

# 68 - F - 214 M

LINCOLN STREET

EXTENSION

WELLAND



BA. 2844  
Site 34-229

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REPORT ON  
SOIL AND FOUNDATION INVESTIGATION  
FOR PROPOSED  
LINCOLN STREET EXTENSION  
WELLAND, ONTARIO.

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PREPARED FOR:

DAMAS AND SMITH LIMITED,  
Consulting Engineers.

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A B S T R A C T

The soil and foundation conditions at the site of the proposed Lincoln Street extension over the Welland River are described.

Sixty to eighty feet of soft to stiff clayey overburden is underlain by the Paleozoic bedrock: a dolomitic limestone interbedded with shale and gypsum layers.

The presence of gypsum in the bedrock causes concern in relation to bearing capacity and solution cavities, and in the design of foundation on bedrock allowance should be made for voids. Steel 'H' piles or drilled caissons which transfer small unit stresses to the bedrock are both acceptable alternatives, and the choice between the two should be mainly an economical one.

Timber friction piles embedded 30 to 40 feet in the overburden are also suggested as a safe and probably economical foundation method.

The stability of the proposed 20 feet high embankment on the east shore is controlled by a soft organic clay stratum extending about 15 feet below the ground surface. Near the end slope this organic clay

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will have to be removed in order to secure an acceptable safety factor. Although the side slopes could be constructed with stabilizing berms because of the large anticipated settlements, it may be more desirable to exchange the organic clay throughout with a compacted fill.

## 1.0. INTRODUCTION

Described in this report are the results of a detailed engineering study of the subsurface conditions at the site of the proposed Lincoln Street Extension over the Welland River. The investigation was authorized by Mr. D.H. Landells, City Engineer, for the City of Welland and the Consulting Engineers on this project, Damas & Smith Limited of Toronto, Ontario.

The proposed project, which will connect Lincoln Street on the east side with Webster Road on the west shore of the Welland River, includes the design and construction of nearly 12,000 feet of roadway and the construction of a three-span 350 ft. long structure over the Welland River.

A preliminary soil investigation had been previously carried out in the general area by the Department of Highways in 1965 to investigate alternate crossing areas. The nearest alternate route lies about 3500 ft. to the north of the present site. Although in the preparation of this report reference was made to these earlier investigations in view of the somewhat different subsurface

conditions, the findings of the earlier investigations are of general interest only.

2.0. SCOPE OF THE INVESTIGATION

The purpose of the investigation can be divided broadly into two categories; to obtain sufficient information about the subsurface conditions in the structure area for the safe foundation design of the structure and the approach fills; and to determine the subgrade conditions along the proposed roadways for the purpose of pavement design.

At the structure and approach fill locations, 8 boreholes were put down, all penetrating to the surface or into the bedrock. Borings were located at the abutment and pier locations and those for the piers were carried out from a raft.

For the purpose of easier access, the subsurface conditions along the roadway were explored by a portable hand auger.

The results of the borings can be found in the Borehole Logs and further details of the fieldwork can be found in the Appendix.



### 3.0. SITE & GEOLOGY

The site is located in the south westerly outskirts of the City of Welland near the intersection of Highway No.3A and the Toronto/Hamilton/Buffalo railway line.

The area is characterized by a generally low relief and a poor drainage. There is only minor variation in the ground surface elevation of the terrain which lies generally at elevation 580 ft. above mean sea level. The drainage of the area is controlled by the Welland River which follows a somewhat meandering course in a shallow channel with an approximately 800 ft. wide flood plain. The drainage of the area is so poor that many undrained depressions remain on the higher ground in a distinctive pattern that is clearly visible on aerial photographs.

The site, as much of Welland County, lies within a clay plain occupying the area between the Niagara Escarpment and Lake Erie. The clays are mostly of lacustrine origin deposited by the now extinct glacial Lake Warren although glacial deposits intermixed with these lacustrine clays are quite common. The

glacial Lake phase was possibly interrupted by at least two major retreats of the ice lobe which resulted in distinctively different deposits. Non-stratified relatively homogeneous deposits were laid down with the ice front close, whereas the heavily stratified and much more plastic and clayey deposits were laid down possibly when the ice front had retreated further. The fact that the deposits are lightly over-consolidated indicates that there could have been a short advance of the ice front over-riding the area.

The thickness of the overburden ranges between 50 and over 100 ft. and are underlain by sedimentary rocks belonging to the Silurian system of the Paleozoic era.

#### 4.0. SUMMARIZED SUBSURFACE CONDITIONS

The surface of the bedrock lies at elevation 500 ± ft., and is overlain by 60 to 80 feet of clayey overburden.

With the exception of a thin granular glacial drift overlying the bedrock, the overburden consists of various clay deposits ranging from a homogeneous

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clay till to a stratified lacustrine clay deposit  
or a highly compressible organic clay stratum  
encountered in the flood plains.

The soil stratigraphy at the structure  
location is summarized in a tabular form in Table I  
on the next page.

T A B L E I

Soil Type	Elevation (feet)	Relevant Soil Properties	Remarks
1. Hard Clay Till	580 - 540	$C_u = 2000 - 4000$ p.s.f. Dessicated and slightly weathered and fissured.	West side only
2. Firm Clay	563 - 557	$C_u = 600 - 1000$ p.s.f.	East side only
3. Very Soft to Firm Organic Clay	567 - 548±	$C_u = 200 - 600$ p.s.f. $w = 160\%$ Void ratio $e = 3.3$ Highly compressible	Between Stations 22 + 50 and 27 + 30
4. Firm to Stiff Stratified Clay	545± - 505±	$C_u = 750 - 2000$ p.s.f. Void ratio $e = 0.7$ Medium compressibility	
5. Very dense Sandy Silt to Clayey Silt Till	505 - Bedrock	'N' = 32 to over 100 blows/ft.	
6. Dolomitic limestone bedrock	Surface 502 - 496	Layers of Shale and Gypsum.	

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For further details of the soil conditions, reference should be made to the individual log of boreholes, the subsurface profile shown on Drawing No.2 and to the Appendix where the significant properties of the different soil types are discussed.

#### 5.0. GROUNDWATER CONDITIONS

Groundwater observations were made in the boreholes during the fieldwork and in piezometers installed in the boreholes at various elevations. The records of the water level observations are shown on the borehole logs.

During drilling heavy artesian flow was encountered near the interface of the till and the bedrock emanating from the granular layers in the till or from the bedrock itself. In each case the total head was measured and was found to be at elevation  $570 \pm$  ft. that is 6 to 7 ft. above the ground surface. The rate of flow through the 3 inch diameter casing was measured between 4 to 8 gallons per minute.

After completing the borings, the artesian conditions was stopped and the boreholes filled

an at least 5 ft. long fast setting cement and bentonite clay plug.

In boreholes Nos. S-1 and S-3 a total of four piezometers were installed. Two piezometers were installed at about elevation 510 ft. one at 530 ft. and one at 550 ft. The observations in these piezometers indicate that the artesian head observed in the bedrock gradually dissipates through the overburden and reaches an equilibrium position corresponding to the river level in the upper zones of the clay deposit.

The water level in the river during the major part of the investigation was at elevation 561.9 ft. During the last 24 hours of the field work however, following a heavy rainstorm, the water level in the river rose more than 2 ft. to elevation 564 ft.

Chemical tests on four samples of the groundwater indicate that the water is neutral with pH values ranging between 7.1 and 7.6. In spite of a strong odour, the water contains only 2 to 19 ppm. hydrogen sulphide ( $H_2S$ ). The dissolved gypsum content ( $CaSO_4 \cdot 2H_2O$ ) however is high; 0.33 and 0.34%.

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The temperature of the groundwater is 17°C. The total amount of the solids in solution and the saturation limit of the water at 17°C temperature will also be determined. The results of these tests will be reported later.

The results of the chemical tests on the groundwater are given in the Appendix.

## 6.0. DISCUSSION

### 6.1. STRUCTURE

Details of the proposed structure were obtained from the Consulting Engineers drawings No. 6751-E014 and E015. Accordingly the proposed structure will be a 350 ft. long three-span continuous structure consisting of two 110 ft. long exterior and a 130 ft. centre span.

On the east bank, an approximately 20 ft. high and 600 ft. long approach embankment will be constructed.

#### 6.1.1. Foundations in Overburden

##### a) Spread footings

Because of the low bearing capacity of the subsoil and the anticipated large settlements under the heavy structural loadings, spread footing foundations are not feasible.

##### b) Friction Piles

The structure could be supported on friction piles embedded in the clay overburden. As the adhesion will likely be the highest for timber piles, tapered



timber piles are considered to be the most suitable. The adhesion factor will likely range between 0.6 (Tomlinson, 1957) and 1.0 (Stermac & Lo, 1964), that is the skin friction will be 60 to 100% of the undrained shear strength. Assuming that 80% of the undrained shear strength will be mobilized and that the average undrained shear strength of the clay is 1200 lbs. p.s.f., the estimated working capacity (S.F. = 3.0) of a size 14 timber pile is 15 to 20 tons for 30 and 40 ft. of embedded length respectively. At the east abutment for 30 feet of embedment the piles will have to be 50 feet long. As longer piles will not be readily available, the work load on timber piles in this area will be limited to about 15 tons.

The actual working capacity of the piles however, should be established by two full scale load tests, one carried out at the west abutment location and one near the east abutment. The tests should be carried to at least twice the design load.

That portion of the piles which is above the low water level in the river, should be pressure treated with creosote.

According to available information, the T.H.B. Railway bridge located a few hundred feet south of the site is also supported on timber piles.

6.1.2. Foundations on Bedrock

At the abutment and pier locations the bedrock was penetrated 20 ft. Inspecting the cores it was found that the bedrock is a dolomitic limestone with occasional 1 to 3 inches thick gypsum lenses or layers. The gypsum content increases with depth and below elevation 490 ± ft. the bedrock contains as much as 20 to 25% gypsum with individual layers up to 12 inches thick. The occurrence of the gypsum was found to be in three modes: horizontal layers, non-horizontal lenses or marbled with the parent bedrock. Although gypsum layers were encountered in every borehole, their extent or the continuity of the individual layers could not be determined.

The presence of the gypsum influences the bearing capacity of the bedrock and causes also concern in relation to solution cavities.

Although cavities were not noticed during drilling or in the cores, one cannot exclude the

possibility of existing or the formation of future solution cavities. Potentially dangerous conditions exist under the west shore. Approximately 200 ft. west of the west abutment there is an operating artesian well and several other capped wells were also noticed in the vicinity. The discharge of the operating well is about 20 imperial gallons per minute and the dissolved gypsum content was measured to be 0.34%. Combining these two figures, this single well alone could remove from within its zone of influence, about 980 lbs. of gypsum per day or about 180 tons per year. Although the amount and extent of solution cannot be predicted in detail, it is felt that the design should allow for some voids.

To lessen the effect of small cavities, the stresses in the bedrock should be kept to a minimum by distributing the structural loads over as large volume of the bedrock as possible.

a) Steel 'H' Piles

In order to distribute the loads of the piers and abutments over a large area of the bedrock, the

lightest steel 'H' pile section should be used. Contact pressure between the pile and the bedrock should be limited to 5000 p.s.i. which would limit the design capacity of a 10BP42 section to 62 kips and to 78 kips for a 12BP53 section.

To further increase the volume of rock supporting the foundations, it is suggested that some of the piles be driven on a slight batter.

b) Caissons

Alternatively the structure could be supported on machine drilled caissons which derive the load carrying capacity from side friction between the bedrock and the caisson rather than from end bearing. Such a design would prevent load concentrations and utilise the skin friction on the sides of the piers to distribute the load to the rock strata. This will minimize the effect of weaker strata and bearing capacity even if partial support is lost due to solution.

The bored piers should be at least 30 inches in diameter to allow visual inspection of the rock and adjustment of the embedded length if necessary.

As small diameter piers present more contact area than larger ones for the same volume, the size of the caissons should be limited to 36 inches in diameter.

The recommended design skin friction value is 3000 lbs. p.s.f. (S.F. > 3.0 ). The anticipated carrying capacity of a 30 inch or 36 inch diameter caisson is therefore 12 tons per ft. or 14 tons per foot of embedment respectively.

In view of the high sulphate content of the groundwater and the presence of water soluble sulphates in the overburden, the concrete in the caisson should be vibrated to a high density and constructed of sulphate resistant Portland cement.

## 6.2. Construction

### a) Abutments

The construction of the perched abutments supported on piles will present no unusual problems. At both locations the pile-caps will be constructed above the groundwater table and the subgrade, the very stiff clay till on the west shore and the compacted granular fill on the east shore will provide

a good base for pouring the concrete.

b) Piers

The boreholes and the river soundings, the results of which are presented in the Appendix, indicate that at the pier locations the water depth ranges between 6 and 9 feet and that the riverbed is covered by 1 to 4 feet of very soft river mud. Within the pier areas these soft deposits should be removed (about 7 ft. at the east pier and 12 inches at the west pier). The underside of the pile caps could be established either at the level of subexcavation or else the grade should be brought up to the desired level with a layer of crushed stone or lean concrete.

