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G.I.-30 SEPT. 1976

GEOCRES No. 30MS-109

DIST. 4 REGION CENTRAL

W.P. No. _____

CONT. No. _____

W. O. No. _____

STR. SITE No. 36-211

HWY. No. _____

LOCATION CHATAM ST & T.H. AND
B. RAILWAY, HAMILTON

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____

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Weston, Ontario
241-4644

William A. Trow
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Project: J1801

30m5-109

GEOCRLS No.

Soil Mechanics
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Associates Ltd.

C. C. Parker & Associates Ltd.,
795 Hamilton Street West,
Hamilton, Ontario.

February 18, 1965.

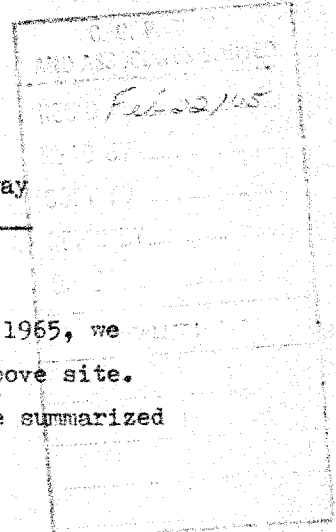
Attention: Mr. D. C. Cramm, P. Eng.

Re: Foundation Investigation
Proposed Grade Separation
Chatham St. at T.H. & B. Railway
Hamilton, Ontario

Dear Sirs:

Following your authorization of January 13, 1965, we have completed our foundation investigation of the above site. Our findings and recommendations at this location are summarized as follows:

- 1) The subsoil consists of a silty clay or clayey silt with a small quantity of sand and gravel. The material is heterogeneous and fine layering is not uncommon. The top portion of this deposit to elevation 285 forms a hard crust - a result of desiccation, while the lower depths have a reduced strength.
- 2) Spread footings founded at the elevations proposed in the text of the report are recommended. These spread footings should be designed using a safe net bearing value of 4 tsf. An alternative to spread type footing is a piled foundation. Tube type piles 12" diameter concrete filled can be designed for a safe load of 50 tons per pile if they meet refusal in the upper crust, or alternatively terminated as a floating pile to elevation 250 ft. Because of the possible variations in pile lengths for tube piles, steel H piles may be substituted. These piles must be driven to elevation 250.



3) Bin type retaining walls should incorporate adequate drainage behind the wall and be designed using an earth pressure co-efficient $K_a = 0.22$.

4) Comments related to possible earth pressures on retaining type abutments are given in the report. No problems associated with embankment stability exist at this location.

These comments have resulted from consideration of the following detail:

THE SITE

The proposed grade separation is located on Chatham Street at its crossing of the Toronto, Hamilton and Buffalo Railway in the City of Hamilton, Ontario. This site is presently occupied by a three span 19 foot wide bridge. The proposed structure will incorporate a 30' roadway and similar structure. The new structure will necessitate widening and a slight increase in height of the present embankment.

FIELD WORK AND SUBSOIL STRATIGRAPHY

A total of three sampled boreholes comprise the field work at this site. Boreholes were advanced using continuous flight auger equipment and uncased holes. Samples of the subsoil were obtained using standard split spoon or Shelby type samplers. The location of the boreholes, as well as an estimated subsoil stratigraphy, appear on Dwg. 1. Detailed information as to the subsoil encountered in the borings has been shown on the borehole logs Dwg. 2 to 4. These logs include, in graphical form, a summary of all laboratory tests performed. Borehole elevations have been referenced to the bench mark shown on Dwg. 1.

Ground water level at this site is estimated at elevation \pm 302. This estimated water level is relatively unimportant since the subsoil is generally impervious. Small sand pockets or seams give rise to variation in borehole water levels, however, the saturated samples of the subsoil confirm the suggested ground water elevations.

The subsoil at this site was found to consist of a hard silt or clay till becoming less stiff with depth. This heterogeneous mixture of silt, clay, sand, and fine gravel is variable in composition throughout the depth of the subsoil investigated. Laboratory strength tests show this deposit to have an upper crust extending to elevation \pm 285. The upper crust has a strength in excess of 4000 psf while measurements in the lower levels indicate this strength to be about 1000 psf. Moisture contents of both upper and lower levels of clay are generally just below 20%.

FOUNDATIONS

Foundations for the proposed structure are recommended to be of the simple spread type. However, since piles may be used to support spill through type abutments, both foundation types have been considered.

a) Spread Footings: Spread footings may be placed on the stiff upper crust. Borehole No. 1 indicates that a suitable elevation is 302 while holes 2 and 3 place footings at elevations 292 and 310 respectively. From this it can be seen that the footing founding level is variable but will be satisfactory provided the dense or hard material is encountered in the excavation.

