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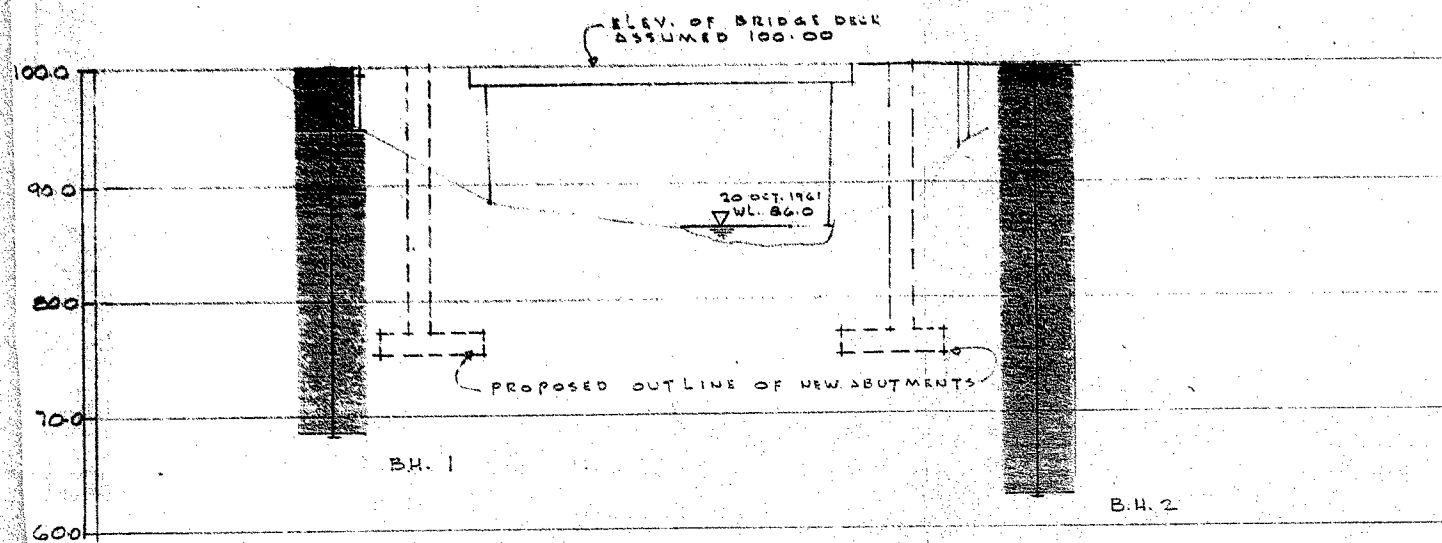
BRIDGE OVER

