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LONDON, ONTARIO  
TELEPHONE GE. 3-8851



FOUNDATION ENGINEERS

P.O. BOX 938  
SAULT STE. MARIE  
ONTARIO  
TELEPHONE AL. 4-2615

London, 15 June 1962.

2-5-L1

Ontario Department of Highways,  
Materials and Research Division,  
Downsview, Ontario.

Attention: Mr. A. Stermac  
Principal Foundation Engineer

Soil Investigation for Proposed Hwy. #126  
Commissioner's Road Underpass, London,  
Ontario, District #2, W.P. 95-62

Dear Sirs:

We have pleasure in enclosing 14 copies of our report on the above project.

We are glad to have had this opportunity to be of service to you and should any questions arise in connection with the report or during excavation for the structure, please do not hesitate to get in touch with us.

Kindly advise us if you have any special instruction for disposal of the soil samples, otherwise they are normally stored for a period of 3 months from the date of issue of the report and thereafter destroyed.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED

Encl.  
JP/mc

James Park, P.Eng.  
London Branch Manager

Mr. A. M. Toye,

June 12, 1962.

Bridge Engineer.

FOURTH (OR INVESTIGATIVE) REPORT

Materials & Research Division,

By: Dominion Soil Investigation  
Limited

(Foundation Section)

Attention: Mr. J. McMillan.

Re: Proposed Hwy. #126, Commissioner's Road,  
Underpass, London, Ontario, District #2,  
(H.R. 25-62.)

Attached, we are sending you the report on the foundation investigation for the above-mentioned structure, submitted by the Consultant, Dominion Soil Investigation, Limited.

We have reviewed the report and have found the factual information well presented. We also agree with the conclusions and recommendations given in the report.

We believe that the data contained in the report should prove to be adequate for your future design work. However, should there be any questions you would like to discuss, please feel free to contact our office.

REL/mar

attach.

cc: Material.

A. M. Toye (2)  
H. A. Trepanier  
H. D. McMillan  
A. Cater  
W. L. Fraser  
J. May  
W. J. Kovach  
J. R. Gussier  
C. E. Saint  
W. Barnes  
A. Watt  
Foundations Office  
Gen. Files.

For:

*sylo*  
H. Y. Jo.

submitting foundation report

A. G. Starnes,  
PRINCIPAL FOUNDATION ENGINEER.

ONTARIO  
DEPARTMENT OF HIGHWAYS  
MATERIALS AND RESEARCH DIVISION

REPORT ON  
SOIL INVESTIGATION  
FOR  
PROPOSED HWY. #126,  
COMMISSIONER'S ROAD UNDERPASS  
LONDON, ONTARIO, DISTRICT #2  
W.P. 95-62

by  
DOMINION SOIL INVESTIGATION LIMITED  
363 Queens Avenue  
LONDON ONTARIO

Reference No. 2-5-L1

May 1962

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## INTRODUCTION

In accordance with a letter of authorization dated May 8th, 1962 from the Ontario Department of Highways, Materials and Research Division, a soil investigation has been carried out at a site in the southeast of the City of London, at the planned intersection of Commissioner's Road and the proposed Highway #126. The new road will be a 4-lane divided highway extending Highbury Avenue southwards to meet Highway #401.

A Department of Highways drawing (Plan E-4087-1) was supplied showing survey data and the proposed line and grade of the intersecting roads. It was indicated that Highway #126 would run 15 feet below the present grade, and that Commissioner's Road would be carried at an elevation 7 or 8 feet above its present level, by a 4-span structure.

The purpose of this investigation was to reveal the subsurface conditions, and to determine the necessary soil properties for the design and construction of the underpass structure.

At the time of this investigation the loading on the structure is not known and no information is available concerning the proposed size or arrangement of footings for any given soil condition. The analyses presented in this report therefore cover intensities of loading which it is believed will not be exceeded and a consideration of stability conditions which may or may not arise. In the latter case it is possible only to analyse a hypothetical condition which may be used as a guide in design.

## I DESCRIPTION OF SITE AND GEOLOGY

The site lies in a rural district which was previously part of the Township of Westminster and is now in the City of London. Commissioner's Road, which has a gravel surface at this point, is a through route connecting Highway #2 in the west with Nilestown and points to the east, while skirting to the south of the built-up area.

The location is near the crest of the Ingersoll Moraine,<sup>1</sup> the first of a series of east-west ridges lying to the south of the River Thames. To the south, west and east the ground surface is undulating without any notable features, while to the north it declines steeply into the spillway valley now occupied by the South Branch of the Thames.

## II FIELD WORK

Field work was carried out during the period 9th to 16th of May 1962 and consisted of five boreholes at the locations shown on enclosure 1. Dynamic cone penetration tests were made adjacent to each borehole and at five other locations, all of which are shown on the location plan. These points correspond to the extremities of the piers and abutments of the underpass structure.

The boreholes were advanced using a 2½-inch diameter auger powered by a diamond drill with a hydraulic drive head. Standard Penetration tests were made at intervals not exceeding 5 feet using a 2-inch O.D. split spoon. A constant driving energy was employed in the Standard and dynamic cone penetration 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 a clay stratum were recovered in 2-inch diameter thin-walled Shelby tubes. The sampling tubes were forced into the soil with a slow continuous motion using the jacking force of the hydraulic head on the drill. In this way it was possible to recover samples of very stiff soil in good condition at elevations where the gravel content is small.

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1 Numbers apply to references given on pages 12 and 13.

Vane shear tests were made in the field using a 2-inch diameter 4-bladed vane. The blades have a length of 4 inches and a thickness of 1/8 inch. The tests were performed by pushing the vane a distance of 12 inches (to the centre of the blades) below the bottom of the hole using the hydraulic head on the drill, and by measuring the torque required to rotate the assembly with a calibrated torque wrench. Both the insitu and remoulded torques were measured and the ratio of these results (S, which may be called the sensitivity) is recorded along with the insitu shear strength values.

In every case recorded the failure torque was clearly observable. Where the torque resistance exceeded 1200 inch-pounds, corresponding to a shear strength of 6000 p.s.f., the test was abandoned.

Soundings to obtain ground water levels were made up to one week after completion of the borings and before the holes were plugged, but because of the low permeability of the soil for a considerable depth the results were not of value. The ground water conditions are described in more detail in the following section on subsurface conditions.

The results of the field tests are recorded on data sheets comprising enclosures 2 to 8. Elevations have been referred to the same datum as that used in Department of Highways plan E-4087-1.

### III SUBSURFACE CONDITIONS

A subsurface profile shown on enclosure 1 illustrates the stratification encountered in the borings. The strata are described in detail in the following paragraphs:

- (i) A very stiff to hard clay till extends from ground surface to a depth varying from 13 to 18 feet. The transition between this stratum and the underlying clay is a gradual one both in colour and in composition. The till is brown to a depth of approximately 12 feet and thereafter changes gradually to grey. Organic material penetrates to a depth varying from 1.0 to 2.5 feet from surface.

The upper 10 feet of this stratum contains between 5 and 10 per cent (by visual inspection) of mostly

rounded gravel generally less than 3/4 inch in diameter. Thereafter the amount of gravel decreases to 3 or 4 per cent. The density of the till is very high (7 samples gave an average value of 1.9 p.c.f.) and the moisture content is approximately 21 per cent.\*

At boreholes 2 and 4 only on the south side of the road, water bearing seams were encountered at 2 to 4 feet from surface, which filled the holes with water overnight.

- (ii) Below the till a stratum of very stiff grey silty clay extends to elevation 836 or 35 feet below the general ground level. Coarse particles generally less than 1/4 inch in diameter are present in a proportion varying from 0 to about 3 per cent. Laboratory tests have shown this material to be an overconsolidated clay of low to intermediate plasticity with a bulk density close to 135 p.c.f. The void ratio is about 0.5 and the liquidity index 0.27. These figures alone provide a good indication that this material, on which some or all of the foundations will bear, is very stiff and dense with a high shear strength and low compressibility.
- (iii) Between 35 and 48 feet from surface there is a layer of very dense silt. The upper 7 or 8 feet is brown, oxidized and slightly sandy. A 6-inch thick sand seam was found within the layer at boreholes 2 and 5. The remainder of the stratum is grey in colour and contains a few isolated thin clay seams, but most of this layer is cohesionless. A free water table was found at or above the elevation of the colour change. Above the water table the silt is almost dry and below, it is fully saturated.
- (iv) Below the bottom of the silt stratum at 48 feet there is a second layer of stiff grey silty clay. By visual examination it is similar to the clay stratum above. Four of the boreholes were terminated in this layer, the exception being borehole 3 which was advanced until a stratum of very dense fine grey sand was encountered at a depth of 60 feet 6 inches. The sand layer was almost dry.

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\* A summary of laboratory test results is given on enclosure 12.



Boreholes 2 and 4 filled with water overnight from seams which could be seen near the top of the holes. Boreholes 1, 3 and 5 caved in at the top of the silt layer, approximately 35 feet below ground, and no free water was found in these holes up to 7 days after drilling. From laboratory tests it is known that the grey silty clay stratum is fully saturated at least as deep as elevation 895 and the moisture content from samples of the till layer above are not appreciably different from those in the clay. It thus appears that the true piezometric head is close to the present ground surface and this has been assumed in calculations.

