

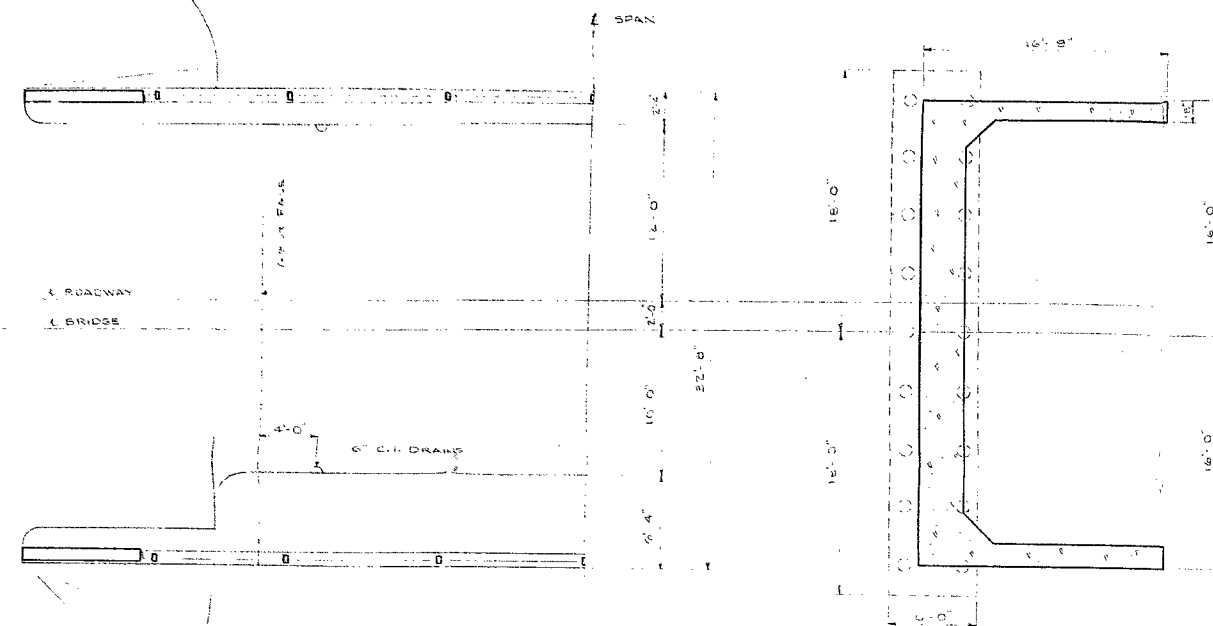
#61-F-25/M

TWP. ROAD BRIDGE

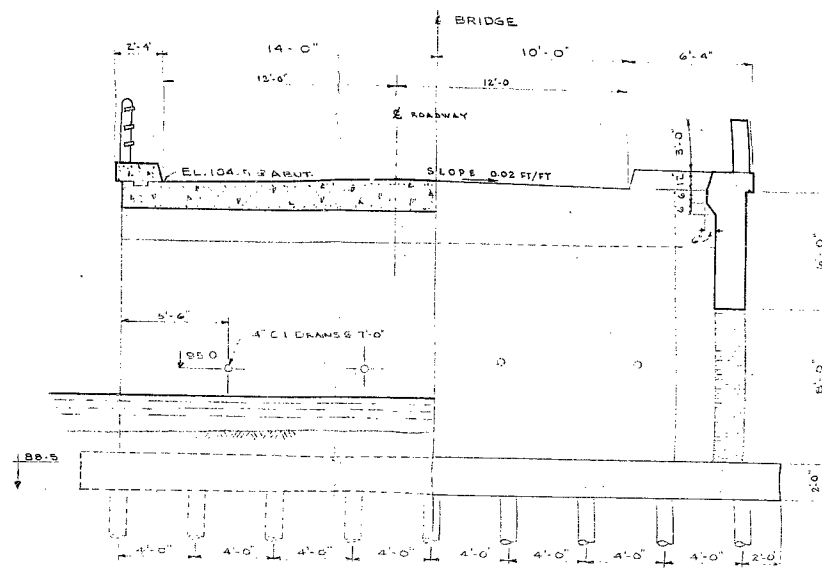
CON. VII/VIII

LOTS 16/17

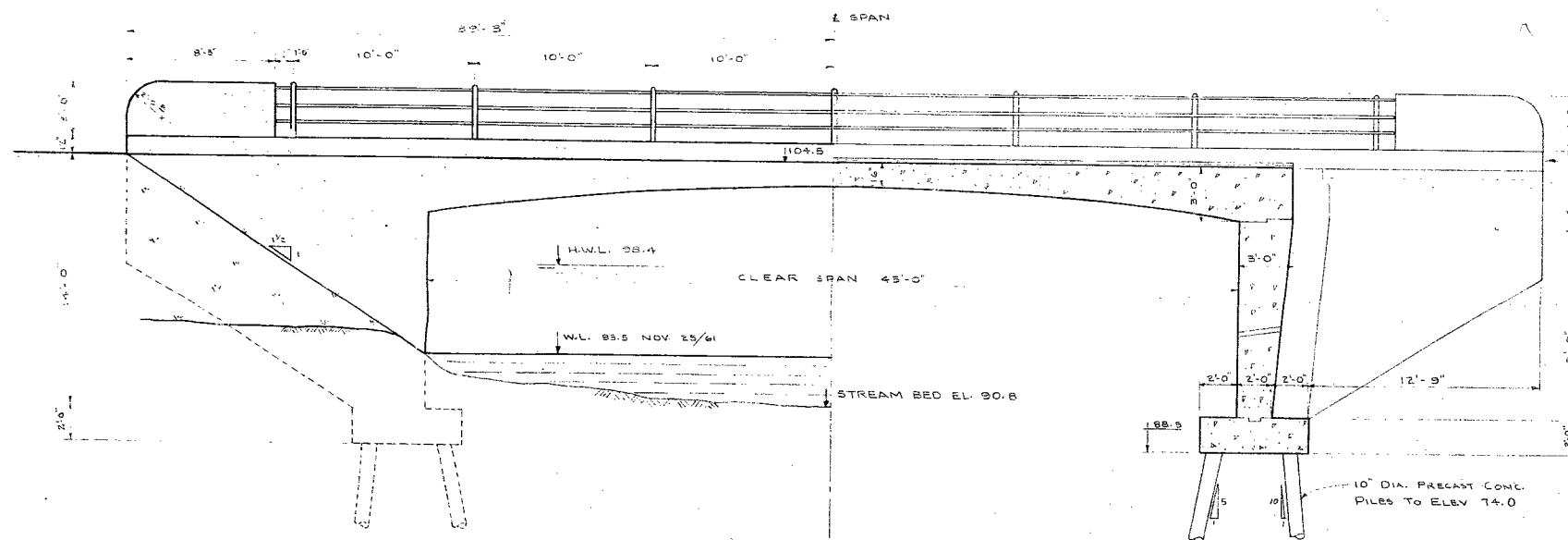
LOBO TWP.



HALF PLAN SCALE $\frac{1}{8}'' = 1'-0''$ HALF SECTION



HALF CROSS SECTION HALF END VIEW SCALE $\frac{1}{4}'' = 1'-0''$



HALF ELEVATION SCALE $\frac{1}{4}'' = 1'-0''$ HALF SECTION

FOR PRELIMINARY
APPROVAL

BEAR CREEK BRIDGE			
TOWNSHIP OF LEE	APPROVED BY	JOB NO.	DRAWN BY
SCALE AS SHOWN	DATE	6121	REVIEWED
GENERAL PLAN & SECTIONS			
A. M. SPRIET & ASSOCIATES CONSULTING ENGINEERS LONDON, ONTARIO			DRAWING NUMBER 2