The variation in foundation level results from the upper fill deposit found in hole 2. No foundations should be placed on this fill material. From the results of the other holes and the general topography of the site, it is felt that this depth of fill (as indicated in hole 2) does not continue under the proposed piers.

The safe net bearing value to apply for footings founded as suggested is 4 tsf. This value can be confirmed theoretically using laboratory strength measurements as follows:

$$q_s = \frac{C \cdot N_c}{S.F.}$$

where q_s = safe net bearing value

N_c a bearing capacity factor = 6 for these footings

S.F. = 3 the safety factor

C = 4000 psf, a conservative estimate of the shear strength for a distance beneath the footing equal to the footing width. (Based on laboratory strength tests). Solving this expression, the safe net bearing value of 4 tsf is confirmed.

It is not likely that deeper founding depths will be required but if footings must be founded below elevation 292, the bearing value must be reduced linearly from 4 tsf at elevation 292 to 1.5 tsf at elevation 285.0.

The settlement of spread footings will depend not only on the load on the footing but also on the influence from any embankment loading. Since embankments have been in place for a numbers of years and only widening is required, any future settlement should be of relatively small magnitude. Footing loads will dissipate mainly through the dense upper crust which is relatively incompressible. Therefore the total settlement expected for bridge footings is slight. Based on past records for structures in this area, it is estimated that this settlement will not exceed 2 inches and will probably be in the range of 1 inch.

No ground water problems will exist during excavation since the soil is relatively impermeable. Any ground water that does enter the excavation will drain from sand or silt pockets or seams and will rapidly diminish in quantity to near zero.

b) Piled Foundation: If piles are selected to support the proposed structure, they may be either the steel tube type or steel H type pile.

Steel tube piles 12 inches in diameter concrete filled are generally used to support the spill through type abutment. Because of variation in the strength of the upper crust, it is possible that some piles will become end bearing in this crust, while others will pass through the crust.

For piles meeting refusal near elevation \pm 290 (in this crust), a safe design load of 50 tons/pile or a load equal to the structural capacity of the pile as a short column, may be used. Refusal may be defined as a penetration of less than 1 inch when the pile is subjected to 8 blows from a hammer having a driving energy of not less than 20,000 ft. lb/blow.

If piles pass through the crust they will not meet refusal but should be driven to elevation 250. These piles will derive their carrying capacity from the friction along their sides and can be designed for a safe load of 50 tons/pile. The design value of 50 tons for this friction pile incorporates a safety factor of 2.5, however a pile load test is suggested to confirm this carrying capacity.

The condition of the fill material in the existing embankment was not determined because of access problems and financial obligations. It is generally found that this old fill is in a medium dense to loose state and can be penetrated by tube piles.

Because of the possible variations in pile lengths for tube piles, it may be considered advantageous to use steel H piles driven to a specific elevation. This elevation can be maintained because of the ease of penetration of H sections. All such piles would derive their carrying capacity from friction along the sides and should be driven to elevation 250. The safe load on these piles would then be 50 tons/pile. As for the tube type pile a pile load test should be carried out to confirm this capacity. Steel H piles 10" @ 42 lb/ft. or 10" @ 57 lb/ft. will be suitable. Steel H piles driven to refusal have not been suggested because of the long length required. This length was not determined at this site but refusal depths at the Main Street crossing were near elevation 180*.

RETAINING WALLS

Bin type retaining walls have been proposed for support of the sides of the approach fills. These bin walls are necessary because of the limitations on available property.

Foundations for the proposed bin walls can be placed on original soil provided all loose material and topsoil is first removed and a minimum depth of 2 feet of compacted free draining material is placed between the footing and the ground surface. If it is necessary to place these foundations on existing fill material, the fill material must be removed and replaced in a well compacted state prior to placing the footings.

Settlement of the retaining wall will be small, and unimportant because of the flexibility of this type of structure. No problems associated with groundwater during installation of the bin retaining walls are present.

* - Foundation Investigation, Main Street and Highway 403, D.H.O.

Provision for drainage behind the retaining walls is recommended. Despite the fact that the wall is above groundwater level surface run-off, water should not be allowed to accumulate behind the retaining walls. To avoid this, backfill immediately adjacent to the walls should be free draining granular material.

A 4" farm tile drain must be installed in the granular layer beneath the bin wall to guarantee drainage. This tile should lead to a frost-free sump for disposal of the water.

For the purpose of estimating the active earth pressure on the retaining wall, the fill material and the granular fill adjacent to the retaining wall are assumed to be cohesionless with $\phi' = 38^\circ$, and to have an in-place unit weight of 120 pcf. The slope of the fill is taken as 2 horizontal to 1 vertical or 26.5° . The slope of the back of the retaining wall is assumed to slope into the fill at 1 horizontal to 6 vertical or approximately 10° . The wall friction developed at the retaining wall face is assumed to be 19° .