#### IV LABORATORY WORK AND SHEAR STRENGTH

A laboratory testing programme has been carried out as follows:

- (a) Determination of bulk density and natural moisture content of samples from the till and grey silty clay strata,
- (b) Determination of Atterberg limits from samples of the clay for the purpose of classification and as a guide to soil behaviour,
- (c) Consolidation tests on two samples of the clay to obtain data on compressibility and preconsolidation,
- (d) Unconfined compression tests on 4 samples of the clay to determine shear strength.

A summary of results from (a), (b) and (d) is given on enclosure 12 and, where appropriate, they are also plotted on the data sheets (enclosures 2 to 6). Pressure-void ratio curves from the consolidation tests are plotted on enclosures 9 and 10 along with calculated values for the coefficient of consolidation. The estimated preconsolidation load which is determined from the shape of the e-p curve gives an over-consolidation ratio of 2.5 and 2.7 in the cases of borehole 3 sample 5 and borehole 4 sample 4 respectively.

The following summary of unconfined compression and vane shear test results provides a comparison from which a shear strength value has been selected.

## Shear strength (p.s.f.)

Borehole No.	Unconfined comp. test	Nearest vane test	Other vane tests
1	-	-	5000, 5400
2	5000 (Sa. 5)	4200	-
3	2910 (Sa. 5)	3600	-
4	3920 (Sa. 4)	3750	4200
5	<u>2130</u> (Sa. 7)	<u>3750</u>	4200
Average	3490	Average 3825	

The disturbance associated with sampling often tends to lower unconfined compression test results, whereas the field vane tests may tend to be high because of frictional resistance caused by granular particles in the soil. In this case, however, except for borehole 5 sample 7, there is a good agreement between the field and laboratory test results. Two of the laboratory results are higher than the nearest vane test figures by 19 per cent and 5 per cent, while one is lower by 19 per cent. Also, the vane test results are quite consistent.

The average of the unconfined compression test values is 3490 p.s.f. while all the vane shear results are in excess of 3500 p.s.f. On the basis of the foregoing figures a value of 3500 p.s.f. will be used for the shear strength of the soil in estimating bearing capacity or stability.

## V

BEARING CAPACITY AND SETTLEMENT OF FOOTINGS

The very stiff strata encountered will provide ample support for conventional spread footings. The elevation of the footings for the intermediate or "pier" supports of the 4-span structure will be governed by the level of Highway #126 which will run 15 to 16 feet below the present grade, so that the footings will be located in the stiff silty clay stratum. The elevation of the abutment supports may be governed by factors which are not known at the time of preparing this report. The footings for the abutments may be located at the same level as those of the piers, or at some higher elevation in the clay till. Both of these possibilities will be considered in the following analysis.

(a) Pier Supports

The grade level of Highway #126 has been given as El. 906 feet. In the very stiff soil at this level, the footings need only be deep enough for frost protection. A minimum of 5 feet of cover is proposed, giving a footing elevation of 901 and leaving a 15 foot thickness of stiff grey silty clay between the footings and the very dense, brown, dry, sandy silt at El. 886. The ultimate bearing capacity of square and rectangular footings at El. 901 has been calculated, in accordance with Meyerhof's theory,<sup>5</sup> for a range of footing sizes. In the case of rectangular footings the length of the footing has been taken as 66 feet in all cases. The following table gives recommended maximum gross soil pressures which are taken as 1/3 of the calculated ultimate values. The cohesion of the soil has been taken as 3500 p.s.f. and the bulk density as 135 p.c.f. Although these values have been calculated for El. 901 feet, they represent safe or conservative figures for any elevation within the grey silty clay stratum.

MAXIMUM ALLOWABLE (GROSS) SOIL PRESSURES (P.S.F.)  
FOR FOOTINGS AT EL. 901 FEET

Width of footing (feet)	Square footing	Rectangular footing
6	9410 (0.48")	8060 (1.4")
8	8990 (0.66")	7920 (1.7")
10	8760 (0.90")	7820 (1.9")

The figures in brackets in the foregoing table are the calculated consolidation settlements for each size of

footing loaded to the maximum recommended value.\* The calculated time periods for 50% and 90% consolidation are 7 months and 30 months respectively.

It is anticipated that the actual loads created by the structure will be considerably less than the maximum loading conditions treated above. Also, the consolidation settlement will arise mainly from dead loading which will represent only a proportion of the total applied load. Once the size of a footing has been determined from the total load and the maximum soil pressures quoted above, the consolidation settlement can be estimated approximately by expressing the dead load pressure as a fraction of the maximum allowable (gross) pressure and applying this ratio to the settlement figures quoted.

#### (b) Abutment Supports

If the abutment footings are located at the same level as those for the piers, the recommendations in the foregoing paragraphs on pier supports are applicable. If, however, the requirements of the structure allow the abutments to be supported at a higher level, a considerable amount of excavation can be avoided.

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\* The distribution of vertical stress below the footing has been computed according to Csutovich.<sup>7</sup> The oedometer settlement has been calculated from the consolidation test results and adjusted using the Skempton-Bjerrum correction factor modified, in the case of long footings, by Muir-Wood.<sup>6</sup> The overconsolidation ratio is estimated to be 2.6, in view of which a value of 0.25 has been assumed for the pore pressure coefficient  $A_{v,8}$ .

In the case of the 10 foot square footing, the distribution of stress below the footing has also been calculated according to Steinbrenner<sup>3</sup> for the purpose of comparison. The resulting settlement is 0.63 inches compared with 0.90 using the Csutovich theory. The latter method is preferred because it takes into account variations in the elastic properties of the soil which will affect the distribution of stress, whereas Steinbrenner's method recognises only one possible distribution for all soils.

It is proposed that the abutment footings should be placed not higher than El. 917 feet. This will ensure that the footing grade is well below the level of organic or weathered material. Also the slope between the abutment and the roadway should be graded in such a way that the abutment footing is stable and that sufficient cover is given for frost protection. The arrangement illustrated on enclosure 11 satisfies both of these conditions and will be examined in detail in the following paragraphs.

The clay till is a hard material with a proportion of gravel which precludes the recovery of undisturbed samples by ordinary soil sampling methods. The strength and deformation characteristics can be assessed only by comparison with other materials which are more amenable to sampling, or by reference to published data from special tests. To determine the bearing capacity of a footing the assumption will be made first that the till is at least as strong as the underlying clay. Comparing the results of Standard Penetration tests in the two materials, this is a conservative assumption. Because the long term stability of the footing on the slope is of interest, the till will also be considered as a frictional material with a  $\phi$  (drained) value. Reference is made to published data from consolidated, drained, triaxial tests on remoulded samples of New England tills.<sup>9</sup> Values for the angle of internal friction  $\phi$  ranged from  $32^\circ$  to  $38^\circ$  with cohesion values between 0 and 0.4 t.s.f. These materials were compacted at or near their optimum moisture content giving a wet density in the range of 137 to 144 p.c.f. compared with an average measured density of 139 p.c.f. in the present case. For the purpose of this analysis a value of  $\phi = 36^\circ$  will be assumed with a cohesion value of zero. Because this is a naturally compacted and not a remoulded till these values are probably conservative.

The bearing capacity of footings 66 feet long with widths of 6, 8 and 10 feet has been evaluated in terms of  $c = 3500$  p.s.f., and  $\phi = 0$ . The depth of footing is taken as zero because of its position adjacent to a slope. The following maximum allowable gross soil pressures are  $1/3$  of the ultimate values calculated according to Meyerhof. The consolidation settlement anticipated with this loading is one half of that calculated for footings of the same size in the grey silty clay stratum. This prediction is based on a comparison of Standard Penetration test results in the two materials.

Width of footing	Maximum allowable gross soil pressure (p.s.f.)
6 feet	6100
8 feet	6170
10 feet	6300

The foregoing calculations do not take into account the removal of material from one side of the footing which will alter the distribution of stress, so it is necessary to consider now the possibility of a slip failure on the slope.

Assuming again that  $c = 3500$  p.s.f.,  $\phi = 0$ , the stability of 5 possible slip circles, as shown on enclosure 11, has been determined. The factors of safety are 3.3, 3.7, 3.8, 3.2 and 3.5. Thus in terms of this  $\phi = 0$  analysis there is adequate stability.

The next step has been to examine the stability in terms of  $\phi = 36^\circ$  and  $c = 0$ . It has been assumed that the soil is fully saturated from ground surface to the bottom of the grey silty clay layer. The distribution of pore pressures after the cut is made is not known. Conceivably the removal of lateral soil pressure will allow a swelling in the till which will reduce pore pressures to a negative value, but in the worst case these pressures could remain at their original positive value for some time after the abutment is constructed. The stability of No. 4 circle has been examined for this condition and the factor of safety is found to be 1.6 which is adequate. If in time the pore pressures dissipate to zero, the safety factor will increase to 2.2, which is a high value in terms of slope stability.

The foregoing analyses have been based on assumptions as to the strength of the soil, which are believed to be conservative. Also they apply to loading conditions whose intensity will probably never be realised. Many other combinations of loading and position are possible but it is impracticable to attempt to deal with all of them here. The foregoing example serves to illustrate the degree of safety inherent in such an arrangement for the soil conditions encountered on this site. It should be used as a guide in the design of the structure, and the final proposed design arrangement should be checked for stability.

## VI APPROACH EMBANKMENTS

The raising of Commissioner's Road by 7 or 8 feet will require embankments of this height at the approaches to the structure. Providing all organic material is removed before the fill is placed the settlement due to consolidation of the till will be negligible.

