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STR. SITE No.

HWY. No. 17

LOCATION Slope Instability
South Bank of Ottawa River

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

CONT 74-121

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

MEMORANDUM

TO:	Mr. J. M. Childs, District Engineer, District #9, Ottawa, Ontario.	FROM:	Foundations Office, Design Services Branch, West Bldg., Downsview.
ATTENTION:		DATE:	November 22, 1973.
OUR FILE REF.		IN REPLY TO	DEC - 3 1973

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Slope Instability, South Bank of
Ottawa River
(Hwy. 17 Between Station 262 and Station 380)
Township of Cumberland
Regional Municipality of Ottawa-Carleton
W.O. 73-11053(X) - W.P. 159-73-01

Attached we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above-mentioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

AGStermac

A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

AGS/ao
Attch.
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Foundations Files
Documents

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FOUNDATION INVESTIGATION REPORT
For
Slope Instability, South Bank of
Ottawa River
(Hwy. 17 Between Station 262 and Station 380)
Township of Cumberland
Regional Municipality of Ottawa-Carleton
W.O. 73-11053(X) - W.P.

1. INTRODUCTION:

In the spring of 1973, a slope failure took place on the south bank of the Ottawa River approximately 1 mile east of the Village of Cumberland (Station 370 to Station 373 of Hwy. 17). The distressed area was approximately 250 feet long and its upper limits were, in places, dangerously close to the north shoulder of the Hwy. #17. In addition, longitudinal cracks up to 100 feet long and 1 foot wide were observed on the pavement of Hwy. 17 in the distressed zone. Because of the proximity of the slide and the existence of the potential danger to the highway, a stretch of Hwy. 17 approximately 4 miles long was closed to traffic and remedial measures recommended by this office were promptly carried out. During the construction of the remedial work, a site meeting, attended by personnel from Foundations Office, Kingston Regional Office and Ottawa District Office, was held to discuss the construction details. At that time, Mr. J.B. Wilkes, Executive Director of Design Division, and Mr. L. R. Eadie, Executive Director of Operations Division, were also present. They were in complete agreement with our suggestions that the Foundations Office should carry out a detailed investigation in this general area (Hwy. 17, Station 262 to Station 380) in order to provide necessary recommendations to ensure the long term stability of the river bank in the vicinity of the Village of Cumberland.

Subsequently, the Foundations Office carried out a detailed subsurface investigation in this area to determine the subsoil, bedrock and groundwater conditions. This report contains the factual data obtained from this investigation together with slope stability analyses of the river bank and also suggested remedial measures to ensure the long term stability of the natural slopes along the Ottawa River in this area.

2. SITE AND GEOLOGY:

The site is located on the south bank of Ottawa River along Highway 17, from 1.5 miles west to 2 miles east of the Village of Cumberland, in the Township of Cumberland, Regional Municipality of Ottawa-Carleton. In this area, Hwy. 17 is in close proximity to the Ottawa River Channel, except within the limits of Cumberland Village. The average slope of the river bank ranges from 2-1/4:1 to 3-1/2:1. The slopes are generally bush-covered. Photographs showing the surficial features of this area are included in the Appendix I of this report.

Physiographically the site is located in the "Ottawa Valley Clay Plain." This region was invaded by one or more ice sheets advancing from the north during Pleistocene time*. The pre-glacial land surface was modified by glacial erosion and by deposition, in places, of material eroded by the ice sheet. Near the close of the Pleistocene time, when the ice sheet began to retreat, the area was, in large part, below the sea level. As the ice retreated and melted back, the sea entered and overspread the Ottawa Valley to a depth, in places, of several hundred feet. In this arm of the sea, known as Champlain Sea, thick deposit of clay was laid down. As the ice sheet further retreated, uplift took place; the land gradually emerged from the sea. The thickness of the clay beds in this area varies randomly from 30 feet to over 200 feet. The clay deposit is underlain, in general, by glacial

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*Johnston, W.A. "Pleistocene and Recent Deposits in the Vicinity of Ottawa, with a Description of the Soils." Geological Surveys #84, Dept. of Mines.

till, which in turn is underlain by limestone bedrock of the Leray beds of the Trenton and Black River formation, Palaezoic Period.

3. FIELD INVESTIGATION AND LABORATORY PROCEDURES:

3.1) Field Investigation:

The field work consisted of fourteen boreholes, three of which were accompanied by dynamic cone penetration tests. The boreholes were drilled by two conventional diamond drill rigs adapted for soil sampling purposes. One of the two machines was mounted on a raft in order to carry out the boreholes located within the Ottawa River channel.

Relatively undisturbed samples of the cohesive soil were obtained by means of two- and three-inch I.D. Shelby Tubes manually pushed into the soil. Disturbed samples in all deposits were recovered by means of a standard 2-inch O.D. split spoon sampler driven into the soil with an energy of 350 ft.-lb. per blow according to the specifications of Standard Penetration Test. The same method was used to advance the dynamic cone penetration test. BX size rock core samples were obtained at only one boring location to prove bedrock conditions.

Field vane tests were carried out, wherever possible, to determine the in-situ undrained shear strength of the cohesive deposit, using a standard M.T.C. 2-5/8" diameter field vane having such dimensions that the undrained shear strength of the soil equals twenty times the applied torque at failure. The vane tip was advanced eighteen inches into the undisturbed soil in a single thrust without rotating the rod. The torque was then applied and the value at failure recorded. The vane was then rotated six complete turns and a remoulded test carried out in the same fashion. The ratio of the undisturbed strength to the remolded strength gives the sensitivity of the cohesive soil.

Groundwater level observations were carried out, during the period of the investigation, in the open boreholes. In

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addition, twelve piezometers of "Terra Test Type", manufactured by Terra Test Piezometers, Toronto, Ontario were installed at seven boring locations. The tip elevations of the various piezometers were shown on individual Record of Borehole sheets, contained in Appendix IIof this report. Piezometric water level readings were taken periodically during and after the field investigation and the results are also shown on the Record of Borehole sheets.

The soil, bedrock and groundwater conditions encountered at the boring locations, are presented in the Record of Borehole sheets. The location and elevation of the boreholes were surveyed in the field by personnel from Ottawa District Office. The elevations in this report are referenced to a Geodetic Datum, Boring locations and elevations are shown on Drawing No. 73-11053A & B, together with estimated stratigraphical section through five critical areas.

3.2) Laboratory Procedures:

All samples were subjected to a careful visual examination in the field and subsequently in the laboratory. Following the examination, laboratory tests were carried out on representative samples to determine the identity and physical properties of the overburden; namely,

Natural Moisture Contents
Atterberg Limits
Grain Size Distributions
Bulk Unit Weights
Undrained Shear Strength Measurements
Shear Strength Measurements in terms of Effective Stress (C.I.U. Tests with Pore Pressure Measurements)
Consolidation Characteristics

The results of the laboratory testing were plotted on the individual Record of Borehole sheets and also summarized on Figures No. 1 to 15, inclusive, all of which are appended to this report.

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4. FIELD OBSERVATIONS:

During August 20 and 21, 1973, Messrs. M. Devata and C.S. Poon of the Foundations Office, together with Maintenance Personnel from Ottawa District Office, reviewed critically all the potential problem areas where some remedial measures may be warranted. This review was carried out by means of a boat trip along the south side of the Ottawa River. A resume of our observations is as follows.

4.1) From Station 262 to Station 272
(Area Just West of the Cumberland Road Side Park):

i) This stretch of Hwy. 17 is located on the top of the Ottawa River bank. It was approximately 45 feet above the river water level during the time of the field observations. The overall slope of the river bank in this area ranges from 3:1 to 4:1. Heavy growth of trees and bushes exist at the lower half of the slope surface.

ii) Active toe erosion was evident, resulted with localized near-vertical slopes of up to 10 feet high.

iii) Two old failures (Station 263 and Station 265) were visible at mid-height of the river bank slope.

iv) A distinctive toe failure was observed at Station 271 immediately west of the Cumberland Park. The size of this failure is approximately 50' wide by 20' deep. Grey marine Leda Clay was exposed (Refer to Plate Nos. 1, 2 and 11).

4.2) From Station 272 to Station 290 (Cumberland Park):

Numerous slides have occurred in this area including the Cumberland Park slide that took place in May 1962. Remedial measures incorporating flattening of the slope to 4:1 and rock fill toe berms were adopted. The slopes in this area appear to be performing satisfactorily without any signs of distress.

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4.3) Station 290 to Station 301 (East of Cumberland Roadside Park):

The rock slopes of the river bank are generally flatter than 3:1. In addition, the toe of the slopes was protected with rock rip rap. It appears that this protective measure was incorporated to ensure the stability of the Hydro transmission towers in this area. There is no evidence of any toe erosion and the slopes appear to be stable.

4.4) From Station 301 to Station 307:

i) This stretch of Hwy. 17 is in a cut section on the top of the river bank. The roadway is some 55 feet above the Ottawa River water level (September 1973). The portion of the river bank slope immediately north of the highway has an average slope of 2-1/2:1 to 3-1/2:1.

ii) Toe erosion is evident, as can be seen in Plate No. 12 of Appendix I.

iii) Local sloughings of the slope surface at mid-height were also observed at various locations.

4.5) Station 307 to Station 309 (Failed During 1972-73):

A recent slide took place in this area and remedial measures similar to those discussed elsewhere for the slopes between Station 367 and Station 373 have been adopted. The slope appears to be performing satisfactorily without any further signs of distress.

4.6) Station 309 to Station 360:

The Hwy. 17 in this area is generally situated far away (up to 2000 ft.) from the Ottawa River. Between Station 309 and Station 325, outcrops of limestone bedrock can be seen immediately above the river water level. Between Station 325 and Station 360 the river bank is generally very flat and at certain locations, protected by rock rip rap at the toe by local residents.

4.7) Station 360 to Station 367:

i) Hwy. 17 in this area is situated on the top of the river bank. It is very close to the river channel and is approximately 45 feet higher than the river water level. The average slope of the river bank is about 2-1/2:1. The lower half of the slope is bush-covered. The trees at the toe of the slope are generally tilted, indicating some signs of distress.

ii) Active toe erosion was also observed.

4.8) Station 367 to Station 373 (Failed during April 1973):

A major failure took place during spring of 1973 which resulted in closing of Hwy. 17 to traffic. This office provided the necessary recommendations for the remedial work to stabilize the river banks in this area. The suggested remedial measures consisted of flattening the slopes to 3:1 and a rock fill toe berm approximately above the maximum high water level. Typical sections showing these details are included in Appendix II of this report.

Observations made during August 1973 after the completion of the remedial work, revealed that the slopes were generally satisfactory (Refer Plate No. 5 and No. 6).

4.9) Station 373 to Station 380:

i) Hwy. 17 in this area is in a cut section on the top of the river bank with the roadway being situated some 40 feet above the river water level. The overall slopes of the river banks are as steep as 2-1/4:1.

ii) The trees near the toe of the slopes were generally tilted, showing some signs of slope instability.

iii) Old failures are visible at the following stations:

a) 373+50 to 374+50
b) 376+00 to 377+00
c) 378+50 to 380+00

5. SUBSOIL AND BEDROCK CONDITIONS:

5.1) General:

The predominant deposit in this area is a sensitive silty clay to clay with traces of organics, whose thickness was found to be quite variable. This cohesive stratum is underlain by a thin deposit of sand, silt and gravel, which in turn is followed by limestone bedrock. At some locations, the cohesive stratum is overlain by fill material up to 23 feet thick. However, at one location near the toe of the river bank, the cohesive stratum is overlain by a thin granular deposit of sand with some silt.

The boundaries between the various deposits, as determined at the boring locations, are shown on the accompanying Record of Borehole sheets. The stratigraphical sections, as shown on Drawings 73-11053A and B, have been inferred from this data.

From ground surface downward, the various soil types encountered are described as follows:

5.2) Fill Material (Silty Clay With Some Sand and Gravel, With Pockets or Layers of Sand and Gravel):

Fill material up to 23 feet thick was found at several boring locations put down on the top of the river banks. In general, the fill material is a silty clay with some sand and gravel, with pockets or layers of sand and gravel. At B.H. #13 which penetrated through the shoulder of the roadway, only sand and gravel fill was found and is believed to be the granular base material. Grain-size distribution curves for the samples of the cohesive and cohesionless portion of the fill material are presented on Figure Nos. 2 and 3, respectively.

Standard penetration testing was carried out within the fill material and the results were plotted on the Record of Borehole sheets. The 'N' values generally vary from 2 to 40 blows per foot, indicating that the fill material has been subjected to poor to moderate compactive efforts.

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5.3) Sand With Some Silt:

At Borehole #2 which is located near the toe of the river bank, approximately three feet of sand with some silt was found immediately below the river bed. Standard penetration testing carried out within this granular deposit gave 'N' values of 1 to 4 blows per foot. The relative density of this stratum is therefore estimated to be very loose to loose.

5.4) Silty Clay to Clay With Traces of Sand and Occasional Inclusions of Organic Matters:

This is the predominant stratum found in all boring locations. The thickness of this deposit ranges from 33 feet (B.H. #5) to over 140 feet (B.H. #7). It is a sensitive marine silty clay to clay, with traces of sand and occasional inclusions of organic matters. Samples recovered from this stratum indicate that the clay deposit is grey in colour; however, occasional bands and pockets of brown clay are present in the upper portion of this deposit. Thin seams of silt up to 1/2 inch thick are also found randomly within this stratum. Grain size distribution curves for the samples of this cohesive stratum are shown on Figure No. 1, in an envelope form.

The engineering properties of the cohesive deposit were summarized on Figures No. 6 and 7 contained in the Appendix, and also tabulated below:

Index Properties	Range	(Average)
Natural Moisture Contents W (%)	31 - 73	(59)
Liquid Limits W_L (%)	42 - 83	(60)
Plastic Limits W_p (%)	22 - 32	(27)
Liquidity Index I_L	0.58 - 2.25	(1.10)
Bulk Unit Weight γ (p.s.f.)	98 - 113	(103)

Compressibility Characteristics	
Initial Void Ratio (e_o)	1.60 - 1.73
Compression Index (C_c)	0.94 - 1.68
Degree of Preconsolidation (P_c-P_o') p.s.f.	2,500 - 6,200

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Undrained Shear Strength (C_u) p.s.f.	Range
In Situ Vane Tests	480 -> 2,000
Laboratory Vane Tests	565 - 2,335
Unconfined Compression Tests	480 - 1,935
Sensitivity (By In Situ Vanes)	4 - 48

A brief resume of the test results is also given as follows:

Atterberg limit tests were carried out on samples of this deposit. The results, which were plotted on the Plasticity Chart (Figure No. 4), indicate that, in general, the clay is inorganic and of intermediate to high plasticity. The natural water content is generally at or higher than the Liquid Limit.

Referring to Figures No. 6 and 7, it can be seen that the undrained shear strength increases with depth in a linear fashion, as represented by a C_u/P_o' ratio of about 0.4, where Po' is the effective overburden pressure. Based on these results, it is estimated that the consistency of the stratum varies from soft to very stiff, and generally from firm to stiff. The undrained shear strength values obtained from laboratory testing (unconfined compression test) gave consistently lower values than those obtained from the field vane tests. It is considered that this is primarily due to the unavoidable sample disturbance caused by the field and laboratory handling and subsequent testing of the sensitive clay. The sensitivity, defined as the ratio of the undrained shear strength of the soil in an undisturbed state to that of the soil in a remolded condition, as determined in the field in situ vane tests ranges from 4 to 48 with an average of 13. This would indicate that the clay is generally very sensitive.

The consolidation characteristics of the stratum were determined by carrying out four laboratory consolidation tests. The results of this testing, presented on Figure No. 5, indicate that the clay is preconsolidated by about 2,500 to 6,200 p.s.f. in excess of the existing overburden pressure.

Consolidated undrained triaxial testing (C.I.U. Test) with pore pressure measurement was performed on several samples obtained within this stratum in order to determine the shear strength parameters (C' and ϕ') in terms of effective stresses. The results, which are presented on Figures No. 8 to No. 15, are also given below.

$$C' = 240 \text{ p.s.f.} - 500 \text{ p.s.f.}$$
$$\phi' = 22° - 26°$$

The scattering of the above results may be attributed to the non-uniformity and degree of disturbance of the sample and also the presence of silt seams and organic inclusions. This aspect is discussed in detail in Section 7.

5.5) Sand, Some Silt and Gravel (Glacial Till):

This granular deposit was found underlying the clay stratum at four boring locations. Its thickness varies between 1.5 (B.H. #5) and 4 feet (B.H. #6). It is believed that this deposit is of glacial origin. From a limited number of standard penetration tests, it is estimated that the relative density of this granular deposit is from compact to dense.

5.6) Limestone Bedrock:

Bedrock was proven in only B.H. #5 by obtaining 4 ft. of BX size rock core sample. The bedrock was found to be a dark grey bedded limestone. It appears to be in a sound condition, as evidenced by the high percentage of recovery (90% - 95%).

6. GROUNDWATER CONDITIONS:

Groundwater level observations were carried out during the period of the field investigation. In addition, twelve piezometers of "Terra Test Type" were installed within the clay stratum or the limestone bedrock. The tip elevation of the piezometers are shown on the Record of Borehole sheets as well as Drawing Nos. 73-11053A & B, all contained in the Appendix of this report. Piezometric water level readings were taken periodically during and after the period of the field investigation. The observations are also shown on Drawing Nos. 73-11053A & B.

The results of the measurements indicate that the groundwater level within the cohesive clay stratum varies between elevations 160 and 172, which correspond to levels from 9 to 17 feet below the existing ground surface. As mentioned previously, one piezometer was installed within the limestone bedrock at B.H. #5. Observations reveal that the water level within the bedrock was some 31 feet below the existing ground surface; i.e., at elevation 146.

In general, two piezometers were installed in each borehole put down on top of the river bank, at different depths. The observations revealed that the water level reading in the shallow piezometer is at a higher elevation than that in the deep one, indicating that steady lateral seepage exists within the clay stratum towards the Ottawa River, which acts as the drainage sink for this area.

7. DISCUSSIONS:

7.1) General:

The long term stability of natural slopes and cut slopes, as determined by using the total stress approach or the undrained shear strength of the cohesive strata (Ø=0 case) does not necessarily represent the most critical condition. In this method of analysis, the effect of pore pressure changes are not considered and as a result of this, large errors can be introduced to the values of the computed factor of safety. Skempton 1. and Bjerrum 2. reported that the factor of safety computed by using Ø=0 method, may be as high as 4 to 10 for

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1. Skempton, A.W. "Opening Address", Proceedings European Conference, Stability of Earth Slopes, Stockholm, Vol. 3, pp 16-20.

2. Bjerrum, L. "Progressive Failure in Slopes of Over Consolidated Plastic Clay and Clay Shales." A.S.C.E. SM5, 1967, pp 3-49

natural or cut slopes which actually failed. The pore pressure conditions within the natural or cut slopes will change, with time, due to lateral seepage towards the slope faces. Therefore, only the effective stress approach was used to analyse the stability of several slopes along Ottawa River in this area. The selection of the shear strength parameters in terms of effective stresses and the results of the analysis are discussed in the following subsections.

7.2) Shear Strength Parameters:

A number of consolidated undrained triaxial tests with pore pressure measurements was carried out on the clay samples in an attempt to determine the shear strength parameters in terms of effective stresses (C' and Ø'). The test results, which were plotted on Figure No. 8 to No. 15, were scattered. The scattering of the results may be attributed to the nonuniformity and degree of disturbance of the sample tested and also the presence of silt seam and organics within the material. In addition, on the basis of an exhaustive study of the most reliable published material, Skempton* observed that "from the analysis of actual slips in clay, the values of the shear strength parameters as determined by conventional tests do not necessarily bear any relation to the values which must have been operative in the clay at the time of failure".

In view of the foregoing, it is decided to carry out slope stabilityanalysis for a typical cross section within the failure zone which took place in April, 1973. In the analysis, an assumed value of Ø'of 22.5° was used, which is believed to be the most probable value for Leda Clay in this general area**.

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*Skempton, A.W. "Long Term Stability of Clay Slopes". Geotechnique, I.C.E., London, Vol. 14, No. 2, 1964. pp 75-102

**Eden, W.J. E.B. Fletcher & R.J. Mitchell, "South Nation River Landslide, May 16th, 1971". Canadian Geotechnical Journal, Vol. 8, 1971.

In addition, W. J. Eden* proved that ϕ' contributes a relatively small portion of the shear strength mobilized along the potential slip surface and consequently small variation in the value of ϕ' does not significantly affect the computed factor of safety. The chosen slope was then analyzed using Bishop's slices method on the actual ground contours, with various combination of C' and the pore-pressure ratio r_u ($r_u = \frac{u}{\gamma h}$ where u is the excess pore-pressure at any point along the potential failure surface, h, the depth of the point in the soil mass below the soil surface and γ the bulk density of the soil). For $r_u = 0.62$ (groundwater table at surface and horizontal flow only), which is the most critical condition, a value of C' of 250 p.s.f. will yield a factor of safety of 1.0 (Refer to Figure No. 16). For $r_u = 0.5$, which represents the groundwater conditions prevailed in September, 1973, a value of C' of 200 p.s.f. gave a factor of safety of unity (Refer to Figure No. 17).

Field observation was made by Dr. N. R. Gadd of Terrain Sciences Division of Federal Department of Energy, Mines and Resources, immediately after the failure took place. In his report, Dr. Gadd expressed that the drainage ditch on the south side of the highway showed evidence of long term saturation by the abundant growth of cat-tails in the ditch. Water was held in depressions in the vicinity of the distressed zone. In view of this, r_u should be very close to 0.62 at the time the failure took place.

This is in complete agreement with W. J. Eden and R. J. Mitchell** who concluded that it appears that the assumption of full saturation may not be overly conservative in the analysis of the stability of natural slopes in Leda Clays.

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*Eden, W.J. — 1963 "Results of soil testing at Cumberland landslide site", Special Report No. 164, Division of Building Research, N.R.C.

**Eden, W.J. & Mitchell, R.J. — "The Mechanics of Landslides in Leda Clay". Canadian Geotechnical Journal, Vol. 7, 1970 pp 285-296.

In view of the foregoing, it appears to be reasonable to assume a value of C' = 200 to 250 p.s.f. and Ø' = 22°-25° in the stability analysis for the slopes in this general area. In fact the C' values quoted above are approximately 2/3 of the values obtained from our laboratory tests. Crawford* suggested that 2/3 of the value of C' obtained from laboratory test be used in stability analysis of natural slopes in Leda Clay, since the rate of strain used in the laboratory testing is much faster than that occurs in the field.

7.3) Stability Considerations:

Detailed stability analysis in terms of effective stresses has been carried out for four typical sections of the river banks in the vicinity of this area, namely, Stations 265+00, 305+00, 364+00 and 377+00. The shear strength parameters in terms of effective stresses, as determined by laboratory testing are tabulated below. It should be noted that the values of the effective cohesion intercept (C') have been modified as suggested by Crawford.

Location	C'	Ø'
Station 265+00	200 p.s.f.	23°
Station 305+00	200 p.s.f.	24°
Station 364+00	250 p.s.f.	24°-25°
Station 377+00	250 p.s.f.	25°

The results of the stability analysis were shown on Figure No. 18 to 21 inclusive; they are also summarized as follows.

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*Crawford, C.B. 1963 "Cohesion in an undisturbed sensitive clay". Geotechnique, Vol. 13, pp 132-146.

Location	r_u	Factor of Safety
Station 265+00	0.62	1.07
	0.52*	1.22
Station 305+00	0.62⌿	0.93
	0.45*	1.02
Station 364+00	0.62	1.05
	0.50*	1.21
Station 377+00	0.62	1.02
	0.45*	1.19

* r_u values as per piezometric data obtained during summer months, 1973

⌿ Due to the geometry of this section, it is assumed that only the upper slope is fully saturated (Refer to Figure #19)

In the above table, $r_u = 0.62$ represents the most critical condition (full saturation of the slope) and the lower r_u value represents the groundwater conditions as determined by piezometric readings taken in September and October 1973. From the computed values of the factor of safety, it may be seen that all slopes are stable with the present groundwater condition. However, the factor of safety dropped to dangerously close to or less than (Station 305+00) unity if $r_u = 0.62$ is used. As mentioned previously, an assumption of $r_u = 0.62$ in stability analysis is not overly conservative. It may be concluded that the slopes along this stretch of Hwy. 17 (Sta. 262+00 to 272+00, Sta. 301+00 to 307+00, Sta. 360+00 to 367+00 and Sta. 373+00 to 380+00) are in a state of limited equilibrium. It should be pointed out that the mechanism of failure in Leda Clay is retrogressive. Active toe erosion of slopes in this area is also evident. This toe erosion may initiate a small toe failure which in turn will trigger a full scale slope failure in spring time when the most critical groundwater conditions prevail. In view of the foregoing, it is essential that the slopes in the critical areas should be

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stabilized. This aspect will be discussed in detail in Section 8.

8. <u>REMEDIAL MEASURES</u>:

8.1) <u>Station 370+00 to 373+00 (Failure occured during April, 1973)</u>

This portion of the Ottawa River Bank adjacent to Hwy. 17 failed during spring of 1973 after heavy rainfalls. Detailed stability analysis (Figure No. 16 and No. 17) were carried out by this office immediately after the failure and the necessary remedial measures (Figure No. 22 to No. 27) were provided to the Ottawa District. The suggested remedial measures were immediately incorporated. According to our recent field observations, this portion of the river banks appeared to be in a stable condition without any signs of distress.

8.2) <u>Station 262+00 to 272+00, Station 301+00 to 307+00, Station 360+00 to 367+00 and Station 373+00 to 380+00</u>

As discussed elsewhere, the Ottawa River bank along Hwy. 17 at certain locations in the vicinity of the Village of Cumberland is in a state of limited equilibrium. A combination of toe erosion and seasonal fluctuation of groundwater conditions may result in slope instability of the Ottawa River Bank area.

In order to prevent any further toe erosion, rock fill toe berms extending to elevation 152 (approximately 2 feet higher than H.W.L.) will be necessary. It is further suggested that the existing condition of the slope above the berm elevation be conserved as the grass, bushes and trees would probably cut down the surficial erosion and lower the groundwater table, hence increase the stability of the river bank slopes.

Analysis in terms of effective stresses have been carried out, by the use of an electronic computer, to determine the length of the toe berm required to ensure the long term stability of the overall river bank slopes.

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The stability computations including the soil properties are summarized on Figures No. 28 to No. 31 inclusive and the results are tabulated as follows.

Location	Typical Section	Length of Rock Fill Toe Berm	Factor of Safety
Stas. 262 to 272	265+00	30'	1.35
Stas. 301 to 307	305+00	50'	1.30
Stas. 360 to 367	364+00	30'	1.40
Stas. 373 to 380	377+00	30'	1.33

Note: 1. Full saturation within the clay portion of the slope was assumed in the computation of factor of safety.

2. All the berms required have been assumed to be at elevation 152 (approximately 2' above H.W.L.).

It is emphasized that the recommended berm lengths given in the above table are for the most critical sections in the four areas where remedial measures are required. It should be noted that in each area the berm length may not be constant since the existing ground contours and slope configuration could vary from station to station. In view of this, it will be necessary to obtain accurate cross sections at 25 feet intervals within the above mentioned four areas. When this information becomes available, this office will provide detailed geometry for all cross sections similar to those given for the failure took place in April, 1973 as shown in Figures No. 22 to No. 27.

As mentioned elsewhere in this report, the drainage ditch along the south shoulder of the highway has not been functioning properly due to the overgrowth of vegetation. It is therefore recommended that this ditch be modified to provide effective drainage in this area.

9. CONCLUSION:

As discussed previously in this report, the Ottawa

River bank along Hwy. 17 in certain locations, in the vicinity of the Village of Cumberland is in a state of limited equilibrium, since the computed factor of safety is generally very close to unity. Past experience indicates that the mechanism of failure in Leda Clay slopes in this area is retrogressive and is caused by erosion at the toes of the slope and also seasonal fluctuation of groundwater conditions. In several areas, evidence of active toe erosion was observed. In view of this, it may be concluded that if the toe of the slopes in these critical areas are left unprotected, more major slope failures may occur, which may result in the closing of the Hwy. 17 in this area. In addition, incorporating remedial measures after a major failure may be more expensive. In view of this, it is suggested that the toe erosion should be controlled by constructing the rock fill toe berm as discussed in Section 8 to prevent retrogressive failures of the river banks.