As the subsoil is practically impervious during construction, only the water in the river will have to be controlled and excluded from the excavation by a coffer dam or a dyke constructed around the pier locations.

The shear strength profile to be used in the design of steel sheet pile coffer dams can be obtained from Boreholes Nos. S-4 and S-5. Should dykes be considered, it is recommended that at the

east pier, the soft river deposit and organic clay be removed from underneath the dyke. The base of the dyke should be at elevation 545 ft.

In the design of the coffer dam or dyke, allowance should be made for a sudden rise in the river level similar to the one experienced during the fieldwork.

### 6.3. Approach Fill

To meet the grade of the structure a 20 ft. high approach fill will be required over the low lying swampy flood plain on the east side of the river. The soil profile consists of 7 ft. of firm clay underlain by 5 to 11 feet of soft organic clay, 45 feet of firm to stiff stratified clay and finally 10 feet of glacial till over the bedrock.

The 20 ft. of fill is heavy enough to cause concern about the stability of the proposed embankment and possible large settlements in the normally consolidated organic clay and the lightly overconsolidated deep stratified clay deposits. As design criterion for the embankment, a safety factor of 1.3 was adopted provided the calculations were based on

the average low undrained shear strength of the soil.

a) Stability of Side Slopes

It was found that the proposed embankment with 2 to 1 side slopes will most likely fail (safety factor less than unity) along shallow circular failure plains extending into the soft organic clay.

The results of stability calculations based on total stresses are shown on Enclosure No.22. As shown here, 40 feet long and 7½ ft. high stabilizing berms will have to be constructed on both sides of the embankment to achieve a safety factor of 1.3. Stabilizing berms will be required between Station 22 + 50 and Station 25 + 00.

The embankment will also have an adequate safety factor (1.32) if under the embankment the organic clay is removed and the fill extended to elevation 546 ± ft.

b) Stability of End Slopes

As the shear strength of the organic clay decreases in the westerly direction, the stability of the end slopes was found to be more critical.



Even with a double berm extending to the east pier, the safety factor was less than 1.10 and to provide stable conditions the organic clay will have to be removed. The extent of the recommended soil exchange together with the calculated safety factors are shown on Enclosure No.21. The minimum calculated safety factor in terms of total stresses is 1.27 which, considering the conservatism exercised in the selection of the design shear strength parameters of the subsoil, is acceptable.

c) Settlement

The consolidation characteristics of the organic and stratified clay were determined in the laboratory by conventional oedometer tests. The void ratio - pressure curves of these tests are shown on Enclosures Nos. 14 and 15.

The results of the settlement calculations are as follows:

Centreline (Station 22 + 50 to 25 + 00)	30 inches
Toe of slope	10 inches
Toe of berm	2 inches

Much of this settlement - about 21 inches at the centre line - is due to the consolidation settlement of the highly compressible organic clay stratum. Should this stratum be removed and the embankment extended to elevation 546 ± ft., the maximum settlement under the centre line is estimated to be 9.5 inches and about 3.5 inches at the toe.

The rate of consolidation, especially in the organic clay, will be slow and it is estimated that 8 to 13 years will be required for 90% of the consolidation to take place.

With the organic clay removed, the time required for 90% of consolidation will be reduced to about 7 years.

6.4. ROAD CONSTRUCTION

Forming part of this project is the construction of about 1200 feet of roadway as indicated on Drawing No.1. The road is classified as a two-lane arterial road with heavy traffic, and the plans call for a flexible type of pavement.

a) Grade Line

Only minor variations in grade are expected with fills and cuts not exceeding 5 ft. in depth.

b) Subgrade

The subgrade conditions along the proposed roadway are shown in the Appendix under "Composite Log of Boreholes No. R-1 to R-16 inclusive".

Under 3 to 18 inches of topsoil, the subgrade is a light to heavy clay, corresponding to Groups No. 11, 12, and 13 in the U.S.B.S. Soil Classification System.

The liquid limit of the subsoil ranges between 48 and 67% , the plastic limit between 21 and 27% and the plasticity index is 26 - 41. The natural moisture content varies between 23 and 25%.

The grain size distribution characteristics of the subsoil are shown on Enclosures Nos. 18, 19 and 20. These indicate 5 - 20% of fine sand content, 10 - 45% of silt, and 35 - 85% of clay fraction.

The California bearing ratio of the subsoil compacted under the Standard Proctor Compaction effort at natural moisture content and saturated for a period of 90 hours is 2.5.

The groundwater table was observed at 2 to 5 ft. below the ground surface. In low lying areas of the site the water was standing at or above the ground surface.

c) Pavement Design

The total thickness of the bituminous wearing surface should be  $5\frac{1}{2}$  inches consisting of  $\frac{3}{4}$  of an inch sand asphalt binder,  $3\frac{1}{2}$  inches of HL 4, 5, 6 or 8 binder coarse  $1\frac{1}{2}$  inch HL 1, 3, 4 or 5 surface coarse.

This bituminous pavement should be constructed on top of 9 inches of granular 'A' base course and 15 inches of granular 'B' or sand cushion sub base course.

Alternatively a deep strength design could also be considered, which in the present case should consist of 12 inches of asphalt on top of 6 inches of granular 'A' base course.

d) Source of Granular Material

Search for granular borrow material in the area indicates that granular 'A', granular 'B' and sand cushion materials conforming to D.H.O. specifications (Form 314 & 316) are all available within 8 - 10 miles of hauling distance from the construction site.

For granular 'A' base course, most likely quarried, crushed stone will have to be used. The nearest source may be the Port Colborne Quarry about 1-3/4 miles north of Port Colborne or the Law Quarry about 2 miles west of Port Colborne. In addition to this, there are two sand and gravel pits in the Fonthill area which may have natural sand and gravel deposits in sufficient quantity for granular 'A' material.

Granular 'B' material may be available from the Moyer Gravel Pit 4 miles west of Fonthill or from

the Fonthill Sand and Gravel Pit, 2 miles west of Fonthill. Much of this material however, is sold for concrete aggregates and therefore may not be available at economical prices.

Sand cushion is readily available in the area, the nearest sources would be the previously mentioned Moyer and Fonthill Sand and Gravel pits.

e) Embankment

Material available from the excavation of the drainage ditches and cut areas can be used for general fill in the embankments. The natural moisture content of the material within the investigated depth, which was generally 4 ft. below the present grade, is near the optimum moisture content of the clay and therefore suitable without much additional handling for compaction. However, if the construction is carried out during a wet season, the moisture content could increase significantly and be too high for proper compaction. Therefore it is recommended that provision should be made to use an alternative source of fill material which is less susceptible to variations in the moisture content.

The excavated material could be compacted by heavy sheeps foot rollers or heavy rubber-tired compactors. The embankment should be compacted to at least 95% of its Standard Proctor Maximum dry density except in the top 18 inches of the subgrade where the degree of compaction should be 100%.

All organic topsoil should be removed from under the embankment.

f) Drainage

As the area is characterized by poor drainage, one of the major design problems will be to provide adequate drainage for the roadway. The groundwater table should be kept by side ditches, or sub-drains if necessary, at least 4 feet below the pavement.

DOMINION SOIL INVESTIGATION LIMITED

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IPL/jm



A P P E N D I X

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I. PROCEDURES

a) Fieldwork

Fieldwork was carried out between May 27th and June 26th, 1968 during which period a total of 26 boreholes were put down, comprising a total footage of 538 feet. Ten boreholes designated by numbers and the letters 'S' and 'A' were put down at the proposed structure and approach embankment location and 16 boreholes were put down along the centre line of the proposed road construction. The boreholes for the roadway are designated by the letter 'R' and numbered 1 to 16 inclusive.

The boreholes in the field were laid out by the Survey Crew of Dominion Soil Investigation Limited with the aid of the Consulting Engineers Drawing No. 6751-E003. Since at the time of the soil investigation the ground survey for the project has not yet been carried out, the boreholes were tied in to existing features, such as street lines, fence lines, existing structures, etc.

Elevations were referred to two City bench marks supplied to us by the Engineering Department

of the City of Welland. The location of the bench marks and their descriptions are indicated on Drawing No.1.

The borings for the structures were carried out by a skid-mounted diamond drill machine equipped for soil testing and sampling using 3-inch diameter Nx size casings and washboring technique. The borings in the river were performed on a raft.

The sampling consisted of the recovery of disturbed and undisturbed samples, alternating at 2½ ft. intervals. Undisturbed samples were recovered by a 2-inch diameter shelly tube sampler whereas the disturbed samples were recovered by a 2-inch outside diameter split barrel sampler driven into the undisturbed ground by the standard driving energy of 350 ft. lbs. The number of blows required for 12 inches of advancement were recorded as the Standard Penetration Resistance or 'N' values. In between the soil samples, the undrained shear strength of the subsoil was measured by vane tests, using a 2-inch diameter 6 inch long vane.



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BxT and BxL cores were recovered from the bedrock at each abutment and pier location. The bedrock was penetrated for a maximum depth of 20 ft.

Where artesian groundwater conditions were encountered, the pressure head was measured by extending the casing above the ground or river surface. On completion of the boreholes the artesian condition was stopped and the boreholes were plugged immediately above the water bearing strata using fast setting cement and bentonite plugs. The piezometric surfaces at various depths in the overburden were observed by piezometers installed in the boreholes at various elevations. The depths of the piezometers and the observed water levels are indicated on the borehole logs.

Because of the difficulties of getting access to the borehole locations along the proposed roadway, the boreholes for the road construction were carried out by a portable hand auger recovering only disturbed auger samples.



b) Laboratory work

The recovered soil samples were shipped to the laboratory for testing.

The testing programme consisted of a number of identification tests: Liquid and Plastic Limits, Plasticity Index and Grain Size Distribution, determination of the Natural Water Content, the Liquidity Index, the Bulk Unit Weight and the void ratio of the soil.

In addition to these index properties, the shear strength parameters and consolidation characteristics of the different soil types were determined by unconfined compression tests, laboratory torvanes, unconsolidated undrained triaxial tests and consolidated undrained triaxial tests with pore pressure measurements.

Chemical tests were also performed on four samples obtained from the groundwater to determine: the pH, the hydrogen sulphide ( $H_2S$ ) content, the gypsum content, the total amount of solids in solution and the saturation limit of the water.

The laboratory test results are tabulated on Enclosure Nos. 11 to 13 inclusive.

## II. DESCRIPTION OF SOIL TYPES

### Clay Till (CI)

The west bank of the river is made up of a reddish-grey coloured clay till with some embedded gravel. The stratum extends from below the ground surface to elevation about 540 ± ft. and its horizontal extent in the easterly direction is estimated to be about Station 27 + 00, that is approximately at the middle of the Welland River.

The significant index properties of the till are as follows:-

Liquid Limit	35 - 39%
Plastic Limit	18 - 21%
Plasticity Index	15 - 18%
Natural Moisture	
Content	21 - 23%
Liquidity Index	0.1 - 0.3

These tests indicate a clay of intermediate plasticity and of generally very stiff consistency.

The undrained shear strength of the till was measured in-situ by vane tests which gave values

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ranging between 1800 and 4300 lbs.p.s.f. Even higher undrained shear strength values can be inferred from the Standard Penetration tests above elevation 560 ft. ('N' = 37 to 40 blows per foot).

Clay (CI - CH)

The uppermost soil stratum encountered in the flood plain on the east side of the river is 5 to 17 feet thick grey coloured in-organic clay.

In the laboratory the following index properties were determined.

Liquid Limit	47 - 62%
Plastic Limit	26 - 30%
Plasticity Index	18 - 33%
Natural Moisture	
Content	31 - 51%
Liquidity Index	0.65
Unit weight	106 - 128 lbs.p.c.f.

The undrained shear strength of the clay was measured in-situ in the field by vane tests and in the laboratory by Torvane and Unconfined Compression tests.

The measured values are as follows:-

Field Vane Tests	600 - 3000 lbs.p.s.f.
Torvane	900 lbs. p.s.f.
Unconfined shear strength	1700 lbs. p.s.f.



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Organic Clay (OH)

Between Stations 22 + 50 and about 27 + 00 a 6 to 14 ft. thick, dark brown organic clay stratum was encountered. The under side of the stratum lies between elevation 550 and about 546 ft.

The physical properties of the deposit are as follows:-

Liquid Limit	61 - 78%
Plastic Limit	37 - 47%
Plasticity Index	19 - 39
Natural Moisture	
Content	110 - 175%
Liquidity Index	2.3 - 4.7
Natural Unit Weight	77 - 81 lbs.p.c.f.

The shear strength parameters of the soil were determined in the field by in-situ field vane tests and in the laboratory by Torvane, Unconfined Compression Tests, Unconsolidated quick triaxial test and Consolidated Undrained triaxial test with pore pressure measurements. The results are tabulated on Enclosure Nos. 11, 12 and 13, and are as follows:

Field vane tests	250 - 800 lbs.p.s.f.
Torvane	390 lbs. p.s.f.
Unconfined Compression test	135 - 300 lbs. p.s.f.
Undrained quick triaxial test	135 - 280 lbs. p.s.f.