PERCH CREEK

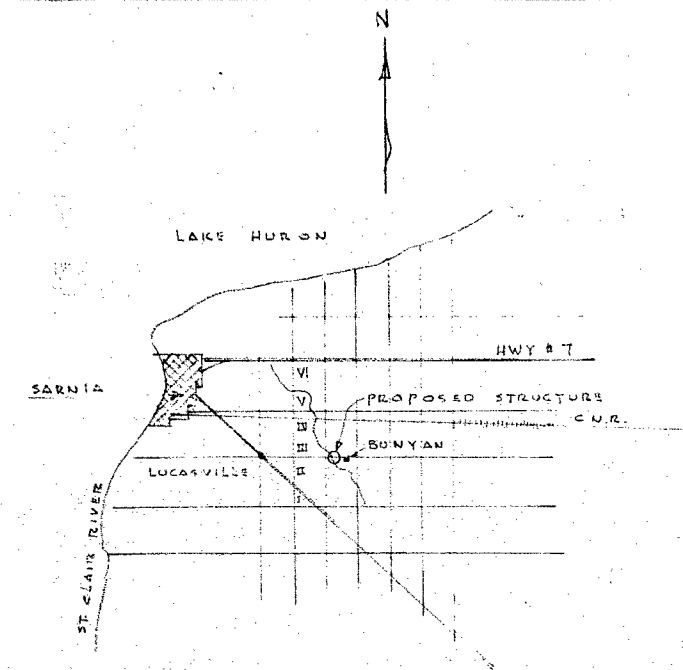
LOT 9 & 10 CON. II. III

HAMBTON

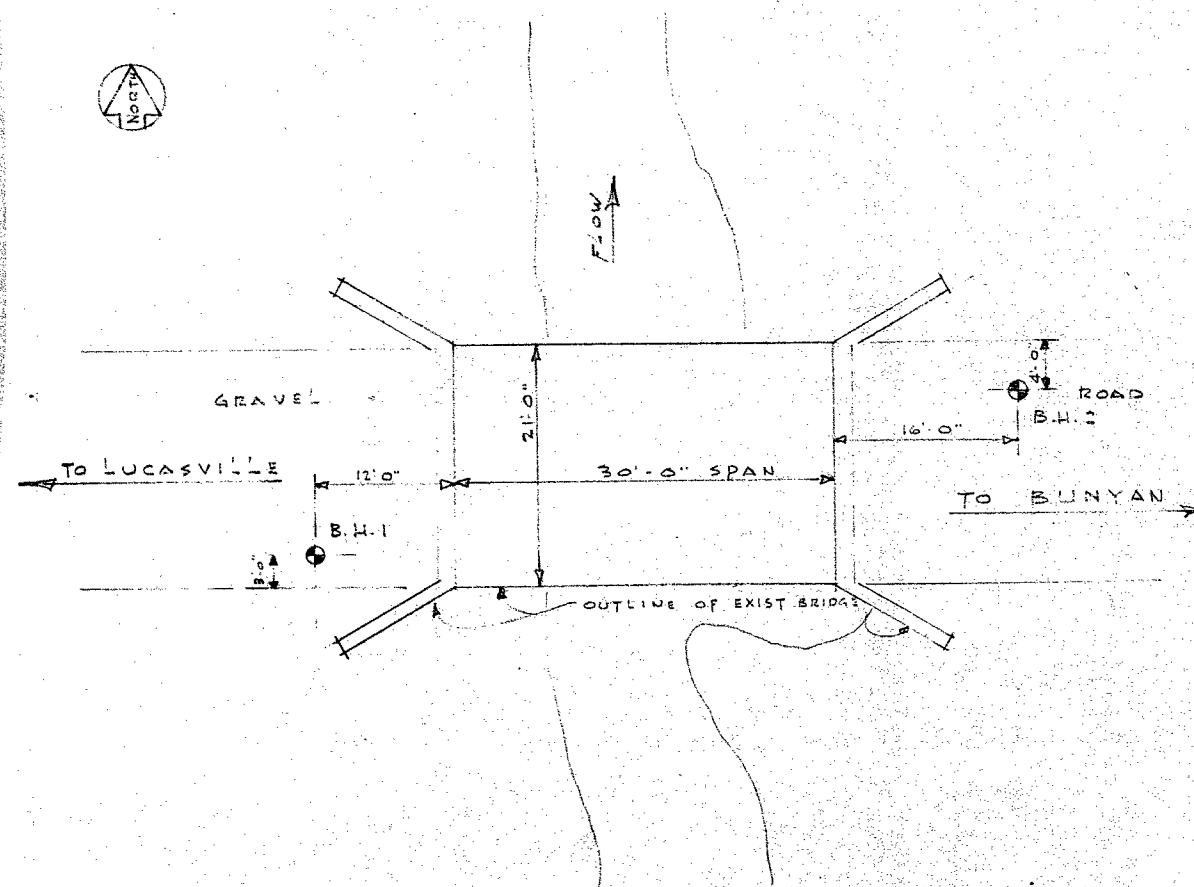
COUNTY



SUBSURFACE PROFILE
SCALE 1" TO 10'-0"



KEY PLAN



LOCATION PLAN
SCALE 1" TO 10'-0"

LEGEND

- GRAVEL ROAD BED
- BROWN SANDY CLAY FILL
- BROWN CLAY TILL
- GREY SILTY CLAY

ENCLOSURE No. 1	COUNTY OF LAMBTON
REF. 1-10-62	SOIL INVESTIGATION FOR PROPOSED ROAD BRIDGE
OCT. 1961	LOT 9, CONCESSION I/II
DRAWN BY J.T.	TOWNSHIP OF SARNIA
CHECKED BY J.P.	DOMINION SOIL INVESTIGATION LIMITED LONDON 363 QUEENS AVE. ONTARIO

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

April 19, 1962.

REVIEW OF FOUNDATION INVESTIGATION
BY DOMINION SOIL INVESTIGATION,
LTD. (Bridge Office Ref. BA 1392)

(D.H.O. District #1)

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

Re: County of Lambton
Bridge over Perch Creek
Township Sarnia
Lot 9/10, Con. II/III.