The calculation of the active earth pressure involves the determination of the active earth pressure coefficient. Various methods of finding this value are available, in some cases a long and tedious calculation is involved, while in others the values can be selected directly from tables. In both cases the end resulting value is essentially the same. It is therefore recommended that, for the determination of the coefficient of active earth pressure, the tables of Caquot and Kerisel * be utilized. Using the values of earth properties listed the resulting value of the active earth pressure coefficient is: $K_a=0.22$ when considering the conditions outlined in the preceding paragraph.

* Caquot & Kerisel (1948) Tables for the Calculation of Passive Pressure, Active Pressure, and Bearing Capacity of Foundations', Paris, Gauthier - Villars, English Translation.

The value of the earth pressure per foot run (P) acting on the retaining wall can be obtained from the simple relationship:

$$P = \frac{1}{2} \gamma H^2 K_a$$

where γ = 120 pcf
 H = total height of wall
 K_a = value as determined

Calculation of passive resistance will not be required since no effective passive force is developed in front of this bin type retaining wall.

It is suggested that the following minimum factors of safety be incorporated into the design:

Factor of safety on bearing capacity (as included previously)	= 3
Factor of safety against sliding on base	= 1.5
Factor of safety against overturning	= 2

When considering sliding on the base the value of ϕ' for use along the base of the wall is recommended as 34° .

EARTH PRESSURE ON ABUTMENTS

If earth retaining type abutments are selected, they must be designed to withstand the earth pressure from the retained fill. It is suggested that for a simply-supported type structure, the earth pressure co-efficient to be used in calculations equals 0.25. This co-efficient assumes a slight yield of the abutment if compaction is such that earth pressures tend to approach the at-rest condition. For rigid frame structures and well compacted backfill, the design co-efficient should be increased to 0.35.

With adequate drainage facilities the earth pressure p at any given depth h can be determined from the expression:

$$p = KYh + Kq$$

where K = the appropriate earth pressure co-efficient
 Y = 120 pcf the estimated unit weight of backfill material
 q = the value of any surcharge (in psf) acting near the abutment

APPROACH FILLS

Because of the dense nature of the subsoil, no problems associated with stability of the approach fills are present. These fills should be constructed with standard procedures and incorporate 2:1 side slopes. Prior to beginning construction the topsoil from the existing embankment should be removed and keys in the form of steps excavated. These keys will prevent any movement between the old and new embankment.

It is hoped that these comments will assist in the design of the foundations for the structure. If we can be of further assistance please do not hesitate to contact this office.

Yours very truly,



K. Peaker, P.Eng.

KP/chm
enclosures

BOREHOLE No. 1
 PROJECT Tramway Bridge
 LOCATION Chatham Street, Hamilton
 HOLE LOCATION See Plan, 1
 HOLE ELEVATION 344.16
 DATUM See Page 1