Material excavated from the cutting, viz. stiff clay and clay till, can be used as fill for the approaches provided it is placed with the correct equipment (sheepsfoot rollers) and under conditions where the moisture content and degree of compaction are carefully controlled. Tests should be made to determine the optimum moisture content and field density required from the chosen material. The intention as to choice of the fill material is not known at this time, and such tests are considered beyond the scope of the present work.

## VII CONSTRUCTION

The prevailing soil conditions should create no unusual construction problems. The material is very stiff and cohesive so that no bracing will be necessary. The only source of seepage will be from water bearing seams within a few feet of the present ground surface, and these can be diverted in trenches draining by gravity to lower ground. Precipitation will accumulate in confined excavations and can be removed by pumping.

It is recommended that the footing grade should be covered with a thin layer of lean concrete as soon as it has been exposed and inspected. This will act as a protection against disturbance by construction equipment or personnel.

## VIII SUMMARY

1. The strata which will be exposed or influenced by the proposed structure are (a) a very stiff to hard clay till extending from surface as far as 18 feet, and (b) a very stiff silty clay reaching 35 feet from surface.
2. Except for water-bearing seams encountered at two boreholes within a few feet of the surface, no freely

moving groundwater was found in the above strata.

3. Soil pressures in the region of 8000 to 9000 p.s.f. are proposed for footings in the clay layer. Tabulated values are given on page 7.
4. Abutments can be supported at a higher level than the intermediate piers, by footings in the till layer. Recommended soil pressures for an arbitrarily chosen arrangement are given on page 10. The stability of such an arrangement has been analysed and found to be safe, and this example can be used as a guide in design. The final design arrangement should also be checked for stability, since it is impracticable to consider all possibilities beforehand.
5. No unusual construction problems are anticipated.
6. The consolidation settlement in the clay till resulting from the weight of the approach fills will be negligible.
7. Material from the till and clay layers can be used to form the approaches if it is compacted in a proper manner.

## IX REFERENCES

1. The Physiography of Southern Ontario by L.J. Chapman and D.F. Putman of the Ontario Research Foundation - University of Toronto Press, 1951.
2. Procedures for Testing Soils, ASTM, April 1958, pp. 186 to 198. (Unified Soil Classification System - by A.A. Wagner)
3. Karl Terzaghi: Theoretical Soil Mechanics, John Wiley and Sons, New York, 1943.
4. The Application of Theories of Elasticity and Plasticity to Foundation Problems. Leo Jurgenson, Sc.D., Boston Society of Civil Engineers, May 1934.
5. The Ultimate Bearing Capacity of Foundations by G.G. Meyerhof Geotechnique, Vol. II, 1950 and 1951.



6. A Contribution to the Settlement Analysis of Foundations on Clay by A.W. Skempton and L. Bjerrum - Geotechnique VII (1957) and Amendment Thereto by A.M. Muir-Wood (Correspondence, Geotechnique Vol. IX).
7. N.A. Csutovich, Calculation of Settlement, Extract from "Foundation Engineering" by Ch. Szechy, Budapest 1957, transl. by L.R. Szalatka.
8. The Measurement of Soil Properties in the Triaxial Test, by Bishop and Henkel, London, 1957.
9. K.A. Linell and H.F. Shea, Strength and Deformation Characteristics of Various Glacial Till in New England, ASCE Research Conference on Shear Strength of Cohesive Soils, Colorado, June 1960.



DOMINION SOIL INVESTIGATION LIMITED

A handwritten signature in cursive script, appearing to read "James Park".

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

## ENCLOSURES

### LEGEND

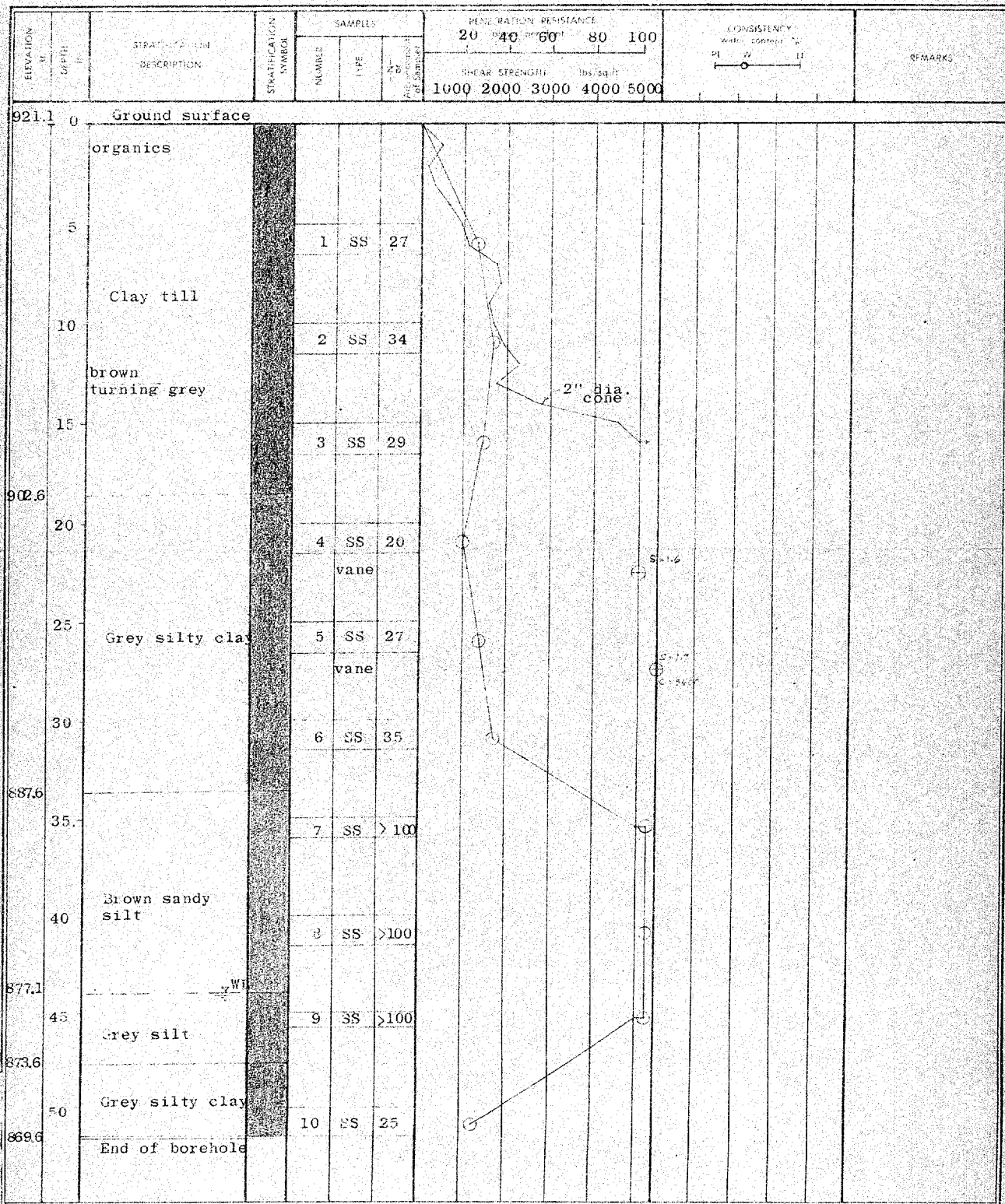
- SS —○— denotes 2-inch O.D. split-spoon  
(disturbed) sample
- TW denotes 2-inch dia. thin-walled  
Shelby tube (undisturbed)  
sample
- vane ⊕ denotes vane shear test
- denotes unconfined compression  
test
- ⊙ denotes moisture content

# GEOTECHNICAL DATA SHEET FOR BOREHOLE 1111

FILE REFERENCE NO. 2-5-L1

CLIENT: Ontario Department of Highways  
 PROJECT: Underpass at proposed highway #126  
 LOCATION: (Highbury Ave.) and Commissioners Rd..  
 DATUM ELEVATION: 921.71 (see encl. 1) London, Ontario

METHOD OF BORING: 2 1/2" dia. power auger  
 DIAMETER OF BOREHOLE:  
 DATE: 10 May 1962  
 RECORD NO. 2



# GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 2 . . .

COR. REFERENCE NO. 2-5-L1

CLIENT: Ontario Department of Highways  
 PROJECT: Underpass at proposed highway #126  
 LOCATION: (Highbury Ave.) and Commissioners Rd..  
 DATUM ELEVATION: (921.71 (see encl. 1) London, Ontario.

METHOD OF BORING: 2½" dia. power auger  
 DIAMETER OF BOREHOLE:  
 DATE: 14 May 1962  
 ENCLOSURE NO. 3

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %				REMARKS
				NUMBER	TYPE	N <sub>60</sub> or No. of blows per foot of sample	20	40	60	80	100	P <sub>u</sub>	W	L <sub>u</sub>		
							SHEAR STRENGTH: lbs/sq. ft.									
							1000	2000	3000	4000	5000	10	20	30	40	
922.4	0	Ground surface														
		organics														
	5			1	SS	29										Bulk density (p.c.f.) 147
		Brown clay till														
	10			2	SS	22										134
909.4																
	15			3	SS	16										136
	20			4	SS	19										132
					vane											
	25	Grey silty clay		5	IV											136
				6	SS	26										
	30															
				7	SS	25										
887.4	35			8	SS	>100										
		Brown sandy silt														
882.4	40	Fine grey sand w/		9	SS	>100										
		brown grey														
	45	Sandy silt		10	SS	>100										
873.9																
	50	Grey silty clay		11	SS	29										
870.9		End of borehole														

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: MC CHD: JP

# GEOTECHNICAL DATA SHEET FOR BOREHOLE...3...

OUR REFERENCE NO. 2-5-L1

CLIENT: Ontario Department of Highways  
 PROJECT: Underpass at proposed highway #126  
 LOCATION: (Highbury Ave.) and Commissioners Rd.,  
 DATUM ELEVATION: 921.71 (see encl. 1) London, Ontario.