The report on the foundation investigation of the above site by Dominion Soil Investigation, Ltd., has been reviewed. We suggest the following modifications to the recommendations made by the consultant:-

- (1) A net allowable bearing pressure of 1.8 T.S.F. may be used.
- (2) The coefficient of active pressure for granular backfill should be taken as 0.3.
- (3) The immediate settlement estimated by the consultant is in error, but will be small in any case, if the allowable bearing pressure is not exceeded.
- (4) The ultimate value of skin friction for piles should be taken as 700 p.s.f.

We believe you will find the above recommendations adequate for your future design work. Should you, however, require additional information, please feel free to contact our office.

KYL/MdeF
cc: Foundations Office
Gen. Files.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.
Per:

P.S. -- Returned herewith is
copy of plan, as requested.

[Signature]
K. Y. Lo,
SUPERVISING FOUNDATION ENGR.

BA 1392

COUNTY OF LAMBTON
MR. OTTO VAN DEURS
COUNTY ENGINEER
COUNTY BUILDING
SARNIA ONTARIO

Report on
SOIL INVESTIGATION

61-F-246 H

for
ROAD BRIDGE
LOT 9, CONCESSIONS II/III
TOWNSHIP OF SARNIA

by
DOMINION SOIL INVESTIGATION LIMITED
363 Queens Avenue
LONDON ONTARIO

Reference No. 1-10-L2

October, 1961

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INTRODUCTION

In accordance with verbal authorization from Mr. Otto Van Deurs, County Engineer, a soil investigation has been carried out at a site on Lot 9 of Sarnia Township where a 30 foot span bridge on II/III Concession Road is to be replaced by a new structure.

The approximate size and shape of the new bridge were described by Mr. J. D. Nisbet, Consulting Engineer, in accordance with the requirements of loading and allowance for erosion. The proposed depth and dimensions of the footings are shown on enclosure 1.

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. Some guidance was also asked for in the use of pile foundations as an alternative design.

I. DESCRIPTION OF SITE & GEOLOGY

Page 2

The site lies approximately 5 miles south-east of Sarnia in the extensive St. Clair clay plains. The topography is flat, and is drained by a number of small streams which flow northward to Lake Huron. At the site of this investigation a stream has cut a gulley 10 to 12 feet deep into the clay.

The nature of the strata as revealed by the field and laboratory tests shows little sign of preconsolidation, which suggests that the material was deposited by Lake Warren, the most recent of the glacial lakes to cover this region.

II. FIELD WORK

Field work was carried out on the 20th and 23rd of October 1961 and consisted of 2 boreholes at the locations shown on enclosure 1. Dynamic cone penetration tests were made adjacent to each borehole. The holes were wash-bored and lined with Bx casing 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. Undisturbed samples of the soil were recovered in 2 inch diameter thin-walled Shelby tubes, and insitu vane shear tests were performed.

The results of the field tests are recorded on data sheets comprising enclosures 2 and 3. Elevations have been referred to the level of the deck of the existing bridge which has been assigned the arbitrary elevation 100.0 feet.

III. SUBSURFACE CONDITIONS

A subsurface profile is shown on enclosure 1. The gravel road bed is 6 inches deep and below this is a sandy clay fill containing 10 to 15% of rounded gravel. This material is a clay till which has been used for backfill. It extends to a depth of 5 feet 6 inches in borehole 1 where it changes to a relatively undisturbed material of the same composition which is probably the original overburden. In borehole 2 the same fill material extends to a depth of 8 feet, and the undisturbed layer is absent.

At 10 feet 6 inches in borehole 1 and 8 feet in borehole 2 a stiff grey silty clay was encountered which extends beyond the limit of the borings, i.e. 37 feet 6 inches. This material contains traces of angular grit particles 2 or 3 millimetres in diameter. A part from a slight increase in strength due to loss of moisture above the water table (El. 86.0 feet) the clay exhibited a very uniform reaction to the Standard penetration and vane shear tests throughout the depth explored.

IV. LABORATORY TESTS

The following is a summary of the results of laboratory tests performed on the clay.

Borehole No.	1	2	2	2
Sample No.	5	3	7	9
Depth (feet)	20	15	24.5	30
Elevation (feet)	80	85	75	70
Unit Weight (p.c.f.)	126.5	130	131	132.5
Void Ratio	0.58	0.54	0.52	0.52
Natural Moisture (%)	20.0	18.8	19.0	19.2
Liquid Limit (%)	33.0	29.5	35.0	30.0
Plastic Limit (%)	16.8	17.0	17.3	17.5
Plasticity Index	16.2	12.5	17.7	12.5
Liquidity Index	0.2	0.14	0.1	0.14
Unconfined compressive Strength (t.s.f)	1.18	2.26	2.82	1.93

A consolidation test has been performed on sample 9 from borehole 2 and the results are shown on enclosure 4.