PENETRATION RESISTANCE

2" O.D. SPLIT TUBE
 2" I.D. SHELBY TUBE
 2" DIA. CONE

SHEAR STRENGTH

UNDRAINED TRIAXIAL
 AT OVERBURDEN PRESSURE
 UNCONFINED COMPRESSION
 VANE TEST AND SENSITIVITY IS, $\frac{1}{2}$

NATURAL MOISTURE CONTENT

ATTERBERG LIMITS

LIQUID LIMIT

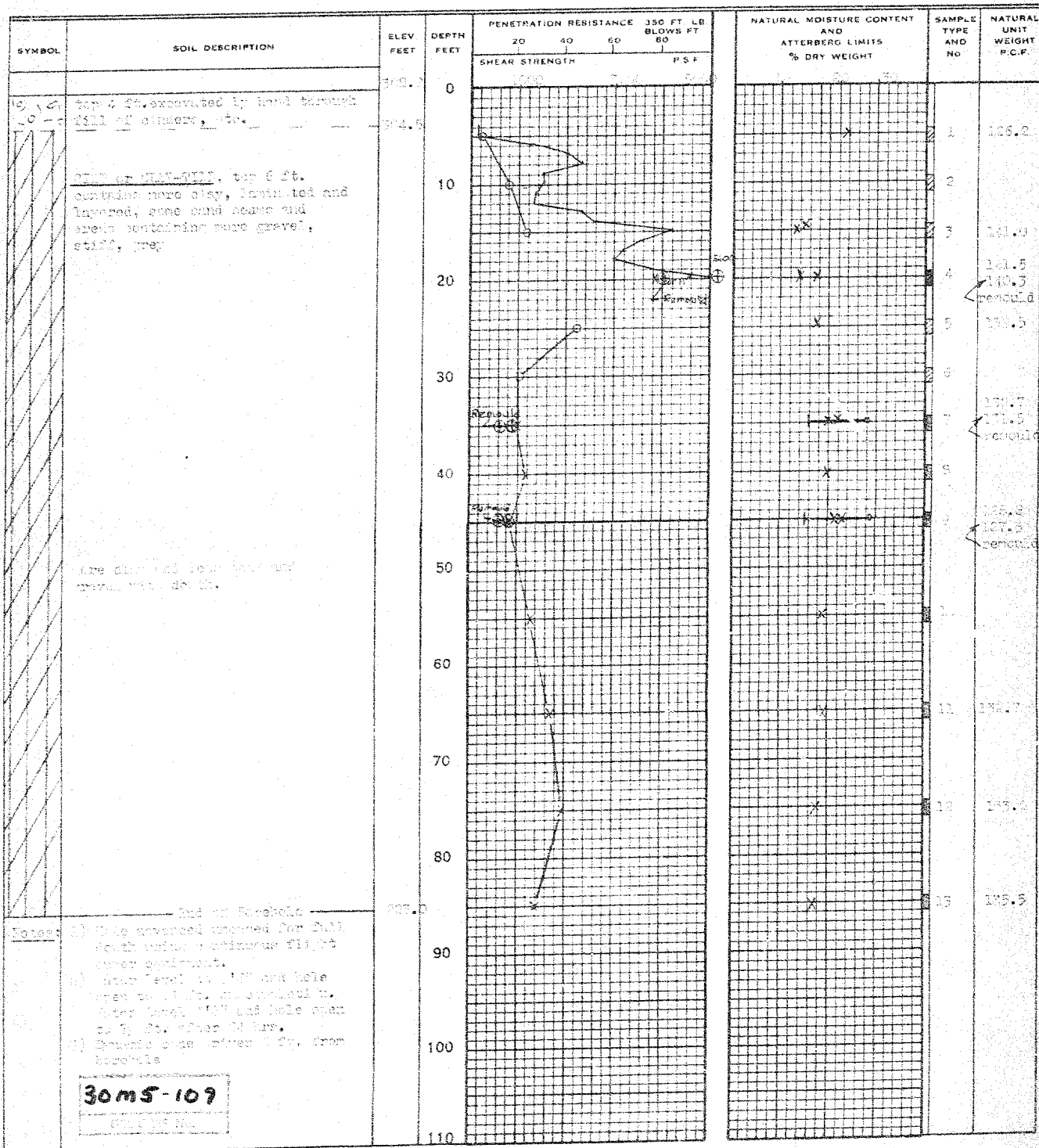
PLASTIC LIMIT

SAMPLE TYPE

2" O.D. SPLIT TUBE

2" I.D. SHELBY TUBE

3" O.D. SHELBY TUBE



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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