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From the tests results it can be concluded that the undrained shear strength of the soil increases in the easterly direction.

The results of the Consolidated Undrained Triaxial test with pore pressure measurements indicate an effective angle of shearing resistance  $\phi' = 38^\circ$ . The cohesion intercept  $c'$  was equal to zero consistent with normally consolidated clays.

The consolidation properties of the organic clay are shown on Enclosure No. 14 indicating a normally consolidated clay with an initial void ratio of 3.3. The modulus of compressibility for the pressure ranges to be considered under the proposed embankment is ( $K = \frac{1}{\sigma_v}$ ) 5 tons per square foot.

#### Stratified Clay (CL - CH)

The significant soil deposit underlying the site is reddish-brown and grey coloured heavily stratified clay deposit, the surface of which was encountered between elevation 550 and 540 feet.

The thickness of the stratum ranges between 30 and 50 feet. The layers were found to be  $\frac{1}{2}$  to 3 inches thick with occasional very thin layers of grey



- 35 -

clay and silt.

The index properties of the material are summarized below:-

Liquid Limit	26 - 53%
Plastic Limit	17 - 23%
Plasticity Index	9 - 30
Natural Moisture Content	21 - 42%
Liquidity Index	0.5 - 0.9
Unit Weight	108 - 128 lbs.p.c.f.

The consistency of the stratum is generally firm to stiff with 'N' values ranging between 3 and 8 blows per foot. The undrained shear strength of the deposit appears to be decreasing in the easterly direction as indicated on Drawing No.2.

The following undrained shear strength values were measured:-

Field vane shear strength	750 - 2000 lbs.p.s.f.
Torvane shear strength	530 - 700 lbs. p.s.f.
Unconfined shear strength	430 - 810 lbs.p.s.f.

The shear strength of the clay in terms of effective stresses was determined by Consolidated Undrained Triaxial Tests during which pore pressures were measured at the base of the sample. Mohr's circles representing the stress conditions of the samples at failure (maximum deviator stress  $\sigma_1 - \sigma_3$ )

-36 -

are plotted on Enclosure No.16 indicating that the cohesion intercept  $c' = 2.4$  p.s.i. and the angle of shearing resistance  $\phi' = 21^\circ$ .

The consolidation properties of the clay are presented on Enclosure No.15 indicating that the clay is overconsolidated by a pressure in excess of the existing overburden pressure of about 1.5 tons per square foot. The tested sample had a relatively low initial void ratio ( $e_0 = 0.73$ ). The modulus of compressibility,  $K$ , of the clay for a pressure increment of 1 ton per square foot over the existing overburden pressure is about 80 tons per square foot ( $m_v = 0.0125$  square feet per ton).

#### Clay to Sandy Silt Till

The surface elevation of this deposit ranges between 513 and 500 feet. The thickness of the stratum is quite variable, ranging between 2 ft. in Borehole 3-5 to over 22 feet in Borehole 8-1.

The composition of the till varies considerably over the site.

The matrix of the till is a clayey silt or sandy silt with embedded coarse sand or fine gravel. Occasional gravelly sand layers or lenses were also encountered.

Based on the Standard Penetration test results which gave 'N' values generally exceeding 100 blows per foot, the relative density or consistency of the till is estimated to be very dense or hard.

#### Dolomitic Limestone Bedrock

Within the area of the structure, the surface of the bedrock was encountered between elevation 495 and 502 feet.

The bedrock was cored at four locations with Bxt and BxL diamond bits and core barrels to a depth of 20 feet.

The rock has a buff to grey colour and is made up of dolomitic limestone with bands of shaley limestone or shale. Interbedded in the bedrock are layers or lenses of gypsum. The gypsum layers are up to 12 inches thick, but 2 to 5 inch thick layers are more representative.

III. GROUND WATER ANALYSIS

Borehole Number	pH Value.	Hydrogen Sulphide (H <sub>2</sub> S)	Gypsum (CaSO <sub>4</sub> H <sub>2</sub> O)
S-1	7.5	6 ppm	0.33%
S-4	7.1	1.6 ppm	0.33%
S-5	7.5	7 ppm	0.33%
Artesian well	7.2	19 ppm	0.34%

Our Ref. No. 8-5-25.

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IV. COMPOSITE LOG OF BOREHOLES R-1 - R-16  
INCLUSIVE

Borehole No.	Elevation Feet	Depth	Soil Type	Water Level Depth below Ground surface
R-1	581.7	0 - 1.0 ft.	Topsoil	5.0 ft.
		1.0 - 6.0 ft.	Light Clay	
R-2	582.6	0 - 0.5 ft.	Gravelly Sand Fill	5.2 ft.
		0.5 - 1.5 ft.	Clayey Silt Fill	
		1.5 - 3.0 ft.	Silty Clay	
		3.0 - 6.0 ft.	Light Clay	
R-3	570.3	0 - 1.0 ft.	Topsoil	2.7 ft.
		1.0 - 4.0 ft.	Light Clay	
R-4	584.2	0 - 0.9 ft.	Topsoil	Nil
		0.9 - 4.0 ft.	Light Clay	
R-5	581.9	0 - 0.7 ft.	Topsoil	1.8 ft.
		0.7 - 2.0 ft.	Medium Clay	
		2.0 - 6.0 ft.	Heavy Clay	
R-6	578.9	0 - 0.7 ft.	Topsoil	2.0 ft.
		0.7 - 4.0 ft.	Medium Clay	
R-7	581.9	0 - 1.5 ft.	Topsoil	4.8 ft.
		1.5 - 3.0 ft.	Medium Clay	
		3.0 - 6.0 ft.	Heavy Clay	

Our Ref. No. 8-5-25.

Si

Borehole No.	Elevation Feet	Depth	Soil Type	Water Level Depth below Ground surface
R-8	578.8	0 - 1.0 ft.	Topsoil	Nil
		1.0 - 4.0 ft.	Heavy Clay	
R-9	587.1	0 - 0.8 ft.	Topsoil	4.8 ft.
		0.8 - 6.0 ft.	Light Clay	
R-10	569.9	0 - 1.0 ft.	Topsoil	2.5 ft.
		1.0 - 3.0 ft.	Light Clay	
		3.0 - 4.0 ft.	Medium Clay	
R-11	575.8	0 - 1.0 ft.	Topsoil	Nil
		1.0 - 4.0 ft.	Light Clay	
R-12	573.4	0 - 0.7 ft.	Topsoil	3.3 ft.
		0.7 - 4.0 ft.	Light Clay	
R-13	584.1	0 - 0.3 ft.	Topsoil	4.8 ft.
		0 - 4.5 ft.	Light Clay	
		4.5 - 6.0 ft.	Medium Clay	
R-14	587.8	0 - 0.4 ft.	Topsoil	5.0 ft.
		0.4 - 4.5 ft.	Light Clay	
		4.5 - 6.0 ft.	Medium Clay	

Our Ref.No. 8-5-25.



Borehole No.	Elevation Feet	Depth	Soil Type	Water Level Depth below Ground surface
R-15	586.5	0 - 0.7 ft.	Topsoil	
		0.7 - 4.5 ft.	Light Clay	3.8 ft.
		4.5 - 5.5 ft.	Medium Clay	
R-16	587.0	0 - 0.3 ft.	Topsoil	
		0.3 - 4.0 ft.	Light Clay	3.0 ft.



## V. RIVER SOUNDINGS

Sta.	100 ft. south of CL		Centre Line		100 ft. north of CL	
	Water Depth	Thickness of River Mud	Water Depth	Thickness of River Mud	Water Depth	Thickness of River Mud
0 + 00	E A S T   S H O R E   L I N E					
0 + 20	1.5'	0.5'	2.5'	1.0'	1.3'	1.5'
0 + 40	1.7'	4.0'	3.0'	1.0'	2.8'	4.0'
0 + 60	5.8'	4.0'	6.2'	2.0'	6.5'	4.0'
0 + 80	8.0'	4.0'	8.3'	4.0'	8.0'	5.0'
1 + 00	8.9'	4.0'	8.3'	4.0'	9.0'	5.0'
1 + 20	9.0'	4.0'	9.0'	4.0'	10.0'	2.0'
1 + 40	8.8'	4.0'	8.5'	4.0'	7.7'	1.0'
1 + 60	7.9'	2.0'	7.5'	4.0'	7.2'	0.5'
1 + 80	6.5'	1.0'	6.4'	4.0'	5.9'	0.5'
2 + 00	5.5'	1.0'	5.5'	1.0'	4.0'	0.5'
2 + 20	3.5'	0.5'	4.0'	1.0'	2.2'	0.5'
2 + 40	2.0'	0.5'	2.0'	0.5'	1.5'	1.0'
2 + 55	-	-	-	-	WEST SHORE LINE	
2 + 60	0.6'	0.5'	1.0'	1.0'	WEST SHORE LINE	
2 + 66	W E S T   S H O R E   L I N E				-	-



Our Ref. No. 8-5-25.

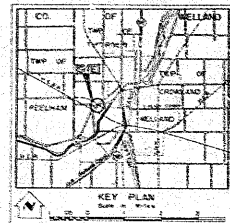
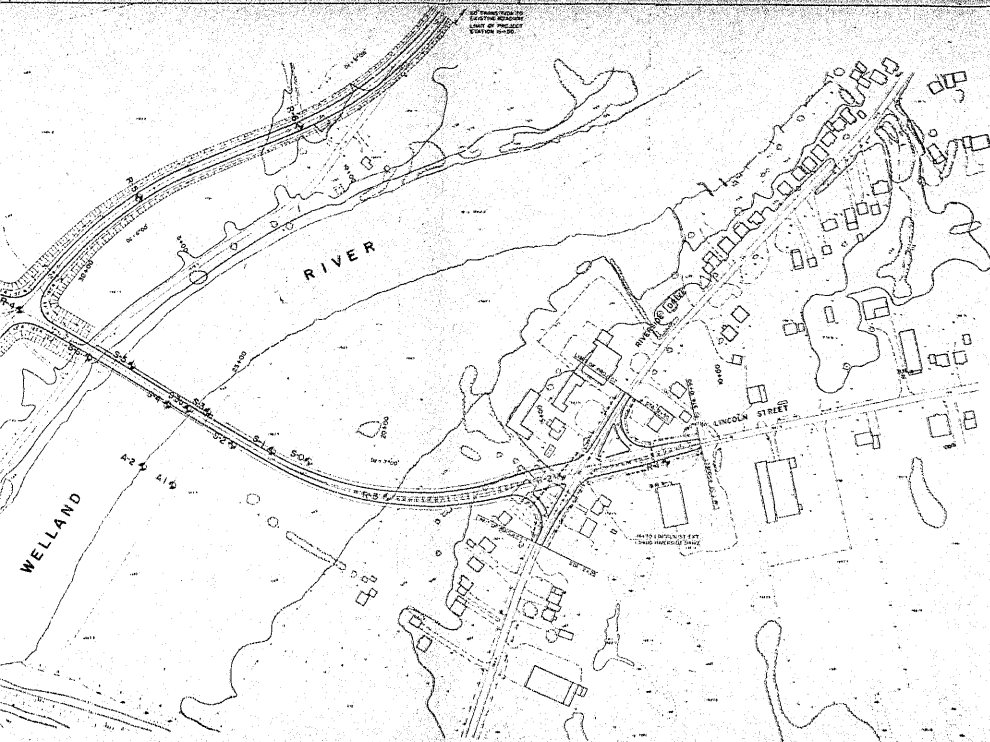


R E F E R E N C E S

1. "Preliminary Foundation Investigation for Functional Study for Connecting Link between City of Welland and Future Highway #406" Department of Highways Report, 1966.
2. "The Adhesion of Piles Driven in Clay Soils" by M.J. Tomlinson.  
Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, 1957.
3. "Some Pile Loading Tests in Stiff Clay" by K.Y.Lo and A.G. Stermac.  
Canadian Geotechnical Journal Vol.I No.2, 1964.

i n c l o s u r e s .