METHOD OF BORING: 2½" dia. power auger  
 DIAMETER OF BOREHOLE: ENCLOSURE NO. 4A  
 DATE: 9, 11 & 12 May 1962

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %				REMARKS
				NUMBER	TYPE	2-b Admixture of Sand/Silt	20	40	60	80	100	10	20	30	40	
920.2	0	organics														
	5	Clay till		1	SS	30										Bulk density (p.c.f.) 148
	10	brown grey		2	SS	19										145
907.2	15			3	SS	17										
	20	Grey silty clay		4	SS	18										145
	25			5	TW											135
	30			6	SS	17										Vane resistance greater than 6000 p.s.f.
	35			7	SS	24										
886.2	40	Silt clay seam		8	SS	100										
	45	brown, sandy grey		9	SS	100										
873.2	50	Grey silty clay		10	SS	96										
		Borehole continued on Enclosure														

# GEOTECHNICAL DATA SHEET FOR BOREHOLE .3. . . .

OUR REFERENCE NO. 2-5 L1

CLIENT Ontario Department of Highways

PROJECT Underpass at proposed highway #126

LOCATION (Highbury Ave.) and Commissioners Rd.,

DATUM ELEVATION (921.71 (see encl. 1) London, Ontario.

METHOD OF BORING 2 1/2" dia. power auger

DIAMETER OF BOREHOLE

DATE 9, 11 & 12 May 1962

ENCLOSURE NO. 4B

ELEVATION IN FEET	STRATIFICATION SYMBOL	STRATIFICATION DESCRIPTION	SAMPLES			PENETRATION RESISTANCE Blows per foot					CONSISTENCY Water content %		REMARKS
			NUMBER	TYPE	NUMBER OF SAMPLES	20	40	60	80	100	PL	W	
8702.50			11	SS	20								Vane would not penetrate below 52'6"
55		Grey silty clay	12	SS	23								
859.6		Fine grey silty sand	13	SS	>100								
858.7		End of borehole											
65													



# GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 4 . . .

CLIENT REFERENCE NO. 2-5-L1

CLIENT Ontario Department of Highways

PROJECT Underpass at proposed highway #126

LOCATION (Highbury Ave.) and Commissioners Road

DATUM ELEVATION 921.71 (see encl. 1)

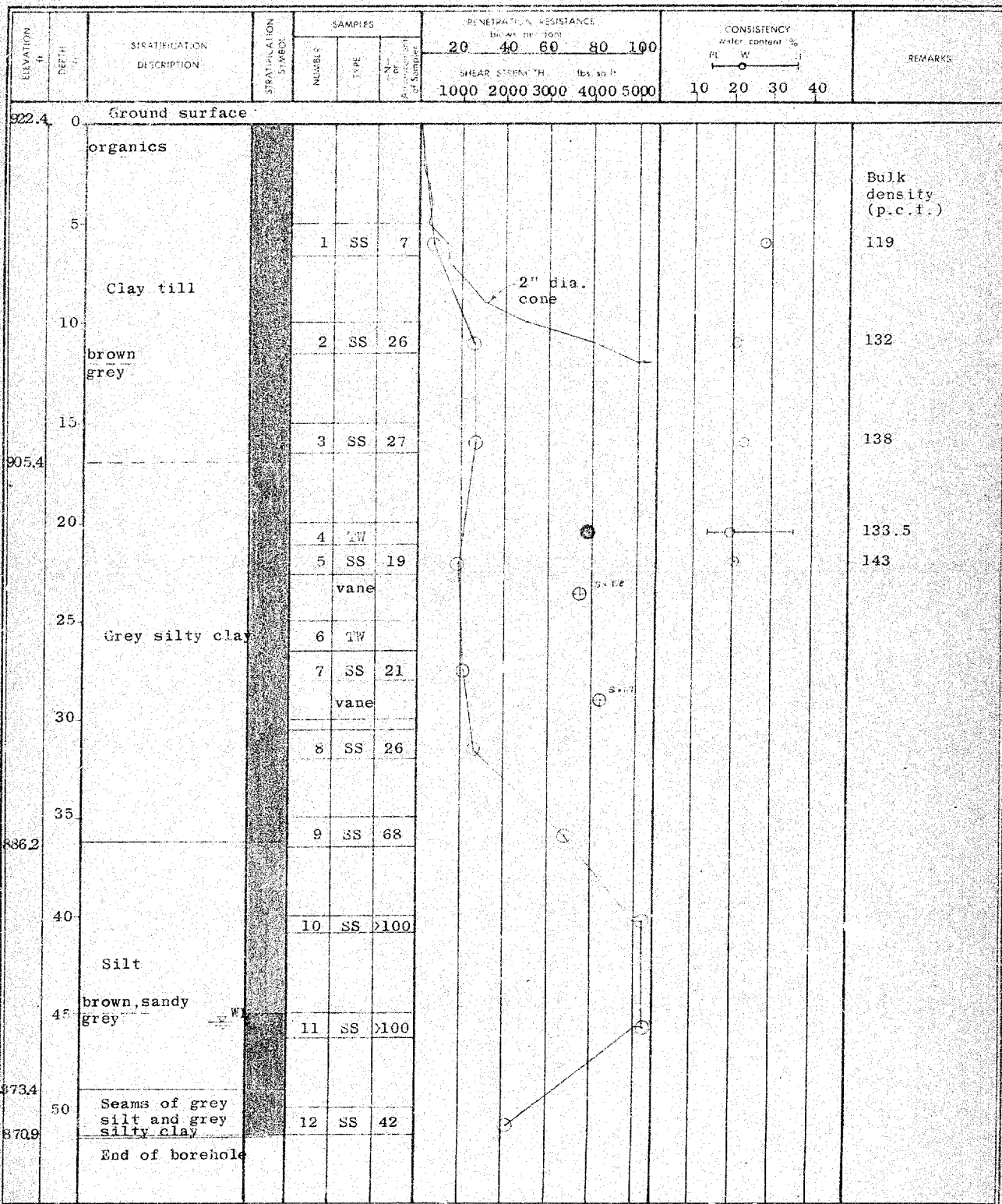
London, Ontario

METHOD OF BORING 2½" dia. power auger

DIAMETER OF BOREHOLE

DATE 15 May 1962

ENCLOSURE NO. 5



VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: MC

CH'D: JP

OUR REFERENCE NO. 2-5-L1

# GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . . .

CLIENT: Ontario Department of Highways

METHOD OF BORING: 2 1/2" dia. power auger

PROJECT: Underpass at proposed highway #126

DIAMETER OF BOREHOLE

ENCLOSURE NO. 6

LOCATION: (Highbury Ave.) and Commissioners Rd.,

DATE: 9 & 10 May 1962

DATUM: ELEVATION: 921.71 (see encl. 1)

London, Ontario

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE Blows per foot					CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	NO. OF ADJUSTMENTS to Standard	20	40	60	80	100	FL	W	
							SHEAR STRENGTH lbs. sq. ft.							
							1000	2000	3000	4000	5000			
920.5	0	organics		1	SS	14								
	5			2	SS	40								
				3	SS	43								
	10	Clay till		4	SS	42								
		brown grey												
	15			5	SS	27								
902.5	20			6	SS	18								
					vane									
	25	Grey silty clay		7	TW									
				8	SS	26								
					vane									
	30			9	SS	26								
885.5	35			10	SS	100								
		Brown sandy silt												
880.3	40	Fine grey sand		11	SS	100								
		brown, sandy grey												
	45	Silt		12	SS	86								
872.0														
	50	Grey silty clay		13	SS	34								
869.0														