10. MISCELLANEOUS:

This project was carried out between August 20th and September 6th, 1973, under the immediate supervision of Mr. C. S. Poon, Project Foundations Engineer, who also prepared this report.

The drilling equipment used was owned and operated by. F. E. Johnston Drilling Company, Ottawa.

Mr. A. M. Batten, Senior Soils Supervisor, with the co-operation of Ottawa District Personnel prepared the cross sections and plans of the potential problem areas in order to assist this office in carrying out this project.

This project was under the overall supervision of Mr. M. Devata, Supervising Foundations Engineer, ~~who~~ also reviewed this report.

C. S. Poon, P. Eng.

M. Devata, P. Eng.

CSP/ji
Nov. 21, 1973.

APPENDIX I

PHOTOGRAPHS

PLATE No. 1

TOE FAILURE AT STATION 271

PLATE No. 2

TOE FAILURE AT STATION 271

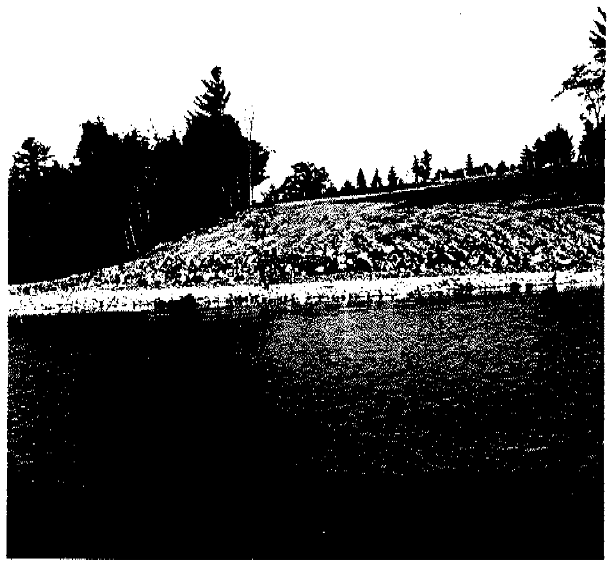
PLATE No. 3

LIMESTONE BEDROCK OUTCROP

AT STATION 320

PLATE No 4
TOE EROSION
STATION 262 TO STATION 267

PLATE No. 5
STATION 367 TO STATION 370



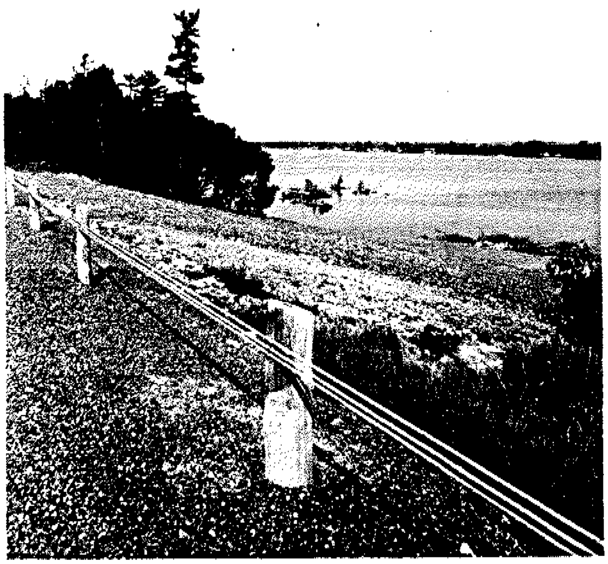


PLATE No 6
STATION 370 TO STATION 373

PLATE No. 7
TILTING OF TREES
STATION 373 TO STATION 380

PLATE No. 8
SCAR OF OLD FAILURE

PLATE No. 9
SCAR OF OLD FAILURE

PLATE No. 10 SET UP OF DRILL AT STATION 305

PLATE No. 11 TOE FAILURE AT STATION 271

PLATE No. 12 TOE EROSION STATION 301 TO 307

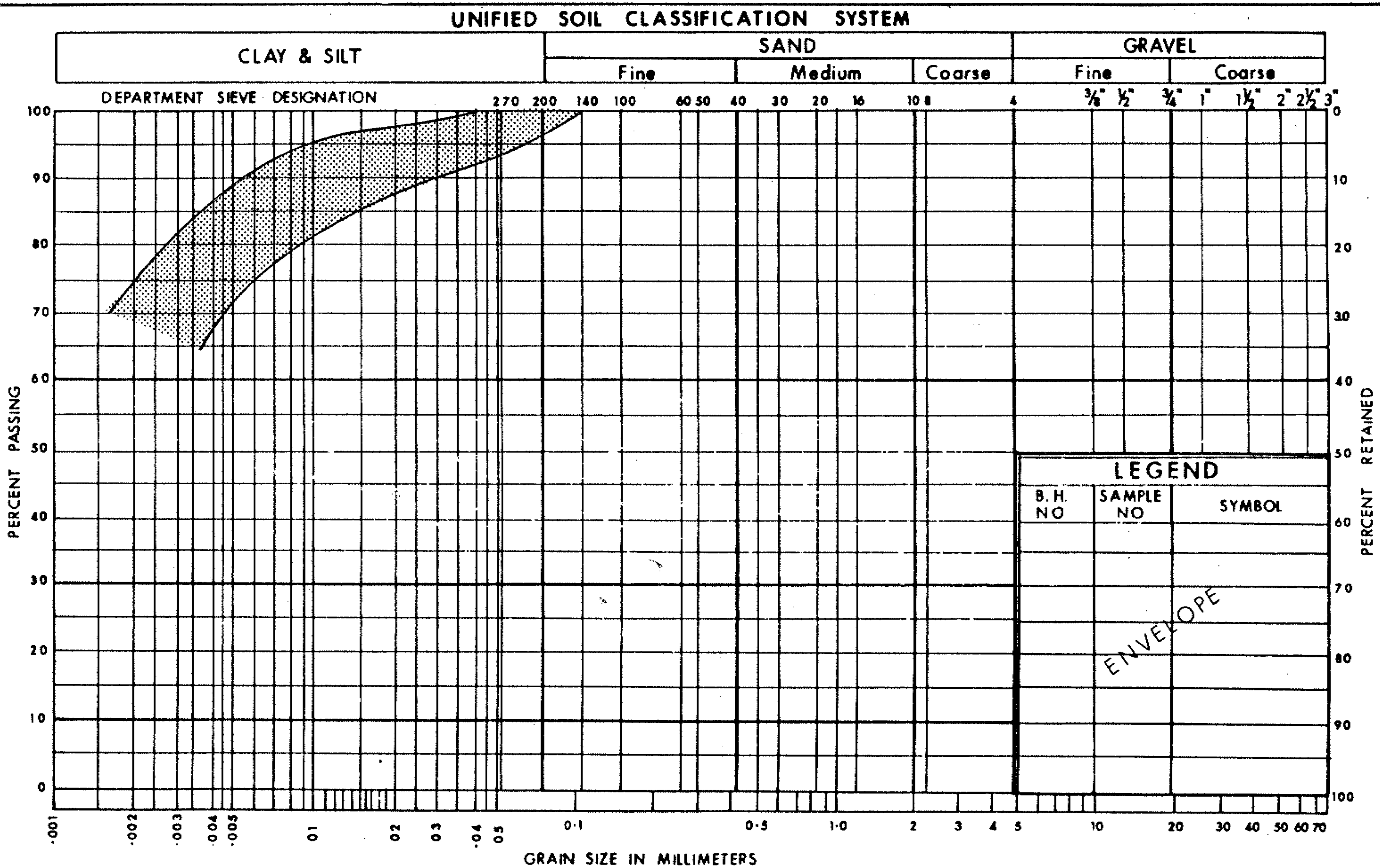
PLATE No. 13 STATION 370 TO STATION 373

PLATE No. 14 GENERAL VIEW BETWEEN STATION 370 & STATION 380

PLATE No. 15 SCAR OF OLD FAILURE

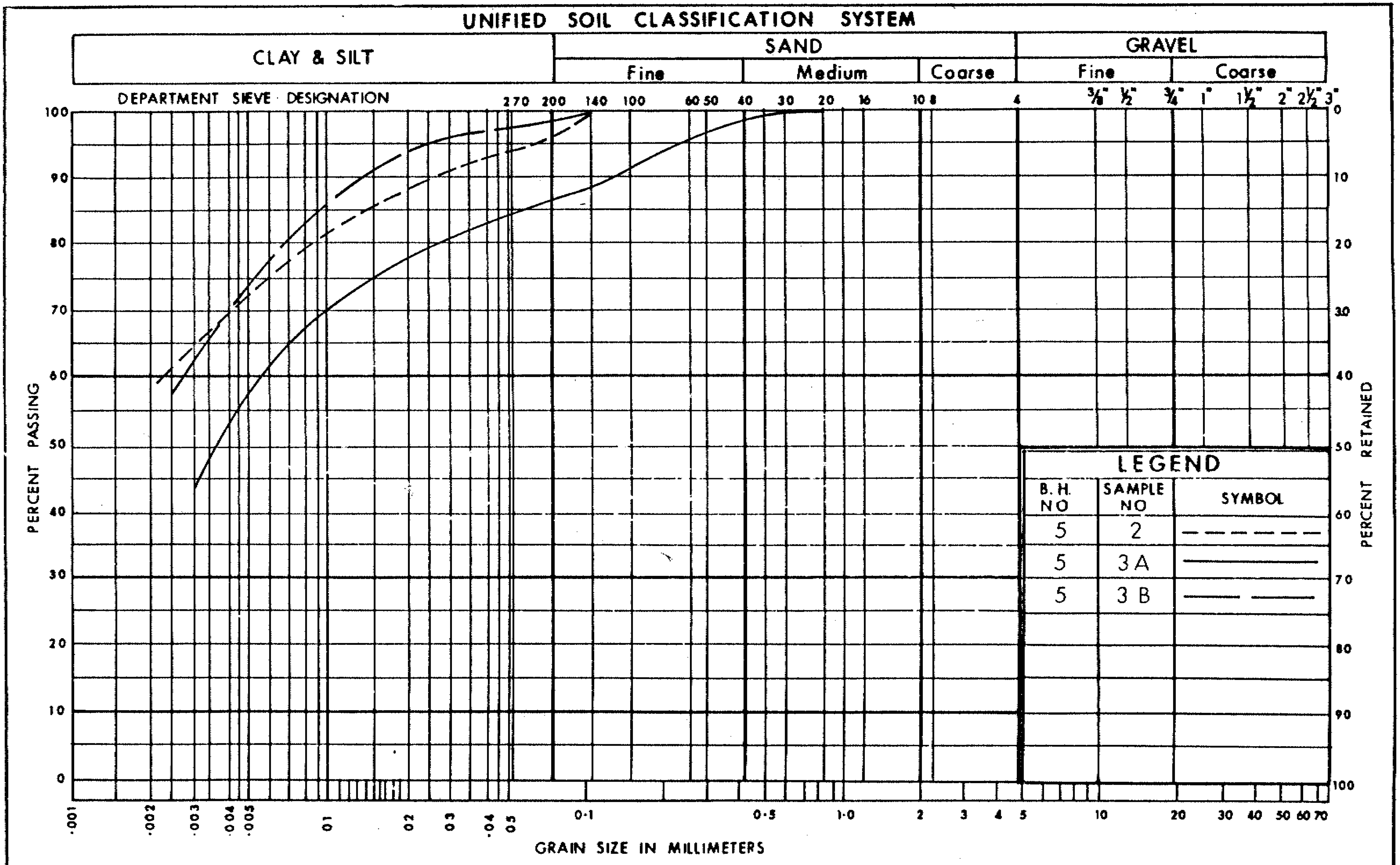
APPENDIX II

FORM OD-MT-17
REV. AUG. 1971



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS DESIGN SERVICES BRANCH ONTARIO	GRAIN SIZE DISTRIBUTION SILTY CLAY TO CLAY TRACE OF SAND	W.P. No. —
		JOB No. 73-11053 X
		FIG. 1

FORM OD-MT-1
REV. AUG. 1971



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

ONTARIO

DESIGN SERVICES BRANCH

GRAIN SIZE DISTRIBUTION

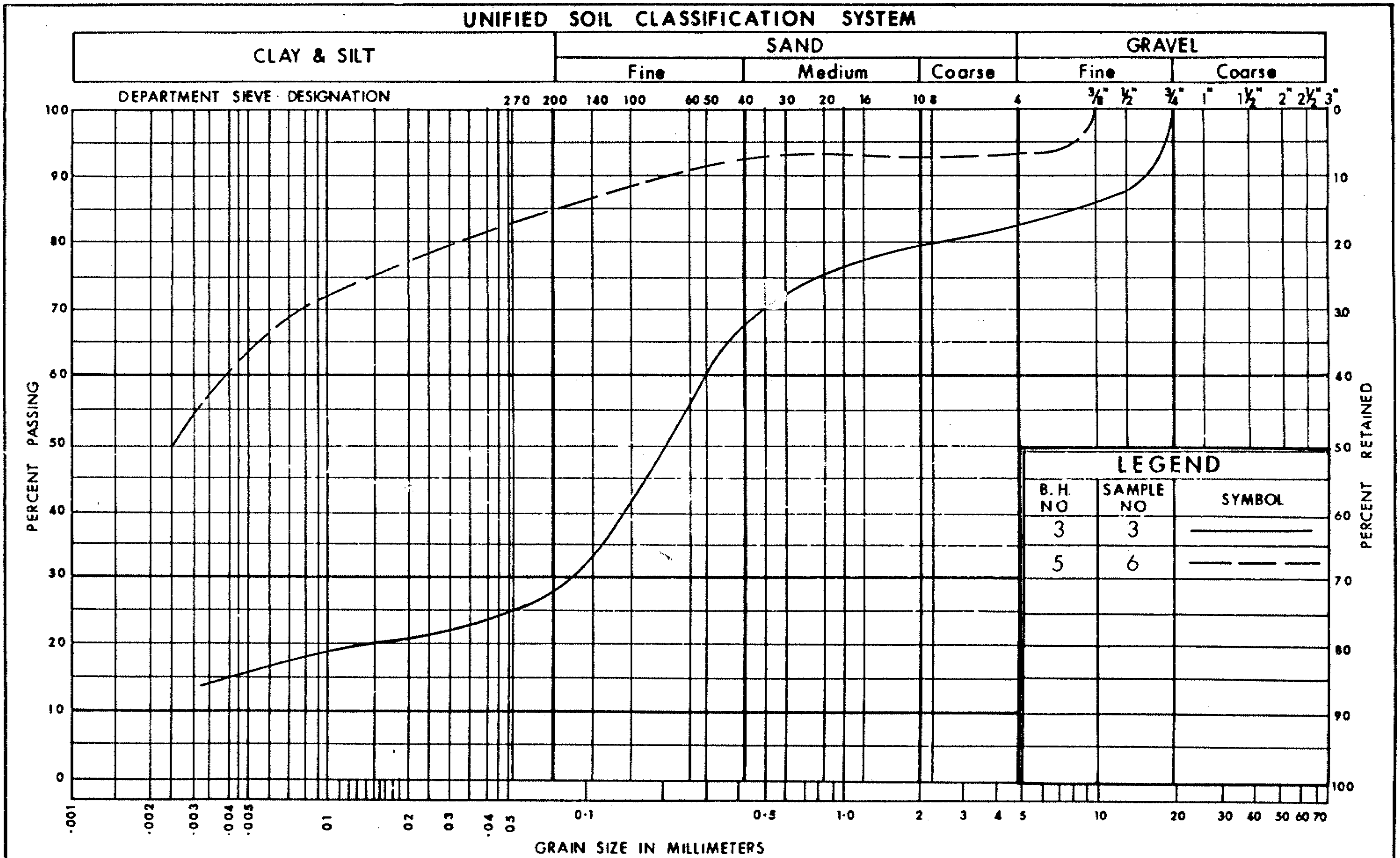
FILL MATERIAL

SILTY CLAY, SOME SAND

W.P. No.	——
JOB No.	73-11053 X

FIG. 2

FORM OD-MT-1
REV. AUG. 1971



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

ONTARIO

DESIGN SERVICES BRANCH

GRAIN SIZE DISTRIBUTION

FILL MATERIAL

SAND, SOME GRAVEL & SILT

W.P. No.	—
JOB No.	73 - 11053 X

FIG. 3

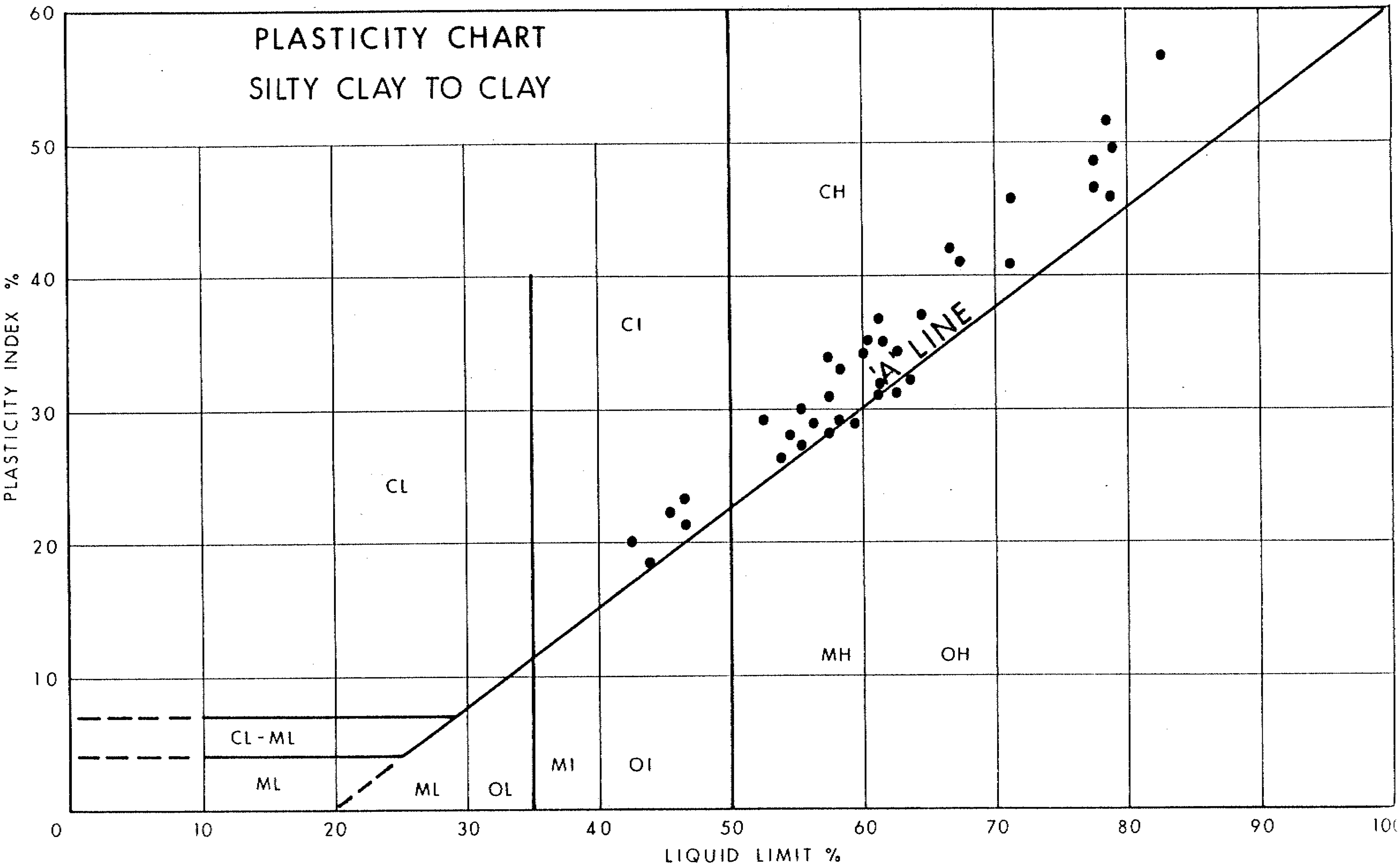


FIG. 4

W.O. 73-11053X

VOID RATIO—PRESSURE CURVES

JOB NO. 73 - 11053 X

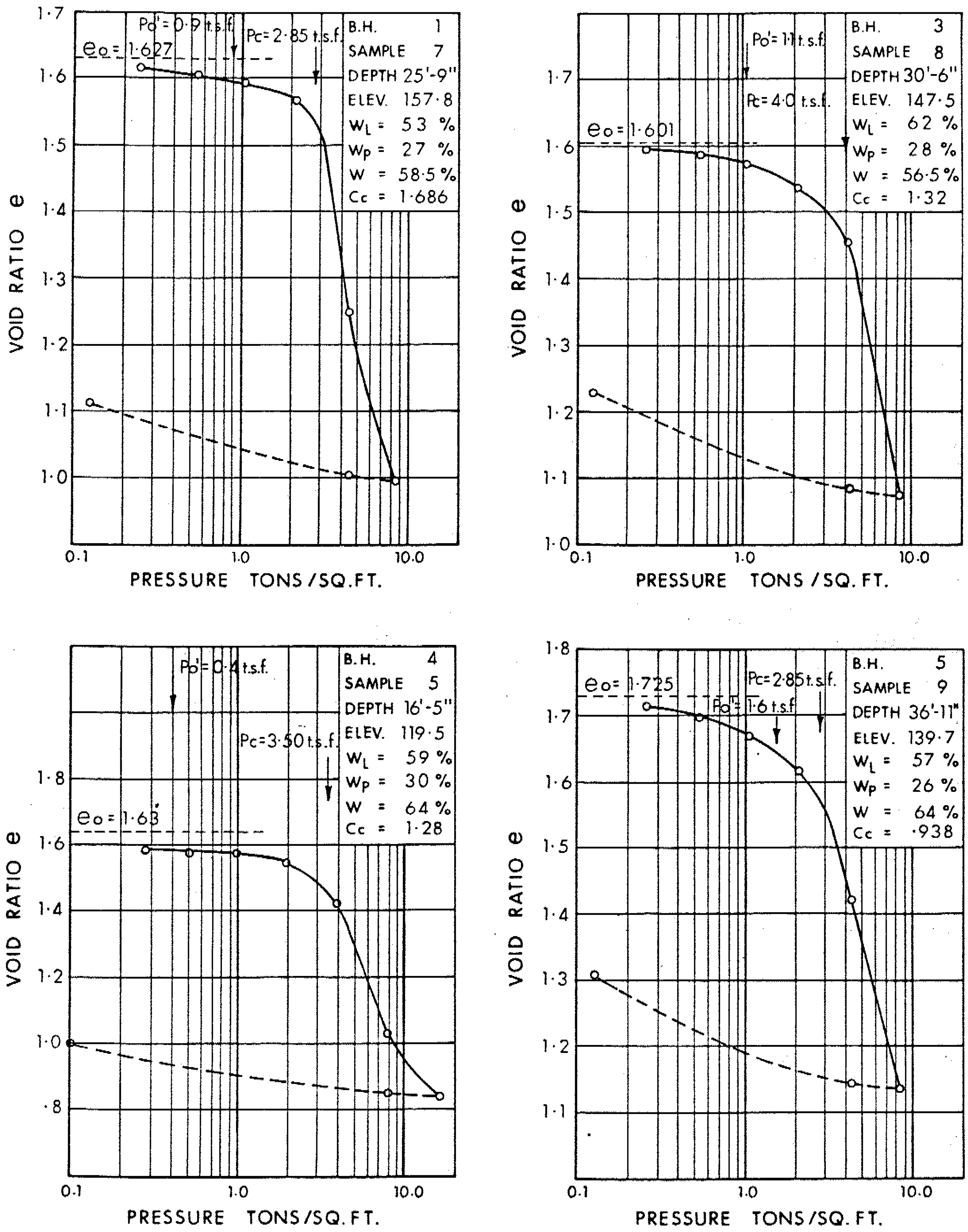
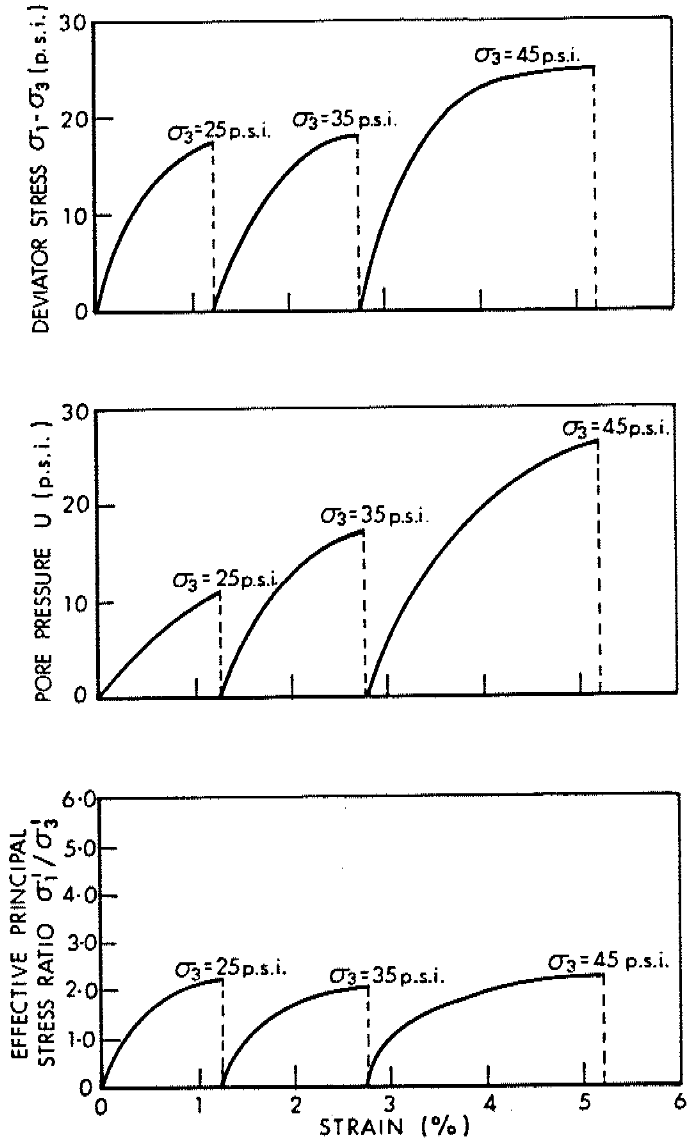
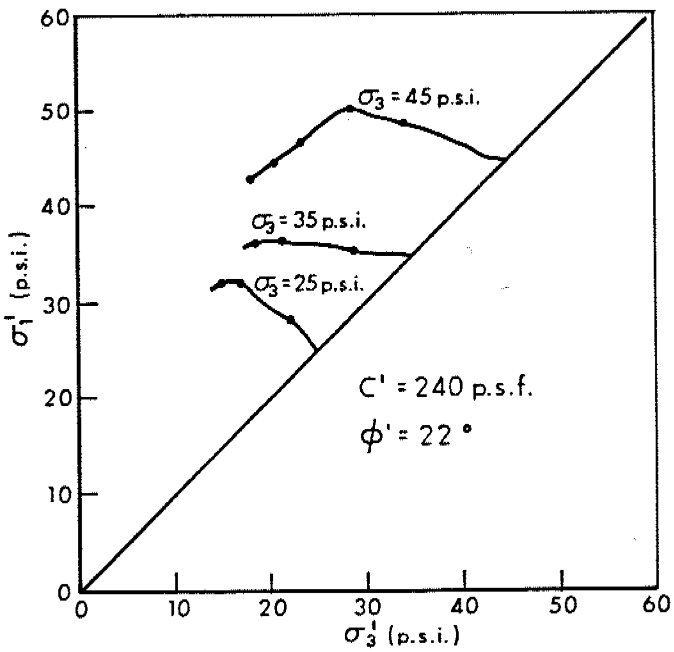


FIG. 5





BORE HOLE 1 SAMPLE 6

σ_3 CONSTANT

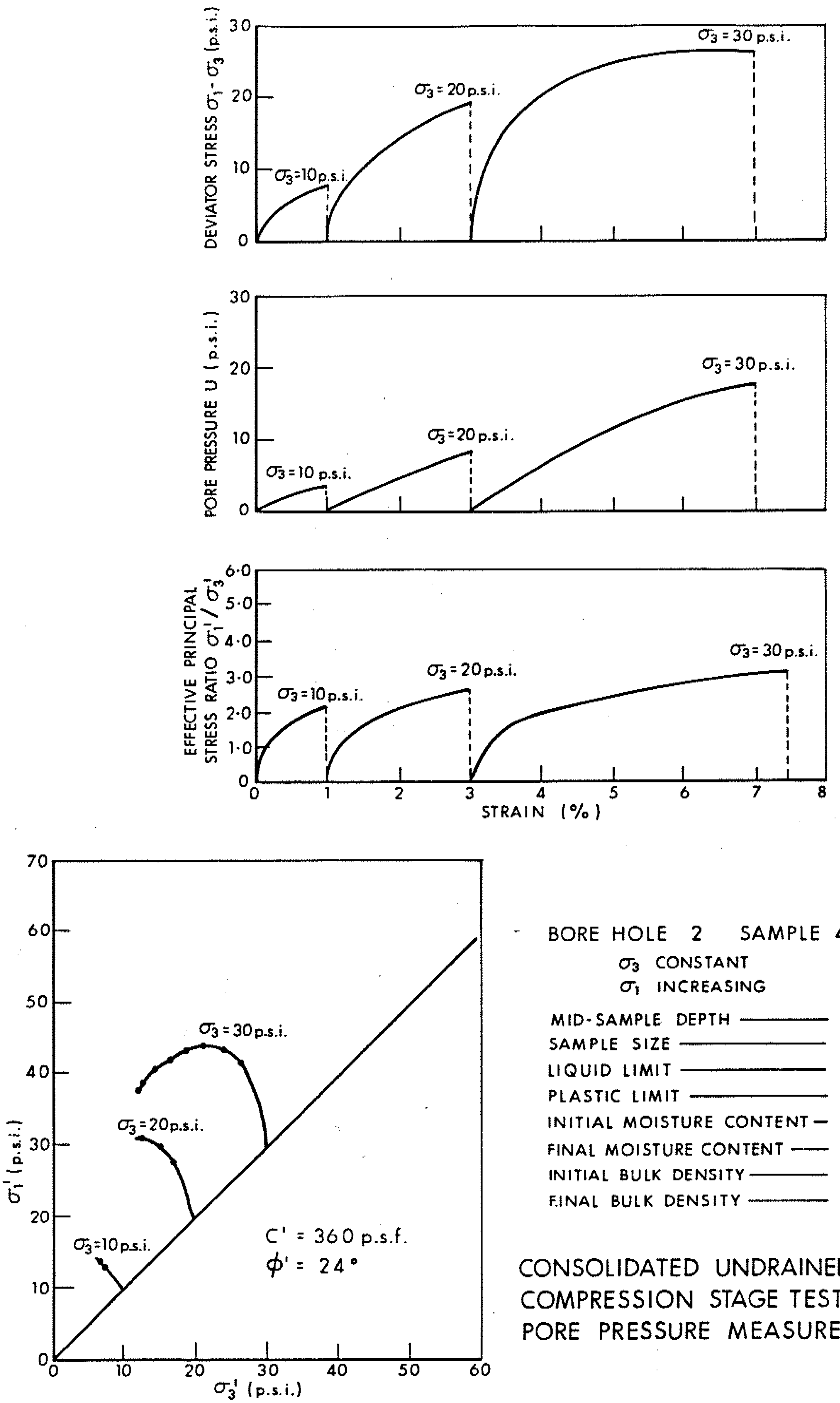
σ_1 INCREASING

MID-SAMPLE DEPTH	20' - 7"
SAMPLE SIZE	2" × 4"
LIQUID LIMIT	53 %
PLASTIC LIMIT	27 %
INITIAL MOISTURE CONTENT	59·4 %
FINAL MOISTURE CONTENT	44·8 %
INITIAL BULK DENSITY	103 p.c.f.
FINAL BULK DENSITY	111 p.c.f.