BA 1348

MR. A. M. SPRIET
CONSULTING ENGINEER
234 QUEENS AVENUE
LONDON ONTARIO

Report on
SOIL INVESTIGATION

for

TOWNSHIP ROAD BRIDGE
(Concessions VII/VIII, Lots 16/17)

TOWNSHIP OF LOBO

by

DOMINION SOIL INVESTIGATION LIMITED
363 Queens Avenue
LONDON ONTARIO

Reference No. 1-3-L6

October, 1961

61-F-251

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INTRODUCTION

In accordance with verbal authorisation from Mr. A. M. Spriet, a soil investigation has been conducted at a site in Lobo Township where it is proposed to replace an existing township road bridge with a new structure. The existing bridge has a span of 43'-6" and a width of 13'-6".

The purpose of the investigation was to reveal the sub-surface conditions and determine the necessary soil properties for the design and construction of foundations.

I. DESCRIPTION OF SITE & GEOLOGY

The site is located in Lobo Township about 3 miles north of Lobo, and near the source of the East Branch of the Sydenham River. It lies in the broad shallow valley of an early glacial stream which flowed southward between the Seaforth Moraine on the west and the Lucan Moraine on the east. The surrounding district is agricultural, partly wooded and sparsely populated.

II. FIELD WORK

Field work was carried out on the 30th and 31st of August and the 1st and 5th of September, 1961, and comprised 2 boreholes at the locations shown on enclosure 1. Dynamic cone penetration tests were made adjacent to each borehole. The holes were advanced by washboring, and Standard Penetration Tests were made at frequent intervals using a 2 inch O.D. split spoon. A constant driving energy was employed in the Standard Penetration and dynamic cone tests using a 140 pound hammer dropping 30 inches. The former test provided disturbed samples of the strata and the latter a continuous record of soil density.

The results of the field tests are recorded on engineering data sheets comprising enclosures 2 and 3. Elevations have been referred to the top of the south-west wing-wall of the existing bridge, which has been assigned the arbitrary elevation 100.00 feet.

III. SUBSURFACE CONDITIONS

A subsurface profile is shown on enclosure 1. Five strata were encountered within a depth of 30 to 40 feet.

- (i) Embankment fill, consisting of a brown silty clay containing traces of sand, gravel and organics, extended to a depth of 7 feet.
- (ii) Below this for 2 to 3 feet the soil is an irregular mixture of rounded and subangular gravel, sand, clay and silt. The clay and silt fraction may rise as high as 20%, but the layer is predominantly granular, and thus pervious. Its condition is compact.
- (iii) A layer of stiff grey clayey silt till, 4 feet thick in borehole 1 and $1\frac{1}{2}$ feet thick in borehole 2. This material contains traces of rounded gravel, 10 to 15% of medium angular sand and sufficient clay to make it cohesive. Samples exhibited considerable dry strength.

- (iv) Starting at 12 to 13 feet a deposit of dense, grey-brown sand extends to approximately 30 feet. The grain size varies from fine to coarse, there are traces of rounded gravel and occasional seams of silt and clay 1 or 2 inches thick.
- (v) Below the sand layer is a very dense stratum of almost pure grey silt in which both boreholes were terminated.

Water was encountered in the boreholes at the same elevation as the stream, i.e. 92.9 feet

IV. ANALYSIS

The choice of foundation will be influenced not only by the bearing capacity of the soil, but by the feasibility of carrying out the work in the prevailing subsurface conditions. Three types of foundation have been considered, and each is discussed in this section.

(a) Spread Footings

On the basis of the Standard Penetration tests the following values are recommended for maximum allowable soil pressure under spread footings.

	Depth (feet)	Elevation (feet)	Soil Pressure (p.s.f.)
Borehole 1	7	93	1600
	8	92	2000
	9 or lower	91 or lower	4000
Borehole 2	7	92.5	1000
	10	89.5	1800
	11 or lower	88.5 or lower	4000

Providing the footings can be placed on an undisturbed grade, settlement is not expected to exceed one inch for footings up to 7 feet wide. For wider footings the allowable soil pressure should be reduced by 5% for every additional foot of width.