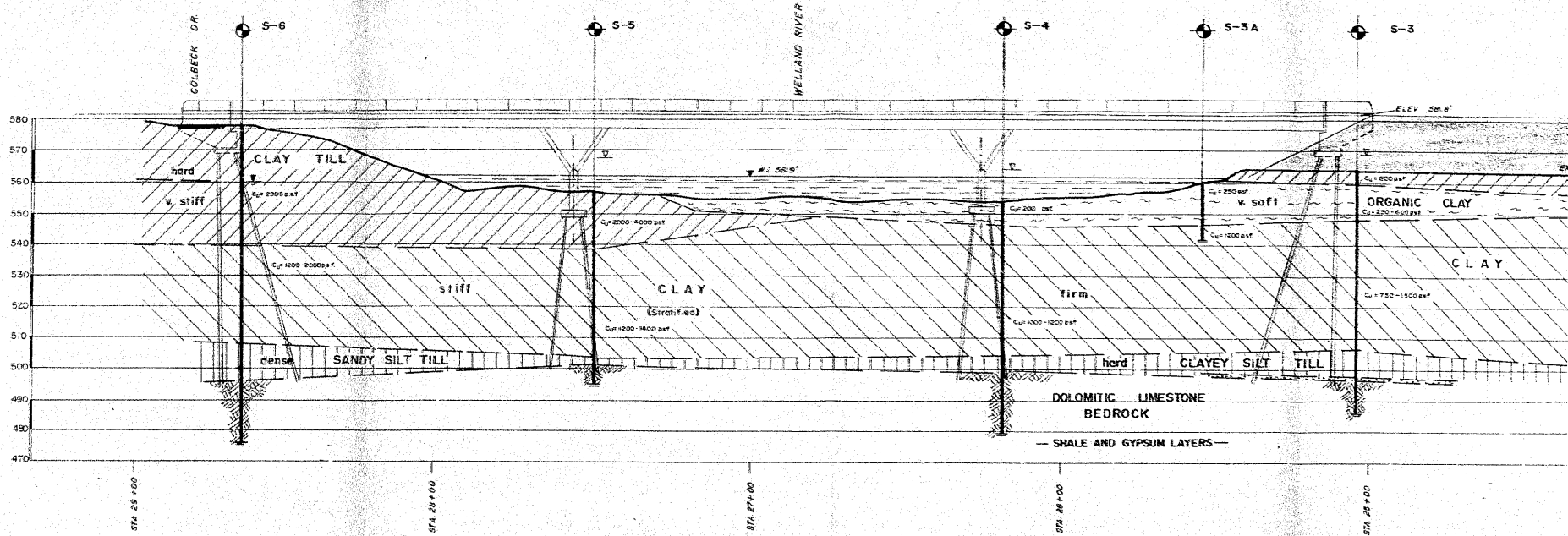
NUMBER	ELEVATION	STATION	OFF-SET
R-1	151.75	11+90	25' LT
R-2	152.64	14+45	"
R-3	150.25	15+65	"
R-4	154.61	12+00	15' RT
R-5	153.41	23+00	15' RT
R-6	152.26	24+00	"
R-7	148.19	25+00	20' RT
R-8	151.90	25+60	"
R-9	141.30	26+30	25' LT
R-10	145.90	27+35	25' LT
R-11	157.25	28+75	25' LT
R-12	154.19	31+00	"
R-13	152.06	32+20	"
R-14	149.96	40+00	"
R-15	155.60	45+20	"
R-16	153.44	52+00	"
R-17	154.42	55+20	"
R-18	157.40	60+20	"
R-19	146.51	63+20	"
R-20	157.73	70+20	"
R-21	157.64	74+00	"
R-22	151.80	79+70	"
R-23	151.43	84+00	"
R-24	154.19	84+00	"
R-25	151.90	84+00	"
R-26	151.90	84+00	"
R-27	151.90	84+00	"
R-28	151.90	84+00	"
R-29	151.90	84+00	"
R-30	151.90	84+00	"

#### SEARCH MARKS

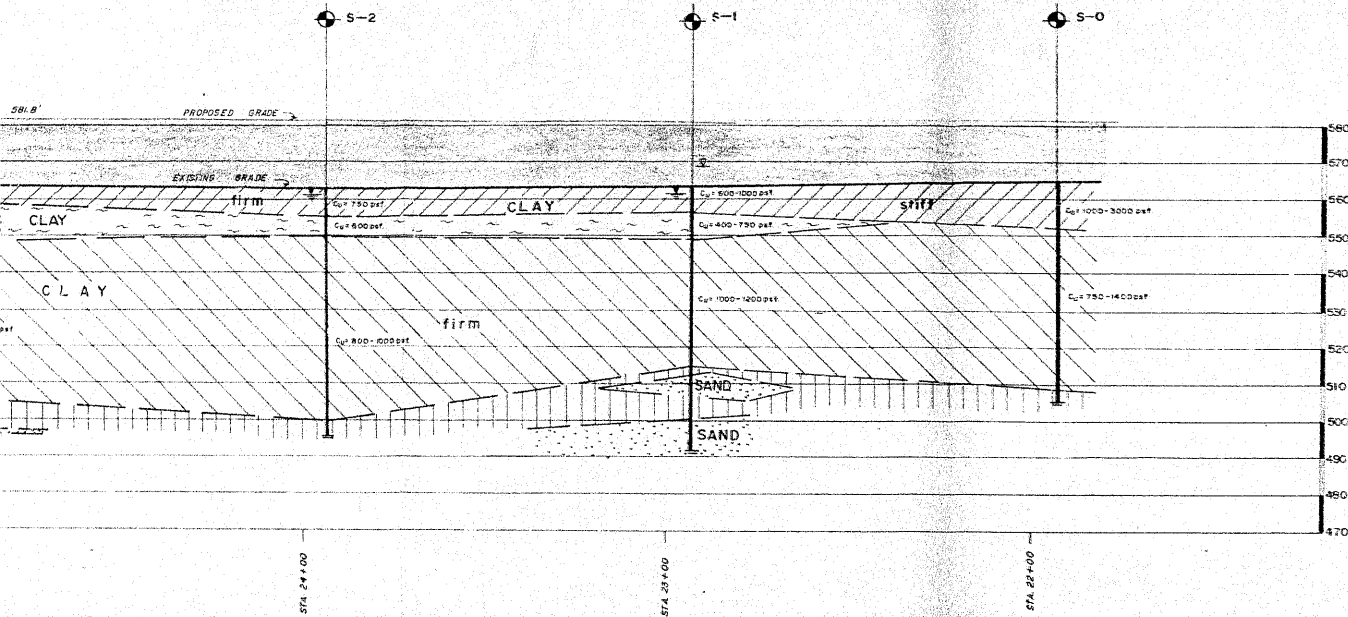
SM #1 - SOUTH WEST CORNER OF CONCRETE  
PAD (CONCRETE WALL)  
ELEVATION 152.10 FEET

SM #2 - EAST SIDE WALL OF FRONT DOOR, EAST  
SIDE (CONCRETE WALL)  
ELEVATION 152.10 FEET

§ DOWNSIDE SOIL INVESTIGATION LIMITED, CONSULTING SOIL & FOUNDATION ENGINEERS <b>DAMAS AND SMITH LIMITED</b> CONSULTING ENGINEERS TORONTO - LONDON - WINNIPEG				<b>CITY OF WELLAND</b> <b>LINCOLN STREET EXTENSION</b> <b>BORHOLE LOCATION PLAN</b>			
DATE	BY	CHKD	APP'D	DATE	BY	CHKD	APP'D



SCALE 1" = 20'



-LEGEND-

- OBSERVED ARTESIAN HEAD
- ▽ GROUND WATER TABLE
- $C_u$  = UNDRAINED SHEAR STRENGTH
- END OF BOREHOLE

DAMAS AND SMITH LIMITED

CITY OF WELLAND—  
LINCOLN STREET EXTENSION

SUBSURFACE PROFILE

ALONG CENTRELINE

DOMINION SOIL INVESTIGATION LIMITED  
CONSULTING ENGINEERS

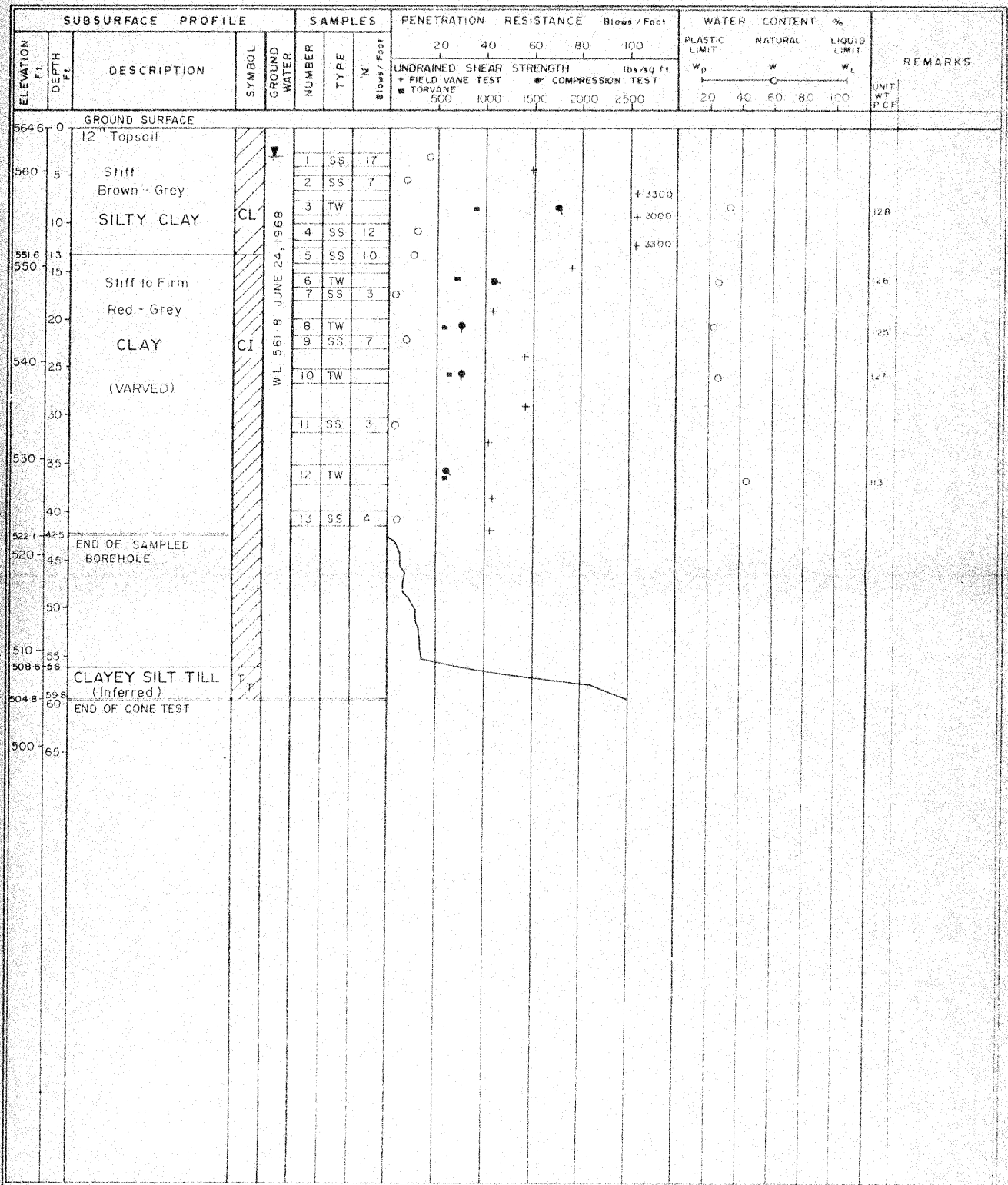
DATE	DRAWN BY	CHECKED BY	APPROVED BY	6-5-25
JUNE 1968	20			DWG. 2

# LOG OF BOREHOLE S0

Our Reference No. 8-5-25  
CLIENT: DAMAS & SMITH LTD  
PROJECT: LINCOLN ST EXTENSION  
LOCATION: WELLAND, ONT  
DATUM ELEVATION: CITY

DRILLING DATA  
Method: WASHBORING  
Diameter: 3"  
Date: JUNE 24, 1968

Enclosure No. 1



# LOG OF BOREHOLE SI.....

Our Reference No 8-5-25

Enclosure No 2

CLIENT: DAMAS & SMITH LIMITED  
PROJECT: LINCOLN ST. EXTENSION  
LOCATION: WELLAND, ONT.  
DATUM ELEVATION: CITY

## DRILLING DATA

Method: WASHBORING  
Diameter: 3 1/2" (NX1)  
Date: JUNE 4-7 1968

SUBSURFACE PROFILE		SAMPLES		PENETRATION RESISTANCE					WATER CONTENT %			REMARKS					
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows / Foot	Blows / Foot					PLASTIC LIMIT	NATURAL	LIQUID LIMIT		
								20	40	60	80		100	W <sub>p</sub>	W	W <sub>L</sub>	
								UNDRAINED SHEAR STRENGTH					COMPRESSION TEST				
								+ FIELD VANE TEST					lbs./sq. ft.				
								500 1000 1500 2000 2500									



# LOG OF BOREHOLE S2

Our Reference No. 8-5-25

Enclosure No. 3

CLIENT DAMAS & SMITH LIMITED  
PROJECT LINCOLN ST EXTENSION  
LOCATION WELLAND, ONT.  
DATUM ELEVATION: CITY

## DRILLING DATA

Method: WASHBORING  
Diameter: 3 1/2" (NX)  
Date: JUNE 3-4, 1968

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE					WATER CONTENT			REMARKS	
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N Blows / Foot	Blows / Foot					%			
								20	40	60	80	100	PLASTIC LIMIT	NATURAL		LIQUID LIMIT
								UNDRAINED SHEAR STRENGTH								
								lb/sq. ft.								
								+ FIELD VANE TEST					W <sub>p</sub> W      W <sub>L</sub>			
								■ TORQUE					20    40    60    80    100			
								500    1000    1500    2000    2500					UNIT WT P.C.F.			