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS

FIG. 8

W.O. 73-11053X



BORE HOLE 2 SAMPLE 4

σ_3 CONSTANT

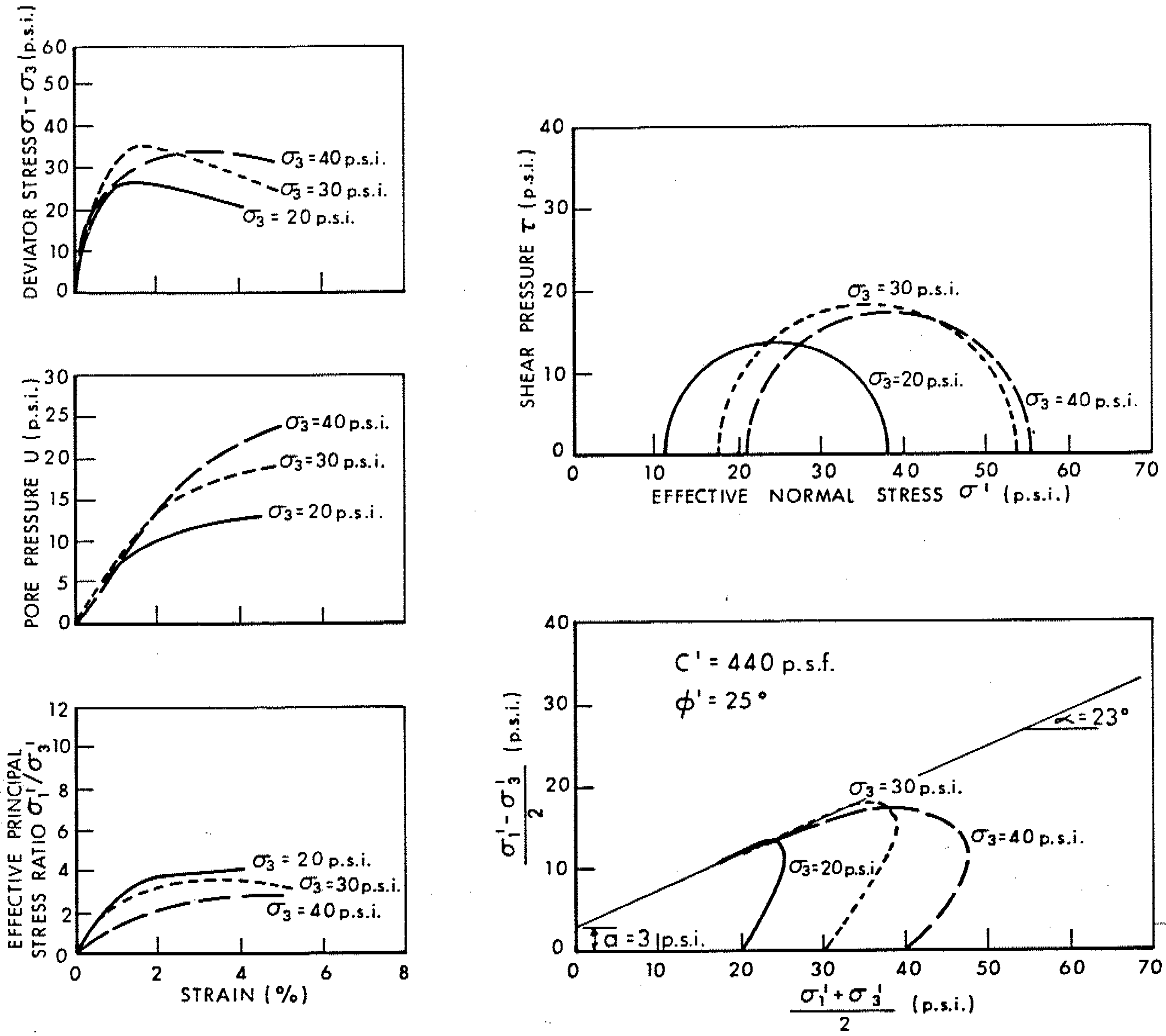
σ_1 INCREASING

MID-SAMPLE DEPTH	11' - 9"
SAMPLE SIZE	2" × 4"
LIQUID LIMIT	53 %
PLASTIC LIMIT	29 %
INITIAL MOISTURE CONTENT	62 %
FINAL MOISTURE CONTENT	54 %
INITIAL BULK DENSITY	101 p.c.f.
FINAL BULK DENSITY	105 p.c.f.

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS

FIG. 9

W.O. 73-11053 X



σ_3 CONFINING PRESSURE (p.s.i.)		20	30	40
MID-SAMPLE DEPTH		25'-9"	25'-5"	25'-0·5"
MOISTURE CONTENT %	Initial	53·1	57·6	61·8
	Final	51	54·4	54
BULK DENSITY (p.c.f.)	Initial	105	104	103
	Final	107	106	107
LIQUID LIMIT		62 %		
PLASTIC LIMIT		28 %		

BORE HOLE 3 SAMPLE 7

σ_3 CONSTANT

σ_1 INCREASING

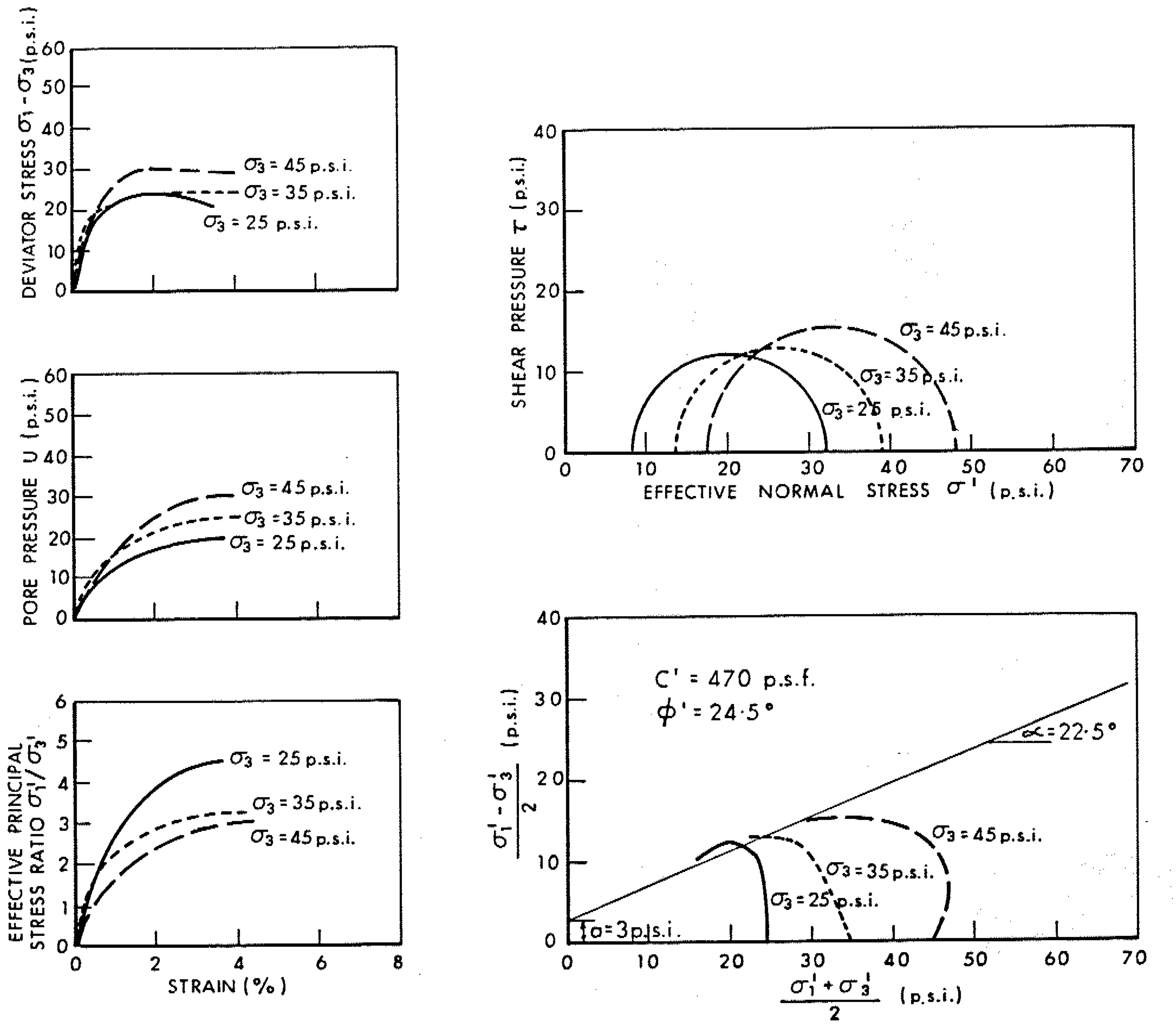
RATE OF STRAIN 0·0005 in./mi

SAMPLE SIZE 1½" × 3"

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS WITH PORE PRESSURE MEASUREMENTS

FIG. 10

W.O. 73-11053X



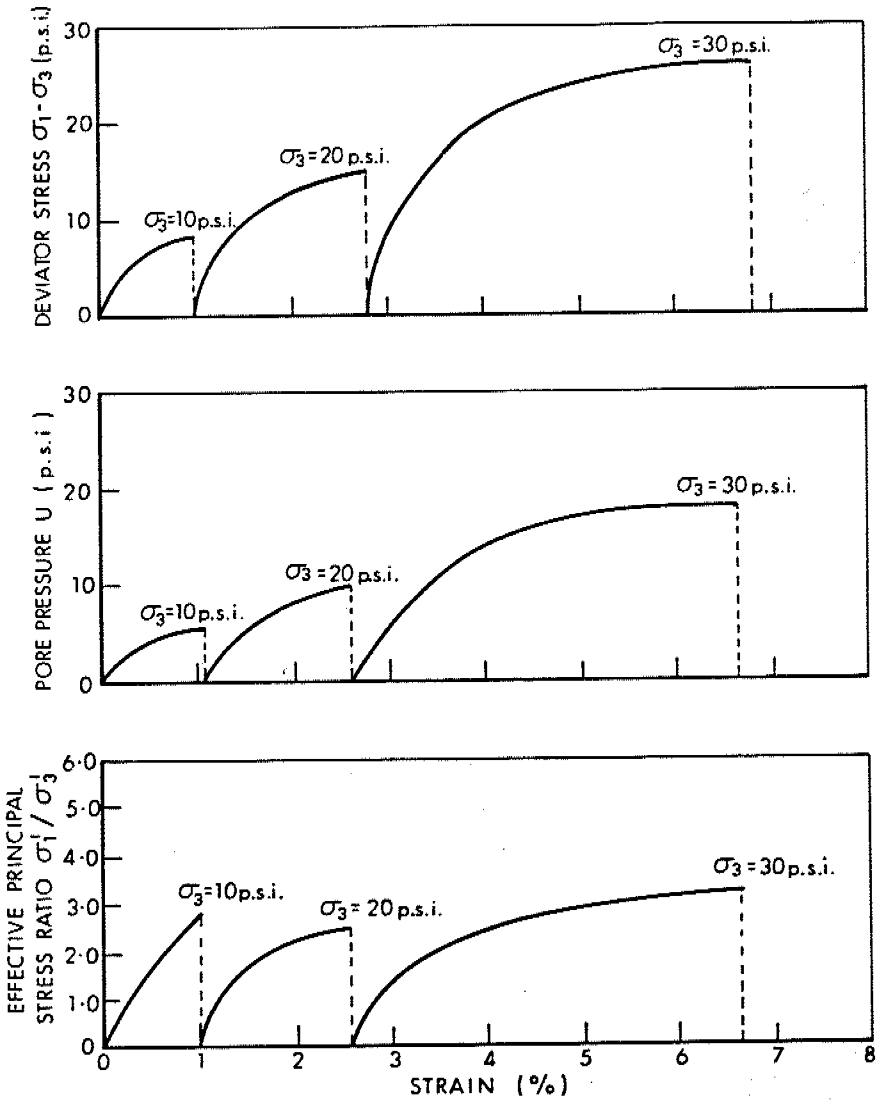
σ_3 CONFINING PRESSURE (p.s.i.)		25	35	45
MID-SAMPLE DEPTH		31'-2½"	30'-10"	30'-5"
MOISTURE CONTENT %	Initial	64·2	63·6	61·7
	Final	60·4	54·4	44·5
BULK DENSITY (p.c.f.)	Initial	101	102	102
	Final	103	106	111
LIQUID LIMIT		57 %		
PLASTIC LIMIT		26 %		

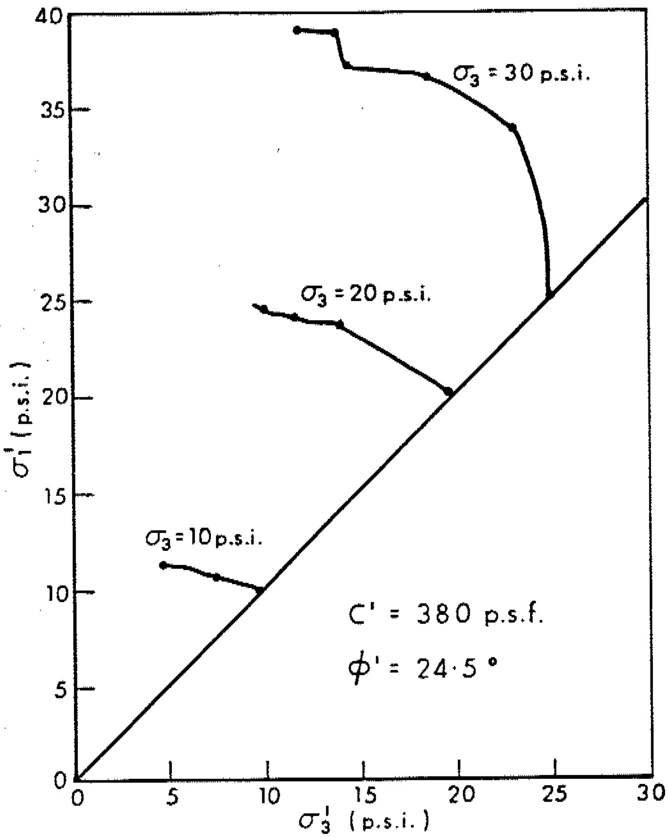
BORE HOLE 5 SAMPLE 8
σ_3 CONSTANT
σ_1 INCREASING
RATE OF STRAIN 0·0005in./mi
SAMPLE SIZE 1½" × 3"

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS WITH PORE PRESSURE MEASUREMENTS

FIG. 11

W.O. 73-11053X





BORE HOLE 6 SAMPLE 3A

σ_3 CONSTANT

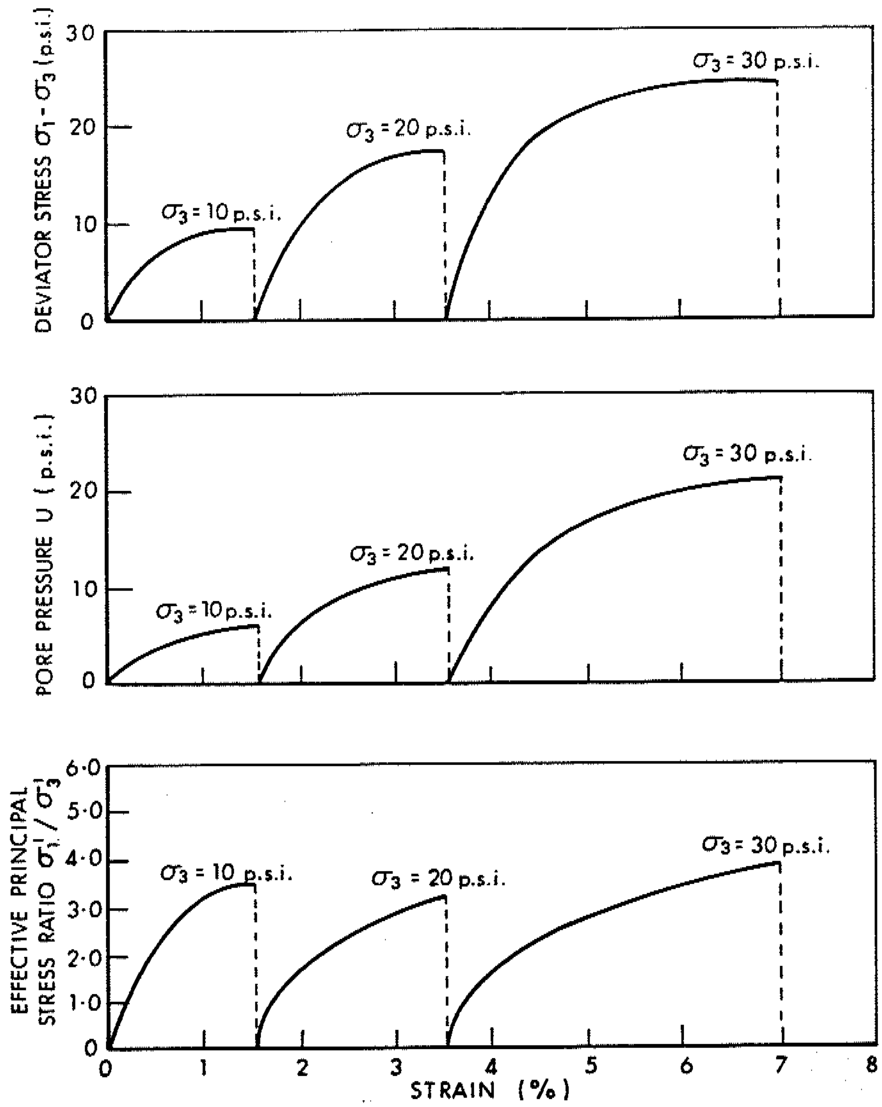
σ_1 INCREASING

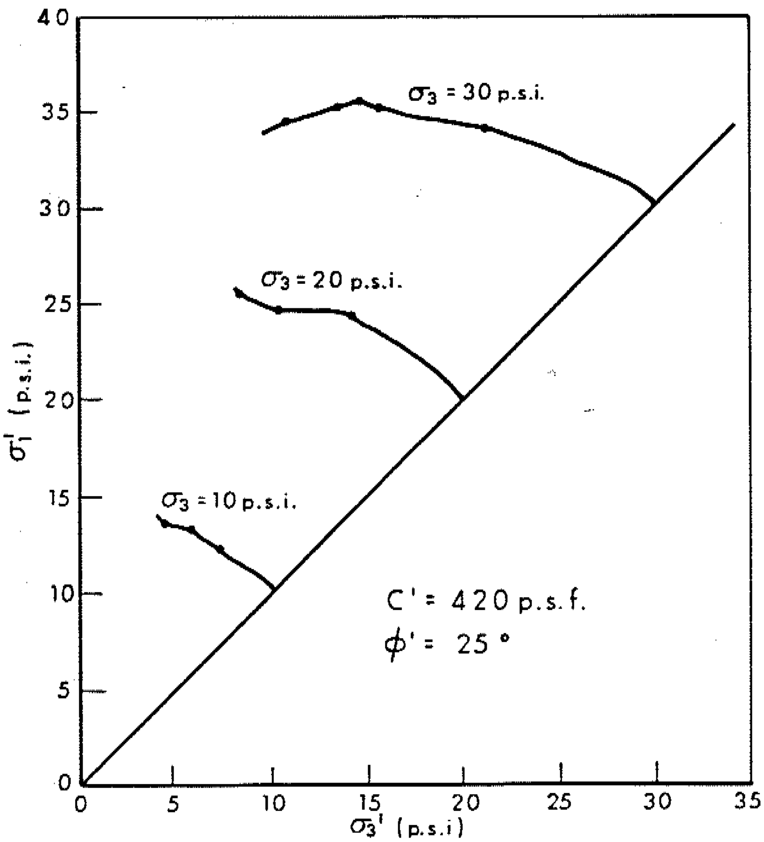
MID-SAMPLE DEPTH	5'-6"
SAMPLE SIZE	2" × 4"
LIQUID LIMIT	61 %
PLASTIC LIMIT	24 %
INITIAL MOISTURE CONTENT	46·4 %
FINAL MOISTURE CONTENT	39·0 %
INITIAL BULK DENSITY	109 p.c.f.
FINAL BULK DENSITY	114 p.c.f.

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS

FIG. 12

W.O. 73-11053 X





BORE HOLE 6   SAMPLE 3 B

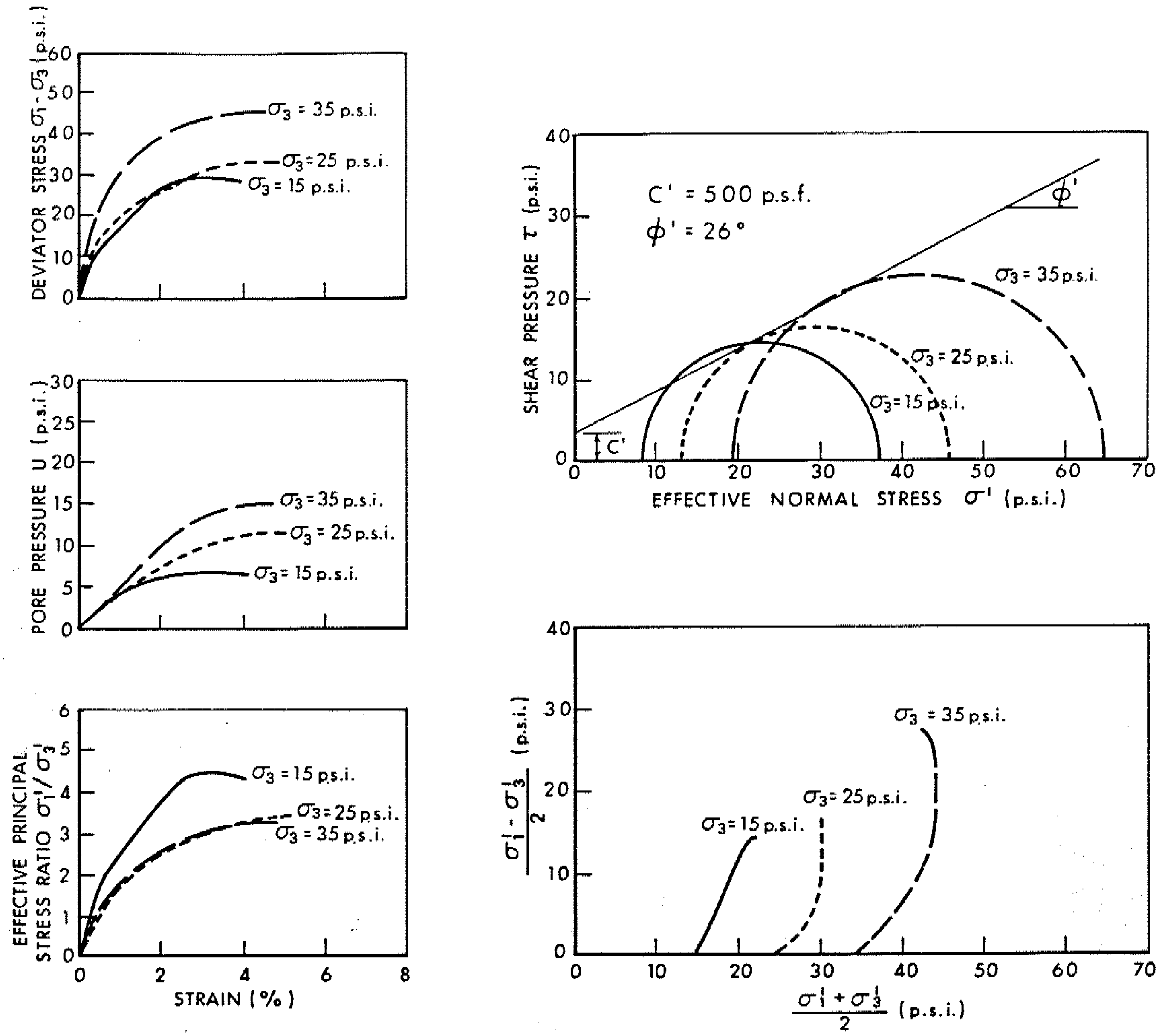
σ_3 CONSTANT
σ_1 INCREASING

MID-SAMPLE DEPTH	6'-9"
SAMPLE SIZE	2"×4"
LIQUID LIMIT	61 %
PLASTIC LIMIT	26 %
INITIAL MOISTURE CONTENT	61·5 %
FINAL MOISTURE CONTENT	52·5%
INITIAL BULK DENSITY	102 p.c.f.
FINAL BULK DENSITY	106 p.c.f.