The most favourable level for the footings is the top of the clayey till layer. In the gravel layer above, the comparatively low allowable soil pressures would necessitate very wide footings and in the sand layer below it would be almost impossible to prevent "boiling" of the grade resulting from an unbalanced hydraulic pressure.

The chief difficulty involved with the construction of this type of foundation will be the control of ground water. Because of the pervious gravel deposit at the level of the creek bed, it may be necessary to drain an extensive area to obtain a dry excavation. This would certainly be expensive and might not be technically feasible.

A solution may be provided by the layer of clayey till lying between the gravel and sand deposits and the sketches shown on enclosure 4 indicate a possible method of forming the footings without disturbing the grade. It is proposed that sheet piles be driven into the till just far enough to form a water seal - say 6 inches. At the location of borehole 1, the till is thick enough to provide sufficient shear strength to resist the unbalanced hydraulic pressure once water has been pumped from the excavation - providing the width of the excavation does not exceed about 6 feet. At borehole 2 where the till is only 1'-6" thick, the soil would probably heave if the excavation were pumped dry. The excavation must therefore be made underwater and 2 to 3 feet of tremie concrete placed on the bottom. When this has hardened the excavation can be dewatered without danger of heaving.

The procedure outlined above will require considerable care and good judgment by the contractor, and some risk is involved in that the till layer may turn out to be thinner in places than was revealed by the borings.

(b) Displacement Piles (wood or precast concrete)

The dewatering problem can be avoided by the use of piles driven into the sand layer. The piles must penetrate through the gravel and till layers, or consolidation of the latter will occur, resulting in settlement which cannot be calculated.

Piles of 10 inch diameter driven to elevation 77 feet (10 feet into sand) may be loaded to 15 tons (provided a satisfactory set is achieved in accordance with an established dynamic pile driving formula). One ton may be added to the working load for each additional foot of penetration. If the pile spacing is not closer than 3 diameters, the allowable load for the pile group may be taken as the working load per pile multiplied by the number of piles. Thus for a loading of 15,000 pounds per lineal foot, two staggered rows of piles at 4 feet centres would be suitable.

For other sizes of pile the following approximate formula may be used:

$$Q_s = \frac{1}{3} (64A + 0.32F) = \text{Working load in tons}$$

where A is the cross-sectional area in feet and F is the circumferential (friction) area within the sand layer.

If timber piles are used they should be provided with a steel shoe. There will still be some risk of the piles breaking before they penetrate the till layer. In this respect precast concrete piles would be more reliable.

The total settlement of the pile group cannot be estimated without loading tests, but in view of the reasonably uniform subsoil conditions, little differential settlement would be expected between abutments.

(c) Steel Piles

The use of steel H-sections would overcome any difficulty in penetrating the upper layers, although the piles would have to be longer to develop sufficient bearing capacity on the smaller cross-sectional area. The highest suitable elevations are El. 71 feet at borehole 1 and El. 64 feet at borehole 2. At these or lower elevations the working loads may be calculated from the approximate formula

$$Q_s = \frac{1}{3} (200 A + 0.2F) \text{ tons}$$

where A and F are as defined previously. Thus for a 10" x 8" H-section with a cross-section of 16.2 inches the working loads are 12 tons at borehole 1 and 13.5 tons at borehole 2. (Again these figures are subject to satisfactory sets being achieved in accordance with dynamic pile driving formulae).

(d) Resume

The use of spread footings will probably be the cheapest solution, but there is some risk attached in that the base of the excavation may "boil" when subjected to an unbalanced hydrostatic pressure.

Timber or concrete piles will perform satisfactorily providing they can be driven without damage. There is some doubt of this in the case of timber piles. The use of steel piles will overcome difficulties in driving, but they would

* The figures and formulae quoted here and in section (c) which follows are based on the work of G.G. Meyerhof. See reference 4.

need to be longer and of smaller working load, and thus more expensive.

Precast concrete piles appear to strike the best balance between technical soundness and cost.