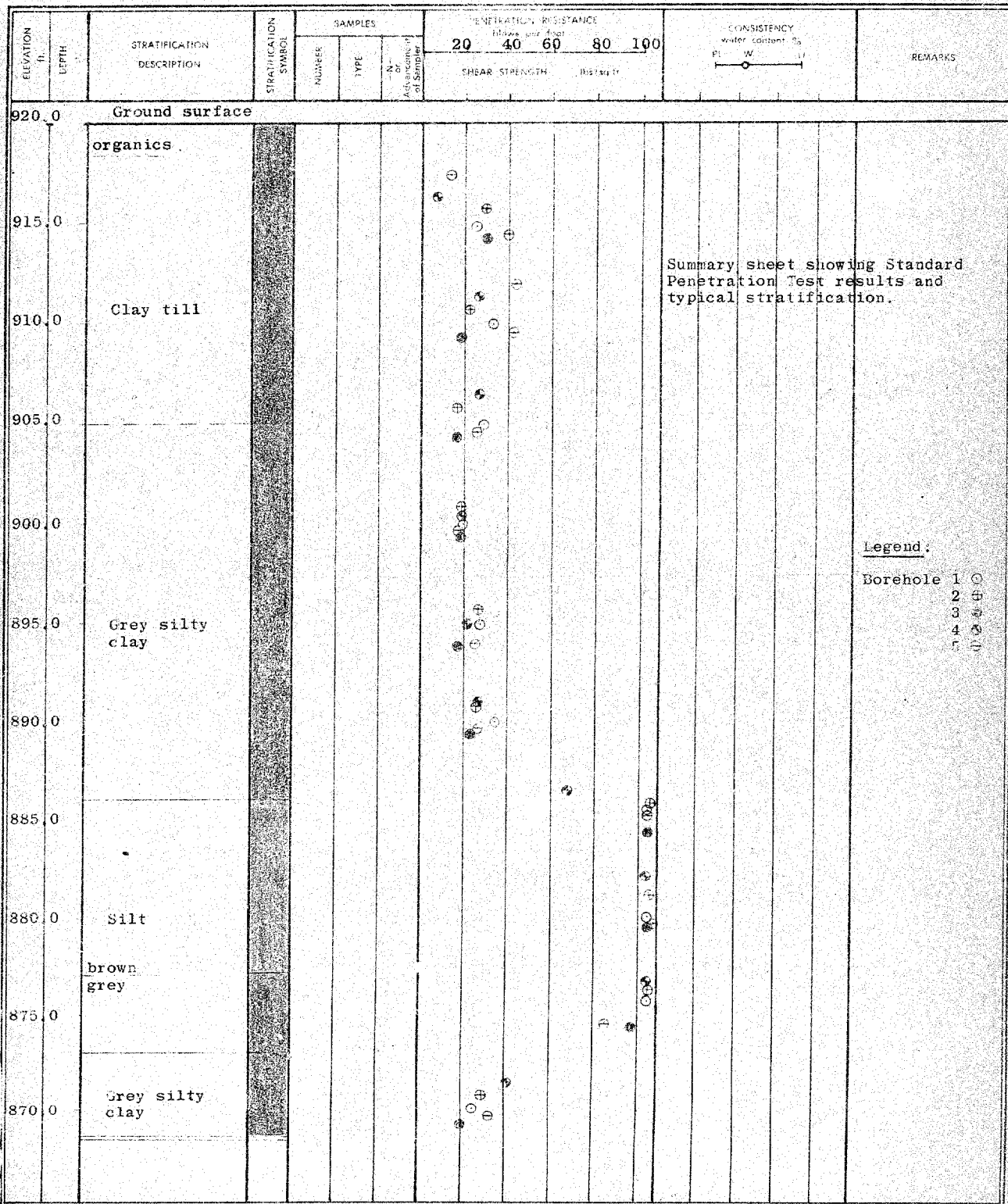


# GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . . .

OUR REFERENCE NO. 2-5-L1

CLIENT: Ontario Department of Highways  
 PROJECT: Underpass at proposed highway #126  
 LOCATION: (Highbury Ave.) and Commissioners Road  
 DATUM ELEVATION: 921.71 (see encl. 1) London, Ontario

METHOD OF BORING 2½" dia. power auger  
 DIAMETER OF BOREHOLE  
 DATE: ENCLOSURE NO. 7



**Dominion Soil Investigation Ltd.**Engineering Data Sheet for ~~Standard~~ Cones 6, 7, 8, 9 and 10

Date:

9 - 15 May 1962

Project: Highbury Underpass at

Location: Commissioners Road

Hole Location: See enclosure 1

Hole Elevation and Datum:

Field Supervisor: JP Prep.: MC

Driller: HD Checked: JP

**LEGEND**

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

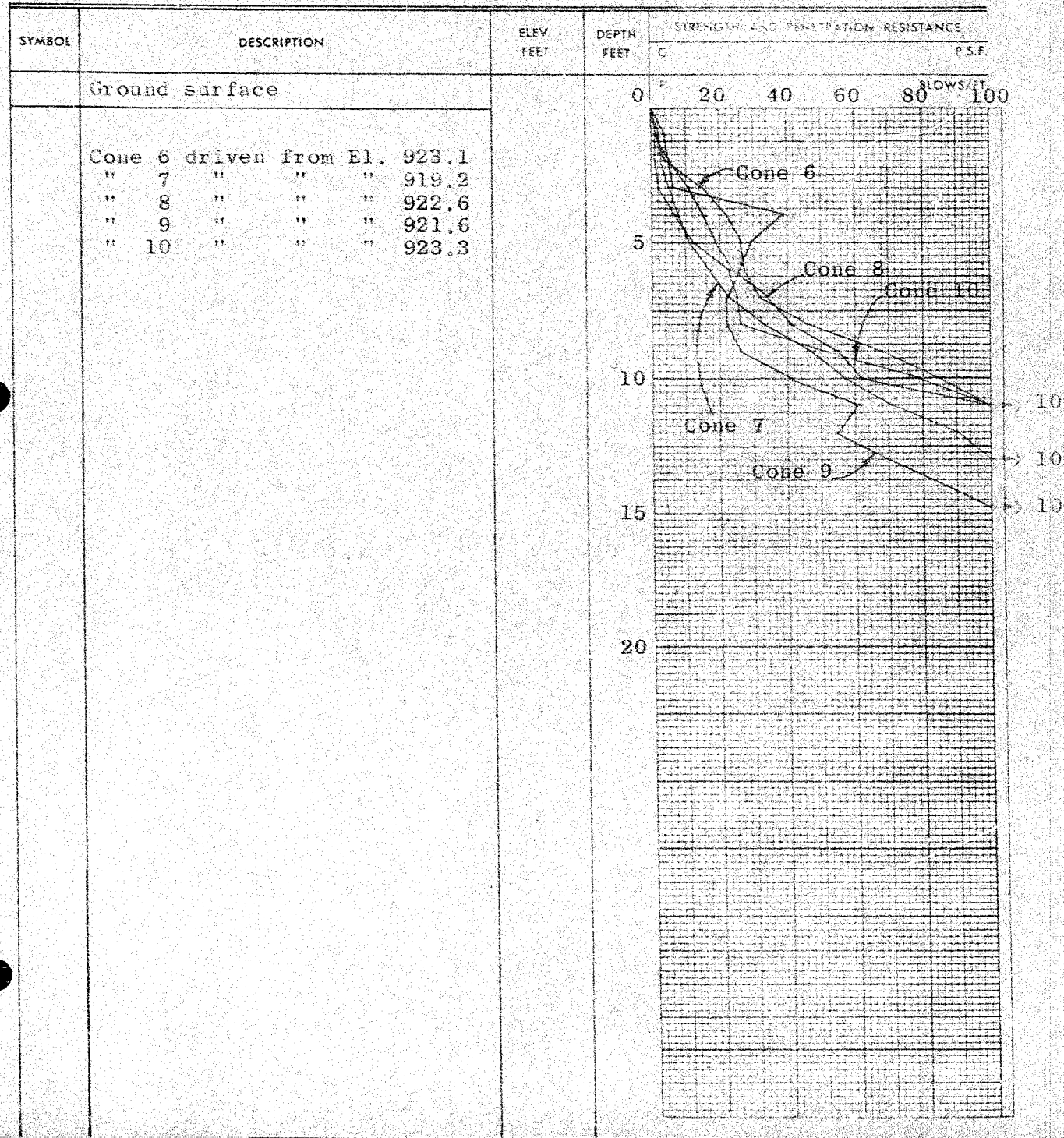
2" Dia. Cone

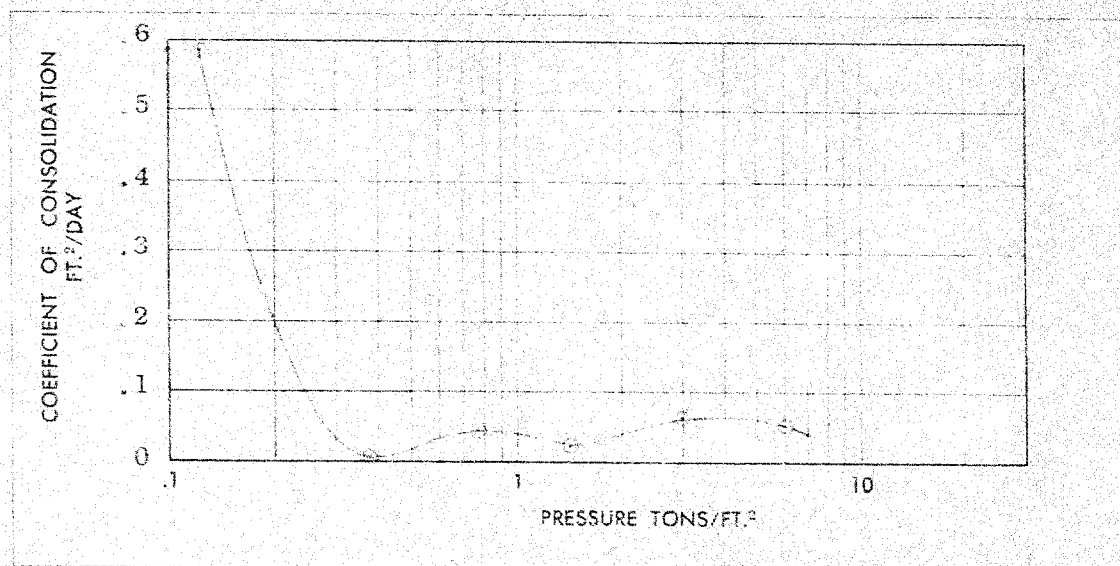
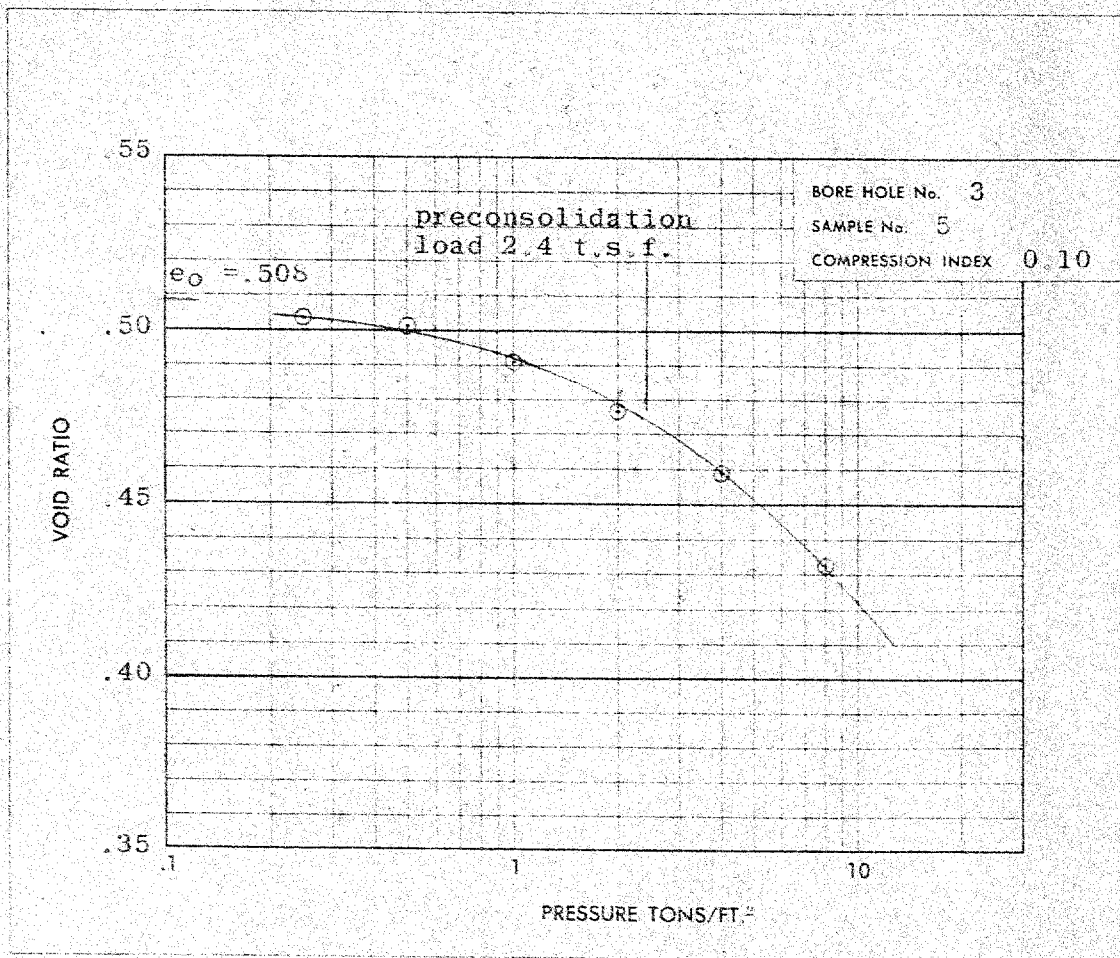
Casing

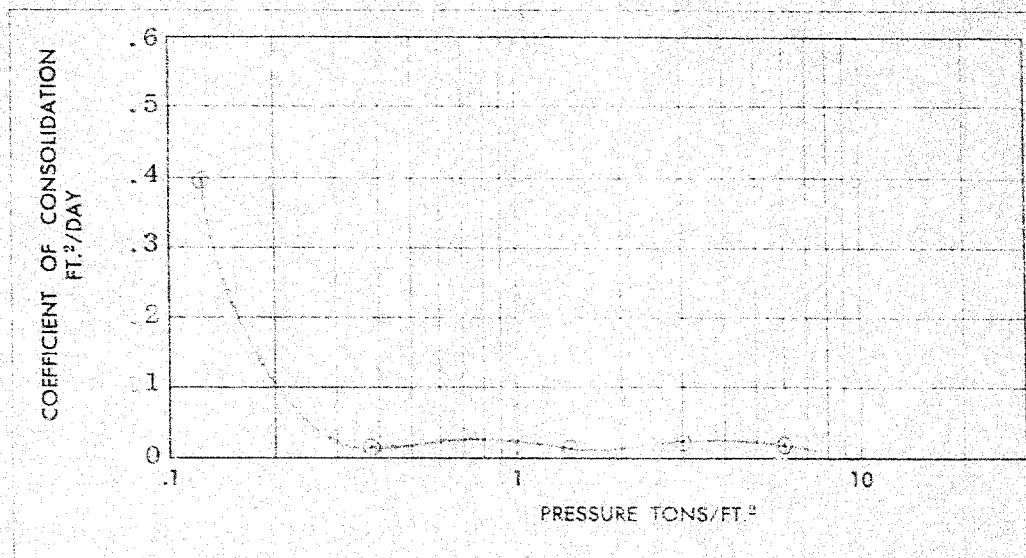
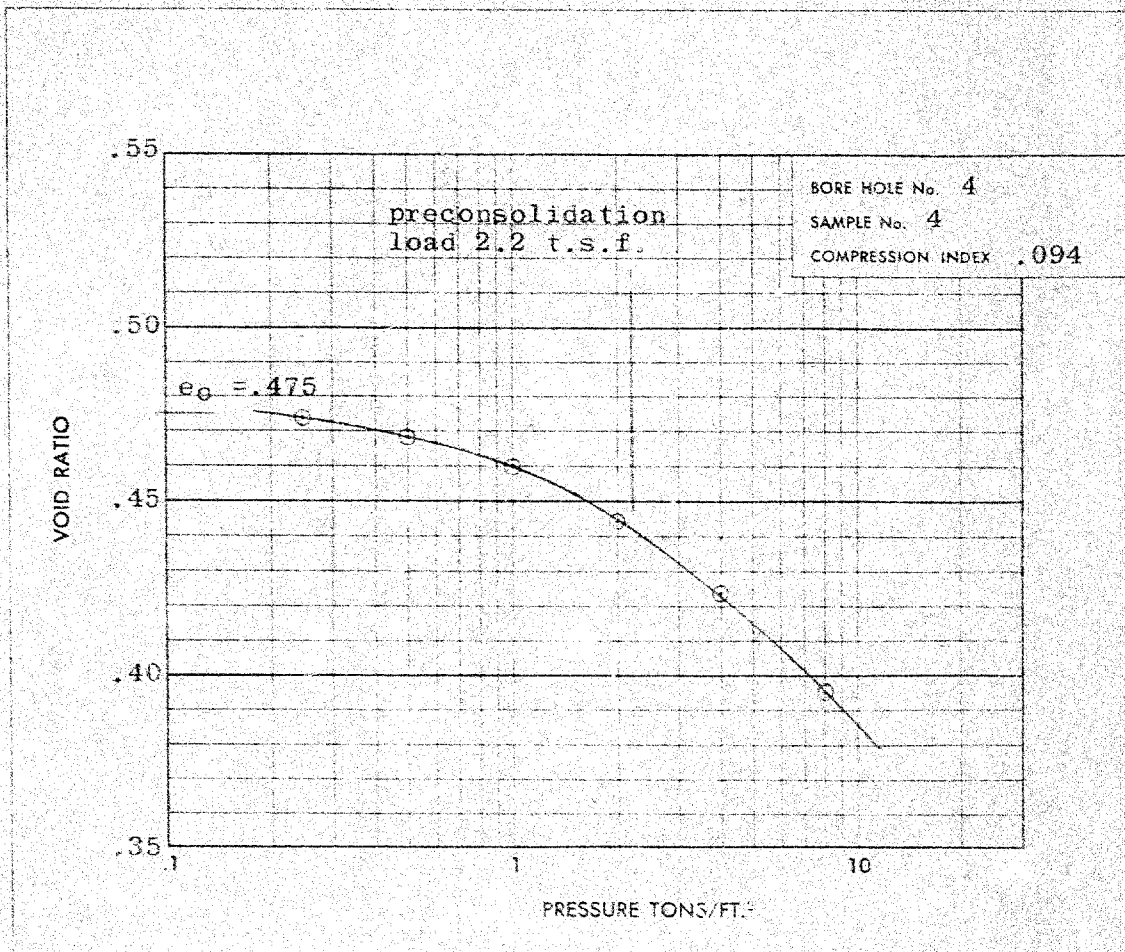
Sampling Method