The data supplied by the field and laboratory tests enable the material to be classified as a stiff grey silty lacustrine clay of low plasticity with a sensitivity of approximately 3. The material is thus fairly characteristic of soils in the region.*

For the purpose of calculations a value of 2000 pounds per square foot will be assumed for the cohesion of the clay. This figure is conservative compared with

* See reference 10.

an average value of 2975 p.s.f. from the vane shear tests and 2050 p.s.f. from the unconfined compression tests, the latter figure being somewhat depressed by a single low value (Sample 5, borehole 1).

V. BEARING CAPACITY

From information supplied as to the proposed size and elevation of the footings, it has been estimated that the approximate dead load of the structure will be 15,000 pounds per lineal foot. The ultimate bearing capacity of a uniformly loaded footing measuring 9 feet by 36 feet has been calculated, according to Meyerhof, to be 19680 pounds per square foot or, applying a safety factor of 3 against shear failure, the maximum allowable soil pressure would be 6560 p.s.f. Assuming a coefficient of active earth pressure of 0.5 from the backfill and, as an extreme condition a lowering of the creek bed by erosion to the level of the footings, the maximum and average soil pressures arising from the horizontal thrust and eccentric loading would be about 3300 and 2500 p.s.f. respectively. There is thus adequate resistance against shear failure.

VI. SETTLEMENT

The settlement of the footings will consist of an immediate settlement due to the elastic deflection of the soil and consolidation settlement over a period of time resulting from the gradual expulsion of pore water under the increased loading. The average value of this loading is assumed to be 2500 p.s.f.

The coefficient of compressibility of the clay in the consolidation test for the pressure range 1 to 2 T.S.F. was found to be 43 T.S.F. from which it is deduced that the modulus of elasticity E of the soil is approximately 175 T.S.F. Using this value the immediate settlement is estimated to be 1.0 inch.

The distribution of vertical stress below the characteristic point (defined by Kany as $0.74 a/2$, $0.74 b/2$ from the centre of a rectangle with sides a and b) of the rectangular footing has been calculated by Steinbrenner's method, and the resulting consolidation settlement calculated from the e -log p curve of the consolidation test.

A value of 2.28 inches was obtained. Applying to this the Skempton-Bjerrum correction factor modified by Wood, the estimated consolidation settlement is 1.4 inches.*

The calculated time periods for 50% and 90% consolidation are:

	Days
50%	100
90%	700

In view of the uniform subsurface conditions, no appreciable differential settlement is expected.

VII. PILE FOUNDATIONS

Only long-term loading tests on piles driven on the site would enable a rational estimate to be made bearing capacity and settlement for a piled foundation. As a first approach, however, it has been observed by Terzaghi & Peck that "the ultimate value of the skin friction is commonly equal to about one-half the unconfined compressive strength of the clay". Thus the ultimate value of skin friction may be taken as 2000 p.s.f. of contact area. The working load should be about 1/3 of this value.

VIII. CONSTRUCTION

The theoretical critical height for a cut in clay possessing a cohesion of 2000 p.s.f. is 33 feet. In this case the required depth of cut will be about 25 feet, which should stand safely without bracing.

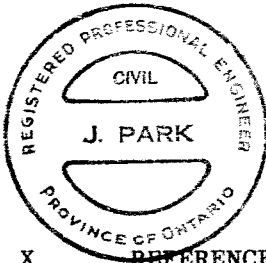
The natural soil is not an ideal material for backfill because it is stiff and impervious. There is the danger that inadequate compaction will result in large voids where water can accumulate, and the formation of ice in cold weather could lead to high horizontal pressures. A granular material would make a more suitable backfill.

Weep holes should be provided through the abutment walls, and tile drains would also help to remove accumulating water.

* The clay has an over consolidation ratio of 1.2, and the pore pressure coefficient "A" has therefore been taken as 0.5. The corresponding correction factor is 0.625.

IX. SUMMARY

1. The strata consist of 8 to 10 feet of brown sandy clay till, much of which is recompacted fill, overlying a bed of stiff grey silty clay.
2. The clay has a cohesion value of at least 2000 p.s.f. Footings of the proposed size will have an adequate safety factor against shear failure.
3. The calculated values of immediate and consolidation settlement are 1.0 and 1.4 inches respectively. Any reduction in the proposed size of the footings will increase these values.
4. If the use of piles is to be considered a value of 2000 p.s.f. may be taken as a first approximation for the ultimate value of skin friction, but loading tests are necessary to determine the load bearing and settlement characteristics.
5. The cut is expected to remain open without bracing to a depth of at least 25 feet.
6. Precautions must be taken to prevent water from accumulating in the backfill.