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION STAGE TESTS WITH PORE PRESSURE MEASUREMENTS

(STAGE TEST)

FIG. 13

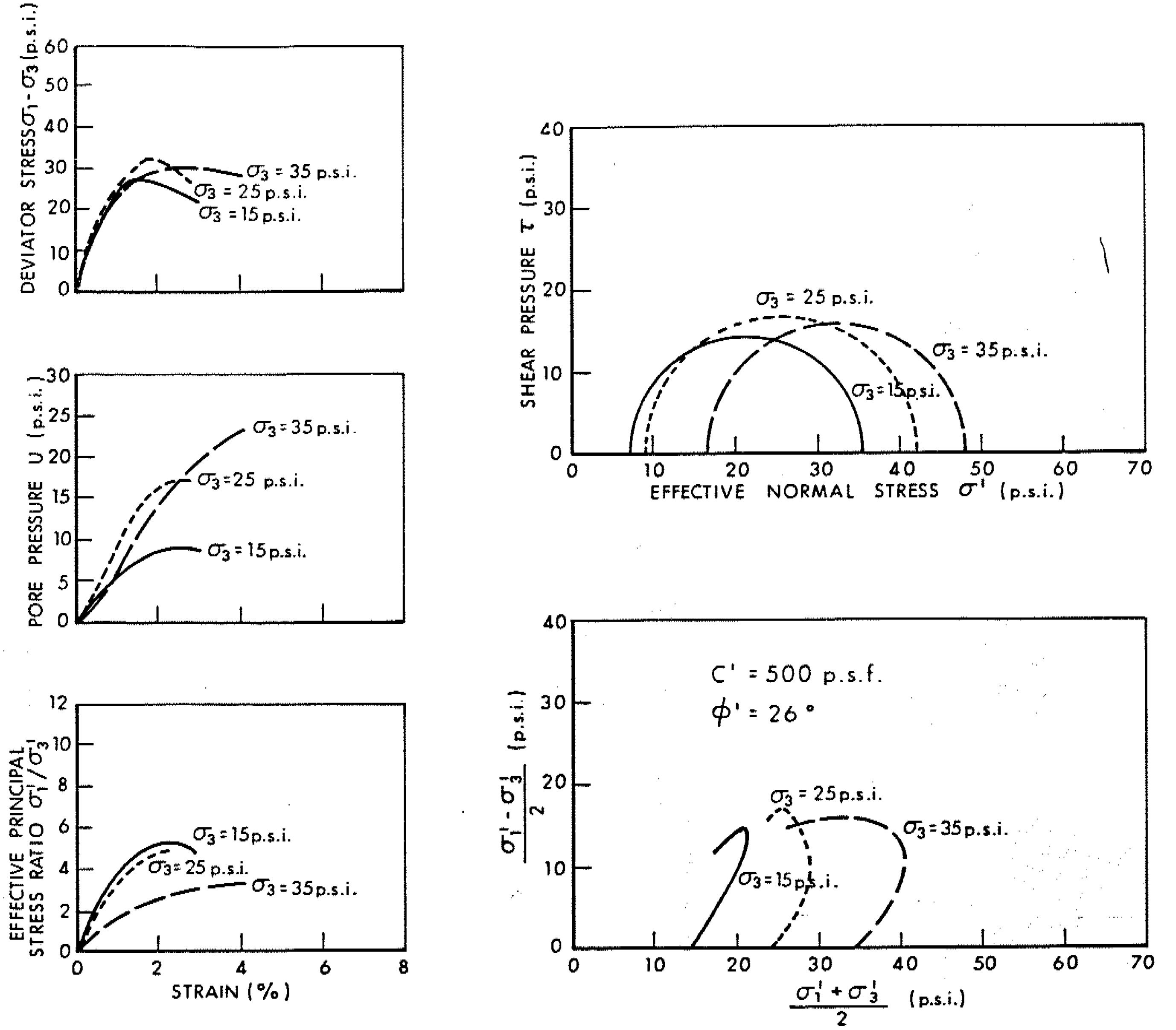


σ_3 CONFINING PRESSURE (p.s.i.)		15	25	35
MID - SAMPLE DEPTH		15'-10"	15'-6"	15'-2"
MOISTURE CONTENT %	Initial	48·5	52	46·5
	Final	47·5	49	43·5
BULK DENSITY (p.c.f.)	Initial	108	107	108·5
	Final	108	108	110
LIQUID LIMIT		62 %		
PLASTIC LIMIT		31 %		

BORE HOLE 7 SAMPLE 4
σ_3 CONSTANT
σ_1 INCREASING
RATE OF STRAIN 0·0005 in/mi
SAMPLE SIZE 1½" × 3"

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS WITH PORE PRESSURE MEASUREMENTS

FIG. 14



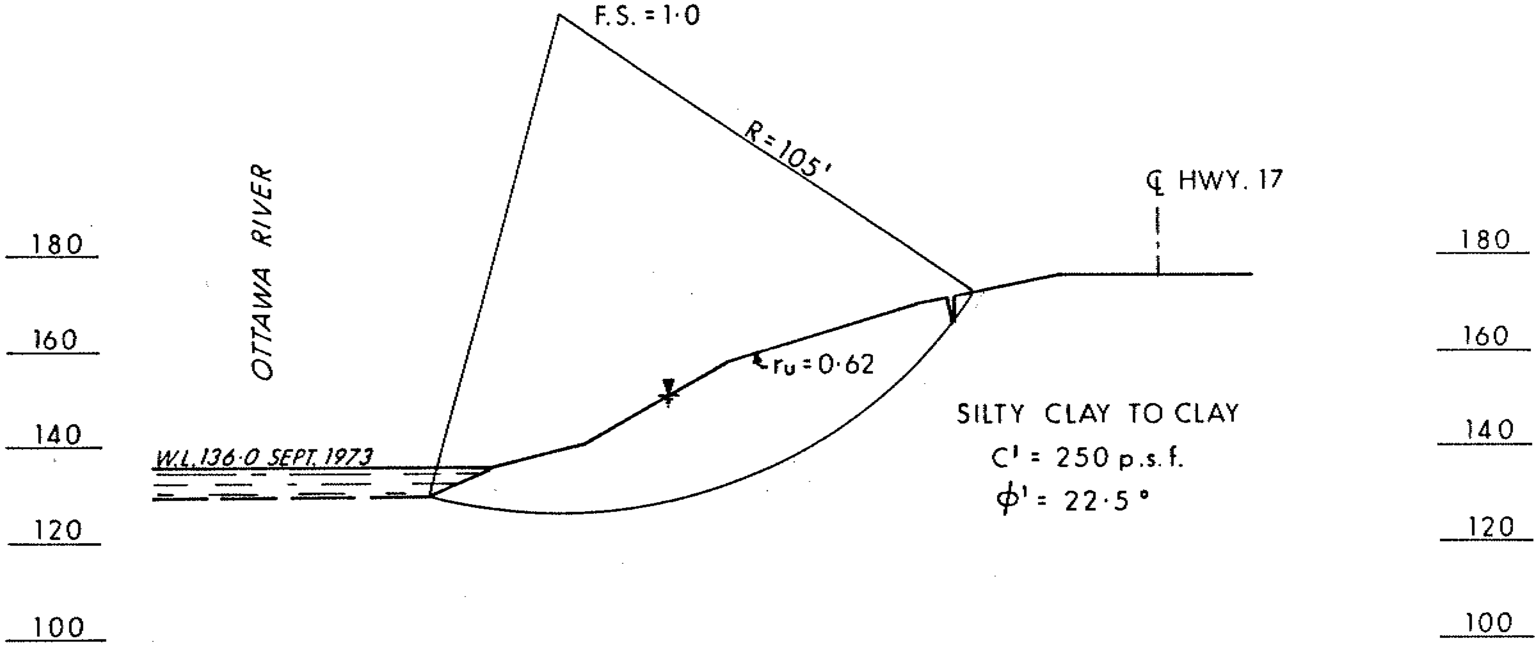
		15	25	35
σ_3 CONFINING PRESSURE (p.s.i.)		15	25	35
MID-SAMPLE DEPTH		21'-2½"	20'-10"	20'-4½"
MOISTURE CONTENT %	Initial	61·4	62·8	66·7
	Final	60·2	60·5	62
BULK DENSITY (p.c.f.)	Initial	102	100	100
	Final	102	101·5	102·5
LIQUID LIMIT		79 %		
PLASTIC LIMIT		27 %		

BORE HOLE 8 SAMPLE 5

σ_3 CONSTANT
σ_1 INCREASING
RATE OF STRAIN 0·0005 in./m
SAMPLE SIZE 1½" × 3"

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS WITH PORE PRESSURE MEASUREMENTS

FIG. 15



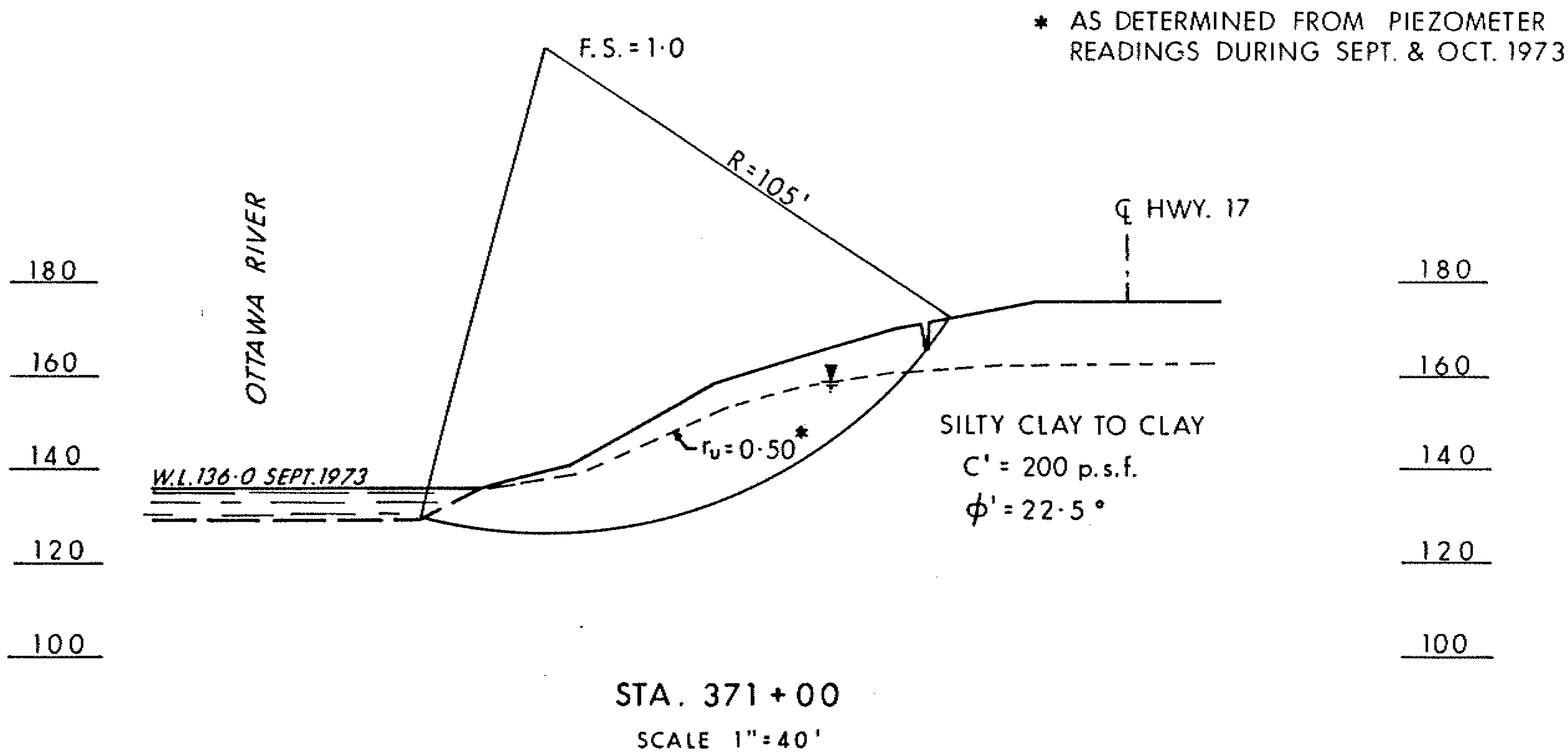
STA. 371 + 00

SCALE 1" = 40'

STABILITY ANALYSIS IN TERMS OF EFFECTIVE STRESS
(NATURAL SLOPE)

FIG. 16

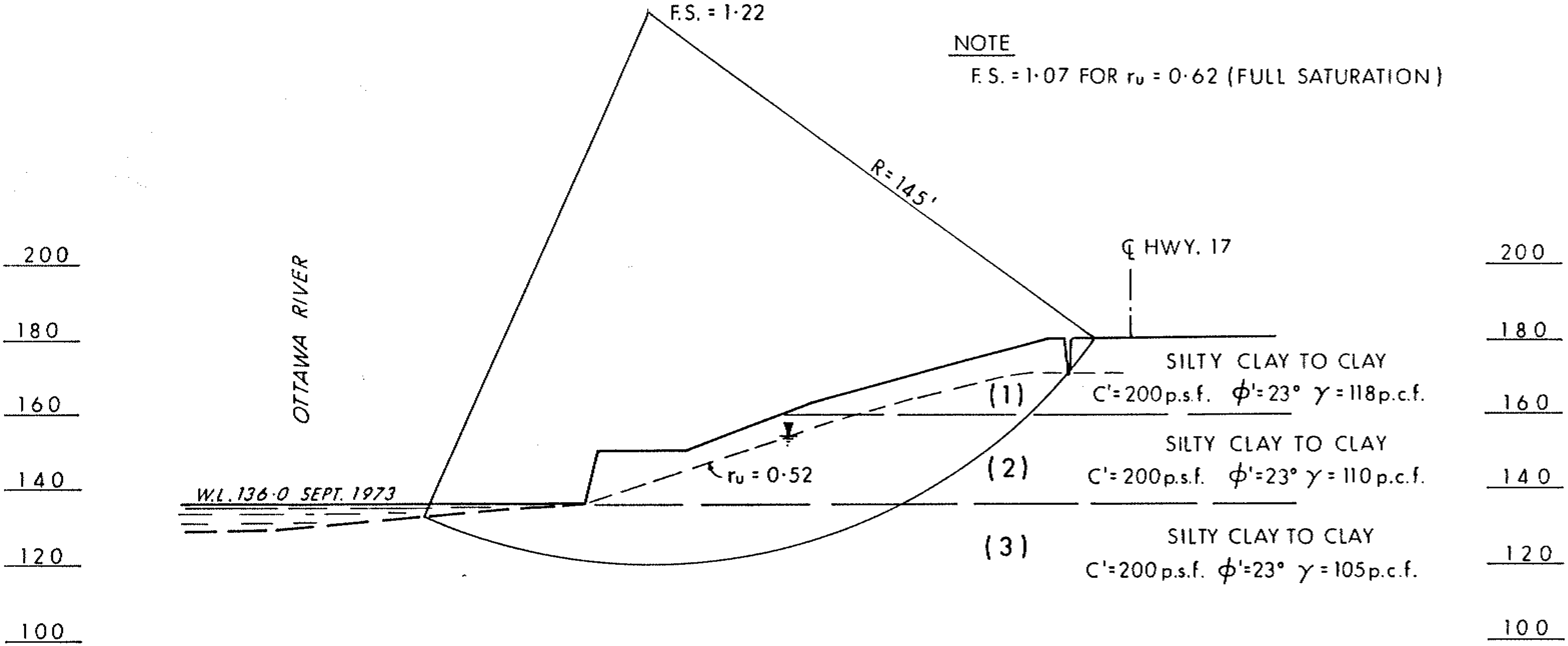
W.O. 73-11053 X



STABILITY ANALYSIS IN TERMS OF EFFECTIVE STRESS
(NATURAL SLOPE)

FIG. 17

W.O. 73 - 11053 X



STA. 265+00

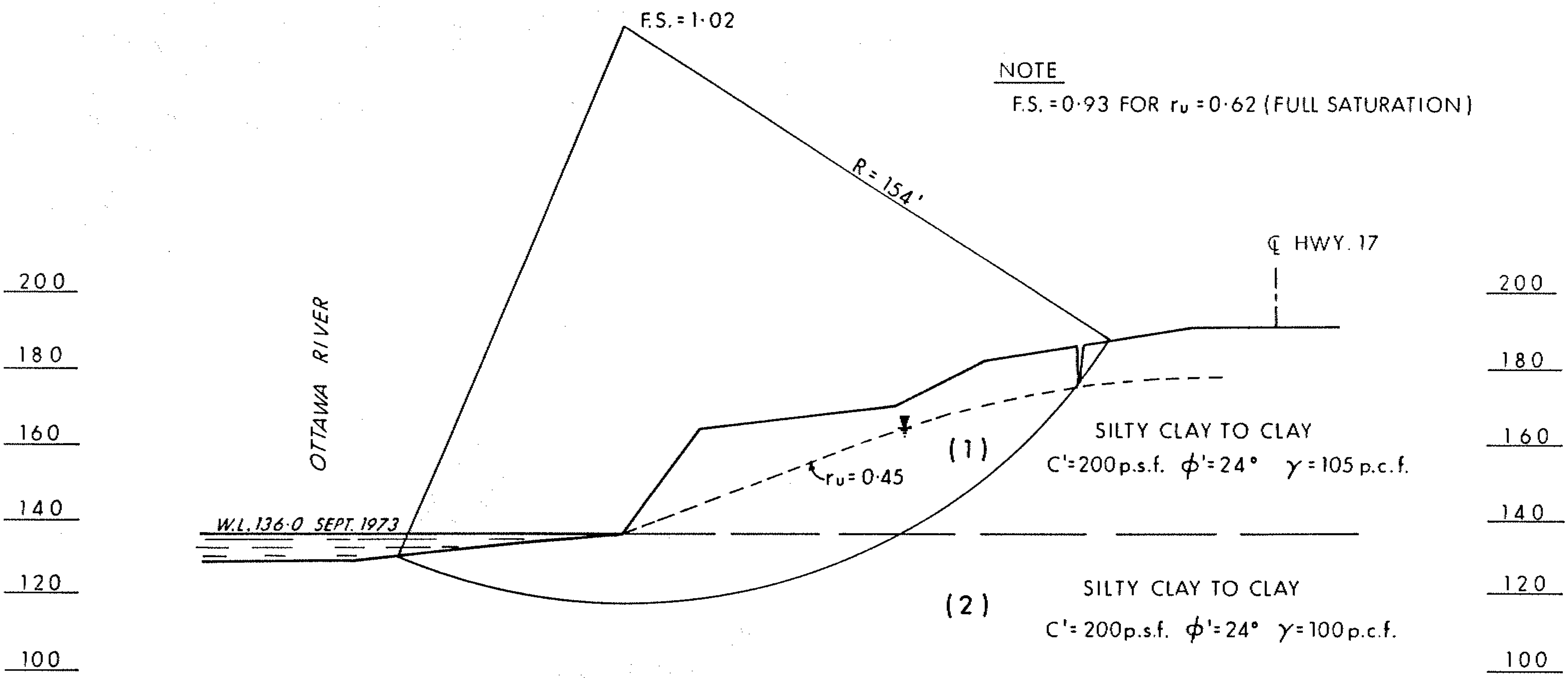
SCALE 1" = 40'

STABILITY ANALYSIS IN TERMS OF EFFECTIVE STRESS

(NATURAL SLOPE)

FIG. 18

W.O. 73-11053X

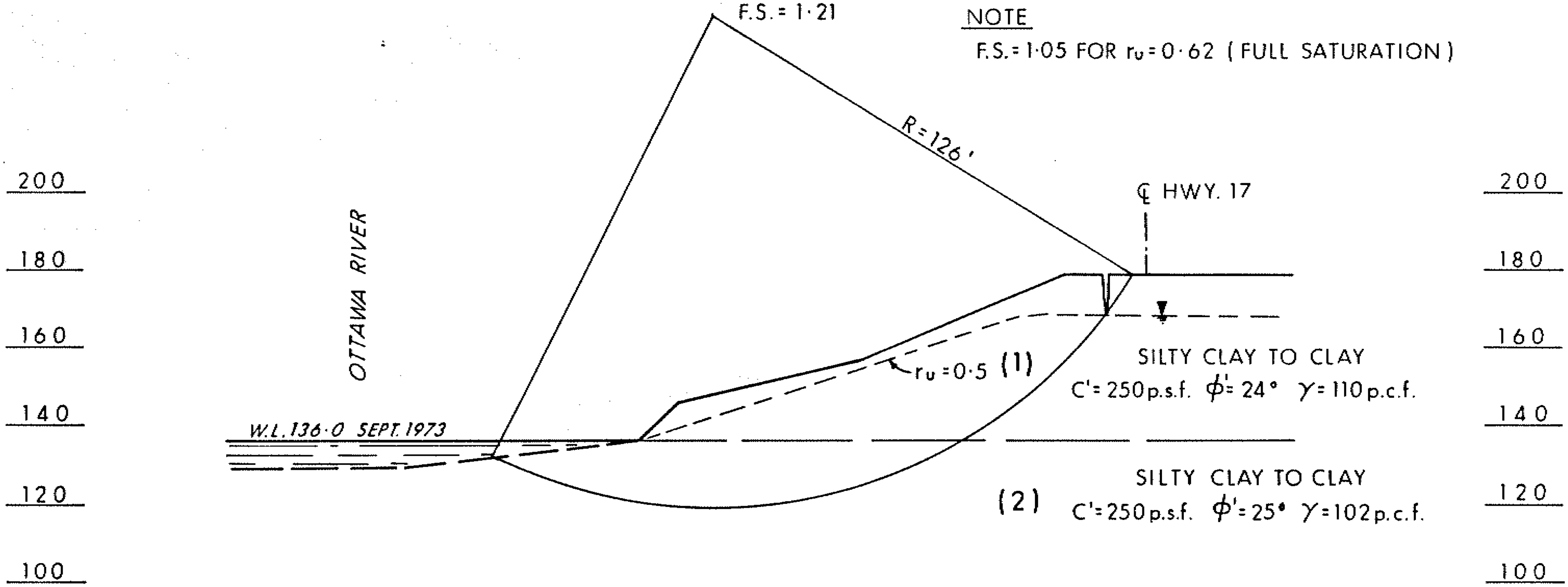


STA. 305+00
SCALE 1" = 40'

STABILITY ANALYSIS IN TERMS OF EFFECTIVE STRESS
(NATURAL SLOPE)

FIG. 19

W.O. 73-11053X

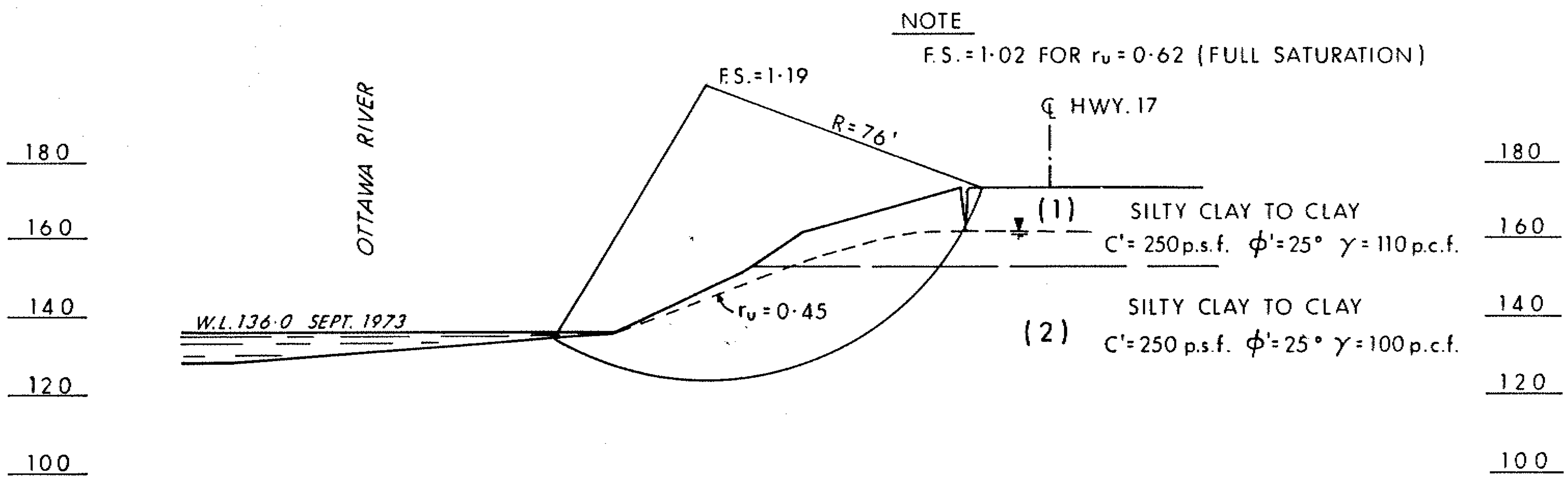


STA. 364 + 00
SCALE 1" = 40'

STABILITY ANALYSIS IN TERMS OF EFFECTIVE STRESS
(NATURAL SLOPE)

FIG. 20

W.O. 73-11053X



STA. 377+00

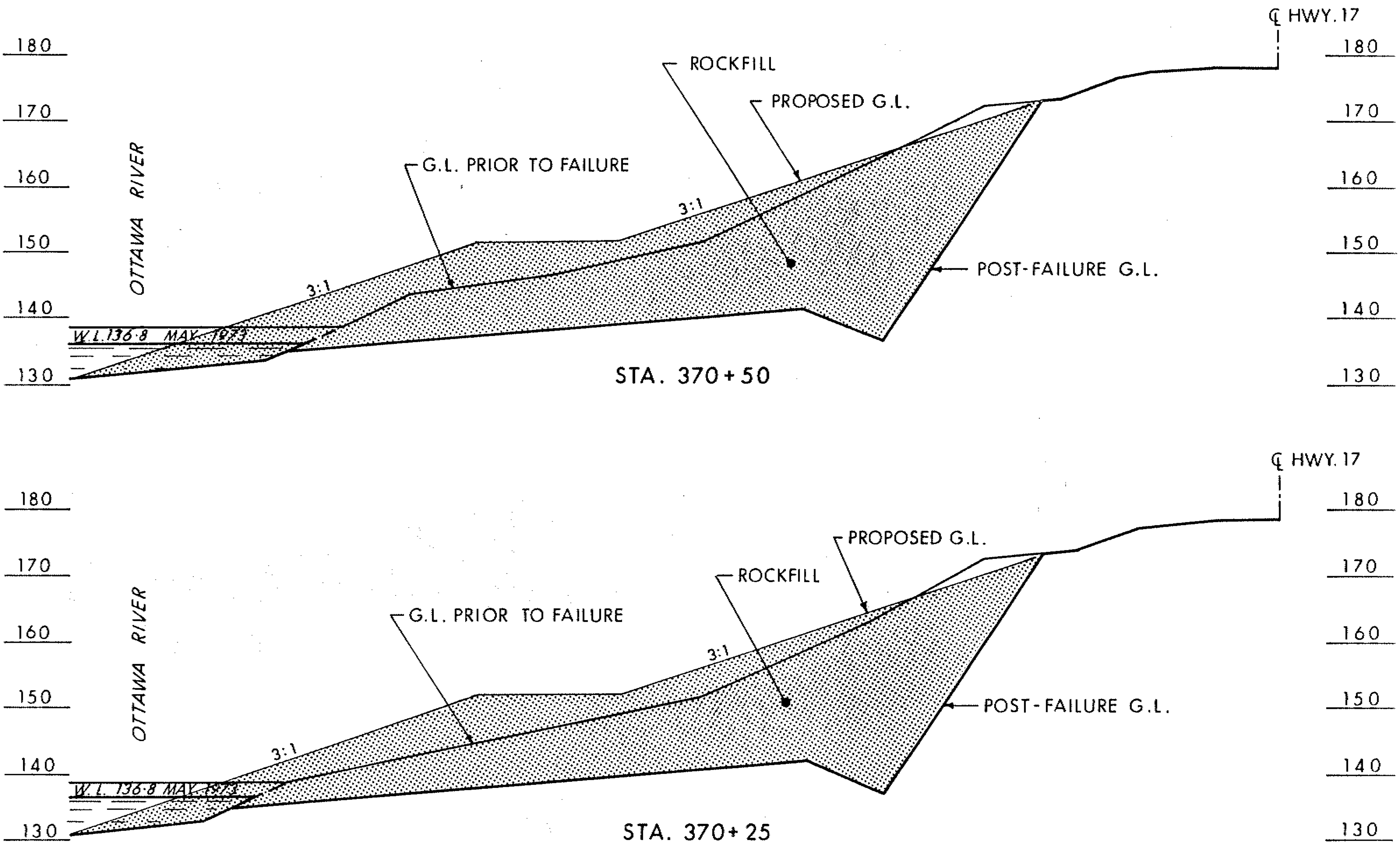
SCALE 1" = 40'