DRAWING No. 3
PROJECT No. J1801

LEGEND

PENETRATION RESISTANCE

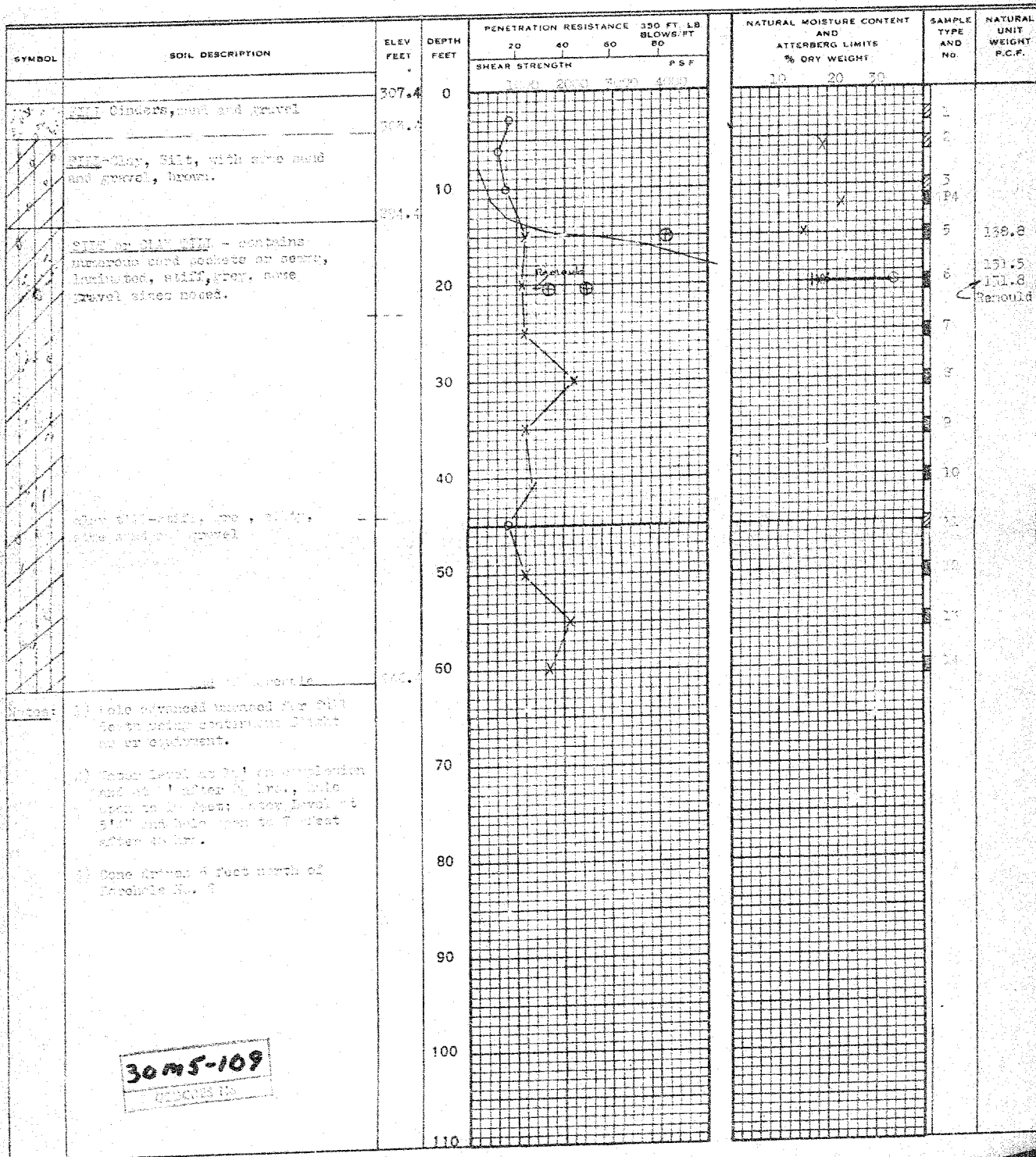
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
2" DIA CONE
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE
UNCONFINED COMPRESSION
VANE TEST AND SENSITIVITY

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

ATTERBERG LIMITS

LIQUID LIMIT
PLASTIC LIMIT
SAMPLE TYPE
2" O.D. SPLIT TUBE
2" I.D. SHELBY TUBE
3" O.D. SHELBY TUBE

BOREHOLE No. 2.
PROJECT Proposed Bridge.
LOCATION Chatham Street, Hamilton.
HOLE LOCATION See DWG. 1.
HOLE ELEVATION 307.40 ft.
DATUM See DWG. 1.



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SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