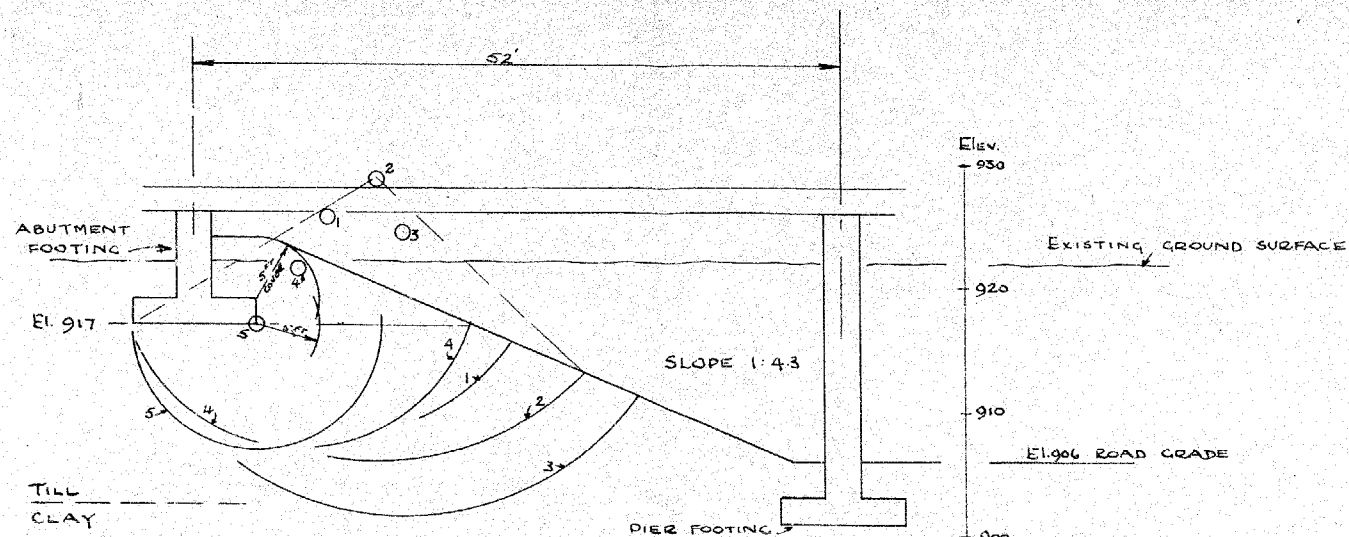
2" Dia. split tube

2" Shelby tube



**Dominion Soil Investigation Ltd.****CONSOLIDATION TEST**

**Dominion Soil Investigation Ltd.****CONSOLIDATION TEST**

C=3500 p.s.f.,  $\phi=0$  ANALYSIS

CIRCLE NO.	FACTOR OF SAFETY
1	3.3
2	3.7
3	3.8
4	3.2
5	3.5

C.O.  $\phi=36^\circ$  ANALYSIS

CIRCLE NO.	FACTOR OF SAFETY
4	1.6 (WITH INITIAL PORE PRESSURE VALUES)
	2.2 (WITH ZERO PORE PRESSURE)

## STABILITY OF TYPICAL ABUTMENT FOOTING

SCALE: 1 INCH TO 10 FEET



SUMMARY OF LABORATORY TEST DATA

Borehole No.	2	3	4	5
Sample No.	5	5	4	7
Depth (feet)	24	25	21	25
Elevation (feet)	897	895	900	895
Liquid limit (%)	-	40.2	35.3	-
Plastic limit (%)	-	12.2	13.0	-
Liquidity index	-	0.26	0.28	-
Plasticity index	-	28.0	22.3	-
Natural moisture content (%)	18.9	19.5	19.2	19.3
Shear strength (p.s.f.)	5000	2910	3920	2130
Void Ratio	0.48	0.49	0.50	0.52
Degree of saturation (%)	100	100	100	100
Elastic modulus (t.s.f.)	83	38	118	38
Bulk density (p.c.f.)	136	135	134	134
Compression index	-	0.10	0.094	-
Group symbol	-	CI	CL	-

(Continued on Enclosure 12a)

## Enclosure 12a

Borehole No.	Sample No.	Natural moisture content (%)	Bulk density (p.c.f.)
2	1	17.4	147
2	2	21.6	134
2	3	20.5	136
2	4	19.8	132
3	1	15.2	148
3	2	21.0	146
3	4	18.6	147
4	1	28.8	119
4	2	21.2	132
4	3	22.8	138
4	5	20.3	143

Materials and Research Division

May 8, 1962.

Dominion Soil Investigation, Ltd.,  
77 Creekford Blvd.,  
Scarborough, Ontario.

Attention: Mr. A. Hogg.

Re: W.P. 95-62 -- Hwy. #126,  
Commissioners Rd. Underpass,  
London, Ontario, District #2.

Dear Sir:-

Please consider this your authority to carry out a foundation investigation at the above site. Plans and profiles were provided to your representative on May 7, 1962.

It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Fourteen copies of the completed foundation report should be submitted to the Foundation Section as soon as possible. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Charges for the work performed will be in accordance with your Schedule of Rates, dated February 17, 1959, and invoice to be addressed to the attention of the undersigned.

Note:- As London is the nearest mobilization point, payment for mobilization will be from there, as discussed with your representative.

MDR/MdOF

Yours very truly,

cc: Messrs. A. McCombie

A. Gater

W. L. Fraser

J. Ray

E. D. Smith (2)

Mrs. I. Tate

Foundations Office ✓

Gen. Files (2)

A. Rutka,

MATERIALS & RESEARCH ENGINEER



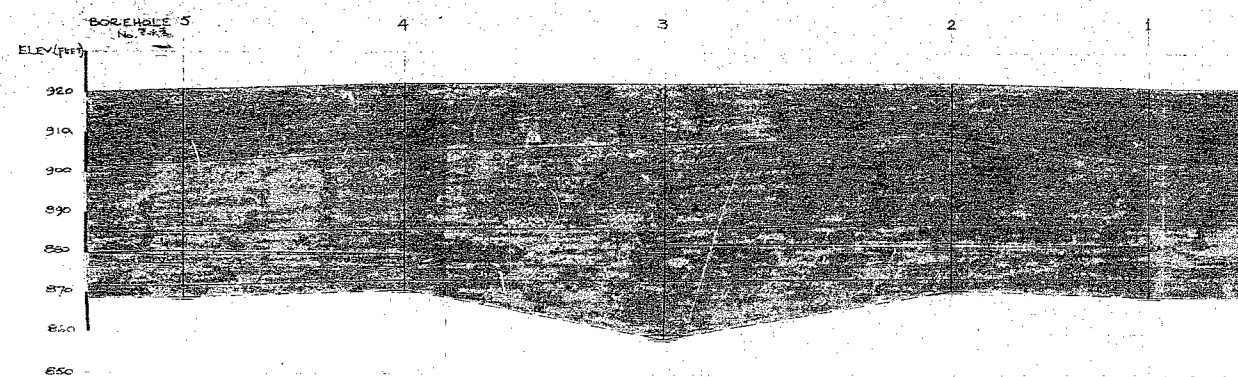
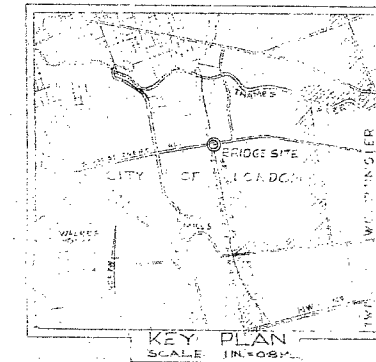
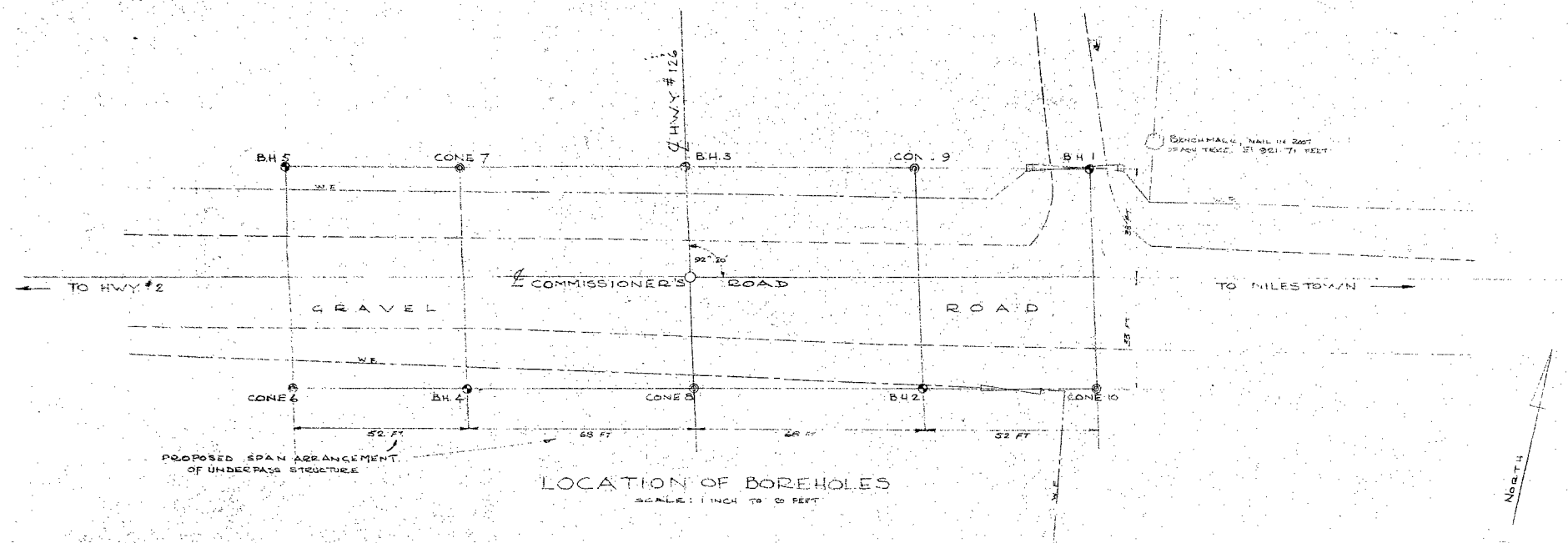
#62-F-212-C

W.P. # 95-62

Hwy. # 126 e'