V. SUMMARY

1. The strata encountered were successive layers of fill, gravel and clayey silt till to an average depth of 12 to 13 feet, overlying a stratum of dense sand to a depth of 30 feet, and thereafter a dense grey silt.
2. Ground water was encountered in the boreholes at the same elevation as in the river, and will constitute the main factor in determining the type of foundation to be used.
3. A full analysis is made of the capacity of the soil to support pile and spread footings. Precast concrete piles appear to offer the best solution.

JP:nr



DOMINION SOIL INVESTIGATION LIMITED,

A handwritten signature in dark ink, appearing to read "James Park".

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

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. p.p. 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.
4. G.G. Meyerhof, Standard Penetration Tests and Bearing Capacity of Cohesionless Soils. Am. Soc. C.E. Paper 866, 1956.

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole:

Date: 30/31 AUG 61

Project: ROAD BRIDGE

Location: L30 TOWNSHIP

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

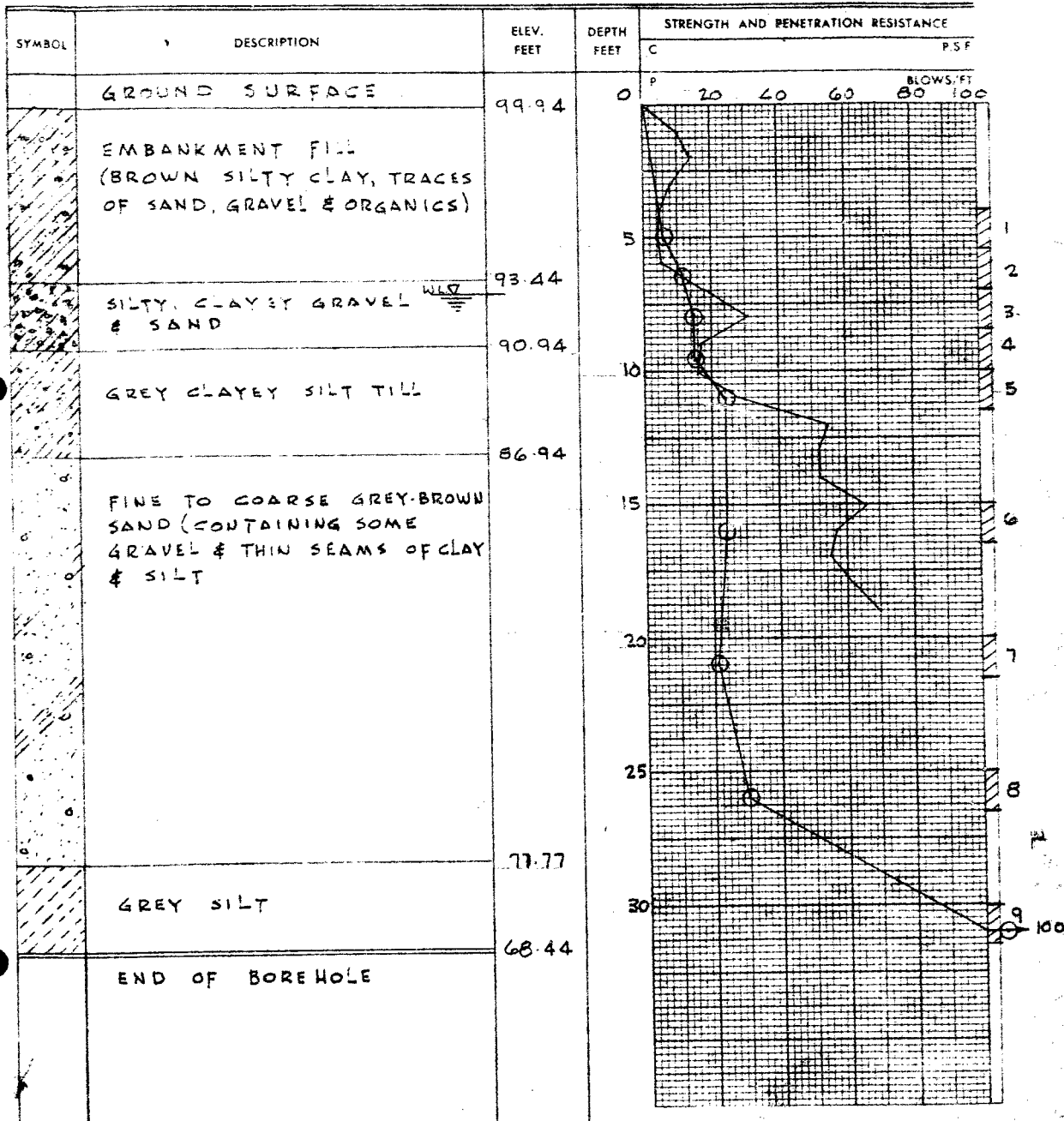
2" Dia. Core

Casing

Sampling Method

2" Dia. split tube

2" Shelby tube



Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 2

Date: 1/5 SEPT 61

Project: ROAD BRIDGE
 Location: LOBO TOWNSHIP
 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)

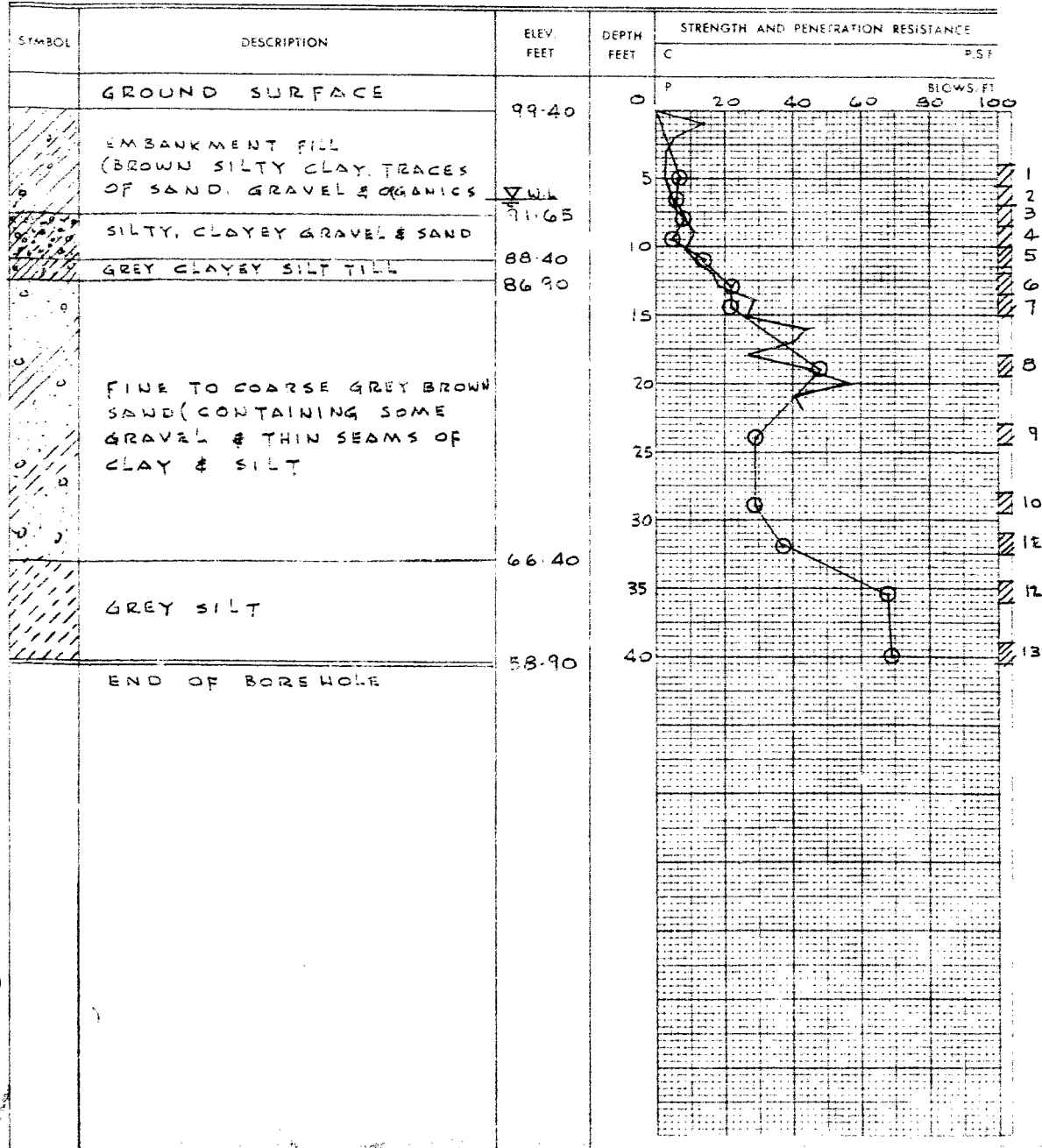
Unconfined compression \oplus
 Vane test and sensitivity (S) \oplus^s