563.0	0	GROUND SURFACE															
560	3	12" Topsoil			1	SS	2	+									
	5	Firm Grey CLAY	CI		2	TW											
555.5	7.5	Trace of Organics			3	SS	1	+									
	10	Soft Dark Brown ORGANIC CLAY	OH		4	TW											
550	13				5	SS	3	+									
	15				6	TW											
	20	Firm Red - Grey CLAY (VARVED)	CI		7	SS	2	+									120 TRIAXIAL COMPR. TEST P.P.M. ENCL. NO. JZ...
540	25		CL		8	TW											123 TRIAXIAL COMPR. TEST P.P.M. ENCL. NO. 15...
	30				9	SS	3	+									129
	35				10	TW											
530	40				11	SS	5	+									
	45				12	TW											
520.4	42.6	END OF SAMPLED BOREHOLE			13	SS	3	+									
510	55	CLAY (Inferred)															
500	63	CLAYEY SILT TILL (Inferred)	TT														
494.6	68.4	END OF CONE TEST															
490	75																
	80																
	85																
	90																

# LOG OF BOREHOLE S3

Our Reference No. 8-5-25

Enclosure No. 4

CLIENT: DAMAS & SMITH LIMITED  
PROJECT: LINCOLN ST EXTENSION  
LOCATION: WELLAND, ONT.  
DATUM ELEVATION: CITY

## DRILLING DATA

Method: WASHBORING  
Diameter: 3 1/2" (NX)  
Date: MAY 28-31, 1968.

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE					WATER CONTENT %			REMARKS		
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N <sub>v</sub> Blows / Foot	Blows / Foot					PLASTIC LIMIT			UNIT WT P.C.F.	
								20	40	60	80	100	NATURAL				
								UNDRAINED SHEAR STRENGTH					LIQUID LIMIT				
								+ FIELD VANE TEST					COMPRESSION TEST		W <sub>p</sub> W      W <sub>L</sub>		
								500   1000   1500   2000   2500					lb/sq. ft.		20   40   60   80   100		
													TRIAXIAL				
GROUND SURFACE																	
563.3	0	12" Topsoil															
560	3	Firm Grey CLAY	CI		1	SS	4										
558.3	5				2	SS	1							0.60%			
		Soft			3	TW								0.65%	7.8		
	10	Dark Brown	OH		4	SS	1							0.60%			
		ORGANIC CLAY			5	TW								0.22%	81		
550	15				6	SS	2							0.80%			
547.3	18.0				7	TW											
	20				8	SS	3								11.9		
	25				9	TW									12.3		
540	25	Firm to Stiff			10	SS	5								COND.		
	30				11	TW									12.1		
		Red-Grey	CI		12	SS	8										
530	35	CLAY	CH		13	TW									10.5		
	40				14	SS	4										
		(VARVED)			15	TW									11.6		
520	45				16	SS	5										
	50				17	TW									12.4		
510	55				18	SS	30										
505.3	58	Hard			19	SS	50										
500	60	Reddish Grey	CL		20	WS											
		CLAYEY SILT			21	RC	40%										
	65	With Embedded Gravel			22	RC	66%										
		(GLACIAL TILL)															
496.3	67	SAND															
495.3	68	Grey															
	70	Shaley Limestone															
490	75	BEDROCK															
485.3	78	END OF BOREHOLE															
480	80																
	85																
	90																

ARTESIAN WATER  
ENCOUNTERED AT  
EL. 496.3 FT.  
MEASURED HEAD  
AT EL. 569.6 FT.

PIZ HEAD EL. 565.3' JUNE 21, 1968

PIZ HEAD EL. 564.7 JUNE 21, 1968

ARTESIAN WATER  
ENCOUNTERED AT  
EL 496.3 FT  
MEASURED HEAD  
AT EL 569.6 FT

PIZ HEAD EL 565.3, JUNE 21, 1968

PIZ HEAD EL 564.7, JUNE 21, 1968

# LOG OF BOREHOLE S-3A

Our Reference No. B-5-25

Enclosure No. 5

CLIENT: DAMAS & SMITH LTD  
PROJECT: LINCOLN ST EXTENSION  
LOCATION: WELLAND, ONT  
DATUM ELEVATION: CITY

## DRILLING DATA

Method: WASHBORING  
Diameter: 3 1/2" (NX)  
Date: JUNE 18, 1968

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE					WATER CONTENT %			REMARKS									
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N Blows / Foot	Blows / Foot					PLASTIC LIMIT	NATURAL		LIQUID LIMIT								
								20	40	60	80	100												
								UNDRAINED SHEAR STRENGTH																
								lb/sq. ft.					Wp			W			WL			UNIT WT. P.C.F.		
								+ FIELD VANE TEST					● COMPRESSION TEST											
								■ TOREVALE					▲ TRIAXIAL											
								500 1000 1500 2000 2500										20 40 60 80 100						
RIVER LEVEL																								
561.9	0	WATER																						
560		Very Soft			1	SS																		
	5	Dark Brown			2	TW																		
		ORGANIC CLAY	OH		3	SS	1										0.76%	79						
	10				4	TW																		
550					5	SS	1										0.64%	20						
	15				6	TW																		
546.4	15.5	Stiff Grey			7	SS	5																	
	20	CLAY	CI															125						
541.9	20	END OF BOREHOLE																						
540																								
	25																							

VERTICAL SCALE: 1 inch to 10 feet

DOMINION SOIL INVESTIGATION LIMITED

MADE: S O

CHECKED:

# LOG OF BOREHOLE S-4

Our Reference No. 8-5-25

Enclosure No. 5

CLIENT: DAMAS & SMITH LIMITED  
PROJECT: LINCOLN ST. EXTENSION  
LOCATION: WELLAND, ONT  
DATUM ELEVATION: CITY

DRILLING DATA  
Method: WASHBORING  
Diameter: 3 1/2" (NX)  
Date: JUNE 13-17, 1968

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE					WATER CONTENT %			REMARKS
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N°	Blows / Foot	UNDRAINED SHEAR + FIELD VANE TEST	STRENGTH	lbs/sq ft.	PLASTIC LIMIT	NATURAL	LIQUID LIMIT	
								20 40 60 80 100							
								500 1000 1500 2000 2500							
561.9	0	RIVER LEVEL													
560	5	WATER													
553.9	8	Very Soft CLAY	CH		A	SS	0		+						
551.9	10	Very Soft Dark Brown ORGANIC CLAY	OH		B	SS	0		+						
546.4	15				b	SS	1								
540	20	Firm to Stiff			3	TW			+						
540	25	Red - Grey	CH		4	SS	4			+					
530	30	CLAY	CI		5	TW				+					
530	35	(VARVED)			6	SS	6			+					
520	40				7	TW									
520	45				8	SS	4			+					
510	50				9	TW				+					
510	55				10	SS	4								
503.9	58	Very Dense			11	TW									
500	60	SANDY SILT TILL	TT		12	SS	4			+					
499.7	62				13	TW									
490	70	DOLOMITE LIMESTONE BEDROCK			14	SS	10/6								
490	75	With Shale and 2" to 12" Thick Gypsum Layers			15	RC	60%								
480	80				16	RC	90%								
479.7	82	END OF BOREHOLE			17	RC	95%								
470	90				18	RC	90%								

ARTESIAN WATER  
ENCOUNTERED AT  
EL 501 FT.  
MEASURED HEAD AT  
EL 564.2 FT

VERTICAL SCALE: 1 inch to 10 feet

DOMINION SOIL INVESTIGATION LIMITED

MADE: S.O. CHECKED:

# LOG OF BOREHOLE S-5

Our Reference No. 8-5-25

Enclosure No. 7

CLIENT: DAMAS & SMITH LIMITED  
PROJECT: LINCOLN ST EXTENSION  
LOCATION: WELLAND, ONT  
DATUM ELEVATION: CITY

## DRILLING DATA

Method: WASHBORING  
Diameter: 3 1/2" (NX)  
Date: JUNE 11-13, 1968

SUBSURFACE PROFILE		SAMPLES		PENETRATION RESISTANCE					WATER CONTENT %			REMARKS				
ELEVATION FT.	DEPTH FT.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N Blows / Foot	Blows / Foot					PLASTIC LIMIT	NATURAL	LIQUID LIMIT	
								UNDRAINED SHEAR STRENGTH								
								+ FIELD VANE TEST      • COMPRESSION TEST								
								20	40	60	80	100				
								500	1000	1500	2000	2500				

561.9	0	RIVER LEVEL														
560		WATER														
556.9	5	6" River Mud	T													
	10	Very Stiff Greyish Red CLAY	T		1	SS	14	0					4200	+		
550			T		2	SS	13	0					4000	+		
	15	With Some Embedded Gravel	T		3	SS	17	0								
			T		4	TW										
	20	(GLACIAL TILL)	T		5	SS	21	0					4380	+		
540			T		6	TW										
538.9	23		T													
	25	Stiff Red - Grey CLAY	T		7	SS	5	0		+						
530			T		8	TW				+						
	35	(VARVED)	T		9	SS	5	0		+						
	40		T		10	TW				+						
520			T													
	45		T		11	SS	5	0		+						
	50		T		12	TW				+						
510			T													
	55		T		13	SS	7	0		+						
503.9	58	CLAYEY SILT TILL	T													
501.9	60	WEATHERED SHALE	T		14	SS	120									
500	61.5	SHALEY LIMESTONE BEDROCK	T		15	RC	67									
495.6	66.3	With Gypsum Layers END OF BOREHOLE	T													
	70															
490																
	75															

ARTESIAN WATER  
ENCOUNTERED AT  
EL. 501.9 FT.  
  
MEASURED HEAD  
AT EL. 567.9 FT.

ARTESIAN WATER  
ENCOUNTERED AT  
EL. 501.9 FT.  
MEASURED HEAD  
AT EL. 567.9 FT.

# LOG OF BOREHOLE S6

Our Reference No. B-5-25

Enclosure No. 8

CLIENT: DAMAS & SMITH LIMITED  
 PROJECT: LINCOLN ST EXTENSION  
 LOCATION: WELLAND, ONT.  
 DATUM ELEVATION: CITY

DRILLING DATA  
 Method: WASHBORING  
 Diameter: 3 1/2" (NX)  
 Date: JUNE 5-6 1968.

SUBSURFACE PROFILE		SAMPLES		PENETRATION RESISTANCE					WATER CONTENT %			REMARKS					
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N Blows / Foot	Blows / Foot					PLASTIC LIMIT	NATURAL W	LIQUID LIMIT		
								UNDRAINED SHEAR STRENGTH + FIELD VANE TEST      COMPRESSION TEST					lbs/sq ft. W <sub>p</sub> W      W <sub>L</sub>				
								500   1000   1500   2000   2500					20   40   60   80   100				
GROUND SURFACE																	
5776.0	0	6" Road Fill															
	5	V Stiff			1	SS	16										
		Hard			2	SS	25										
570	10	Reddish Grey			3	SS	37										
		CLAY			4	SS	40										
	15	With Some Embedded															
		Gravel			5	SS	37										
560	20	V Stiff			6	SS	19										
	25				7	SS	20										
550	30	(GLACIAL TILL)			8	TW											
	35				9	SS	19										
540	38				10	TW											
539.6	40				11	SS	7										
	45	Stiff			12	TW											
530	50	Red - Grey			13	SS	6										
	55	CLAY			14	TW											
		(VARVED)			15	SS	7										
520	60				16	SS	5										
510	65				17	SS	100/6										
508.6	70	Very Dense			18	SS	100/7										
	75	Grey			19	WS											
500	80	SANDY SILT			20	SS	115/5										
		With Some Embedded			21	RC	92%										
		Gravel			21	RC	100%										
496.1	85	(GLACIAL TILL)			22	RC	80%										
	90	DOLOMITE LIMESTONE			23	RC	100%										
		BEDROCK															
490	95	Shale															
		Layers of Gypsum															
		2" to 12" Thick															
480	100																
476.1	101.5	END OF BOREHOLE															
470																	

STEADY PRESSURE  
DURING DRILLING  
NO LOSS OF DRILL  
WATER RETURN

STEADY PRESSURE  
 DURING DRILLING  
 NO LOSS OF DRILL  
 WATER RETURN

VERTICAL SCALE: 1 inch to 10 feet

DOMINION SOIL INVESTIGATION LIMITED

MADE: V. G. H. CHECKED:

# LOG OF BOREHOLE A-1

Our Reference No. 8-5-25

Enclosure No. 9

CLIENT: DAMAS & SMITH LTD  
PROJECT: LINCOLN ST EXTENSION  
LOCATION: WELLAND, ONT  
DATUM: ELEVATION CITY

## DRILLING DATA

Method: WASHBORING  
Diameter: 3"  
Date: JUNE 26, 1968

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE      Blows / Foot					WATER CONTENT %			REMARKS		
ELEVATION Ft	DEPTH Ft	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N Blows / Foot	20      40      60      80      100					PLASTIC LIMIT	NATURAL		LIQUID LIMIT	
								UNDRAINED SHEAR STRENGTH									lbs/sq. ft.
								+ FIELD VANE TEST      • COMPRESSION TEST									
								500      1000      1500      2000      2500									
GROUND SURFACE																	
563.6	0	Topsoil															
560.6	2	Soft Grey	CL		1	SS	4										
560	5	SILTY CLAY	CL		2	TW			+								
556.1	7.5	Soft			3	SS	1										
550	10	Dark Brown	OH		4	TW			+								
548.1	15	ORGANIC CLAY	OH		5	SS	1										
548	15.5	Soft Red-Grey			6	TW			+								
543.6	20	CLAY	CL		7	SS	3		+								
540	25	END OF BOREHOLE															