STABILITY ANALYSIS IN TERMS OF EFFECTIVE STRESS

(NATURAL SLOPE)

FIG. 21

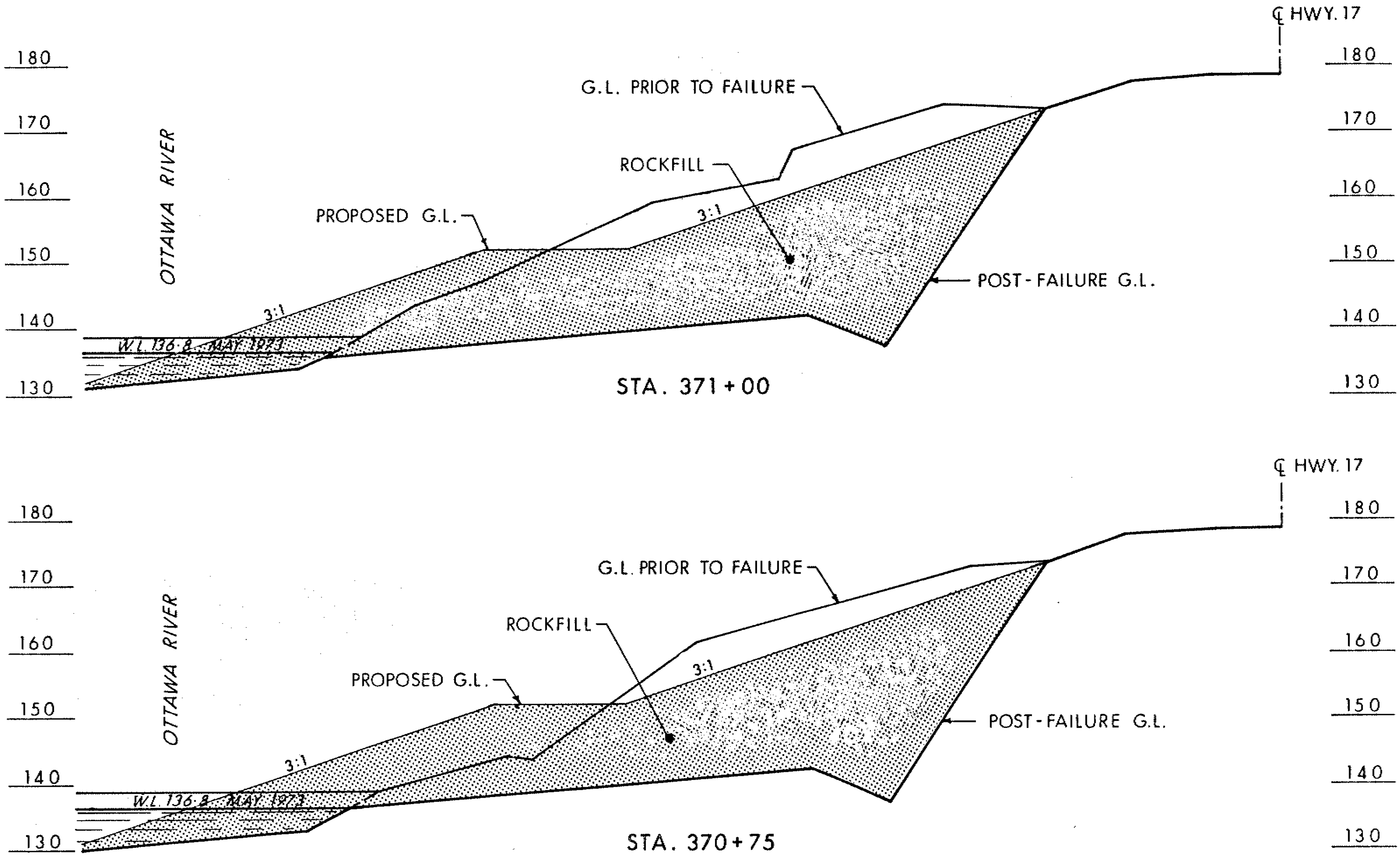
W.O. 73-11053X



RECOMMENDED REMEDIAL MEASURES

FIG. 22

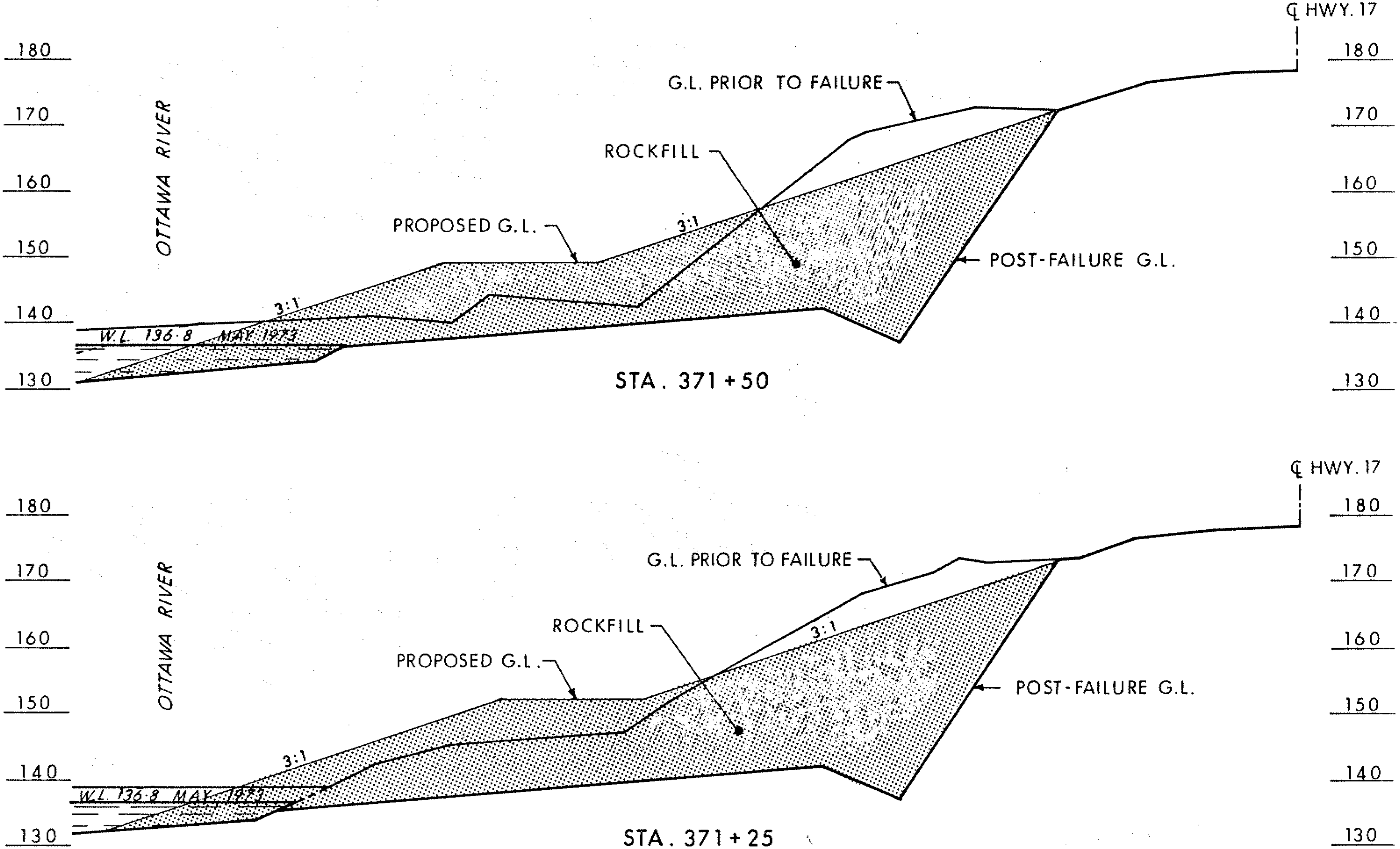
W.O. 73-11053X



RECOMMENDED REMEDIAL MEASURES

FIG. 23

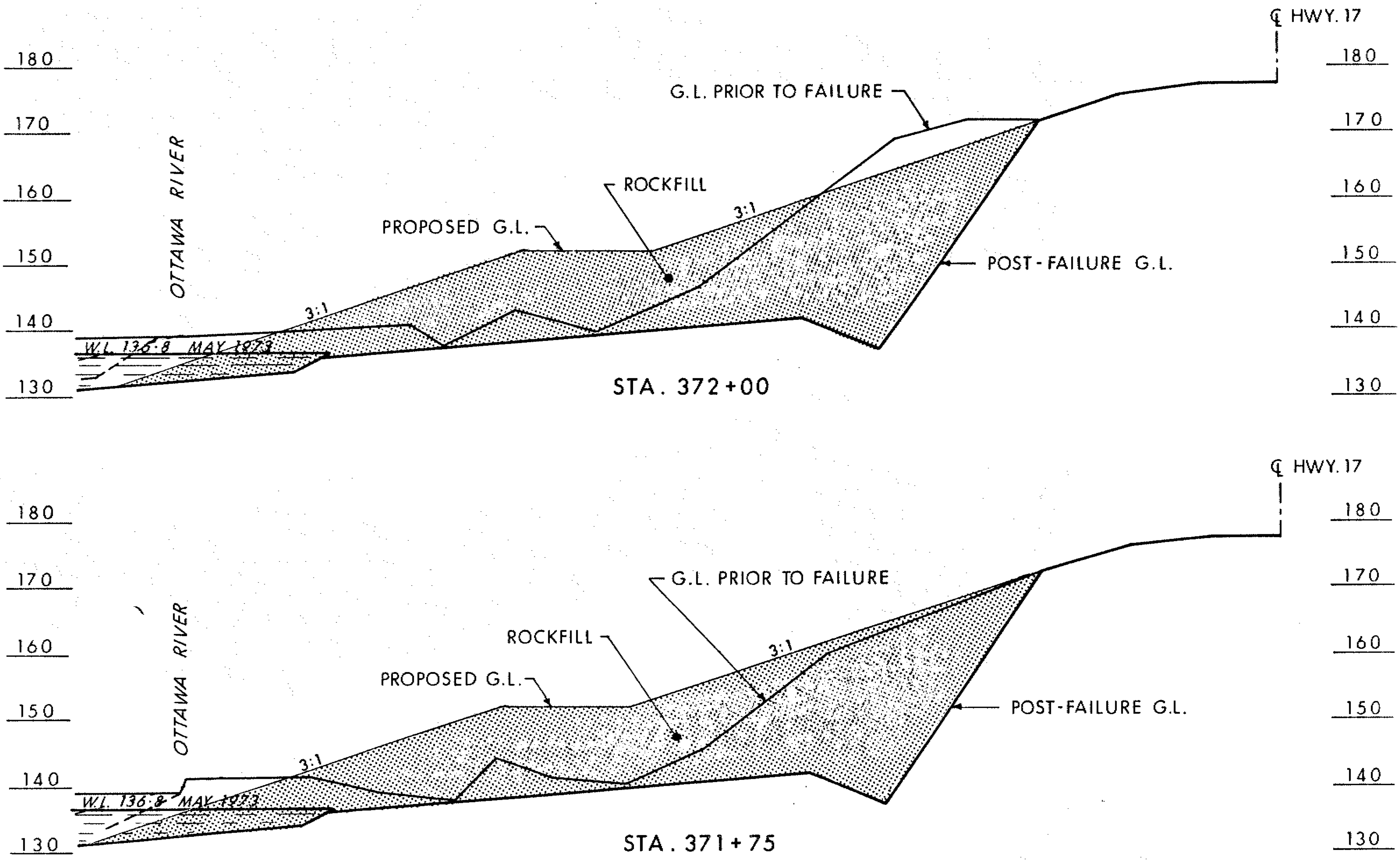
W.O. 73-11053X



RECOMMENDED REMEDIAL MEASURES

FIG. 24

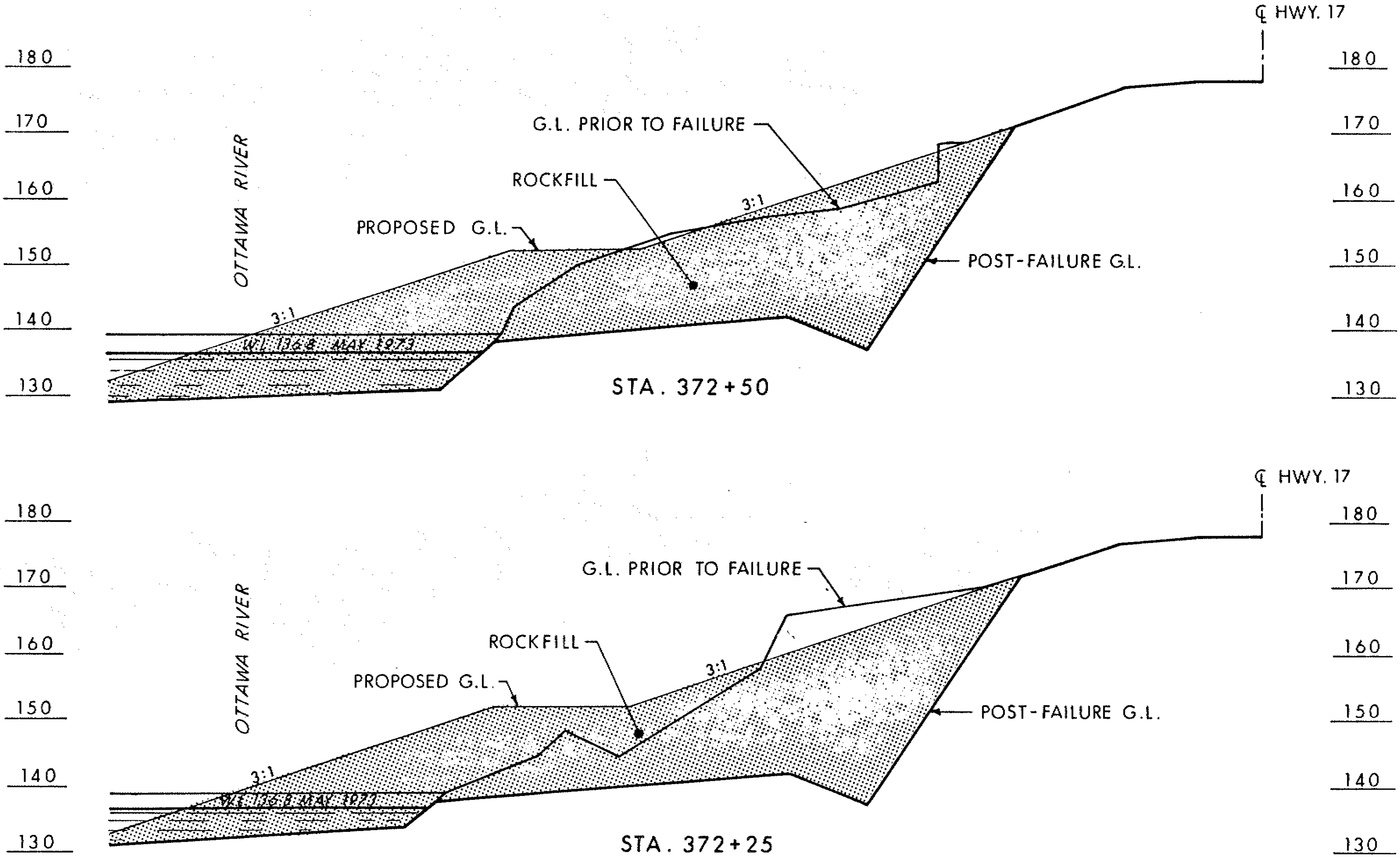
W.O. 73-11053X



RECOMMENDED REMEDIAL MEASURES

FIG. 25

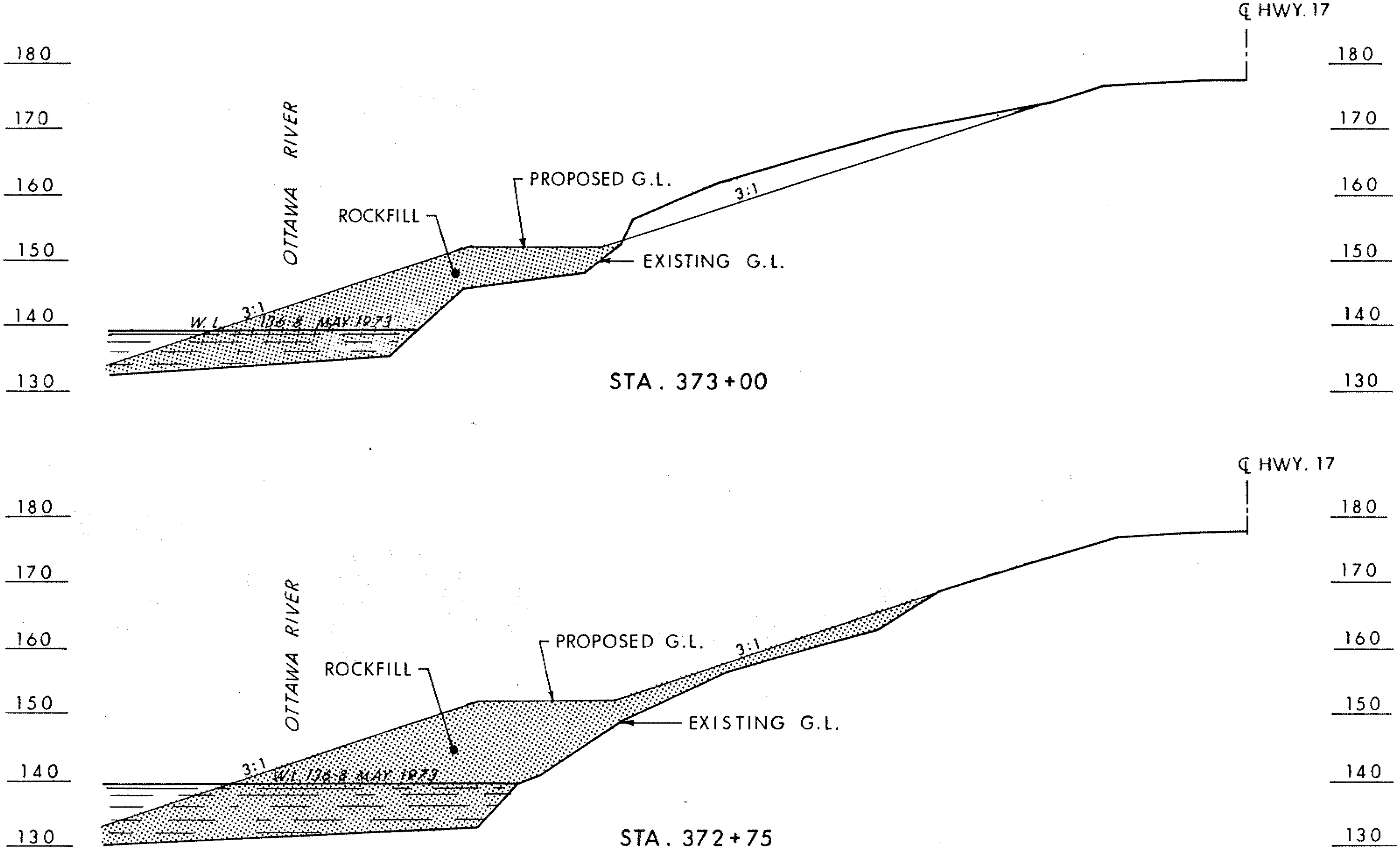
W.O. 73-11053X



RECOMMENDED REMEDIAL MEASURES

FIG. 26

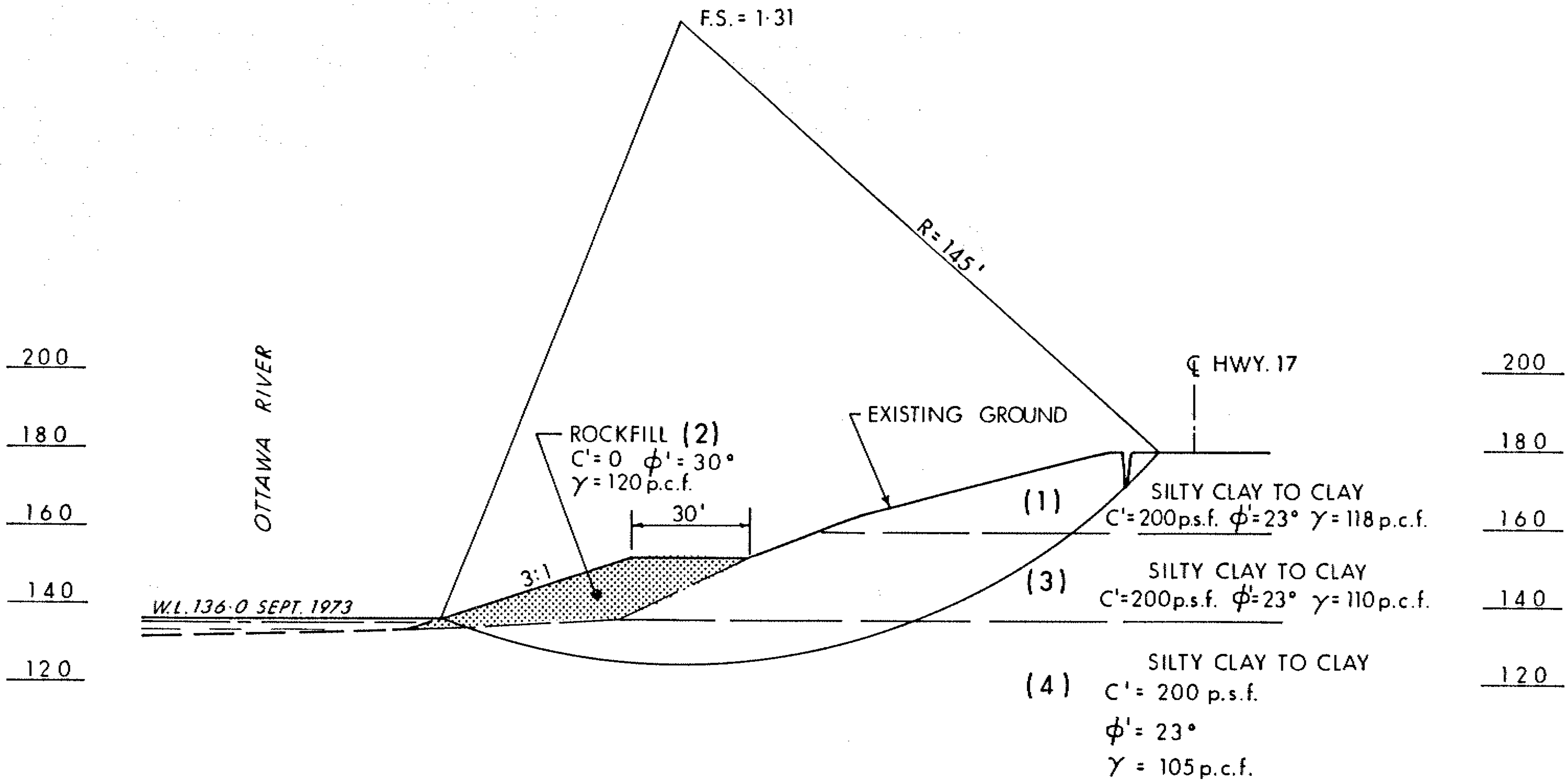
W.O. 73-11053X



RECOMMENDED REMEDIAL MEASURES

FIG. 27

W.O. 73-11053X



STA. 265+00

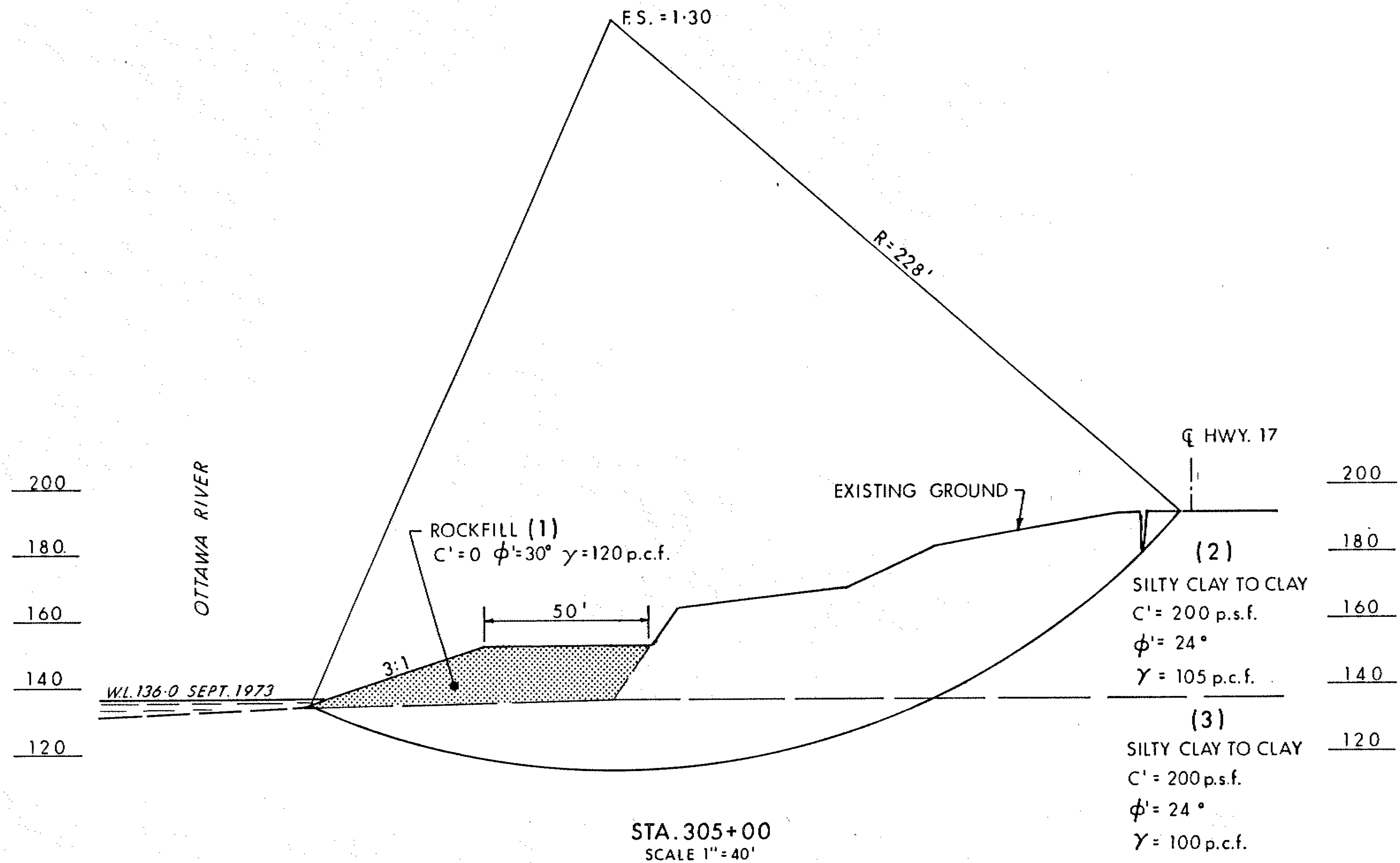
SCALE 1" = 40'

STABILITY ANALYSIS IN TERMS OF EFFECTIVE STRESS

(REMEDIAL MEASURES)

