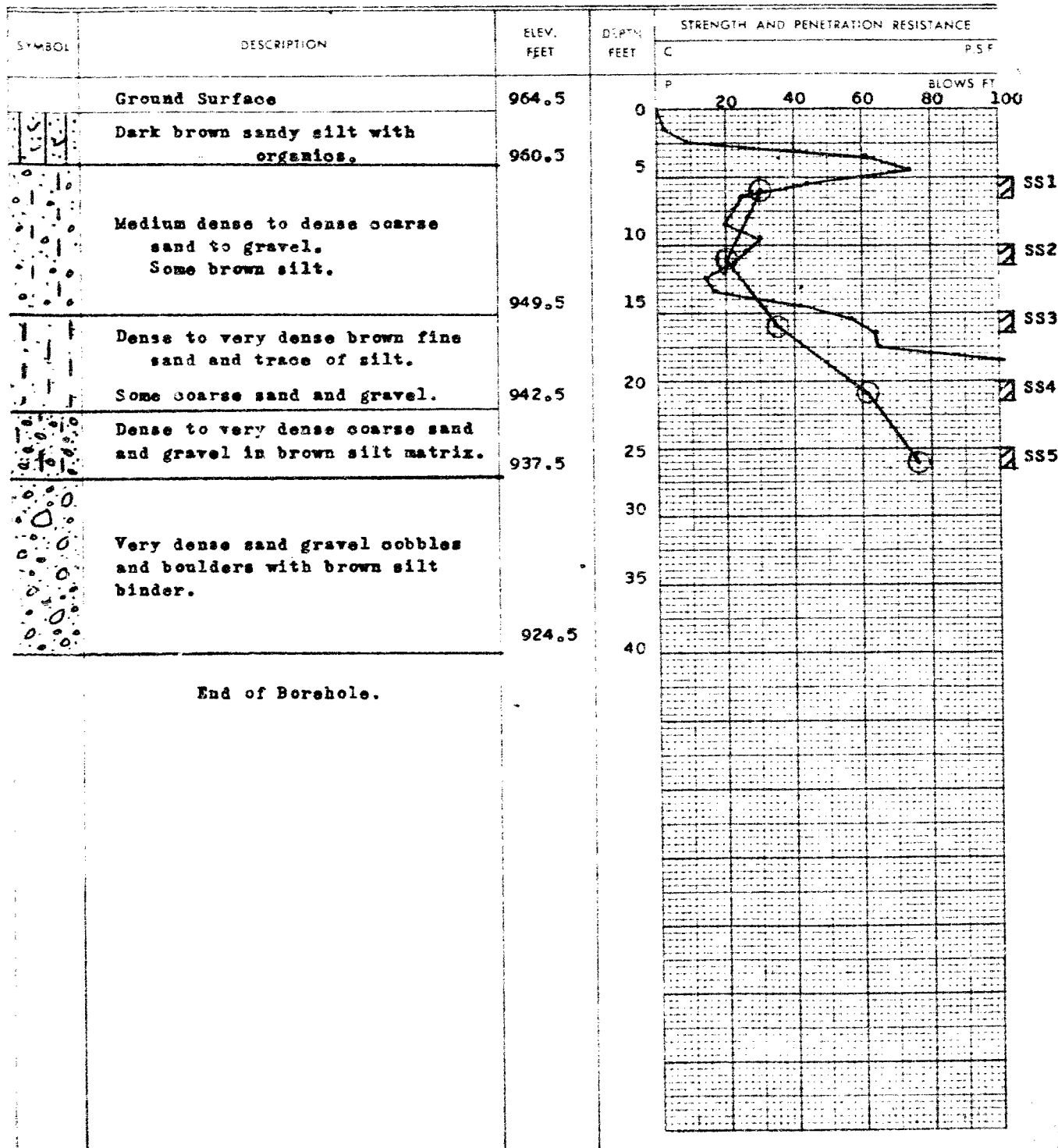
Vane test and sensitivity: S_v

Penetration Resistance: P

2" Split tube

2" Dia. Cone

Casing



RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for ~~Structure~~ Cone #3

Project: Proposed Bridge Site #11,
 Location: Highway #401, Waterloo Township.
 Hole Location: See Enclosure No. 1.
 Hole Elevation and Datum: 964.4 ft.
 Field Supervisor: R.F. Prep.: M.I.B.
 Driller: Checked: J.S. Date:

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

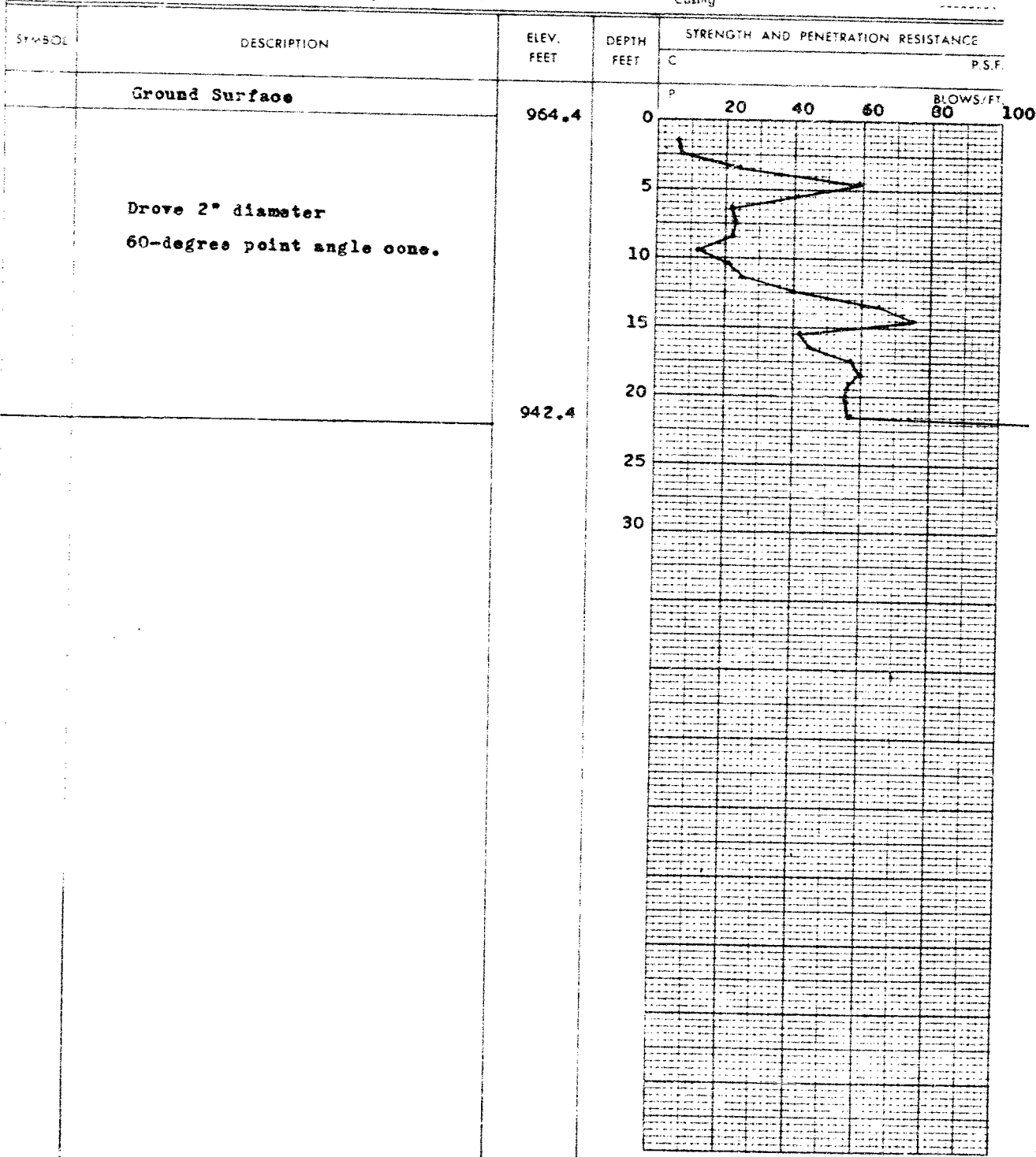
2" Split tube

2" Dia. Cone

Casing

⊕
+3

⊕ ⊕



Foundation Engineering Division

Engineering Data Sheet for Borehole: 4.

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S):

Penetration Resistance : P

2" Split tube

2" Dia. Cone

Casing

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				C	P.S.F.
	Ground Surface	963.4	0		
	Loose to medium dense brown sand and gravel. Trace of silt.	950.4	10		
	Dense brown fine sand. Trace of silt.	943.4	20		
	Very dense brown silt, trace of clay. Packed densely with sand and gravel.	936.9	25		

Graph showing Penetration Resistance (C) and Strength (P.S.F.) versus Depth (Feet). The graph includes a grid with depth from 0 to 40 feet and strength from 0 to 100 P.S.F. The data points are labeled SS1 through SS6, corresponding to the soil layers described in the table.