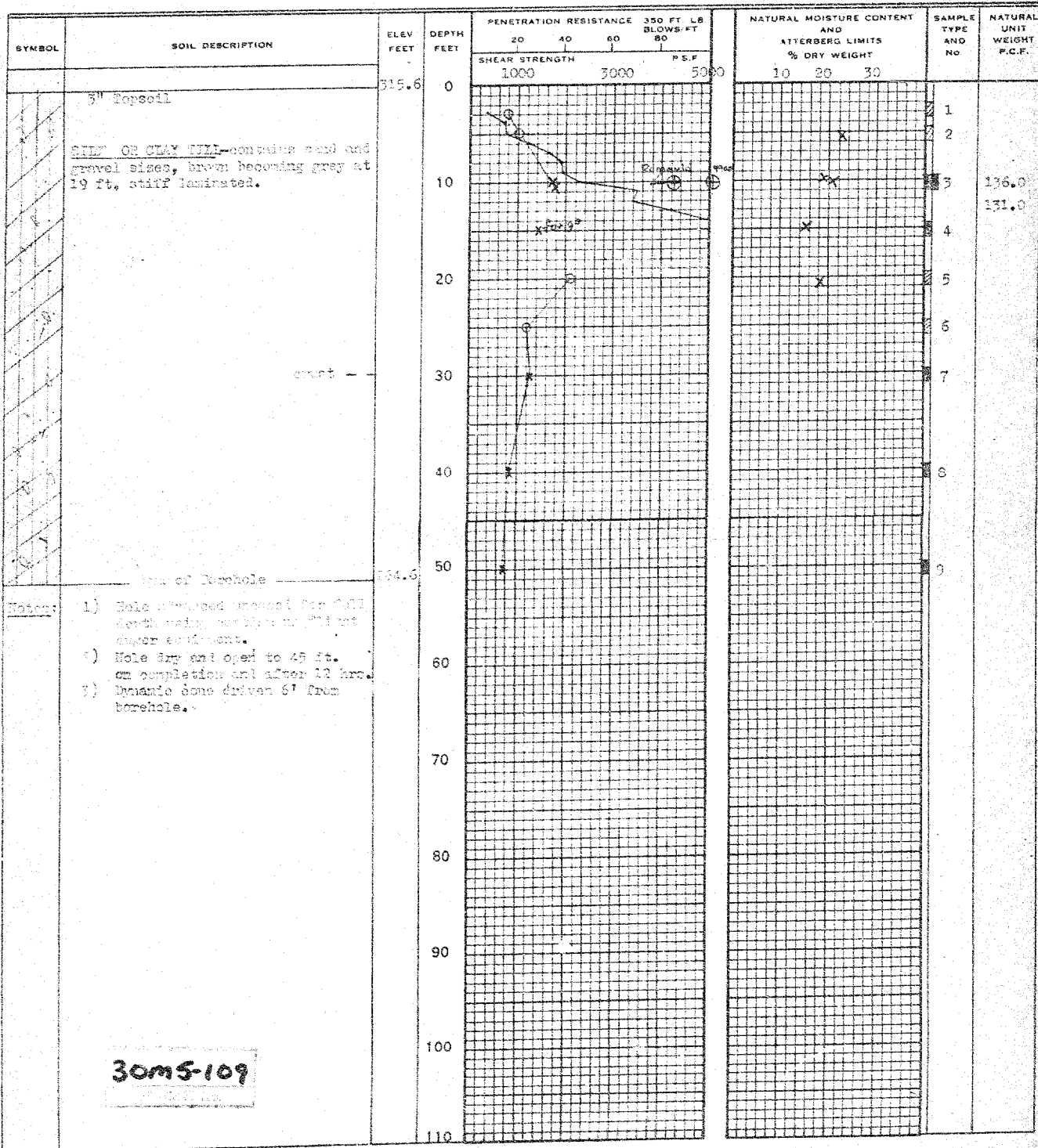
LEGEND

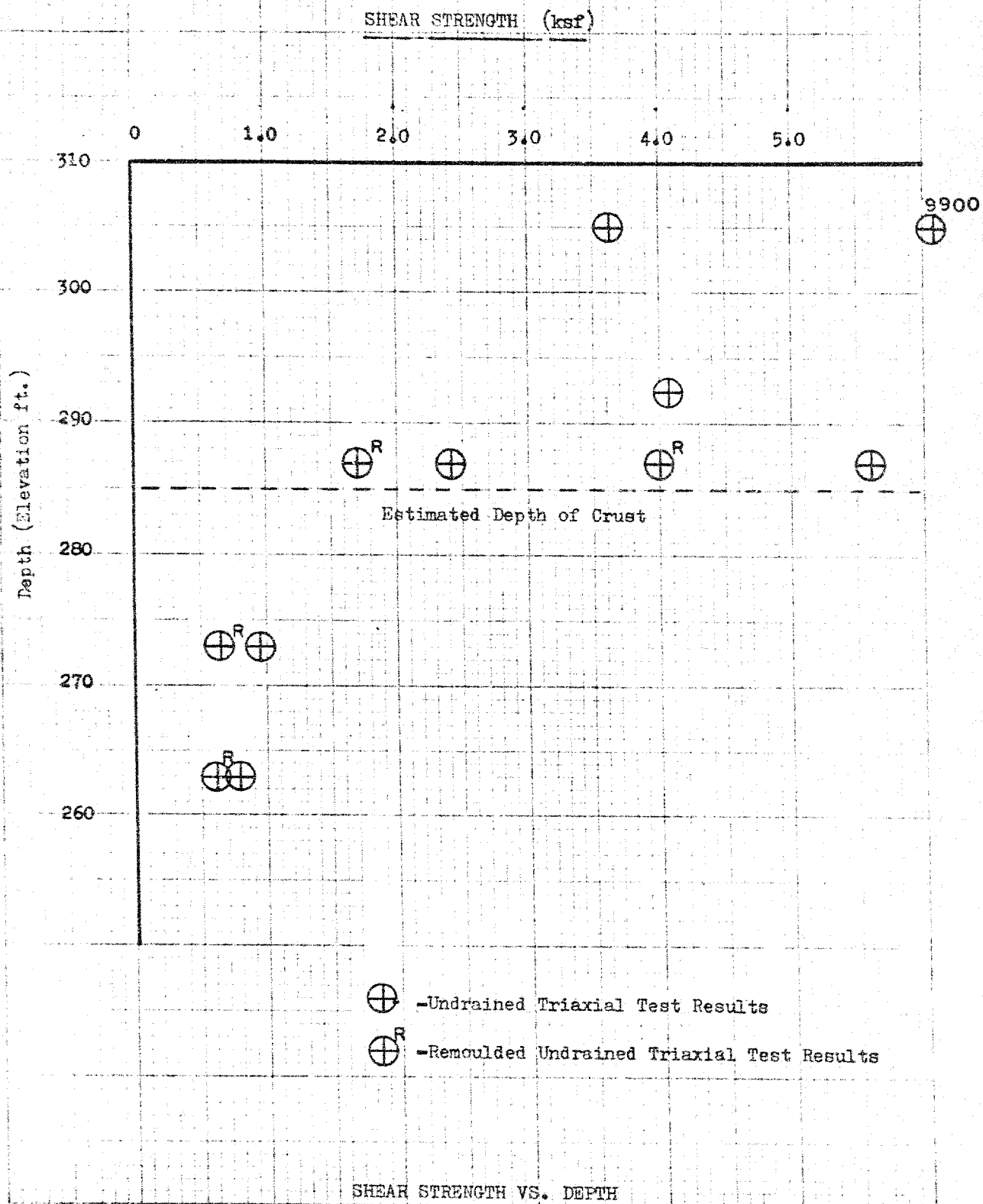
DRAWING NO. 4
PROJECT NO. J1801

BOREHOLE NO. 3.
PROJECT: Proposed Bridge.
LOCATION: Chatham Street, Burlington.
HOLE LOCATION: See Map. 1.
HOLE ELEVATION: 315.6 Ft.
DATUM: See Map. 1.

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —○—○—○—
2" I.D. SHELBY TUBE *—*—*—*—*—*—
2" DIA. CONE —+—+—+—+—+—+—
SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊗
UNCONFINED COMPRESSION ⊕
VANE TEST AND SENSITIVITY (S_v) †

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}
ATTERBERG LIMITS
LIQUID LIMIT —○—
PLASTIC LIMIT —+—
SAMPLE TYPE
2" O.D. SPLIT TUBE —□—
2" I.D. SHELBY TUBE —■—
3" O.D. SHELBY TUBE —▣—





SHEAR STRESS knf

8

6

4

2

Remoulded Sample.

 $C = 4,000 \text{ psf.}$ $\gamma = 140.3 \text{ psf.}$ $w = 17.4\%$ $\sigma_3 = 19 \text{ psi.}$

Depth = 30 ft.

B.H. 1

Remould

Silty Clay - brown.

 $C = 5600 \text{ psf.}$ $\gamma = 141.5 \text{ psf.}$ $w = 13.4\%$ $\sigma_3 = 19 \text{ psi.}$

Depth = 20'

B.H. No. 1

2

4

6

8

10

12

14

16

18

20

% STRAIN

UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

WILLIAM A. TROW AND ASSOCIATES

SHEAR STRESS kef

2

1

Remoulded Sample.

 $C = 700 \text{ psf.}$ $\gamma = 111.5 \text{ pcf.}$ $W = 20.0\%$ $C_u = 32 \text{ psi.}$

Depth = 35 ft.

B.H.1.

Remould

B.H. 1

Silty Clay

 $C = 150 \text{ psf.}$ $\gamma = 130.7 \text{ pcf.}$ $W = 21.8\%$ $C_u = 32 \text{ psi.}$

Depth = 35 ft.

2 4 6 8 10 12 14 16 18 20
* STRAIN

UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

WILLIAM A. TROW AND ASSOCIATES

SHEAR STRESS ksf

1.5

1

0.5

Remoulded sample.

 $C = 650 \text{ psf.}$ $\gamma = 127.5 \text{ pcf.}$ $W = 21.1\%$ $\sigma_3 = 40 \text{ psi.}$

Depth = 45 ft.

B.H.1

Remoulded

Silty Clay - gray.

 $C = 750 \text{ psf.}$ $\gamma = 128.8 \text{ pcf.}$ $W = 22.3\%$ $\sigma_3 = 40 \text{ psi.}$

Depth = 45 ft.

B.H. 1.

% STRAIN

UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

WILLIAM A. TROW AND ASSOCIATES

SHEAR STRESS ksf

3

2

1

Remoulded Sample.
 $C = 1750 \text{ psf.}$
 $\gamma = 131.8 \text{ pcf.}$
 $W = 19.1\%$

$\sigma_3 = 18 \text{ psi.}$
Depth = 20 ft.
B.H. No. 2.