Penetration Resistance (P)

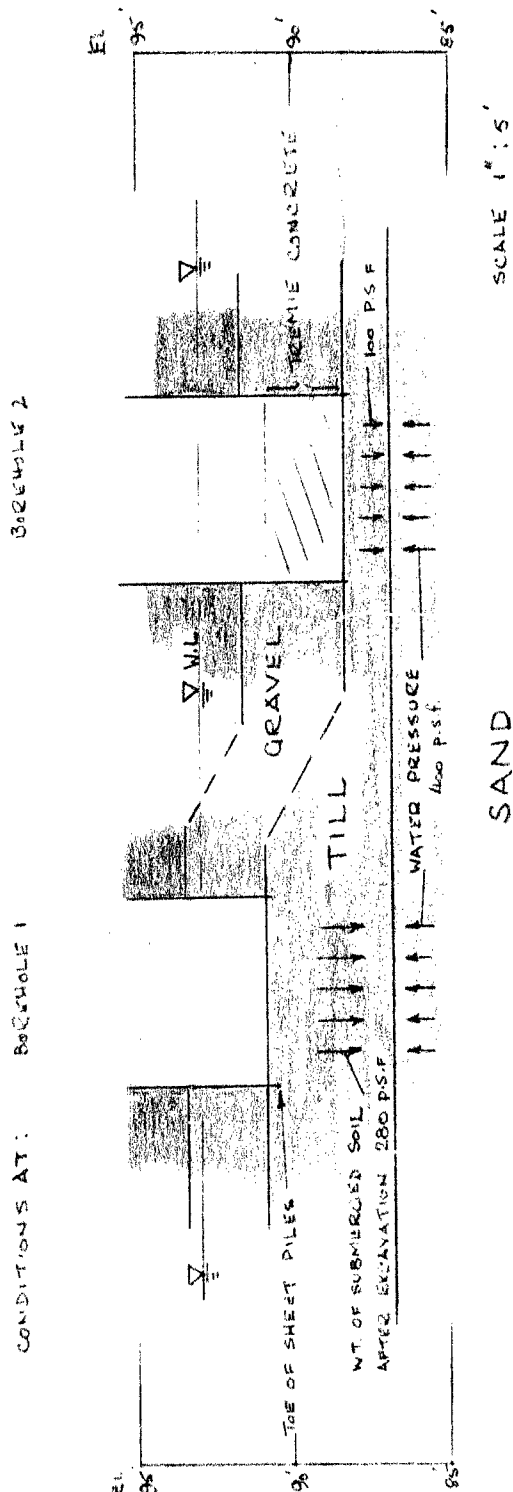
2" Split tube \odot
 2" Dia. Cone \odot
 Casing \cdots

Sampling Method

2" Dia. split tube \square
 2" Shelby tube \blacksquare



Prep. By J.P.



SUGGESTED DEWATERING PROCEDURE FOR
SPREAD FOOTINGS