FIG. 28

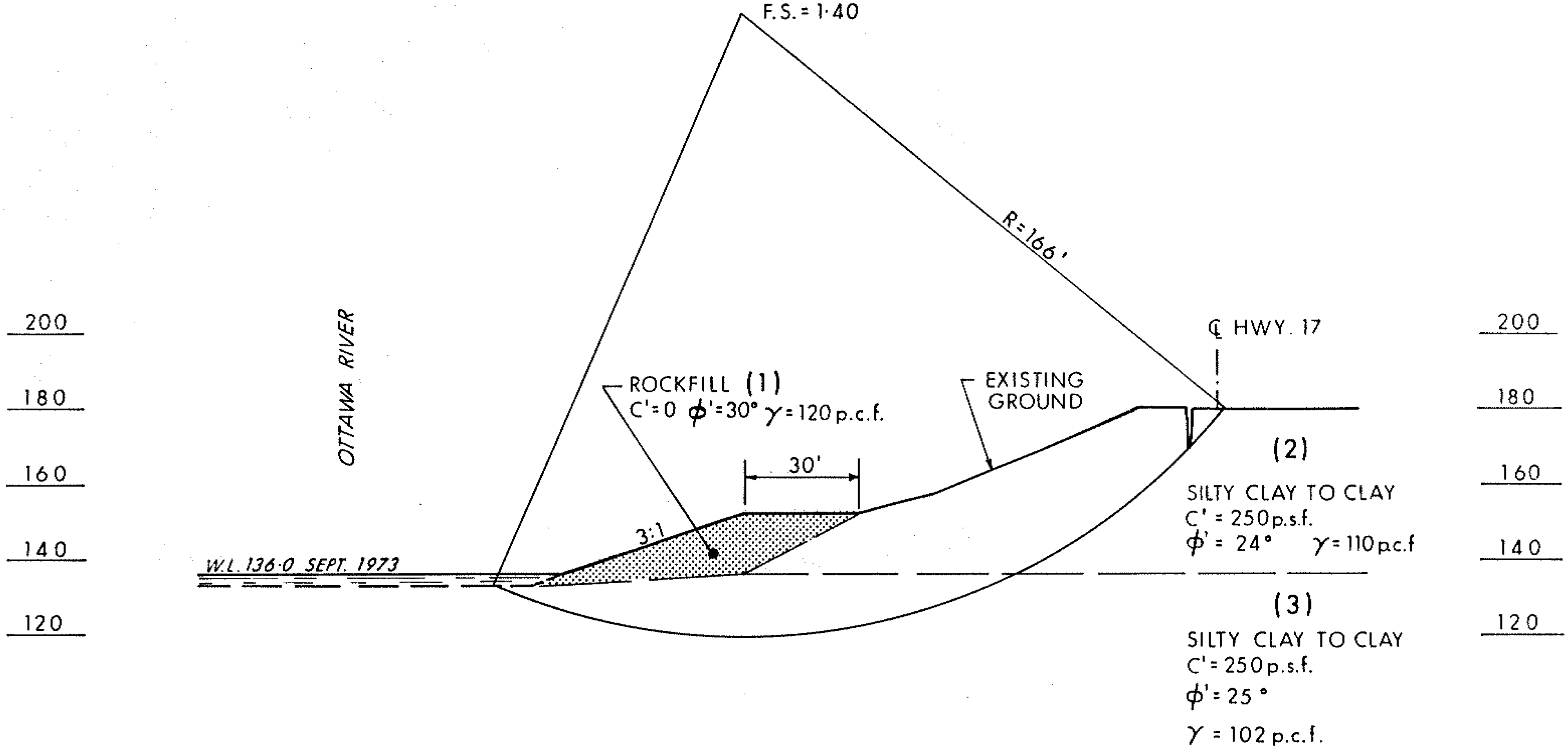
W.O. 73-11053X



STABILITY ANALYSIS IN TERMS OF EFFECTIVE STRESS
(REMEDIAL MEASURES)

FIG. 29

W.O. 73-11053X

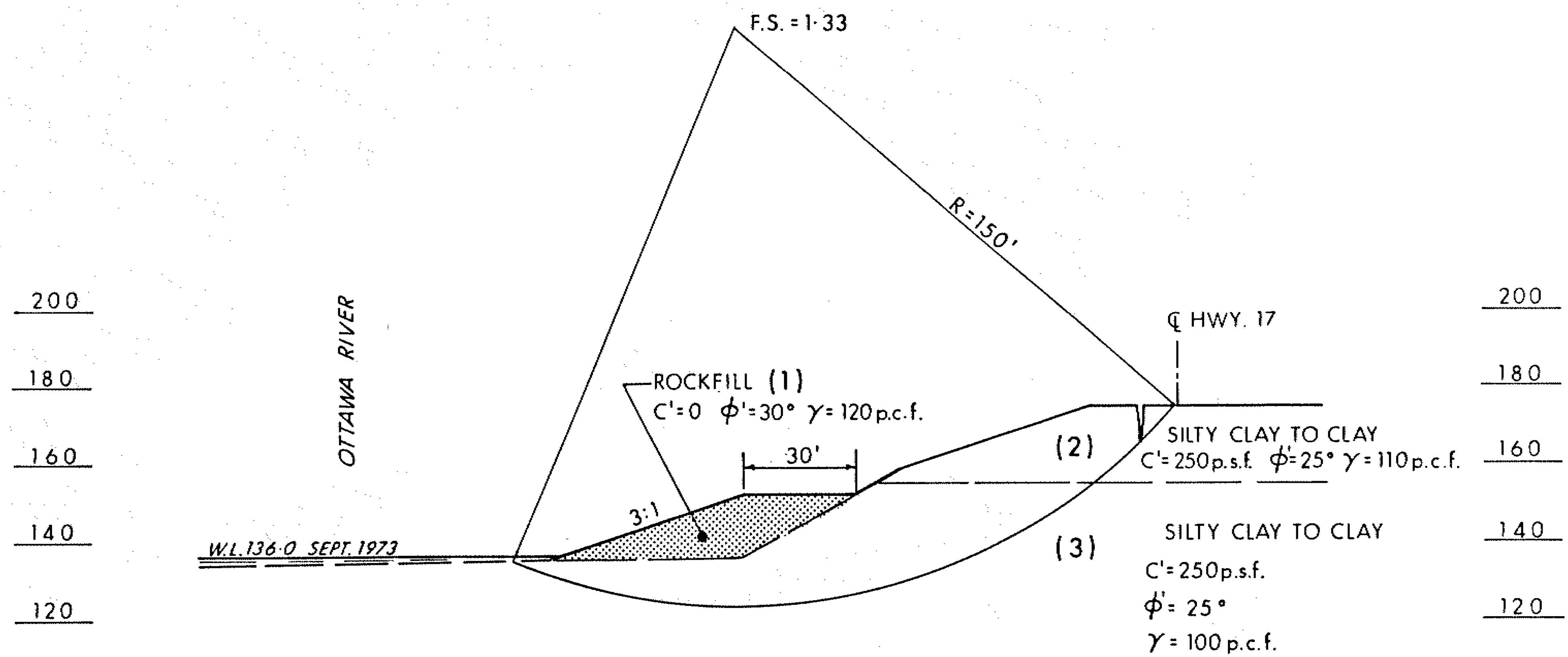


STA. 364+00
SCALE 1" = 40'

STABILITY ANALYSIS IN TERMS OF EFFECTIVE STRESS
(REMEDIAL MEASURES)

FIG. 30

W.O. 73-11053 X



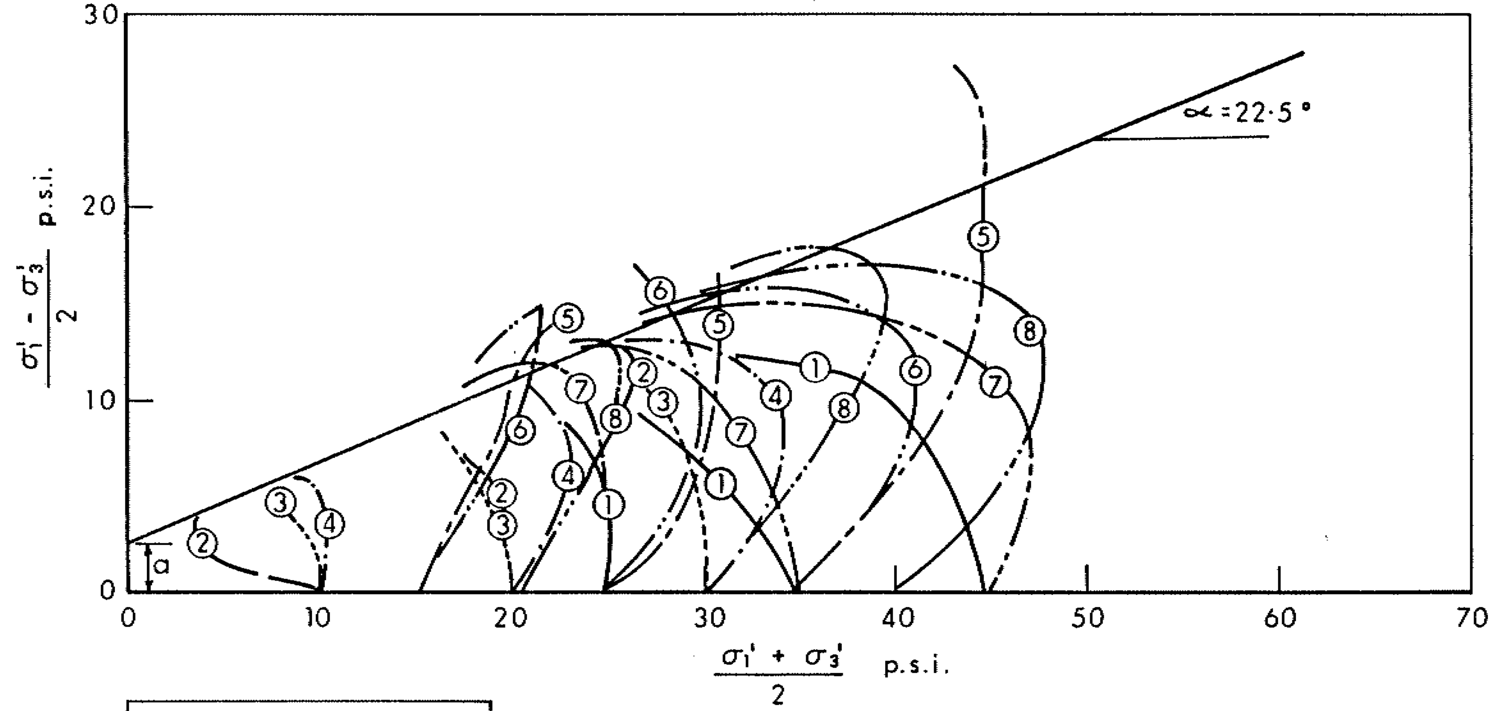
STA. 377+00

SCALE 1" = 40'

STABILITY ANALYSIS IN TERMS OF EFFECTIVE STRESS
(REMEDIAL MEASURES)

FIG. 31

W.O. 73-11053X



LEGEND		
SYMBOL	B. H.	SA.
1 ———	1	6
2 — —	6	3 A
3 - - - - -	6	3 B
4 —·—	2	4 A
5 — – —	7	4
6 —··—	8	5
7 —-·-—	5	8
8 —···—	3	7

NOTE

(1) FOR INDEX PROPERTIES OF SAMPLES SEE RECORD OF BORE HOLE SHEETS.

(2) EFFECTIVE STRESS PARAMETERS FOR SENSITIVE SILTY CLAY TO CLAY STRATUM

$\phi' = \mathrm{SIN}^{-1}\ \mathrm{TAN}\alpha$

$= 24{\cdot}5°$

$C' = a/\mathrm{COS}\,\phi'$

$= 390$ p.s.f.

SUMMARY OF C.I.U. TESTS

SILTY CLAY TO CLAY, WITH OCC. INCLUSIONS OF ORGANICS

FIG. 32

W.O. 73-11053X

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS – ONTARIO

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE No 1

JOB 73-11053 (X) — LOCATION Hwy. #17 Sta. 304 + 90 76LT — ORIGINATED BY CSP

W.P. — BORING DATE August 21 - 23, 1973 — COMPILED BY CSP

DATUM Geodetic — BOREHOLE TYPE Wash Boring and Cone Test — CHECKED BY

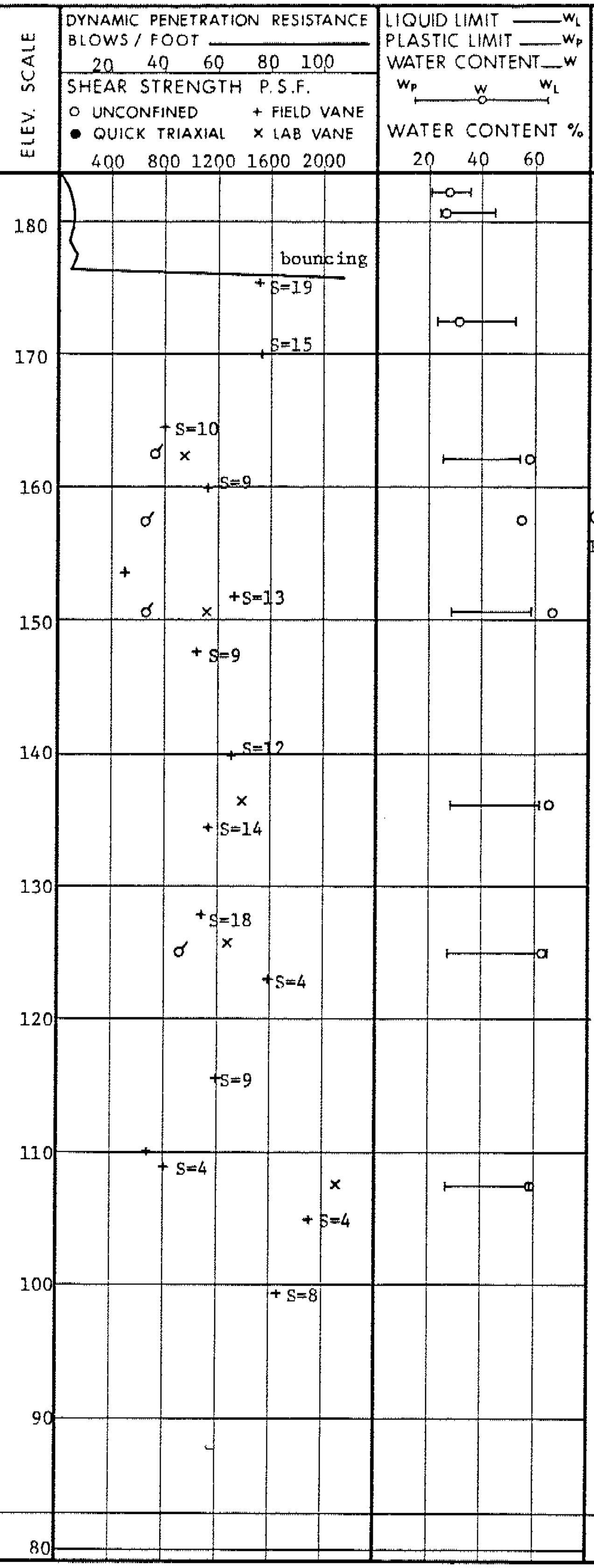
SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
183.5	Ground level
0.7	Top soil
	with pockets and layers of sand
171.5 / 12.0	Brown / Grey
	Silty clay to clay, traces of sand and organics.
	With occasional thin silt seams
	Sensitive
	Firm to stiff
82.7 / 100.8	End of Borehole. Probable bedrock

SAMPLES

NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
1	SS	6		
2	SS	14		
3	TW	PM		
4	SS	25		14 14 29 43
5	TW	PM		
6	TW	PM	104	
7	TW	PM	C_c=1.69, P_c=2.85 tsf	
8	TW	PM	102	0 0 27 73
9	TW	PM		
10	TW	PM	99	0 0 26 74
11	TW	PM		
12	TW	PM	102	0 0 26 74
13	TW	PM		
14	TW	PM		
15	TW	PM		0 0 28 72
16	TW	PM		

Remarks (water levels): 172.0, 169.0, 167.0; P1, P2



20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 1A

JOB 73-11053 (X)	LOCATION Hwy. #17 Sta. 304 + 92 71 LT	ORIGINATED BY CSP
W.P.	BORING DATE September 4, 1973	COMPILED BY CSP
DATUM Geodetic	BOREHOLE TYPE Vane Test	CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. (+ FIELD VANE)	WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
183.9	Ground level									
0.0						180				
	Probably silty clay to clay					170	S=12, S=9			
	Firm to stiff					160	S=14, S=7, S=9, S=6, S=34, S=21			
						150	S=50, S=19, S=11, S=8			
146.5 / 37.4	End of Borehole					140				

SOIL PROFILE | SAMPLES

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.: O UNCONFINED; ● QUICK TRIAXIAL; + FIELD VANE; × LAB VANE — 400 800 1200 1600 2000

LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W

20 / 15 ◊ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 2

JOB 73-11053 (X)
LOCATION Hwy. 17 Sta. 305 + 00 209' Lt
ORIGINATED BY CSP
W.P.
BORING DATE August 25 - 27, 1973
COMPILED BY CSP
DATUM Geodetic
BOREHOLE TYPE Wash Boring
CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	SHEAR STRENGTH P.S.F.	WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
136.0	Water level							
0.0	Water							
1.3	sand with some silt grey loose	1	SS	1/8"				0 84 (16)
132.0 / 4.0		2	SS	4				0 88 (12)
	Silty clay to clay, traces of sand and organics. With occasional thin silt seams. Sensitive. Grey, firm to stiff	3	TW	PM	+ S=7			
		4	TW	PM	+S=9		101	
		5	TW	PM	+S=11			
		6	TW	PM	+ S=14		99	
		7	TW	PM	+S=12			
		8	TW	PM	+S=14		100	
		9	TW	PM	+S=12			
		10	TW	PM	+> ; +S=10 ; + S=12			
82.0 / 54.0	sand, silt and gravel							
79.7	grey							
56.3	End of Borehole. Probably bedrock							

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT
SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
400 800 1200 1600 2000
LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W
20 40 60
ELEV. SCALE 130 120 110 100 90 80 70

20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO

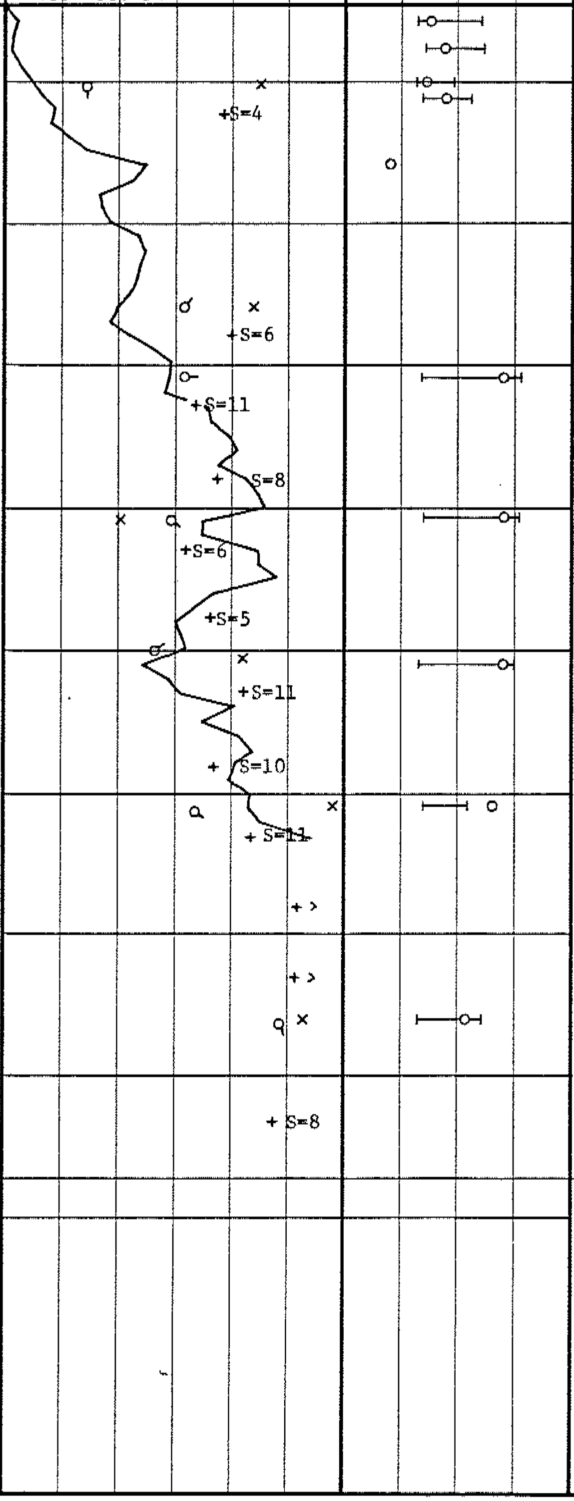
DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 3

JOB 73-11053 (X)	LOCATION Hwy. #17 Sta. 264 + 93 38'Lt	ORIGINATED BY CP
W.P.	BORING DATE August 23, 1973	COMPILED BY SO
DATUM Geodetic	BOREHOLE TYPE Wash Boring and Cone Test	CHECKED BY

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
175.2	Ground level						
0.0	Topsoil						
0.8	Fill Material- silty clay, pockets of sand and gravel some organics	1	SS	9			
		2	SS	12			
		3	TW	PM	170		
167.2	brown soft						166.2
8.0	Sand, some gravel and traces of silt	4	SS	39			17 56 15 12
158.2	grey compact	5	SS	27	160		
17.0	Silty clay to clay, traces of sand and organics.	6	TW	PM			
		7	TW	PM	150	104	0 2 35 63
							C_c=1.32 PI P_c=4tsf
		8	TW	PM			
	With occasional thin silt seams.	9	TW	PM	140	104	0 1 34 65
		10	TW	PM			
		11	TW	PM	130	103.5	0 1 34 65
	Sensitive	12	TW	PM			
		13	TW	PM	120	106	0 0 44 56
		14	TW	PM			
	Grey, firm to stiff	15	TW	PM	110		
		16	TW	PM		112	0 1 46 53
		17	TW	PM	100		
93.2							
82.0	End of Borehole Probably bedrock				90		

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 — SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 400 800 1200 1600 2000 — LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W — WATER CONTENT % 20 40 60



20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH | FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 4

JOB 73-11053 (X) LOCATION Hwy. #17 Sta. 265 + 00 170'Lt ORIGINATED BY CP

W.P. BORING DATE August 24, 1973 COMPILED BY SO

DATUM Geodetic BOREHOLE TYPE Wash Boring CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
136.0	Water level								
0.0	Water								
1.4			1	SS	3				
131.0	dark grey soft		2	SS	3				
5.0			3	TW	PM	130	+S=10		
			4	TW	PM		+S=10	105	
	Silty clay to clay, trace of sand and organics		5	TW	PM	120	+S=13	103 C_c=1.28 P_c=3.5 tsf	
	With occasional thin silt seams		6	TW	PM		+S=8	106	0 1 33 66
	Sensitive		7	TW	PM	110	+S=15		
			8	TW	PM		3,140; +S=12	105	0 2 32 66
	Grey, firm to stiff		9	TW	PM	100	+S=9		
96.0									
40.0	Sand, silt, and gravel		10	SS	4/6"				
92.3	Grey dense		11	SS	118/8"				
43.7	End of Borehole Probably bedrock					90			

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 400 800 1200 1600 2000

LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W — WATER CONTENT % 20 40 60

20 15 ◇ 5 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 5

JOB 73-11053 (X)	LOCATION Hwy. #17 Sta. 363 + 78 31'Lt	ORIGINATED BY CP
W.P.	BORING DATE August 29, 1973	COMPILED BY SO
DATUM Geodetic	BOREHOLE TYPE Wash Boring and BX Rock Cores	CHECKED BY

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
176.7	Ground level
0.0	Sand
174.2 / 2.5	Brown loose
	Fill Material- Silty clay, with some sand and gravel, traces of organics.
	Sand and gravel Compact
	Soft to firm
153.7 / 23.0	Grey
	Silty clay to clay, traces of sand and organics
	With occasional thin silt seams.
	Sensitive
	Stiff
120.7	Grey
119.2	Sand, silt and gravel
57.5	Limestone bedrock
115.2	Grey sound
61.5	End of Borehole

SAMPLES

NUMBER	TYPE	BLOWS/FOOT
1	SS	2
2	SS	2
3	TW	PM
4	SS	18
5	SS	40
6	SS	21
7	TW	PM
8	TW	PM
9	TW	PM
10	TW	PM
11	TW	PM
12	TW	PM
13	TW	PM
14	BX	90% rec
15	BX	95% rec

ELEV. SCALE: 170, 160, 150, 140, 130, 120, 110

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 400 800 1200 1600 2000

LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W — WATER CONTENT % 20 30 40

BULK DENSITY γ P.C.F.

REMARKS GR.SA.SI.CL.

Sample	Field vane	Bulk density (P.C.F.)	Remarks (GR.SA.SI.CL.)
1			0 4 38 58
2			0 13 58 29
3			0 2 48 50
5			159.7
6			7 9 44 40
7	S=7		
8	S=9	102, C_c=0.94, P_c=2.85	145.7
9	S=10		
10	S=13	103	
11	S=9		
12	S=11	110	0 3 48 49
14			P1

20 / 15 ⌖ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 6

JOB 73-11053 (X) LOCATION Hwy. #17 Sta. 364 + 00 155'Lt ORIGINATED BY CP

W.P. BORING DATE August 27, 1973 COMPILED BY SO

DATUM Geodetic BOREHOLE TYPE Wash Boring CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	SHEAR STRENGTH	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
136.0	Water level								
0.0	Water		1	SS	1				
1.0			2	SS	2				0 2 27 71
131.0 / 5.0	Soft		3	TW	PM	130		102	
							S=12		
	Silty clay to clay, traces of sand and organics.		4	TW	PM		S=10		
			5	TW	PM	120	S=8	100	0 0 32 68
	With occasional thin silt seams		6	TW	PM		S=10		
	Sensitive		7	TW	PM	110	S=19	102	0 1 34 65
			8	TW	PM		S=11		
	Grey, Stiff		9	TW	PM	100	S=22		
93.0 / 43.0	Sand, some gravel and silt Grey		10	TW	PM				
89.2 / 46.8	End of Borehole Probably bedrock		11	WS	—	90			
						80			

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F. ○ UNCONFINED ✚ FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 400 800 1200 1600 2000

LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W — WATER CONTENT % 20 40 60

20 15 5 10 % STRAIN AT FAILURE

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

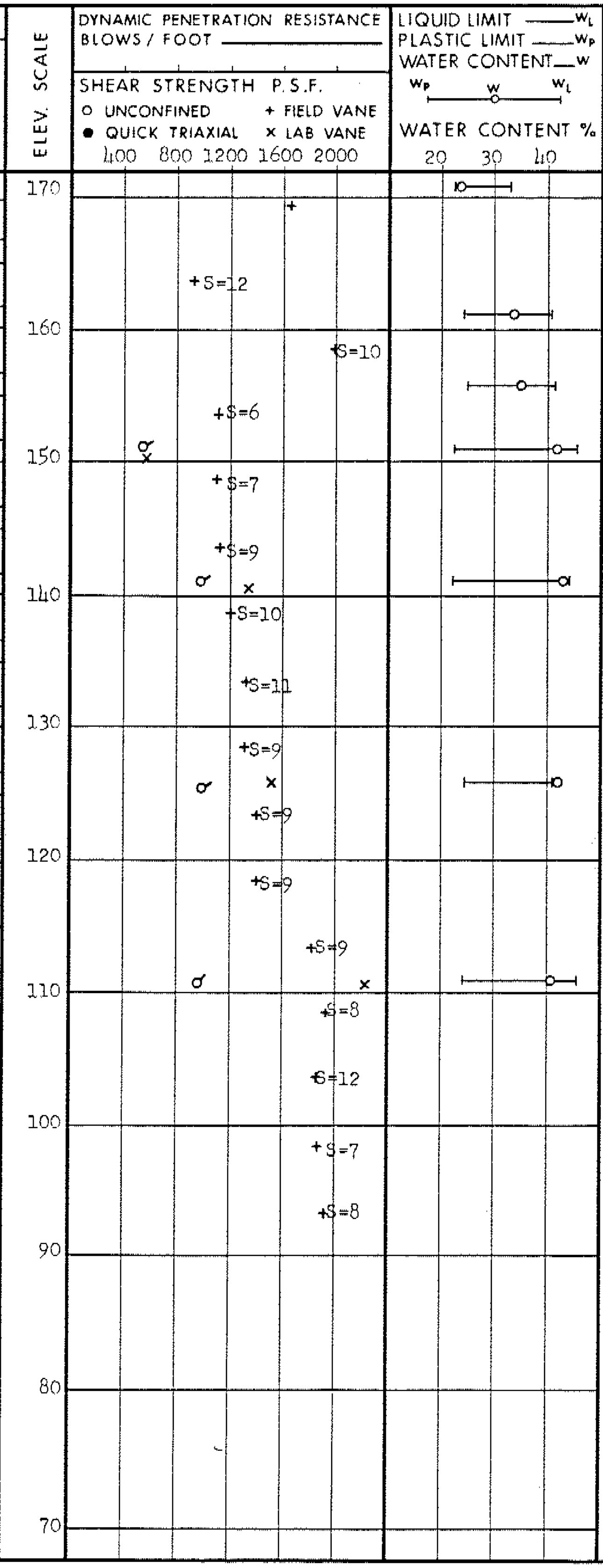
RECORD OF BOREHOLE Nº 7

JOB 73-11053 (X) — LOCATION Hwy #17 Sta. 376 + 91 35'Lt — ORIGINATED BY CP

W.P. — BORING DATE August 27 + 28, 1973 — COMPILED BY SO

DATUM Geodetic — BOREHOLE TYPE Washboring — CHECKED BY

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
171.9	Ground level					
0.0	Topsoil	1	SS	10		
1.0		2	TW	PM		
160.4	Brown	3	TW	PM	107.5	161.9, 160.4
11.5	Grey				108	
		4	TW	PM	107, 108.5	
	Silty clay to clay, traces of sand and organics.	5	TW	PM	98.5	F2; 0 1 38 61
		6	TW	PM		
	With occasional thin silt seams.	7	TW	PM	100	0 0 30 70; P1
		8	TW	PM		
		9	TW	PM		
	SENSITIVE	10	TW	PM	100.5	0 0 31 69
		11	TW	PM		
		12	TW	PM		
		13	TW	PM	101	0 0 30 70
		14	TW	PM		
	Firm to stiff	15	TW	PM		
		16	TW	PM		
67.9	Continued					
104.0						



20
15 ◇ 5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTA N AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 7 Cont.