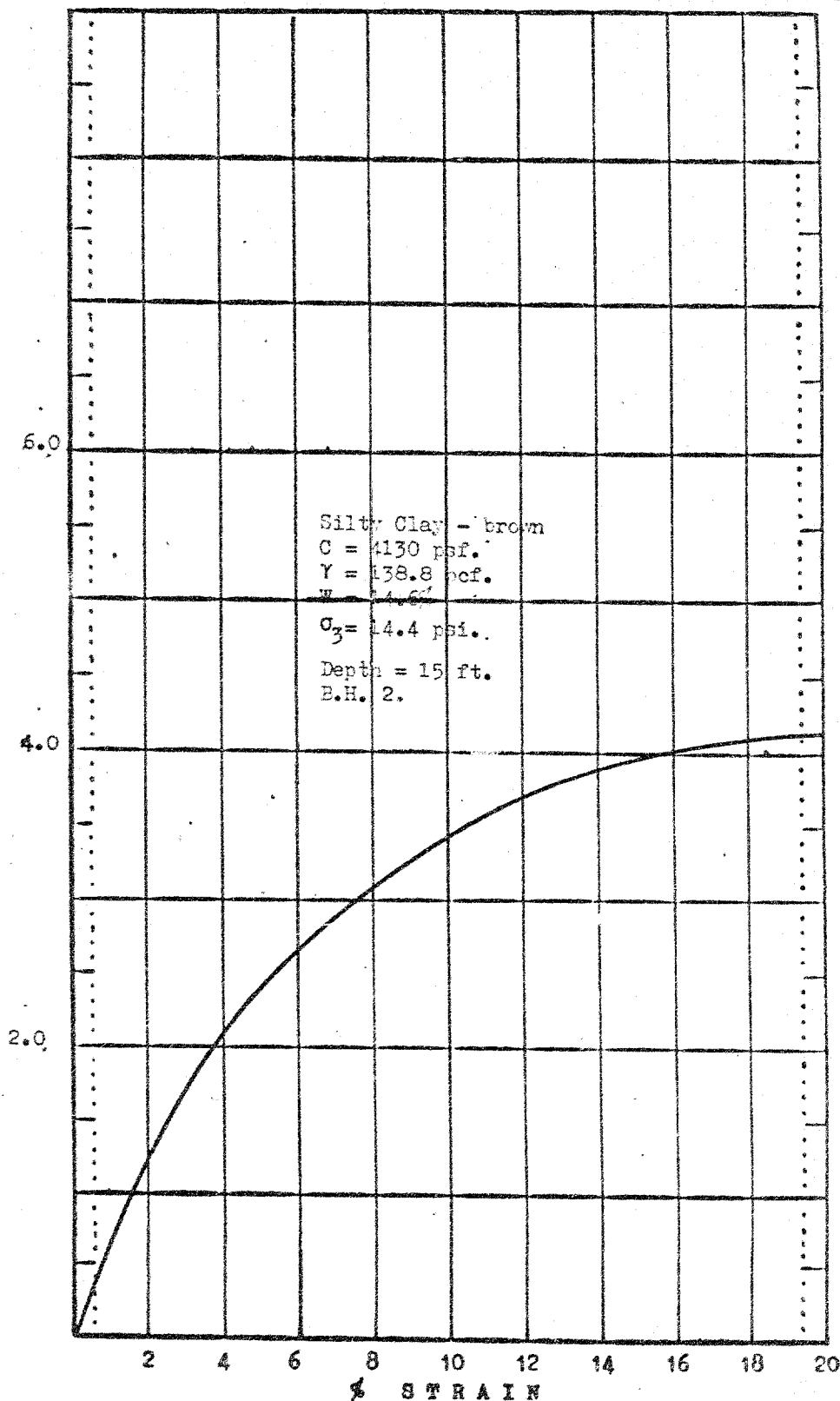
Remould

Silty Clay - grey.
 $C = 2400 \text{ psf.}$
 $\gamma = 131.5 \text{ pcf.}$
 $W = 19.6\%$
 $\sigma_3 = 18.2\%$
Depth = 20 ft.
B.H.2

2 4 6 8 10 12 14 16 18 20
STRAIN

UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

WILLIAM A. TROW AND ASSOCIATES

SHEAR STRESS mf 

UNDRAINED TRIAXIAL TEST RESULTS

WILLIAM A. TROW AND ASSOCIATES

SHEAR STRESS ksf

6.0

4.0

2.0

Silty Clay - greyish brown.
 $C = 4700 \text{ psf.}$
 $\gamma = 111.1 \text{ pcf.}$
 $W = 20.3\%$
 $\sigma_3 = 9.1 \text{ psi.}$
Depth = 10 ft.
B.H. No. 3.

2 4 6 8 10 12 14 16 18 20
% STRAIN

UNDRAINED TRIAXIAL TEST RESULTS

WILLIAM A. TROW AND ASSOCIATES

SHEAR STRESS ksf

10.0

8.0

6.0

4.0

2.0

Silty Clay - grey.

 $C = 9,900 \text{ psf.}$ $\gamma = 126 \text{ pcf.}$ $W = 15.8\%$ $\sigma_3 = 15 \text{ psi.}$

Depth = 15 feet.

B.H. B.

2

4

6

8

10

12

14

16

18

20

% STRAIN

UNDRAINED TRIAXIAL TEST RESULTS

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