COMMISSIONERS

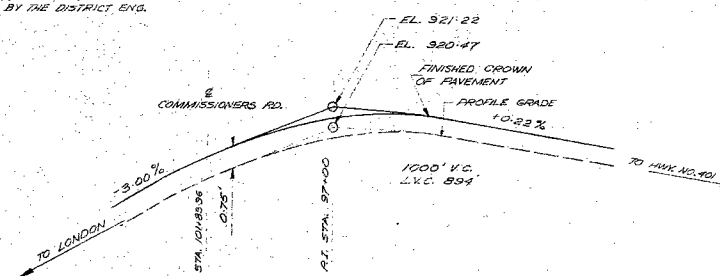
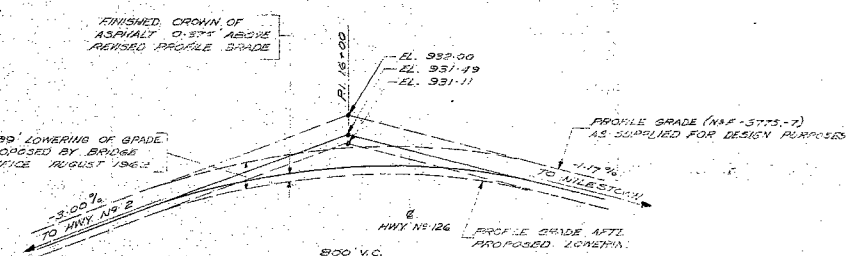
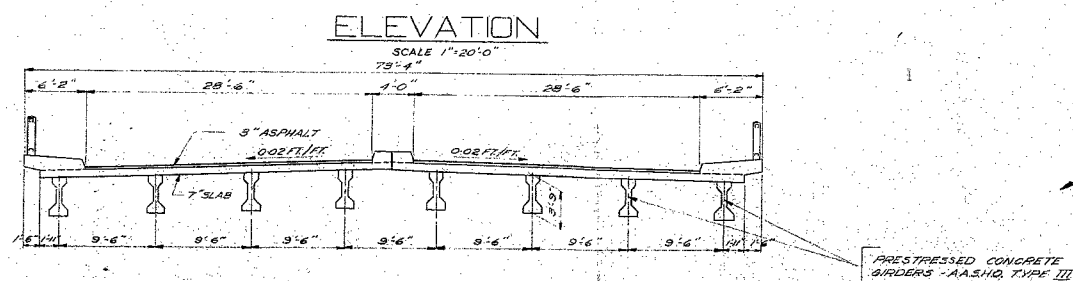
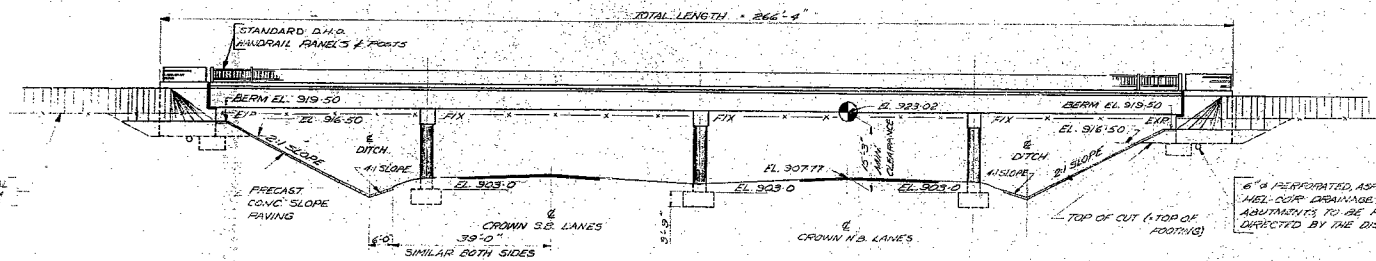
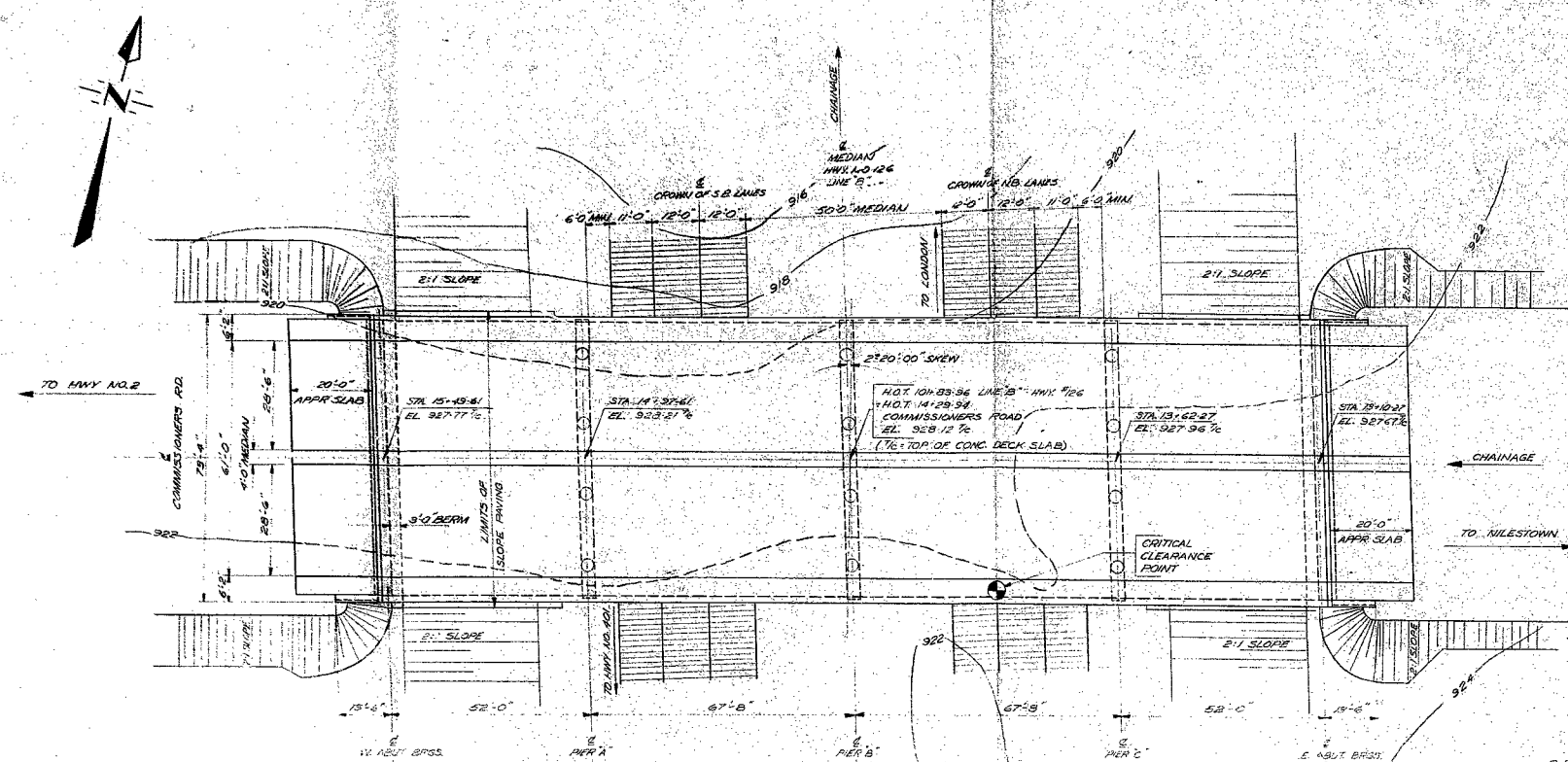
ROAD



#### LEGEND

- VERY STIFF TO HARD BROWN CLAY TILL
- VERY STIFF GREY SILTY CLAY
- BROWN SANDY SILT
- GREY SILT
- FINE GREY SILTY SAND
- DENOTES FREE WATER LEVEL

REF. NO. A-5-11	ONTARIO DEPARTMENT OF HIGHWAYS
DATE MAY 1965	SOIL INVESTIGATION
DRAWN BY M.C.	FOR PROPOSED HWY #2
CHECKED BY M.C.	COMMISSIONER'S ROAD UNDERPASS
	LONDON, ONTARIO, DISTRICT #2, W.P. 95-60
	DOMINION SOIL INVESTIGATION LIMITED
	LONDON 363 QUEENS AVE. ONTARIO

[illegible][illegible]

<u>DEPARTMENT OF HIGHWAYS ONTARIO</u> <u>BRIDGE DIVISION</u>			
<u>COMMISSIONERS ROAD</u> <u>UNDERPASS</u>			
KING'S HIGHWAY No. <u>126</u>		DIST. No. <u>2</u>	
CO. <u>MIDDLESEX</u>		<u>2 MILES NORTH OF ARIY NO 401</u>	
<u>ENR</u> CITY OF LONDON		LOT <u>CON.</u>	
<h2 style="margin: 0;">PRELIMINARY PLAN</h2>			
APPROVED _____		SITE No. <u>20-45;</u>	
BRIDGE ENGINEER		W.P. No. <u>95-62</u>	
DESIGN <u>J.L.K.</u>	CHECK _____	CONTRACT No. _____ <div style="border: 1px solid black; width: 100px; height: 100px; margin: 10px auto;"></div>	
DRAWING <u>R/L</u>	CHECK <u>J.L.K.</u>		
DATE <u>AUGUST 1962</u>	LOADING <u>1100</u> <u>S-16</u>	DRAWING No. <span style="font-size: 2em; font-weight: bold;">D-5106-P1</span>	