VERTICAL SCALE: 1 inch to 10 feet

DOMINION SOIL INVESTIGATION LIMITED

MADE: S O

CHECKED:

DEFECTS IN NEGATIVE PHOTO  
CONDITION OF ORIGINAL DOCUMENT

# LOG OF BOREHOLE A-2

Our Reference No. 8-5-25

Enclosure No. 10

CLIENT: DAMAS & SMITH LTD  
PROJECT: LINCOLN ST EXTENSION  
LOCATION: WELLAND, ONT  
DATUM ELEVATION: CITY

## DRILLING DATA

Method: WASHBORING  
Diameter: 3"  
Date: JUNE 26, 1968

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE					WATER CONTENT %			REMARKS		
ELEVATION Ft	DEPTH Ft	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	N	Blows / Foot					PLASTIC LIMIT	NATURAL		LIQUID LIMIT	
								20	40	60	80	100					
								UNDRAINED SHEAR STRENGTH									COMPRESSION TEST
								+ FIELD VANE TEST					- COMPRESSION TEST				
								500	1000	1500	2000	2500					
GROUND SURFACE																	
563.2	0	Topsoil															
561.7	1.5	Grey SILTY CLAY	CL														
560	3	Soft															
558.7	4.5	Dark Brown															
	6	ORGANIC CLAY	OH														
550	10																
546.2	17	Soft CLAY	CI														
543.2	20	Red-Grey															
540	25	END OF BORE HOLE															

VERTICAL SCALE: 1 inch to 10 feet

DOMINION SOIL INVESTIGATION LIMITED

MADE: S O. CHECKED:



SAMPLE DETAILS				CONSISTENCY					UNDRAINED COMPRESSION		UNIT WEIGHT (P.C.F.T.)	REMARKS
BOREHOLE	SAMPLE	TYPE	AVERAGE DEPTH (FEET)	NATURAL WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX	LIQUIDITY INDEX	SHEAR STRENGTH (P.S.F.T)	AXIAL STRAIN AT FAILURE (%)		
S-0	3	TW	7.5	30.5					1740	8.8	127.5	Torvane: 920 P.S.F.
	6	TW	15.0	25.1					1035	6.3	125.9	"- 700 P.S.F.
	8	TW	20.0	21.2					725	9.4	125.2	"- 530 P.S.F.
	10	TW	25.0	24.1					737	10.6	126.8	"- 580 P.S.F.
	12	TW	35.0	42.0					556	7.5	113.3	"- 550 P.S.F.
S-1	1	TW	2.5	51.4	62.4	29.8	32.6	0.66	694	10.0	106.2	Torvane: 610 P.S.F.
	3	TW	7.5	50.3	47.2	29.4	17.8	1.17	296	11.8	102.0	"- 390 P.S.F.
	5	TW	12.5	26.0					134	8.8	124.0	"- 700 P.S.F.
S-2	1	SS	2.5	41.5	51.4	26.4	25.0	0.64				
	3	SS	7.5	138.0								
	4	TW	10.0	132.2	60.9	41.6	19.3	4.7				Consolidated, Undrained Triaxial Test - with pore pressure measurements.
	5	SS	12.5	29.0								
	6	TW	15.0	30.5	38.8	19.6	19.2	0.57	804	6.9	120.0	
	6	TW		37.9								Consolidated Undrained Triaxial Test with pore pressure measurements.
	7	SS	17.5	28.1								
	8	TW	20.0	29.4	33.8	17.9	15.9	0.72	580	5.6	122.5	
	8	TW	20.0	30.4								Consolidated Undrained Triaxial Test with pore pressure measurements.
	9	SS	22.5	27.9								
	10	TW	25.0	37.9	28.4	16.9	11.5	1.82	814	13.1	128.8	
	10	TW	25.0	23.10								Consolidated Undrained Triaxial Test with pore pressure measurements.
	11	SS	30.0	23.2								
	13	SS	40.0	38.8								
S-3	1	SS	2.5	41.5								
	2	SS	5.0	160.0								
	3	TW	7.5	165.8	78.3	46.8	31.5	3.78	180	8.1	77.5	
	3	TW	7.5	160.8					257	15.0	74.0	Undrained Triaxial Test.

**TABLE OF LABORATORY TEST RESULTS**

SAMPLE DETAILS				CONSISTENCY					UNDRAINED COMPRESSION		UNIT WEIGHT	REMARKS
BOREHOLE	SAMPLE	TYPE	AVERAGE DEPTH (FEET)	NATURAL WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX	LIQUIDITY INDEX	SHEAR STRENGTH (P.S. FT.)	AXIAL STRAIN AT FAILURE (%)	(P.C. FT.)	
S-3	4	SS	10.0	159.6								
	5	TW	12.5	122.1	76.0	37.1	38.9	2.29	135	8.1	81.2	Consolidation Test.
	5	TW	12.5	136.0					279	9.16	80.6	Undrained Triaxial Test.
	6A	SS	15.0	110.2								
	6B	SS	16.0	36.7								
	7	TW	17.5	31.9	36.0	20.1	15.9	0.74	650	6.9	117.5	
	8	SS	20.0	29.6								
	9	TW	22.5	26.4	32.5	19.9	12.6	0.51	802	20.0	122.5	Consolidation Test.
	10	SS	25.0	23.7					580	18.75	121.0	
	11	TW	30.0	27.7								
	12	SS	32.0	42.2								
	13	TW	35.0	40.4	52.8	22.5	30.3	0.59	447	13.75	108.5	
	14	SS	40.0	44.4								
	15	TW	45.0	40.6	43.9	21.1	22.8	0.85	815	8.75	115.5	
	16	SS	50.0	36.0								
	17	TW	55.0	25.3	25.9	16.7	9.2	0.93	431	17.5	124.2	
	18	SS	60.0	10.7	18.4	11.9	6.5	-0.18				
	19	SS	65.0	10.9								
S-3A	2	TW	5.0	175.5					136	13.3	78.6	Undrained Triaxial Test
	4	TW	10.0	164.0					258	15.8	80.0	Undrained Triaxial Test
	6	TW	15.0	26.7					426	3.7	124.7	
S-6	2	SS	5.0	22.5	35.2	19.9	15.25	0.17				
	3	SS	7.5	22.3								
	4	SS	10.0	23.9								
	5	SS	15.0	21.3	35.4	19.9	15.5	0.09				
	6	SS	20.0	22.8								
	7	SS	25.0	23.2	38.9	20.6	18.3	0.14				
	9	SS	35.0	21.6	35.7	17.9	17.8	0.28				

TABLE OF LABORATORY TEST RESULTS

SAMPLE DETAILS				CONSISTENCY					UNDRAINED COMPRESSION		UNIT WEIGHT ( P. C. FT.)	RE MARKS
BOREHOLE	SAMPLE	TYPE	AVERAGE DEPTH ( FEET )	NATURAL WATER CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX	LIQUIDITY INDEX	S HEAR STRENGTH ( P. S. FT )	AXIAL STRAIN AT FAILURE (%)		
S-6	11	SS	45.0	32.8	41.9	20.6	21.3	0.573				
	13	SS	55.0	40.5								
R-3	1	AS		23.0								Sieve & Hydrometer Analysis
R-5	1	AS		28.5	49.4	21.4	28.0	0.25				Sieve & Hydrometer Analysis
R-5	2	AS		22.9	50.4	24.4	26.0	-0.06				Sieve & Hydrometer Analysis
R-7	1	AS		26.5	48.2	21.2	27.0	0.20				
R-7	2	AS		26.8	53.0	24.0	29.0	0.10				
R-9	1	AS		27.4	67.5	27.0	40.6	0.1				Sieve & Hydrometer Analysis

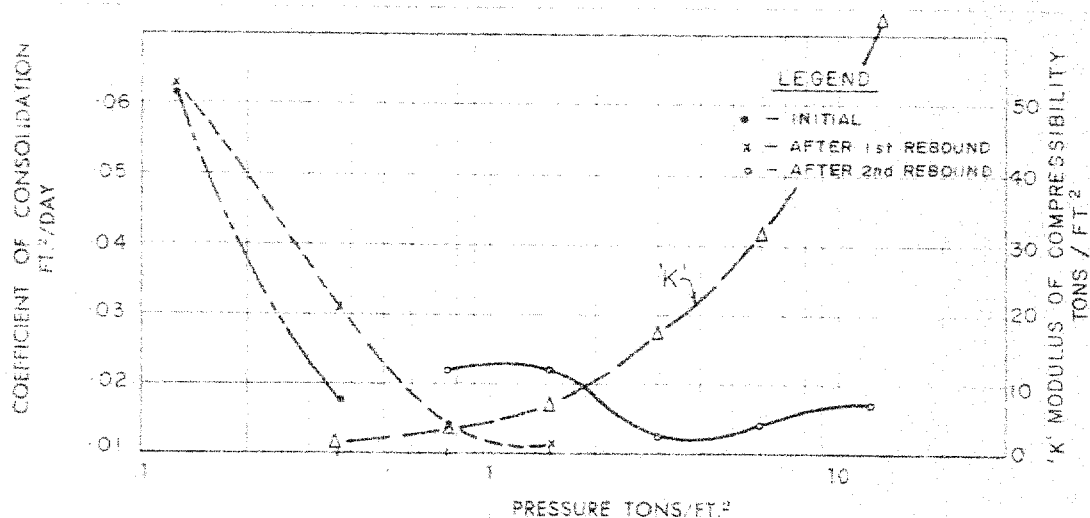
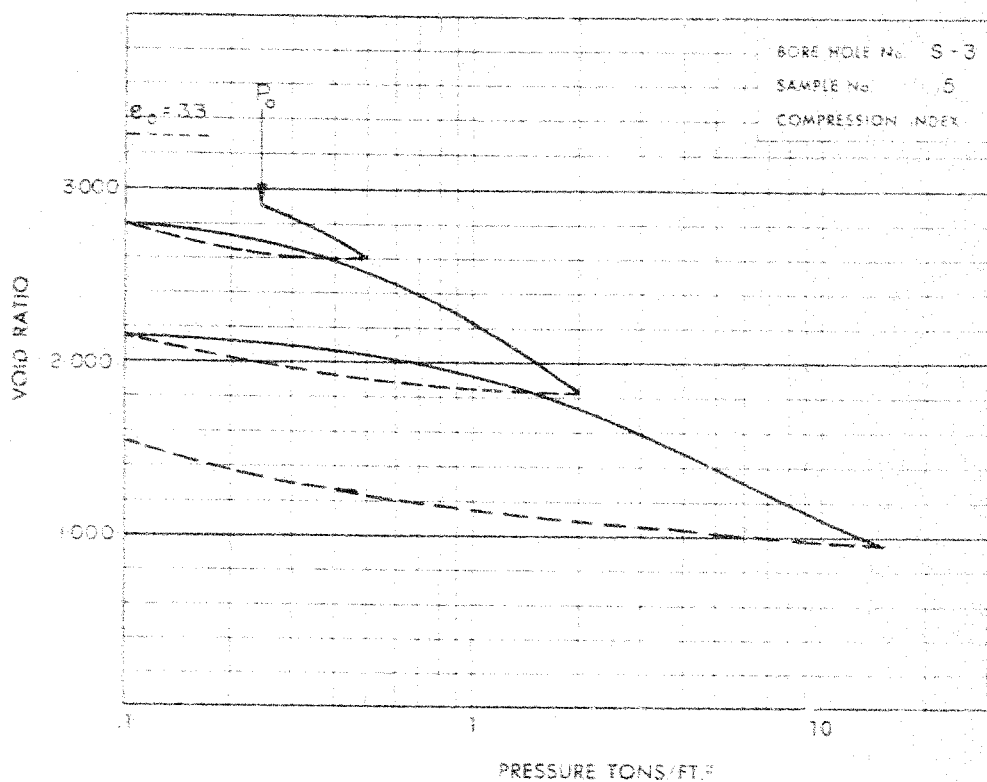
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CONDITION OF ORIGINAL DOCUMENT