DOMINION SOIL INVESTIGATION LIMITED

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

X. REFERENCES

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COUNTY OF LAMINGTON
 ROAD BRIDGE

WASH BORING

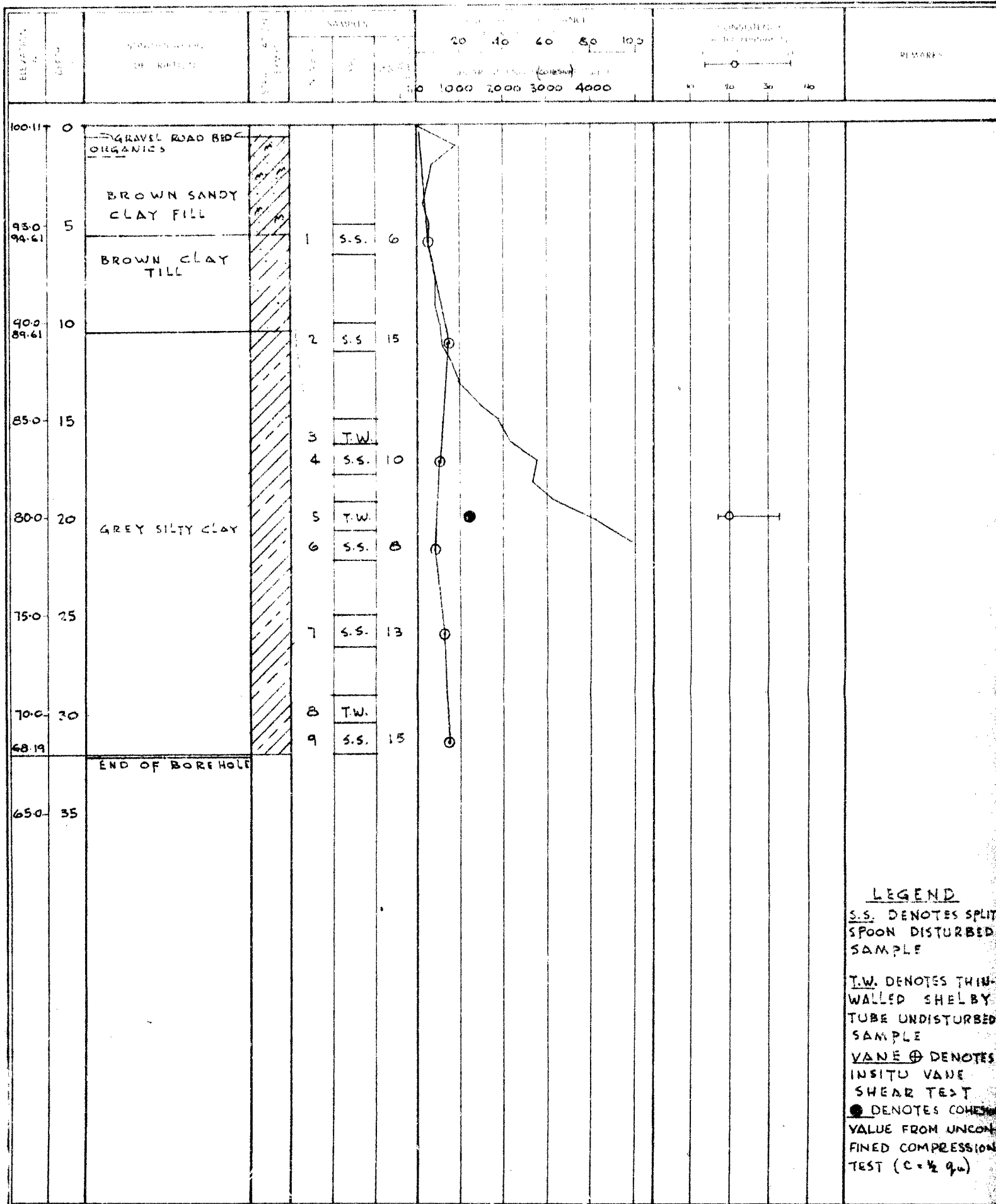
3 INCH

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2

SEE ENCLOSURE #1

BASE VALUE TOP OF BRIDGE DECK ASSUMED 100.0 FEET