JOB 73-11053 (X)
LOCATION
ORIGINATED BY CP

W.P.
BORING DATE
COMPILED BY SO

DATUM Geodetic
BOREHOLE TYPE
CHECKED BY

SOIL PROFILE ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
67.9 / 104.0	Probably silty clay to clay					60				
						50				
						40				
31.9 / 140.0	End of Borehole					30				

20 / 15 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTA N AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH | FOUNDATIONS OFFICE

RECORD OF BOREHOLE № 8

JOB 73-11053 (X) | LOCATION Hwy #17 Sta. 377 + 00 150'Lt | ORIGINATED BY CP

W.P. | BORING DATE August 29, 1973 | COMPILED BY SO

DATUM Geodetic | BOREHOLE TYPE Washboring | CHECKED BY

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
136.0	Water level						
0.0	Water						
1.0	Silty clay to clay, traces of sand and organics.	1	SS	3			
		2	TW	PM	130		
		3	TW	PM		98.5	0 0 34 66
		4	TW	PM	120		
		5	TW	PM		100	
	With occasional thin silt seams	6	TW	PM	110		
		7	TW	PM		102	0 1 30 69
		8	TW	PM	100		
		9	TW	PM			
	SENSITIVE				90		
					80		
	Grey soft to stiff				70		
					60		
					50		
					40		
32.0 104.0	Continued						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE

400 800 1200 1600 2000

S=28, S=30, S=27, S=7, S=6, S=8, S=14, S=18, S=38, S=30, S=19, S=9, S=16, S=19, S=19, S=19, S=10, S=48, S=18, S=6, S=5, S=5, S=7, S=7, S=5, S=14, S=7, S=6

LIQUID LIMIT — W_L
PLASTIC LIMIT — W_P
WATER CONTENT — W

WATER CONTENT %
20 40 60

20
15 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 8 Cont.

JOB 73-11053 (X)
LOCATION
ORIGINATED BY CP
W.P.
BORING DATE
COMPILED BY SO
DATUM Geodetic
BOREHOLE TYPE
CHECKED BY

ELEV. DEPTH 32.0	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
104.0	Probably silty clay to clay					30				
18.9						20				
117.1	End of Borehole Probably bedrock									
						10				

20
15 ⌖ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTA… …N AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH | FOUNDATIONS OFFICE

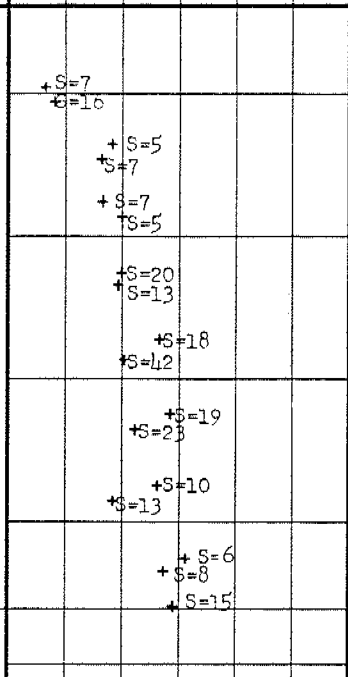
RECORD OF BOREHOLE Nº 9

JOB 73-11053 (X) | LOCATION Hwy #17 Sta. 373 + 00 166'Lt | ORIGINATED BY CP

W.P. | BORING DATE August 30, 1973 | COMPILED BY SO

DATUM Geodetic | BOREHOLE TYPE Vane Test Only | CHECKED BY [signature]

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (400 800 1200 1600 2000)	LIQUID LIMIT — W_L PLASTIC LIMIT — W_P WATER CONTENT — W; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT					
136.0	Water level									
0.0	Water									
1.2	Probably silty clay to clay					130				
						120				
						110				
						100				
94.0 / 42.0	End of Borehole					90				



OFFICE REPORT ON SOIL EXPLORATION

20 / 15 ⌖ 5 / 10 % STRAIN AT FAILURE

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE № 10

JOB 73-11053 (X)
LOCATION Hwy #17 Sta. 306 + 97 47'Lt
ORIGINATED BY CSP
W.P.
BORING DATE September 5, 1973
COMPILED BY CSP
DATUM Geodetic
BOREHOLE TYPE Vane Test and Piezometer
CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. (+ FIELD VANE)	REMARKS
185.6	Ground level							
0.0	Probably silty clay to clay					180		180.1
								176.6
						170		P1
	Firm to stiff						+ S=6	
							+ S=6	
							+ S=9	
						160	+ S=6	
							+ S=8	
							+ S=7	
						150		P2
143.1							+ S=9	
							+ S=7	
42.5	End of Borehole					140		

Column headings: DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE, 400 800 1200 1600 2000; LIQUID LIMIT —W_L PLASTIC LIMIT —W_P WATER CONTENT —W, WATER CONTENT %; BULK DENSITY γ P.C.F.; REMARKS GR SA.SI.CL.

20
15 ◇ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 11

JOB 73-11053 (X)
W.P.
DATUM Geodetic

LOCATION Hwy #17 Sta. 266 + 98 42'Rt
BORING DATE September 6, 1973
BOREHOLE TYPE Wash boring and piezometer

ORIGINATED BY CSP
COMPILED BY CSP
CHECKED BY

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
178.5	Ground level									
0.0	Probably silty clay to clay					170				174.5
						160				P2
						150				No reading
138.5						140				P1
40.0						130				

20
15 ⌖ 5 % STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS – ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE No 12

JOB 73-11053 (X)

W.P.

DATUM Geodetic

LOCATION Hwy #17 Sta. 367 + 05 26' RT

BORING DATE September 6, 1973

BOREHOLE TYPE Wash Boring and Piezometer

ORIGINATED BY C.S.P.

COMPILED BY C.S.P.

CHECKED BY

SOIL PROFILE: ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
177.8	Ground level									
0.0	Probably silty clay to clay					170				168.5
										167.3
										P2
						160				
						150				
137.8						140				P1
40.0	End of Borehole									
						130				

20 / 15 ⌖ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

RECORD OF BOREHOLE Nº 13

JOB	73-11053 (X)	LOCATION	Hwy #17 Sta. 372 + 00 26' Rt	ORIGINATED BY	JB
W.P.		BORING DATE	May 16, 1973	COMPILED BY	PD
DATUM	*Geodetic*	BOREHOLE TYPE	Auger and Cone Test	CHECKED BY	[signature]

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
176.0	Ground level
0.0 / 172.0	Fill Material-sand and gravel Compact
4.0	Silty clay to clay traces of sand and organics. With occasional thin silt seams. SENSITIVE. Grey firm to stiff
101.0 / 75.0	End of Borehole

SAMPLES

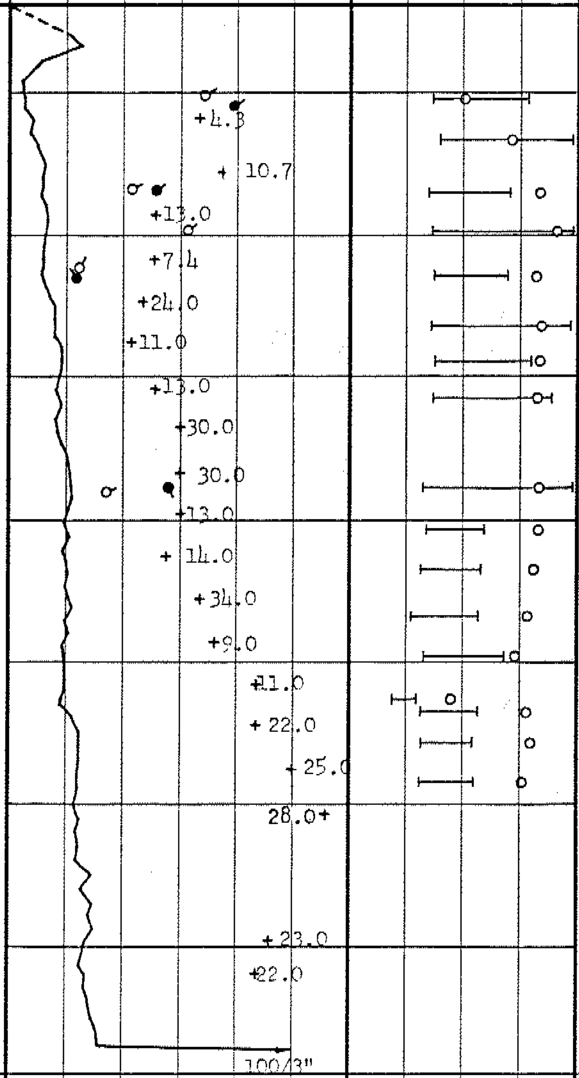
NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
1	SS	20		
2	SS	13		
3	TW	PH	113	0 0 30 70
4	TW	PH		▼ 165.0
5	TW	PH	98 / 100.5	
6	TW	PH	102	
7	TW	PH	100 / 98.5	0 0 68 32
8	TW	PH		
9	TW	PH	98	
10	TW	PH		
11	TW	PH		
12	TW	PH	100	0 0 31 69
13	TW	PH	98.5	
14	TW	PH		
15	TW	PH	100	
16	TW	PH		
17	TW	PH	100.5 / 113.5	0 1 23 76 / 0 0 35 65
18	TW	PH	100 / 101	
19	TW	PH	C'=0 / P'=35°	
20	TW	PH		
21	TW	PH		
22	TW	PH		

ELEV. SCALE: 170, 160, 150, 140, 130, 120, 110, 100

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F. — ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 400 1200 2000

LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; WATER CONTENT % 20 40 60



20 / 15 ◇ 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

PENETRATION RESISTANCE

'N'=STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL. THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

CONSISTENCY	c LB./SQ. FT.
VERY SOFT	0 - 250
SOFT	250 - 500
FIRM	500 - 1000
STIFF	1000 - 2000
VERY STIFF	2000 - 4000
HARD	> 4000

DENSENESS	'N' BLOWS / FT.
VERY LOOSE	0 - 4
LOOSE	4 - 10
COMPACT	10 - 30
DENSE	30 - 50
VERY DENSE	> 50

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10 % , SOME 10–25 % , WITH 25–40 % , > 40 % SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON
W.S.	WASHED SAMPLE
S.T.	SLOTTED TUBE SAMPLE
A.S.	AUGER SAMPLE
C.S.	CHUNK SAMPLE
T.W.	THINWALL OPEN
T.P.	THINWALL PISTON
O.S.	OESTERBERG SAMPLE
F.S.	FOIL SAMPLE
R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL
CIU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL
CID	" " DRAINED "
CAU	" ANISOTROPIC UNDRAINED "
CAD	" " DRAINED "
L.V.	LABORATORY VANE
F.V.	FIELD VANE
C	CONSOLIDATION
S	SENSITIVITY

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \dfrac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_P	PLASTIC LIMIT
I_P	PLASTICITY INDEX
w_S	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX $= \dfrac{w - w_P}{I_P}$
I_C	CONSISTENCY INDEX $= \dfrac{w_L - w}{I_P}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX $= \dfrac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE $= \dfrac{-\Delta e}{(1+e)\Delta\sigma'}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX $= \dfrac{\Delta e}{\Delta \log_{10} \sigma'}$
T_v	TIME FACTOR $= \dfrac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
	c' and ϕ': IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
	c_u and ϕ_u: IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	$= 3{\cdot}1416$
e	BASE OF NATURAL LOGARITHMS 2·7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ε	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

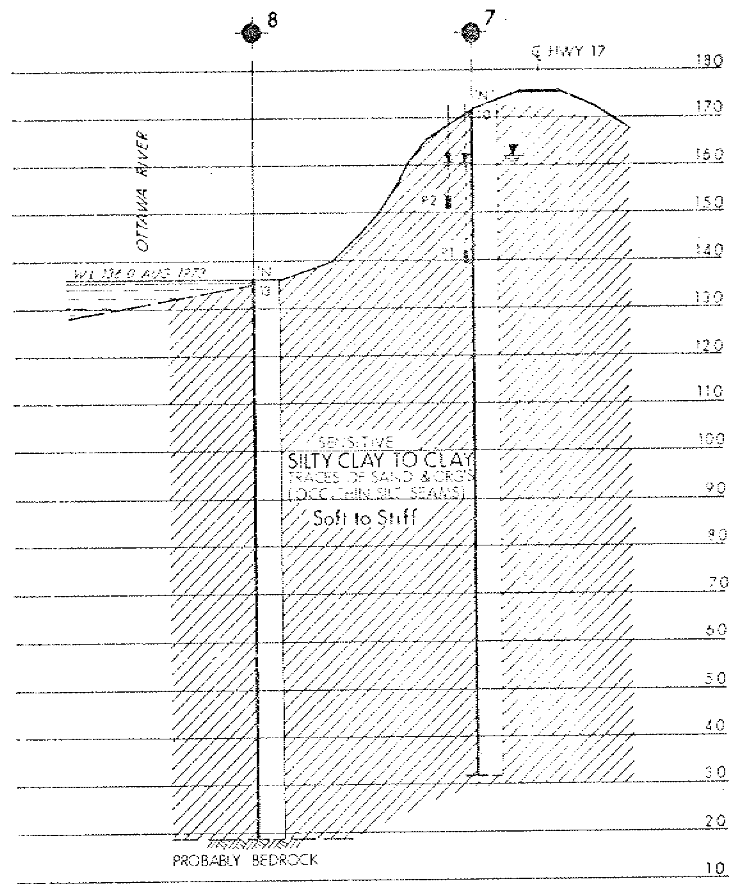
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

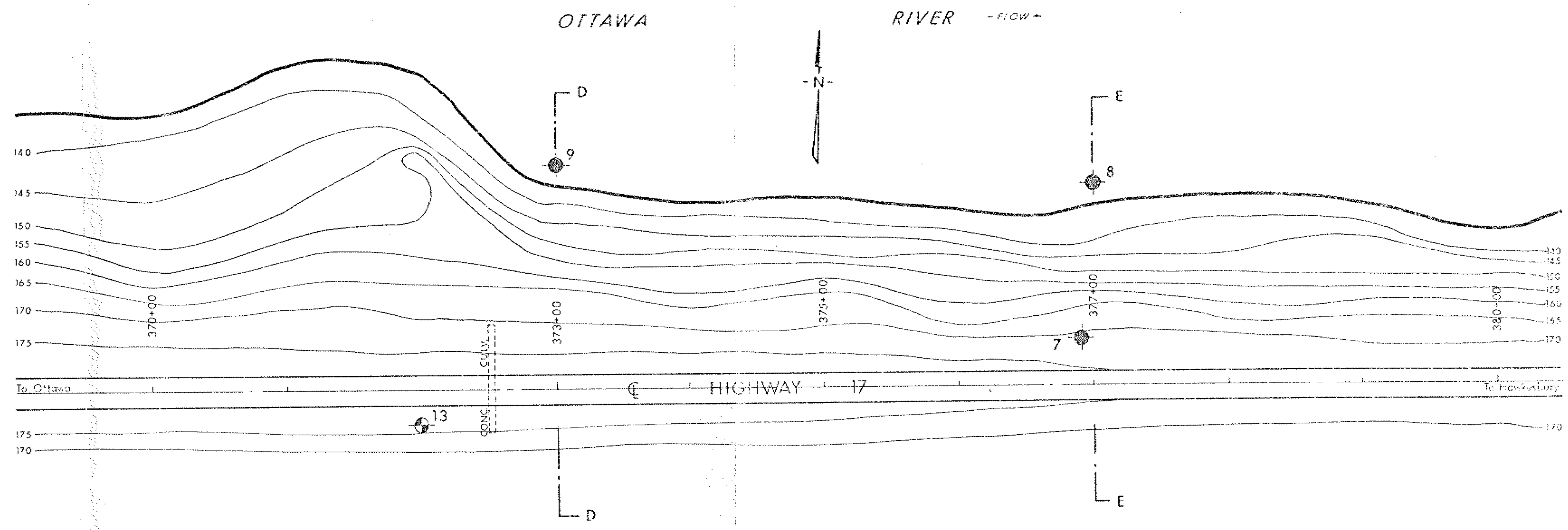
B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY. DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

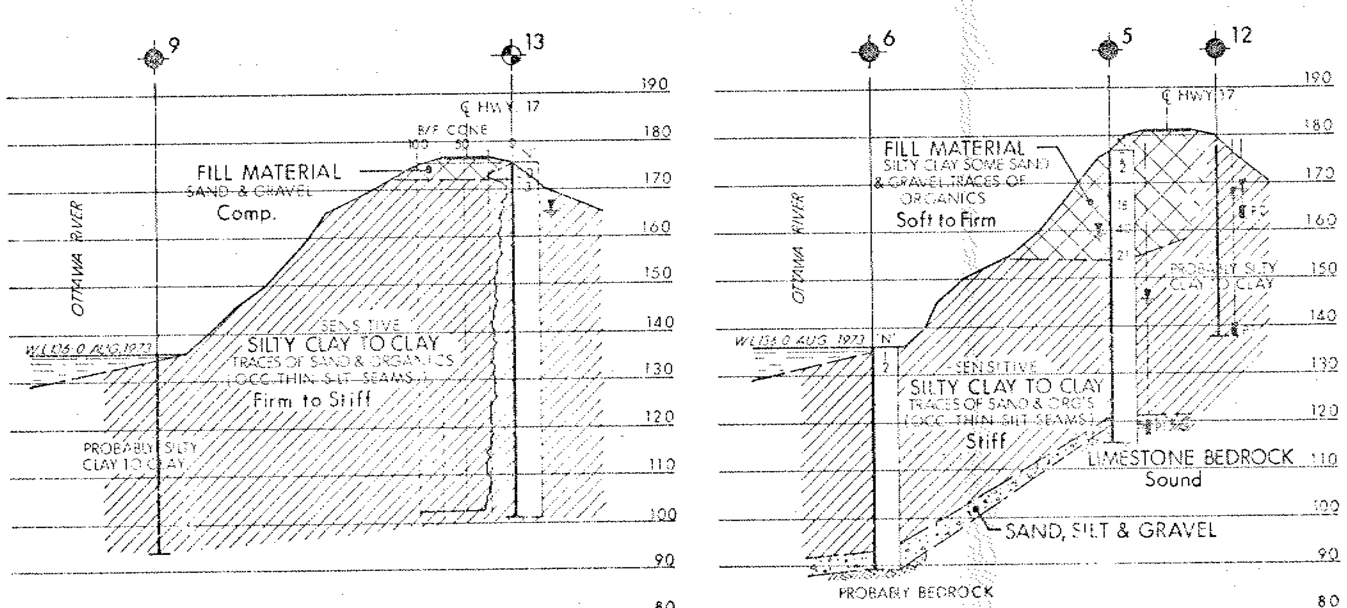
H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL



E - E



SITE 'D'

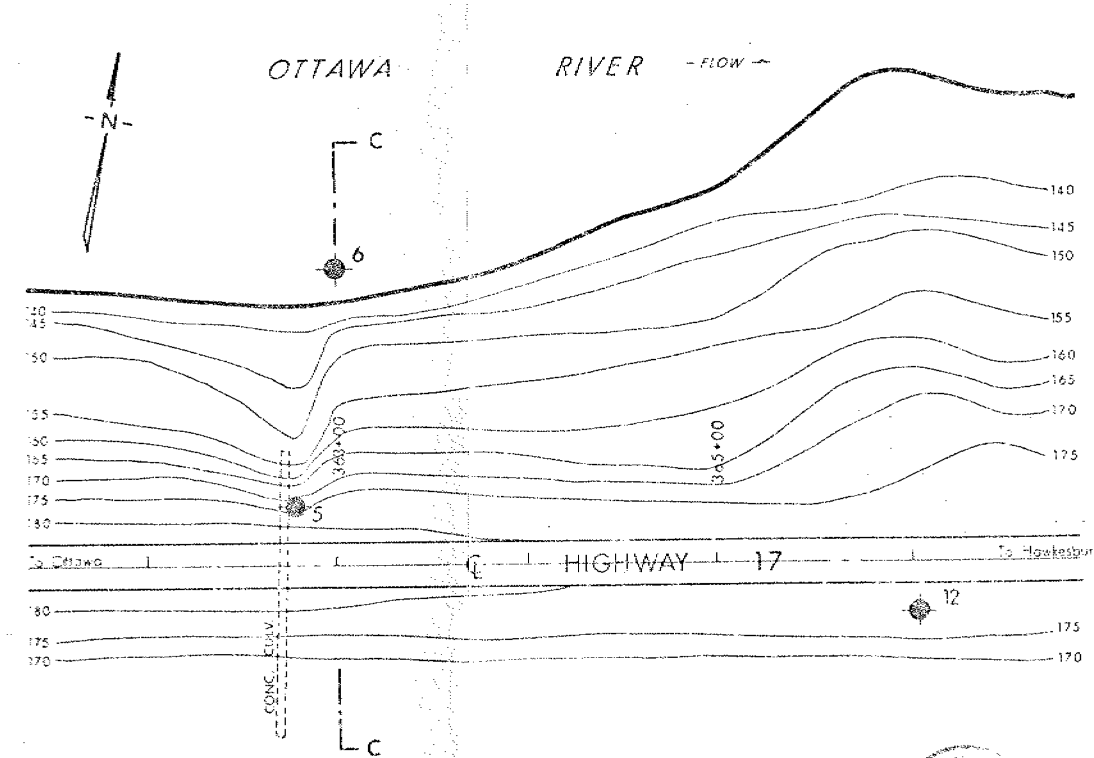


D - D

C - C

SECTIONS

VERT. 20 10 0 SCALE 20 40 FT.
HORIZ. 50 25 0 50 100



SITE 'C'

PLANS

50 25 0 SCALE 50 100 FT.

— NOTE —

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REVISIONS | DATE | BY | DESCRIPTION

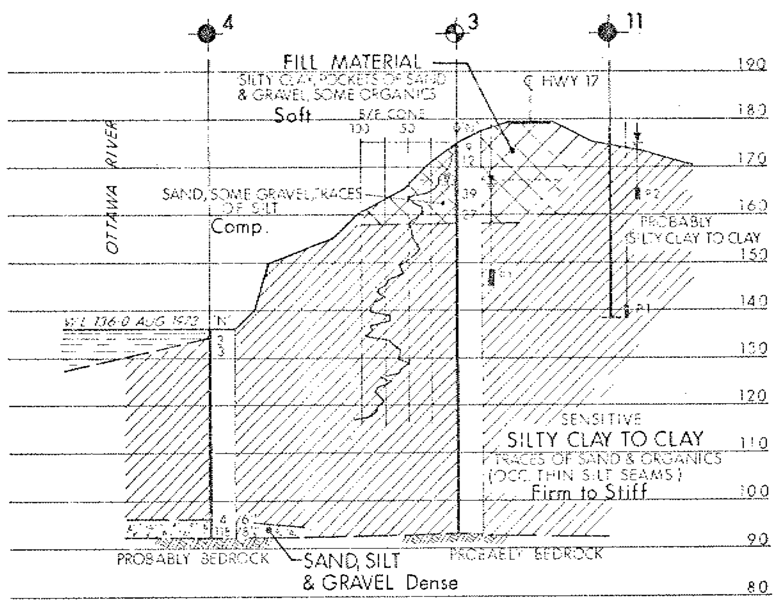
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
DESIGN SERVICES BRANCH—FOUNDATIONS OFFICE

SLOPE STABILITY STUDIES
ALONG OTTAWA RIVER BANK
STA. 361+00 TO STA. 380+00

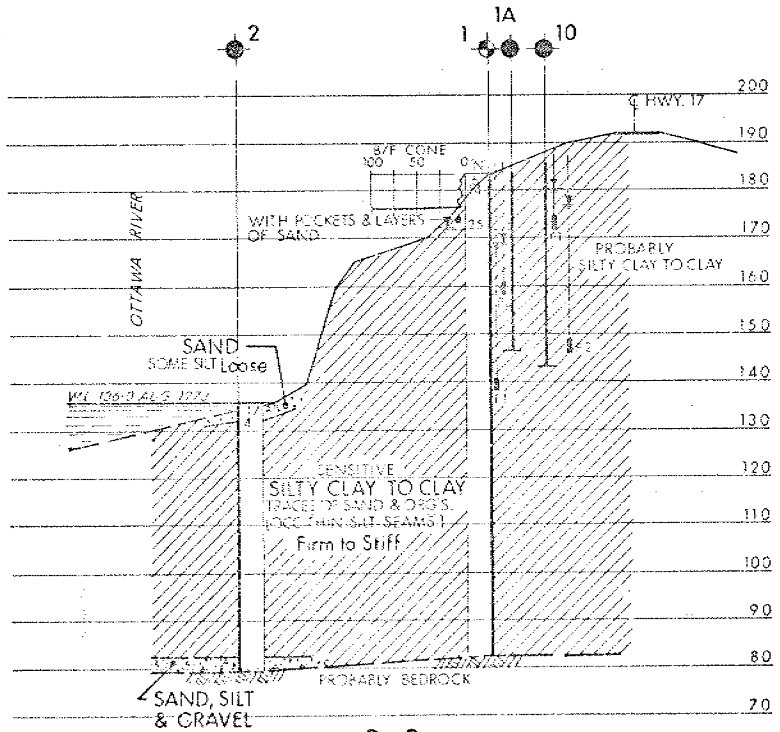
HIGHWAY NO. 17 DIST. NO. 9
REG. MUN. OF OTTAWA - CARLETON
TWP. CUMBERLAND LOT CON

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D C.P.	CHECKED	W.P. NO.	DRAWING NO. 73-11053 B
DRAWN S.D.	CHECKED	W.O. NO. 73-11053A	
DATE 6 NOV 1973		SITE NO.	BRIDGE DRAWING NO.
APPROVED		CONT. NO.	