## TABLE OF LABORATORY TEST RESULTS

# DOMINION SOIL INVESTIGATION LIMITED

## CONSOLIDATION TEST

### ORGANIC CLAY (OH)

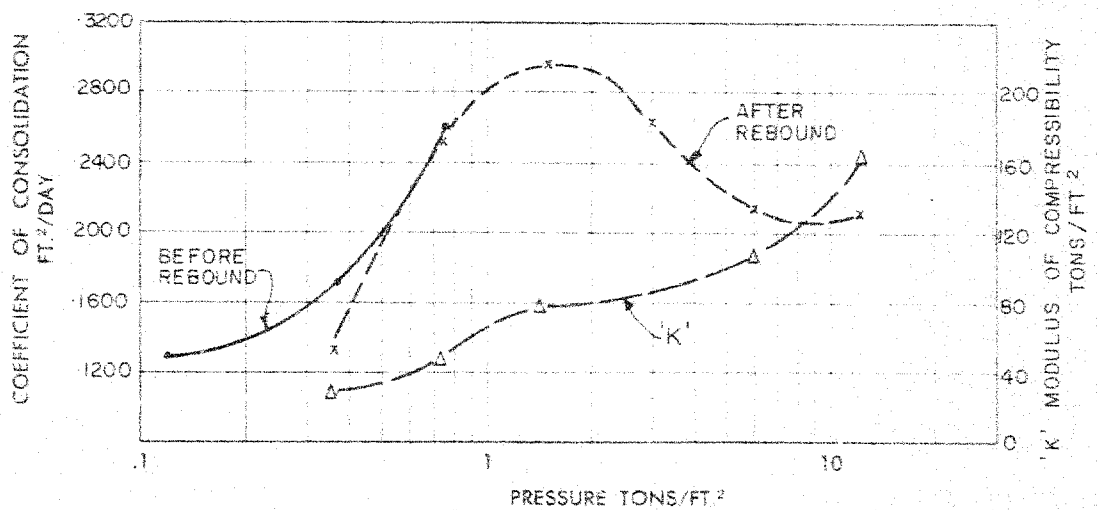
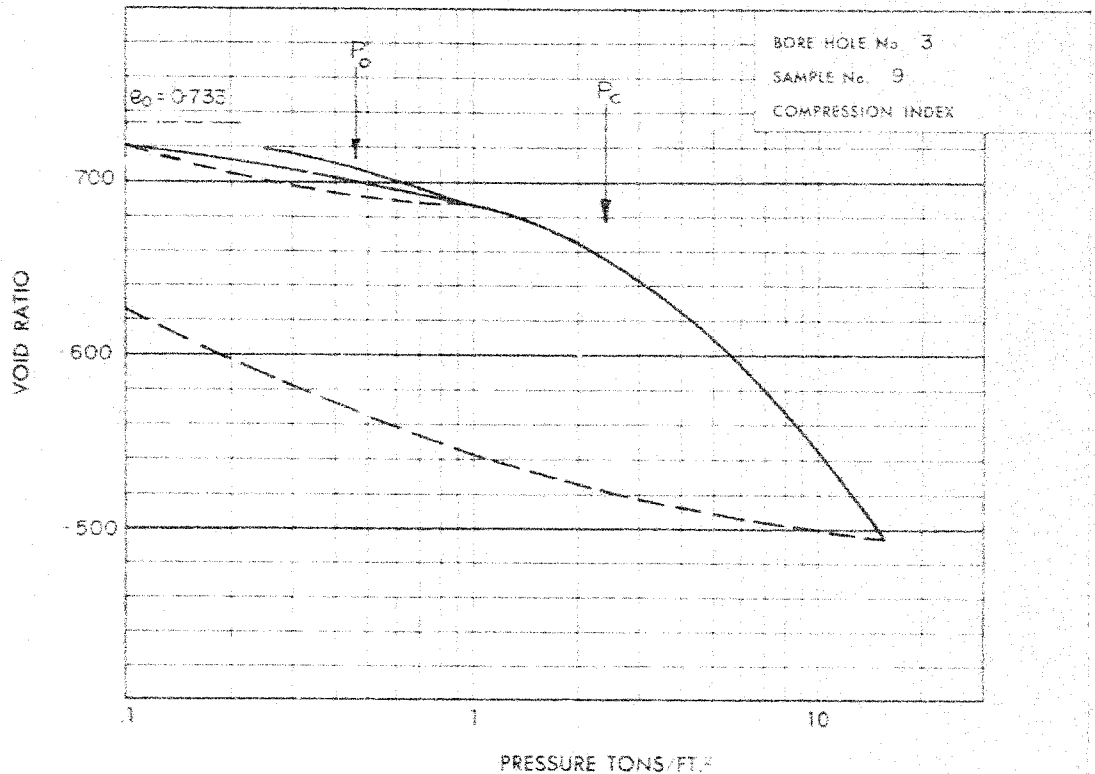


DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

# DOMINION SOIL INVESTIGATION LIMITED

## CONSOLIDATION TEST

### CLAY (CI)



DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST WITH PORE PRESSURE MEASUREMENTS

TEST NO.	CONSOLIDATION PRESSURE (P.S.I.)	CELL PRESSURE (P.S.I.)	DEVIATOR STRESS AT FAILURE $\sigma_1 - \sigma_3$ (P.S.I.)	PORE PRESSURE AT FAILURE (P.S.I.)	$\sigma_1'$ (P.S.I.)	$\sigma_3'$ (P.S.I.)
1.	35	45	8.8	43.4	10.4	1.6
2.	15	20	18.0	3.0	29.2	11.2
3.	25	35	23.6	19.9	38.7	15.1

CLAY (CI)  
BH = S-2

SHEAR STRESS (P.S.I.)

20  
10  
5  
0

0 10 20 30 40

EFFECTIVE PRINCIPAL STRESS ( $\sigma_1'$ ,  $\sigma_3'$ ) P.S.I.

0 10 20 30 40

$C = 2.4 \text{ P.S.I.}$

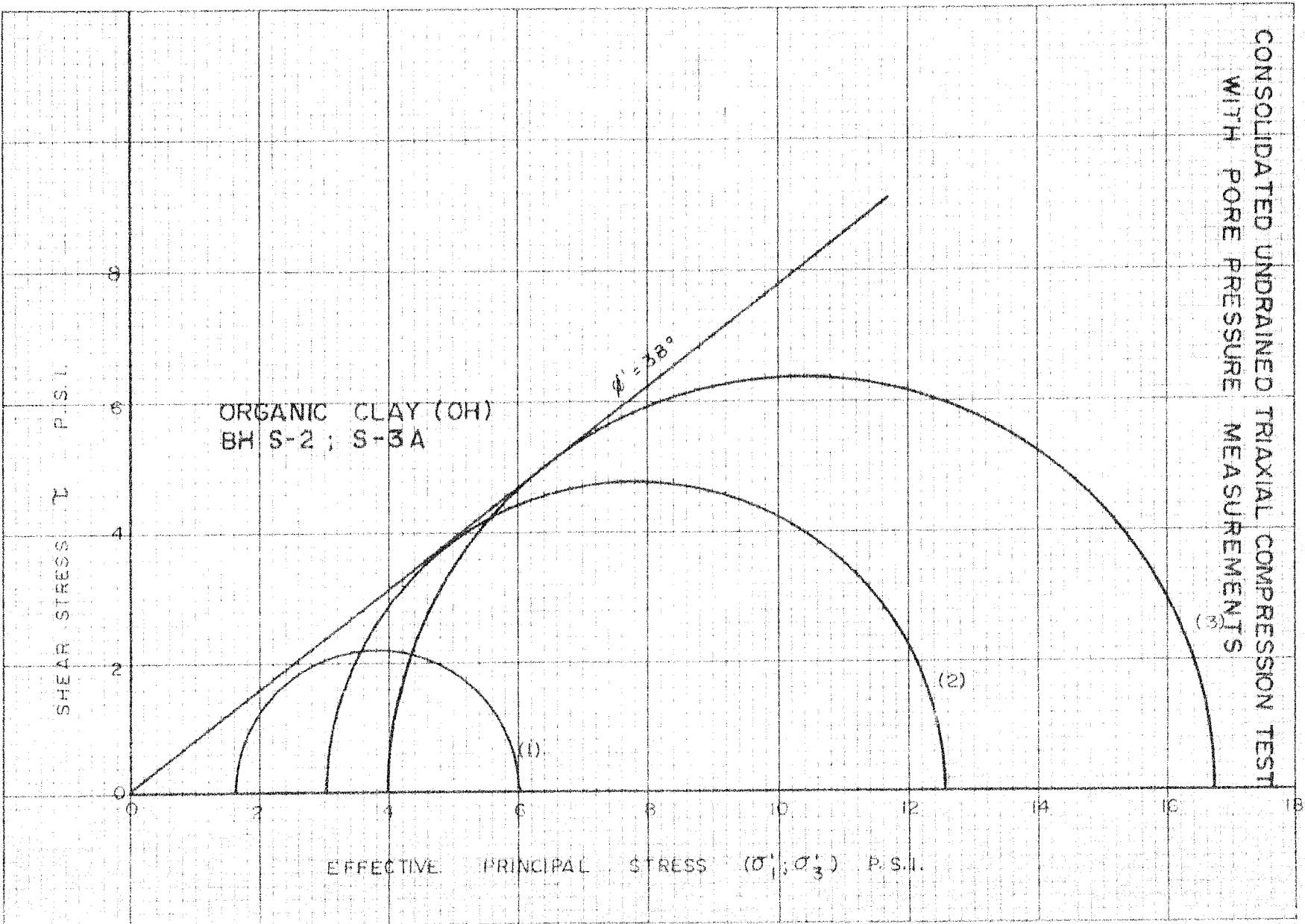
$\phi' = 21^\circ$

(1)

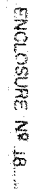
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(3)

# CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST WITH PORE PRESSURE MEASUREMENTS



OUR REFERENCE NO 8-5-25



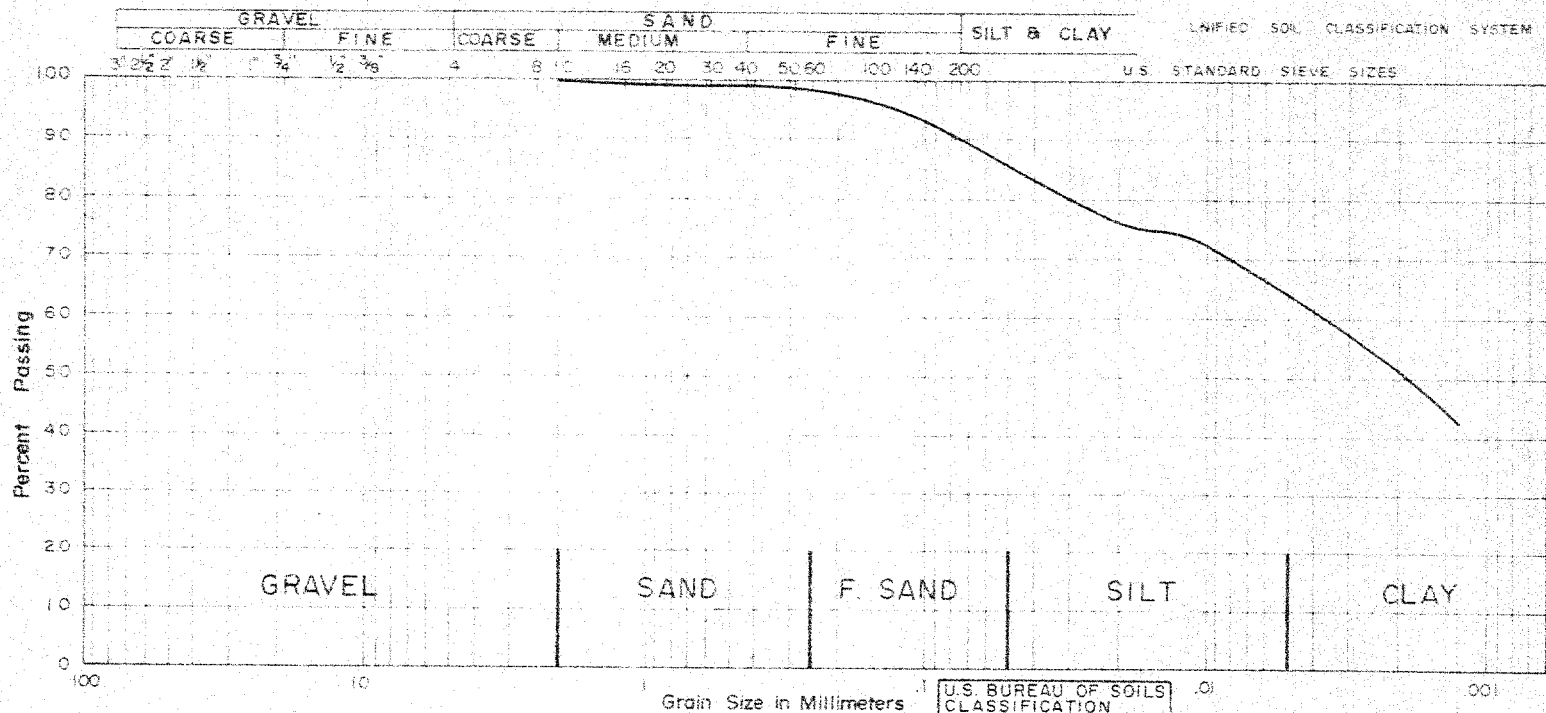
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CONDITION OF ORIGINAL DOCUMENT