1-10-L2

GEOTECHNICAL DATA SHEET FOR BOREHOLE 2

COUNTY OF LAMBERTON
ROAD BRIDGE

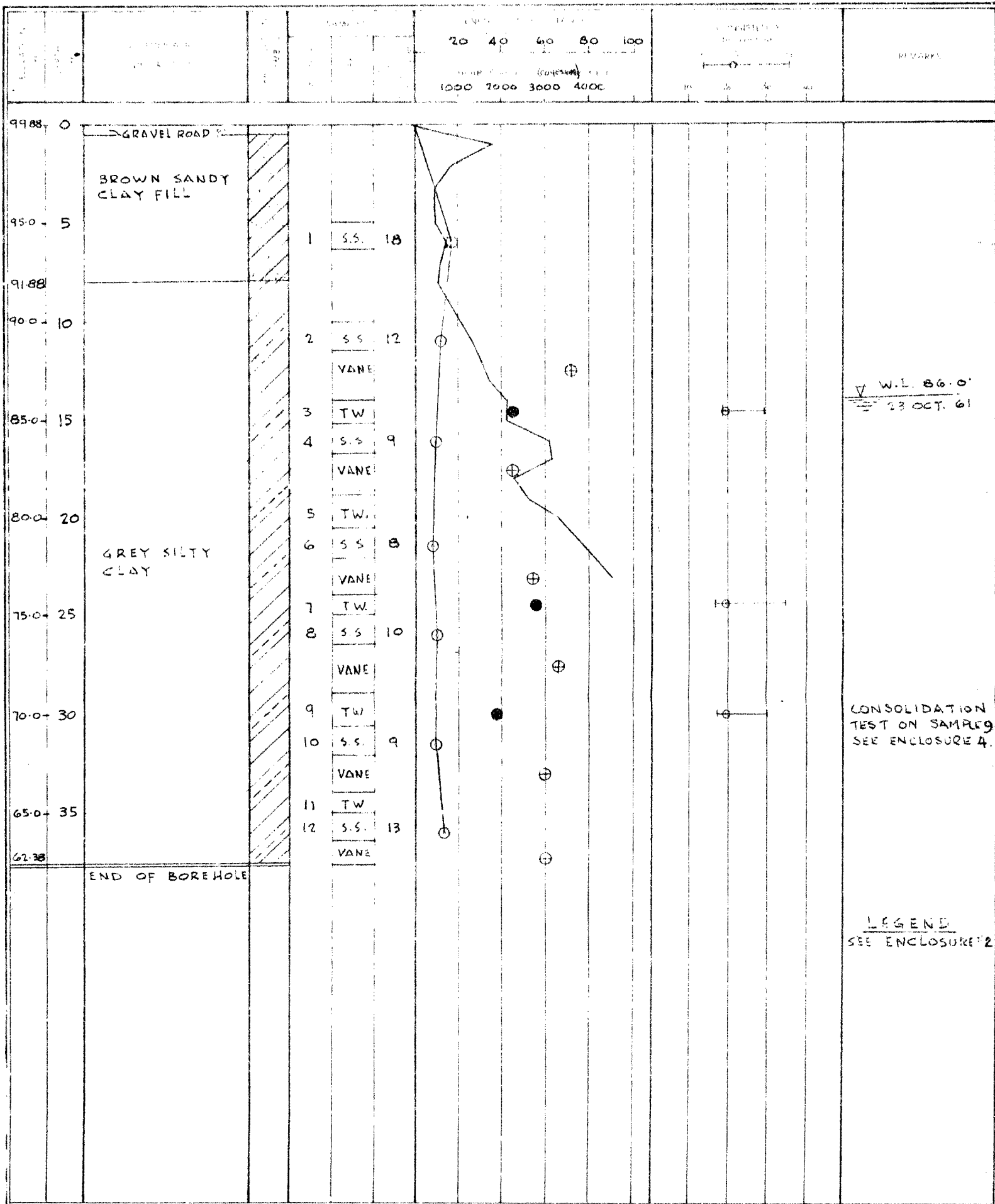
WASH BOREHOLE
3 INCH.

3

SEE ENCLOSURE #1

OCT 23 61

TOP OF BRIDGE DECK ASSUMED ELEV. 100.0 FEET



VERTICAL SCALE: 1 IN. TO 5 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: J. T. CHD: J. P.

Dominion Soil Investigation Ltd.**CONSOLIDATION TEST**