# DOMINION SOIL INVESTIGATION LIMITED

## GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 8-5-25



PROJECT: LINCOLN ST. EXTENSION  
 LOCATION: WELLAND  
 BOREHOLE NO: R-5  
 SAMPLE NO: 1  
 DEPTH:  
 ELEVATION:

COEFFICIENT OF UNIFORMITY:  
 COEFFICIENT OF CURVATURE:

### PLASTIC PROPERTIES

LIQUID LIMIT % = 49  
 PLASTIC LIMIT % = 21  
 PLASTICITY INDEX % = 28  
 MOISTURE CONTENT % = 28

Classification of Sample and Group Symbol:

MEDIUM CLAY

12

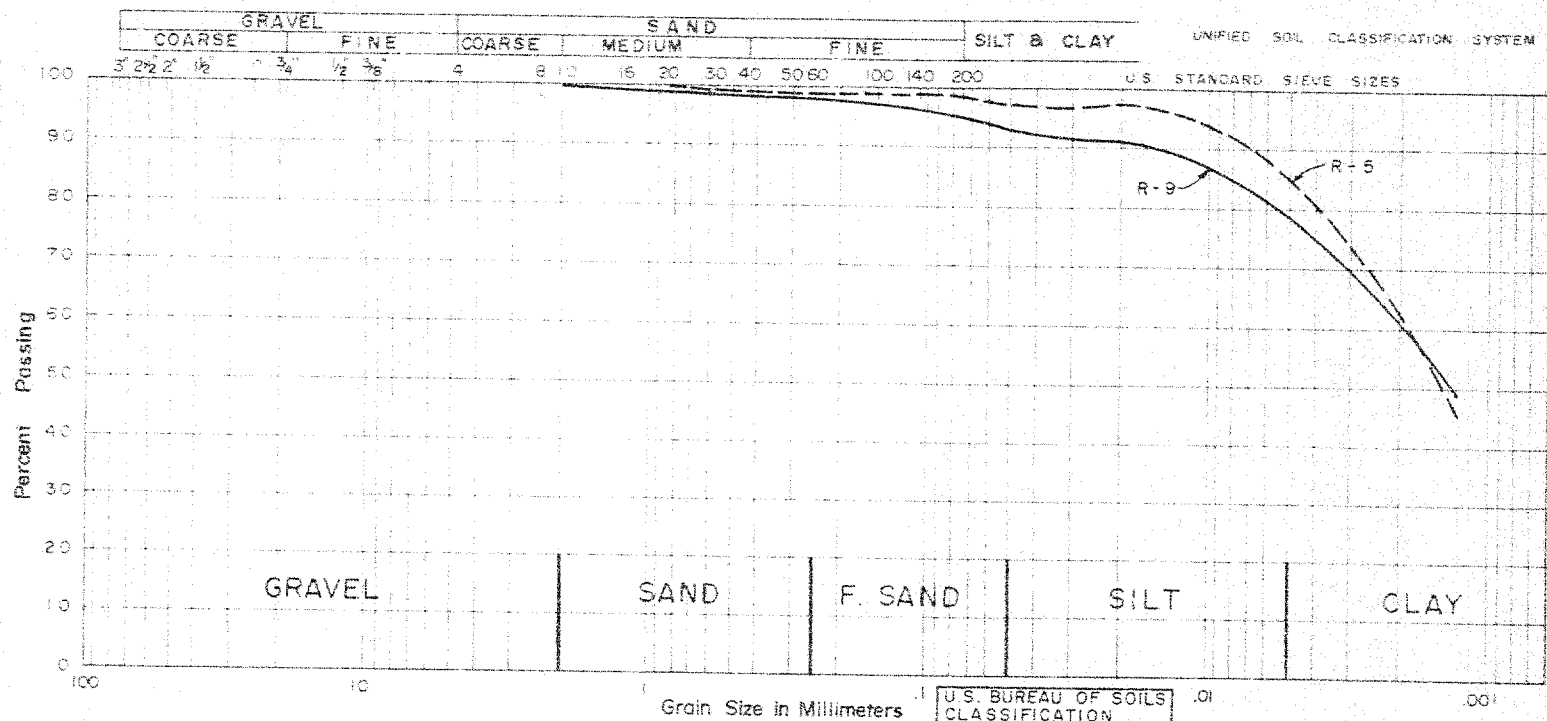
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 CONDITION OF ORIGINAL DOCUMENT

ENCLOSURE NO. 19

# DOMINION SOIL INVESTIGATION LIMITED

## GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 8-5-25



PROJECT: LINCOLN ST. EXTENSION  
 LOCATION: WELLAND  
 BOREHOLE NO: R-5 & R-9  
 SAMPLE NO: 2 B 1  
 DEPTH:  
 ELEVATION:

COEFFICIENT OF UNIFORMITY:  
 COEFFICIENT OF CURVATURE:

Classification of Sample and Group Symbol:

HEAVY CLAY

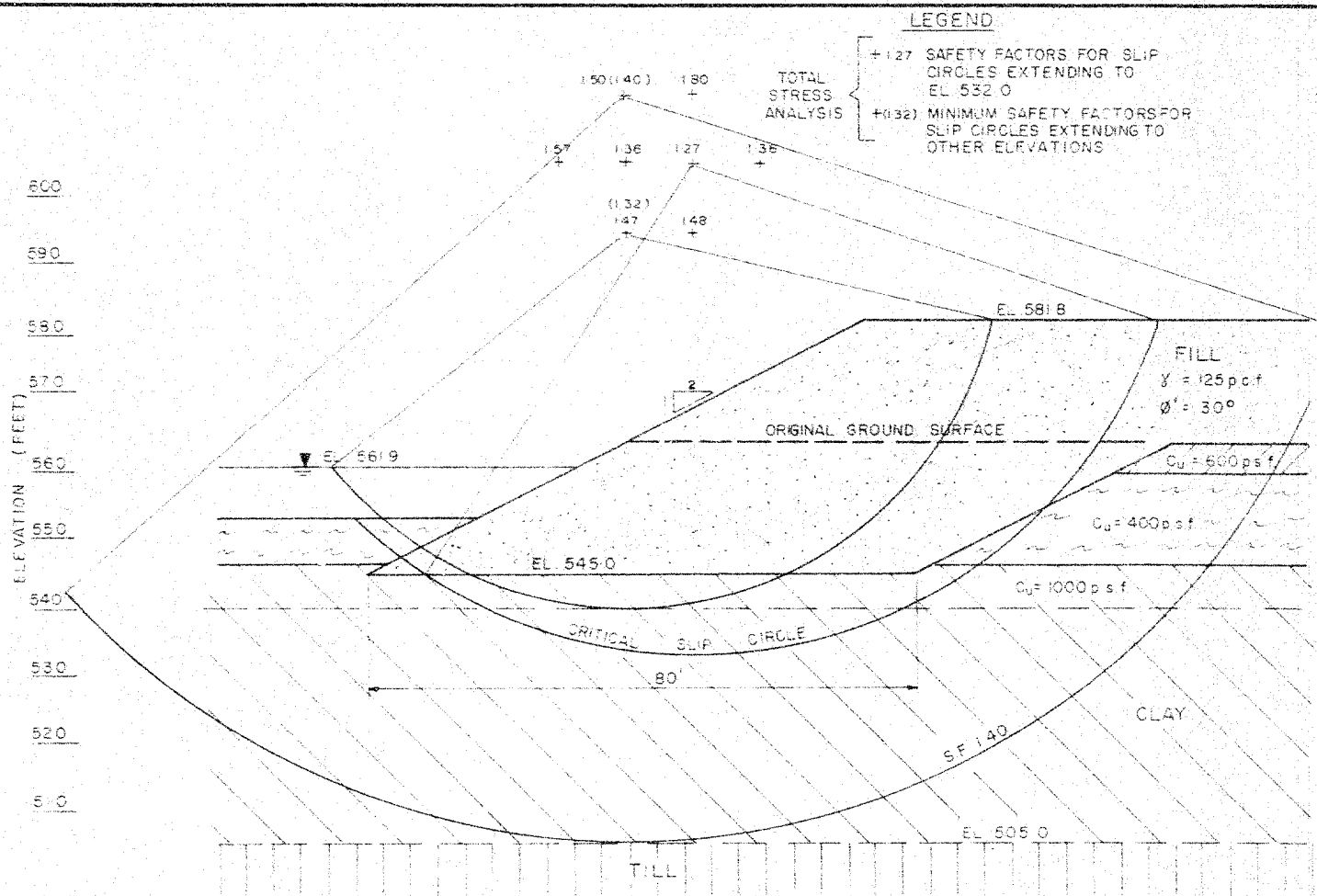
13

PLASTIC PROPERTIES

LIQUID LIMIT % = 50 - 68  
 PLASTIC LIMIT % = 24 - 27  
 PLASTICITY INDEX % = 26 - 41  
 MOISTURE CONTENT % = 23 - 28

DEFECTS IN NEGATIVE DUE TO  
 CONDITION OF ORIGINAL DOCUMENT

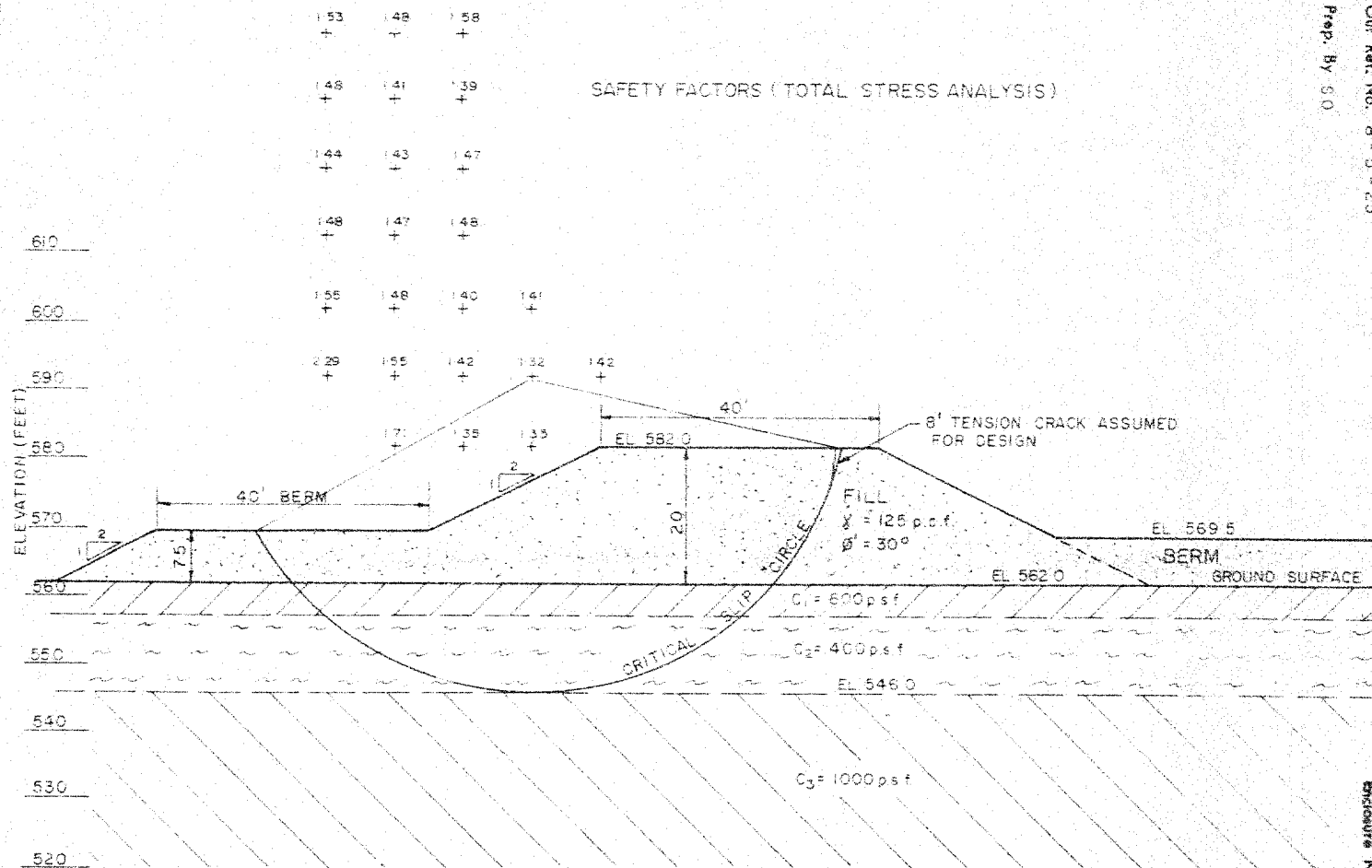
ENCLOSURE NO. 20



PROPOSED END SLOPE CONSTRUCTION AT EAST ABUTMENT (STA. 25+00)

## SAFETY FACTORS (TOTAL STRESS ANALYSIS)

Our Ref. No. 8-5-25  
 Prep. By 50



RECOMMENDED EMBANKMENT SECTION (STA 22+50 to STA 25+00)