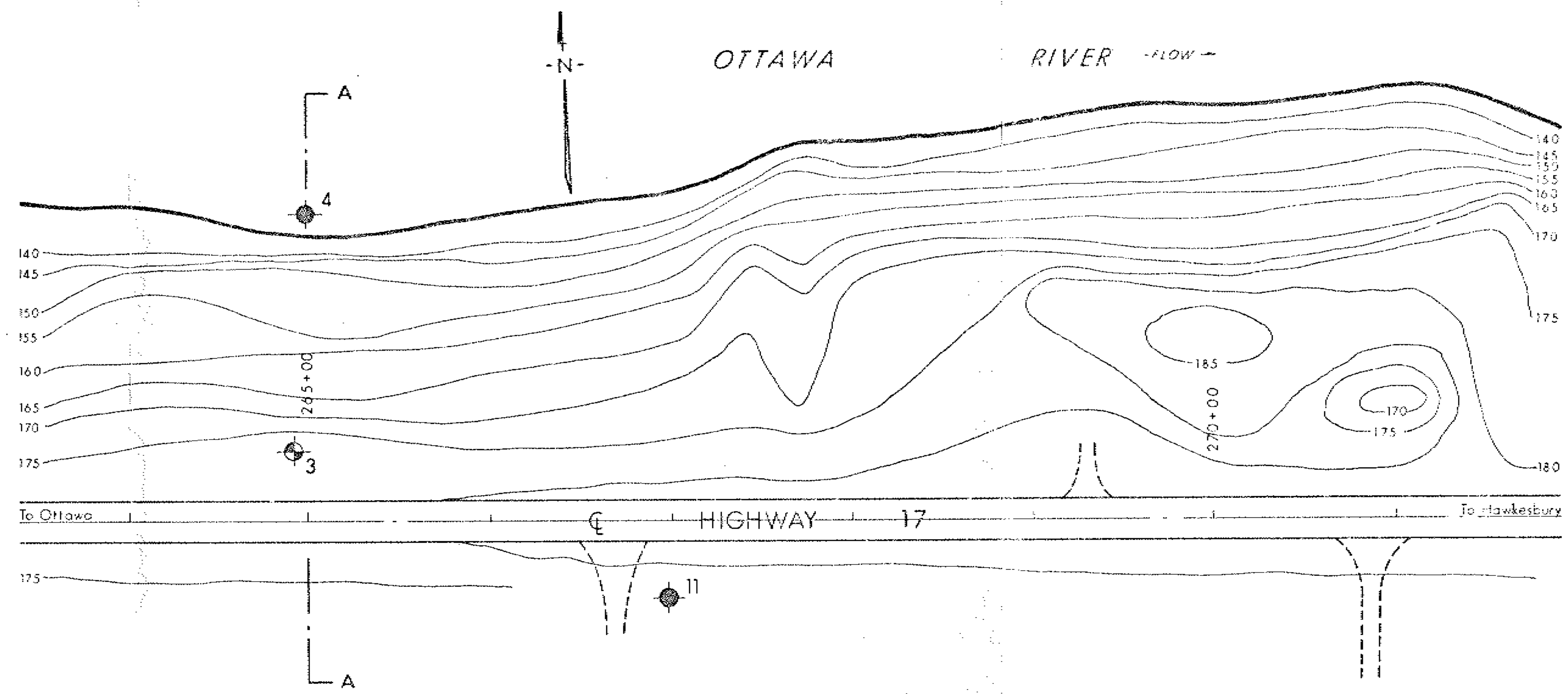
A- A



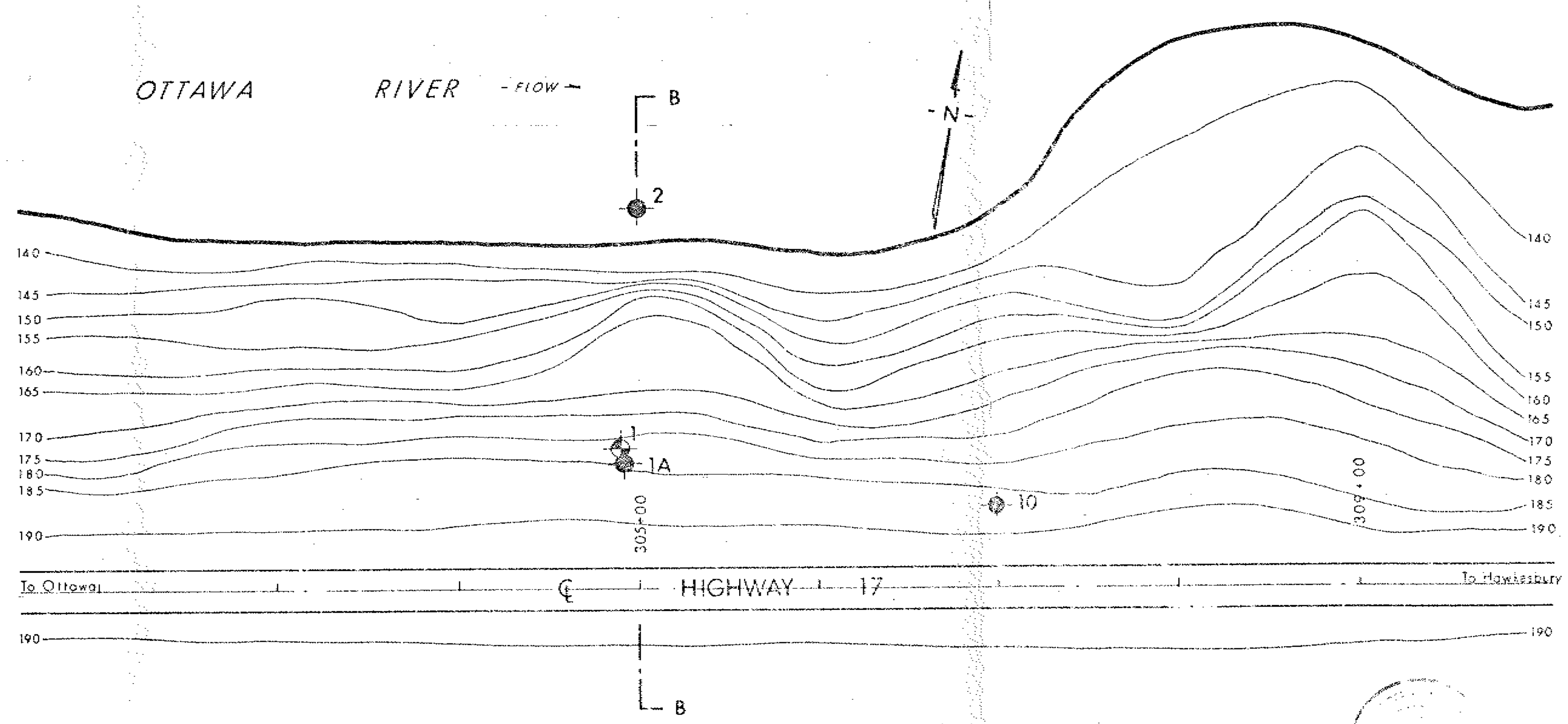
B- B

SECTIONS

VERT. 20 10 0 SCALE 20 40 FT.
HORIZ. 50 25 0 50 100



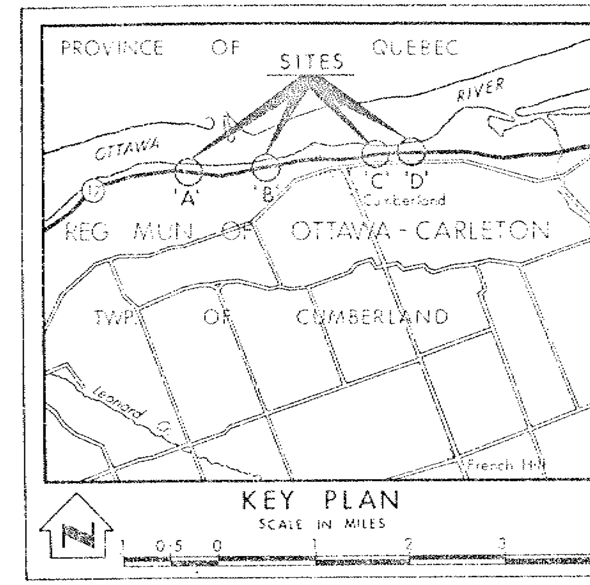
SITE 'A'



SITE 'B'

PLANS

50 25 0 SCALE 50 100 FT.



KEY PLAN

SCALE IN MILES

LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation AUG. 1973

NO.	ELEVATION	STATION	OFFSET
1	183·5	304+90	76' LT.
1A	183·9	304+92	71' LT.
2	136·0	305+00	209' LT.
3	175·2	264+93	38' LT.
4	136·0	265+00	170' LT.
5	176·7	363+78	31' LT.
6	136·0	364+00	155' LT.
7	171·9	376+91	35' LT.
8	136·0	377+00	150' LT.
9	136·0	373+00	166' LT.
10	185·6	306+97	47' LT.
11	178·5	266+98	42' RT.
12	177·8	367+05	26' RT.
13	176·0	372+00	26' RT.

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REVISIONS		
DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

SLOPE STABILITY STUDIES
ALONG OTTAWA RIVER BANK
STA. 265+00 TO STA. 309+00

HIGHWAY NO. 17 DIST. NO. 9
REG. MUN. OF OTTAWA - CARLETON
TWP. CUMBERLAND LOT CON

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D C.F.	CHECKED	W.P. NO.	DRAWING NO. 73-11053 A
DRAWN S.O.	CHECKED	W.O. NO. 73-11053A	
DATE 6 NOV 1973		SITE NO.	BRIDGE DRAWING NO.
APPROVED		CONT. NO.	

EXTRA

PHOTOS

PLATE No.1

TOE FAILURE AT STATION 271

PLATE No.2

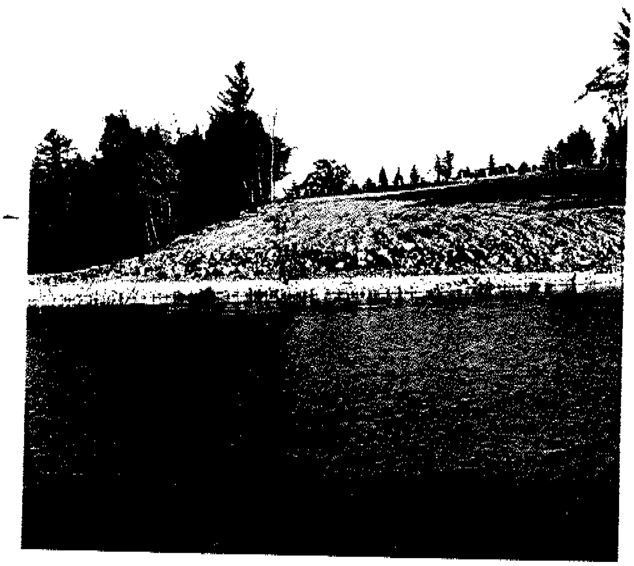
TOE FAILURE AT STATION 271

PLATE No.3

LIMESTONE BEDROCK OUTCROP AT STATION 320

PLATE No 4
TOE EROSION
STATION 262 TO STATION 267

PLATE No. 5
STATION 367 TO STATION 370



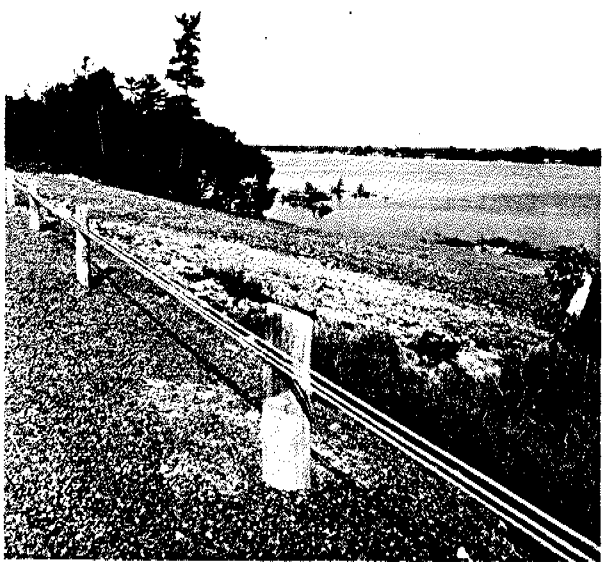


PLATE No 6
STATION 370 TO STATION 373

PLATE No 7
TILTING OF TREES
STATION 373 TO STATION 380

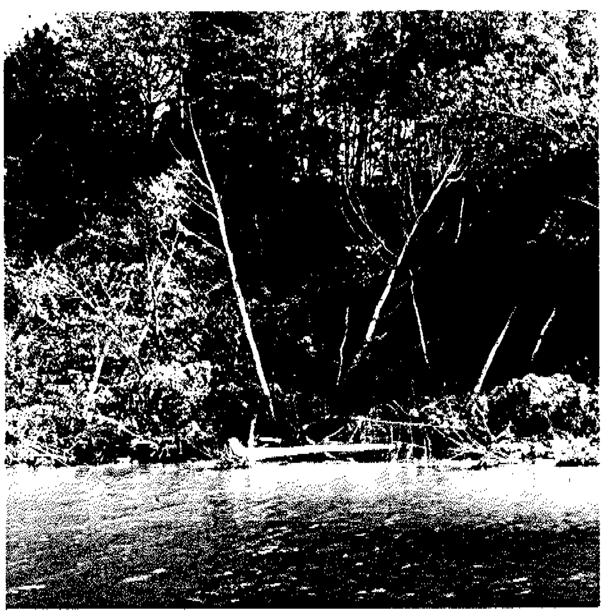
PLATE No 8
SCAR OF OLD FAILURE

PLATE No.9
SCAR OF OLD FAILURE

W.O. 73-11053X

W.O. 73-11053K

W.O. 73-11053X



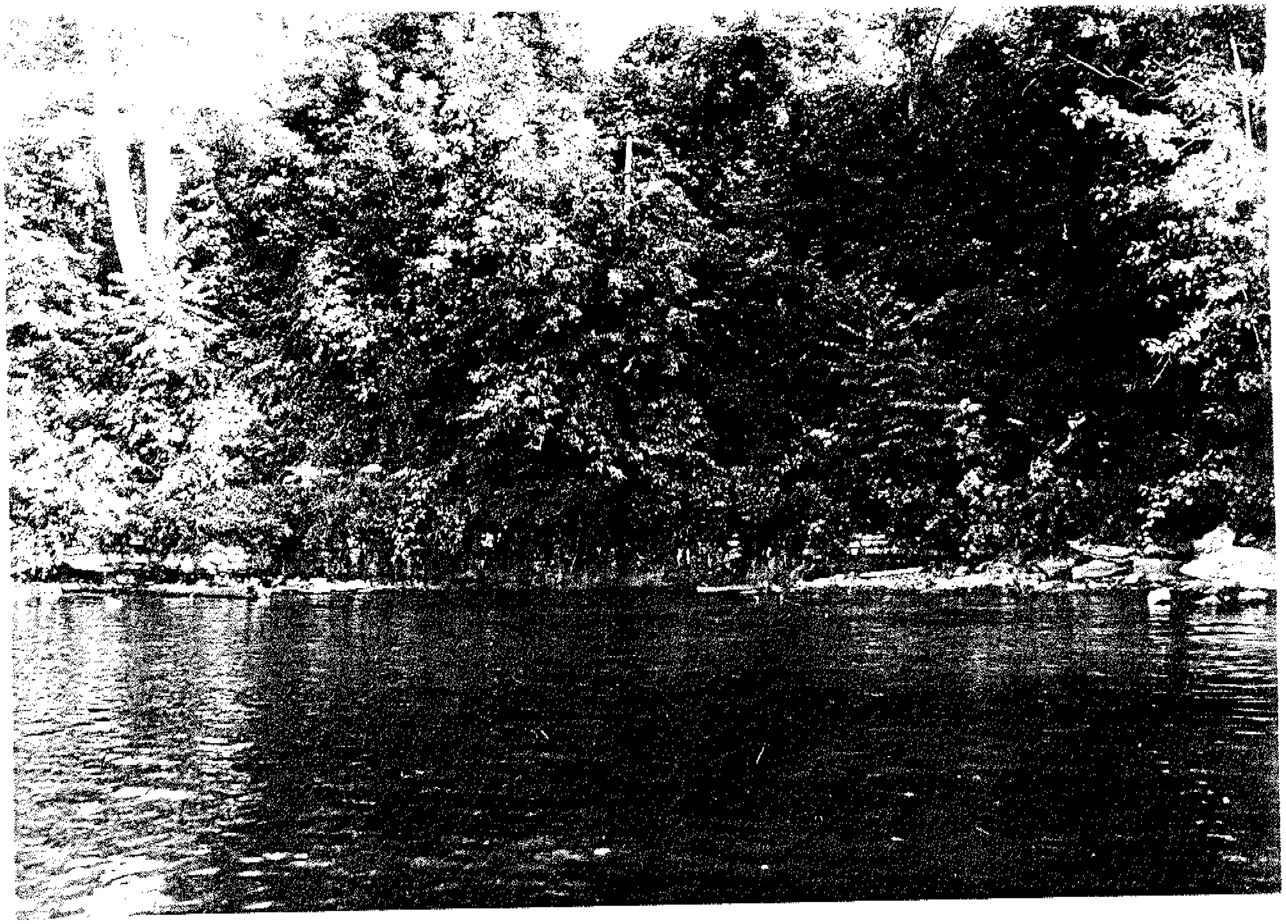
WO. 73-11053X

W.O. 73-11053X

W.O. 73-11053X

W.O. 73-11053X

W.O. 73-11053 X



W.O. 73-11053X

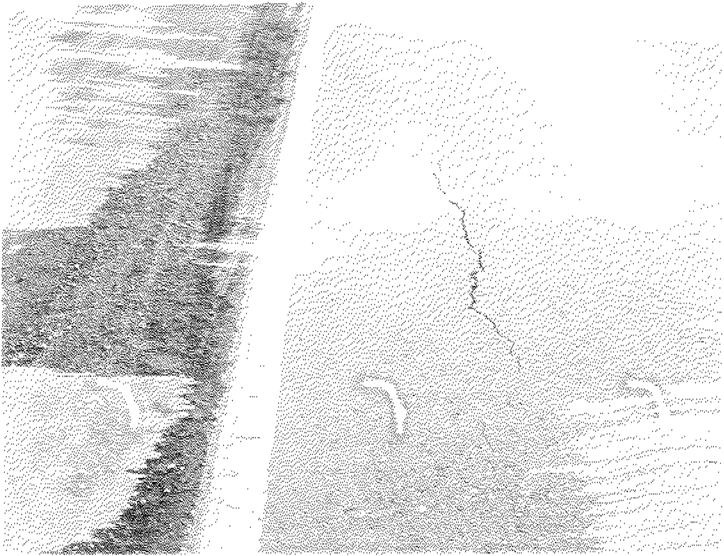
W.O. 73-11053X

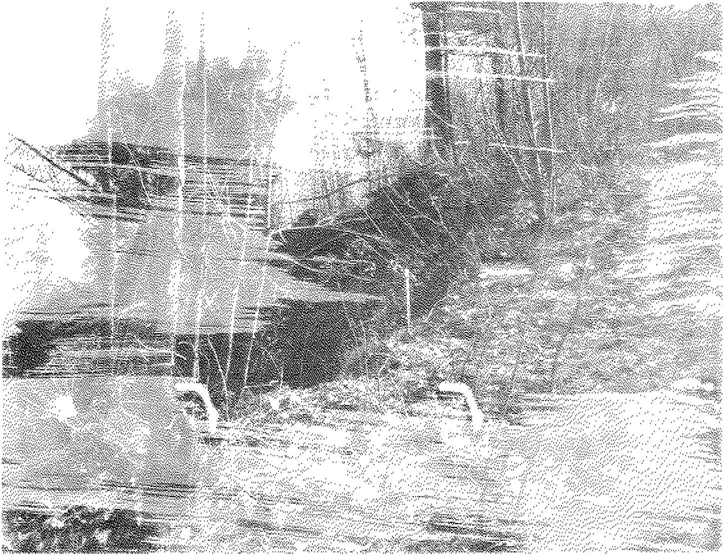
W.O. 73-11053 X

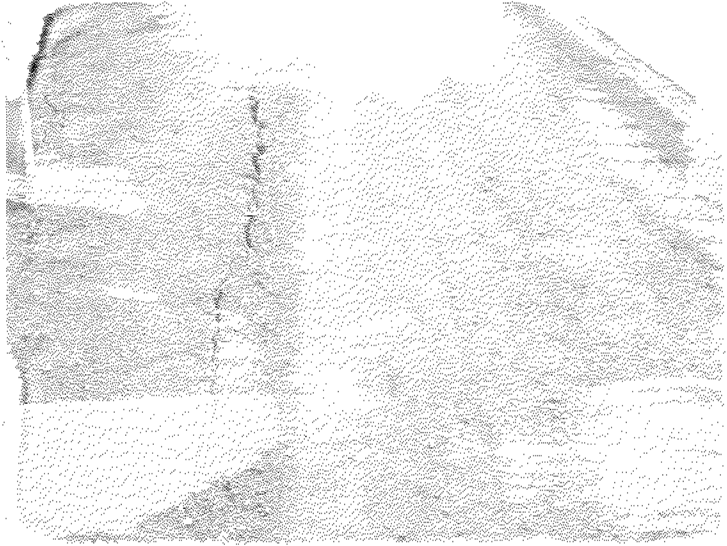
W.O. 73-11053 X

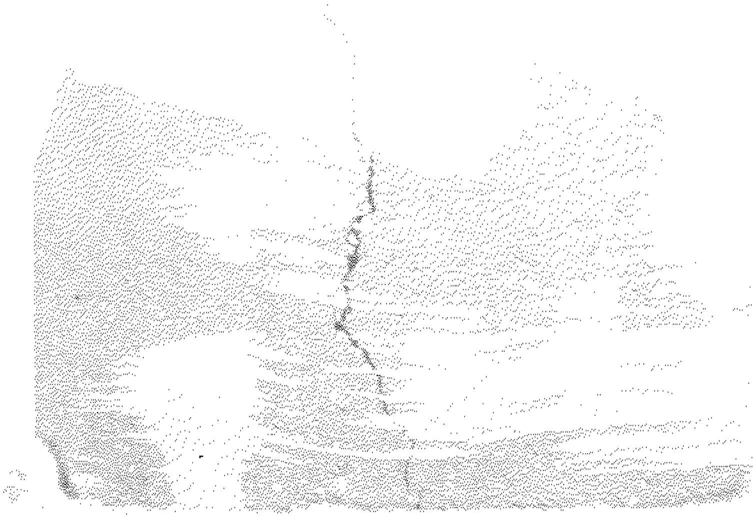
W.O. 73-11053 X

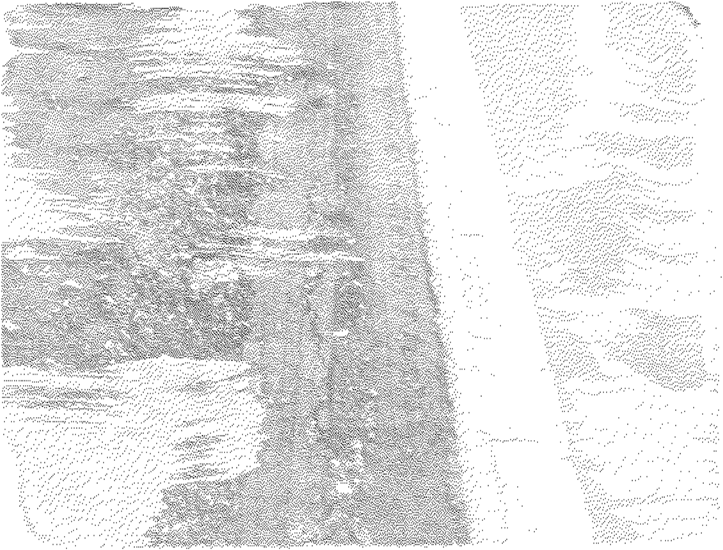
W.O. 73-11053X

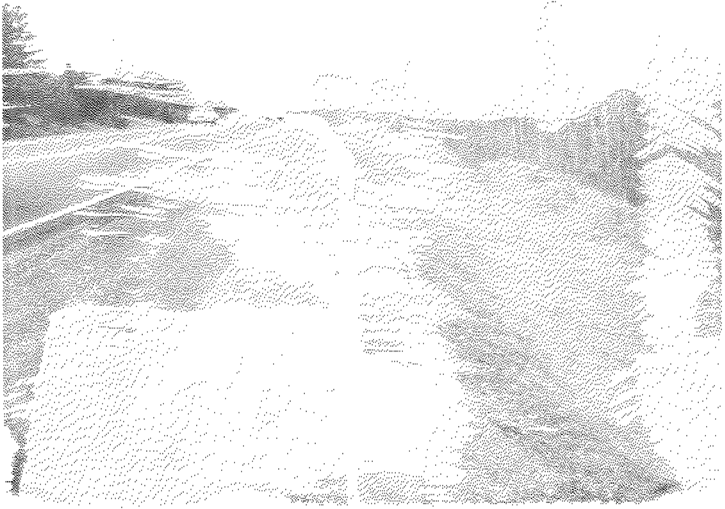












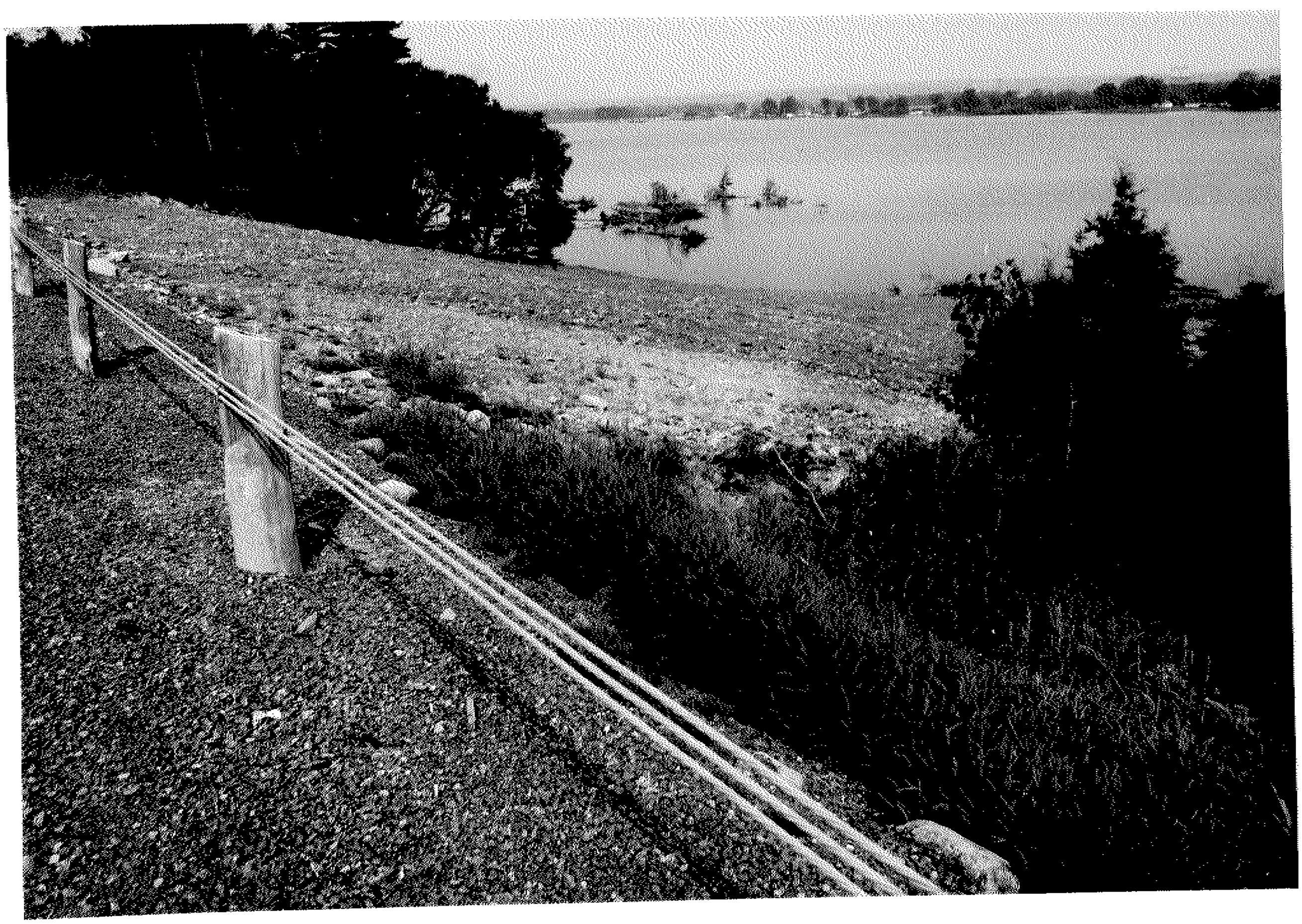
W.O. 73-11053 X

W.O. 73-11053 X

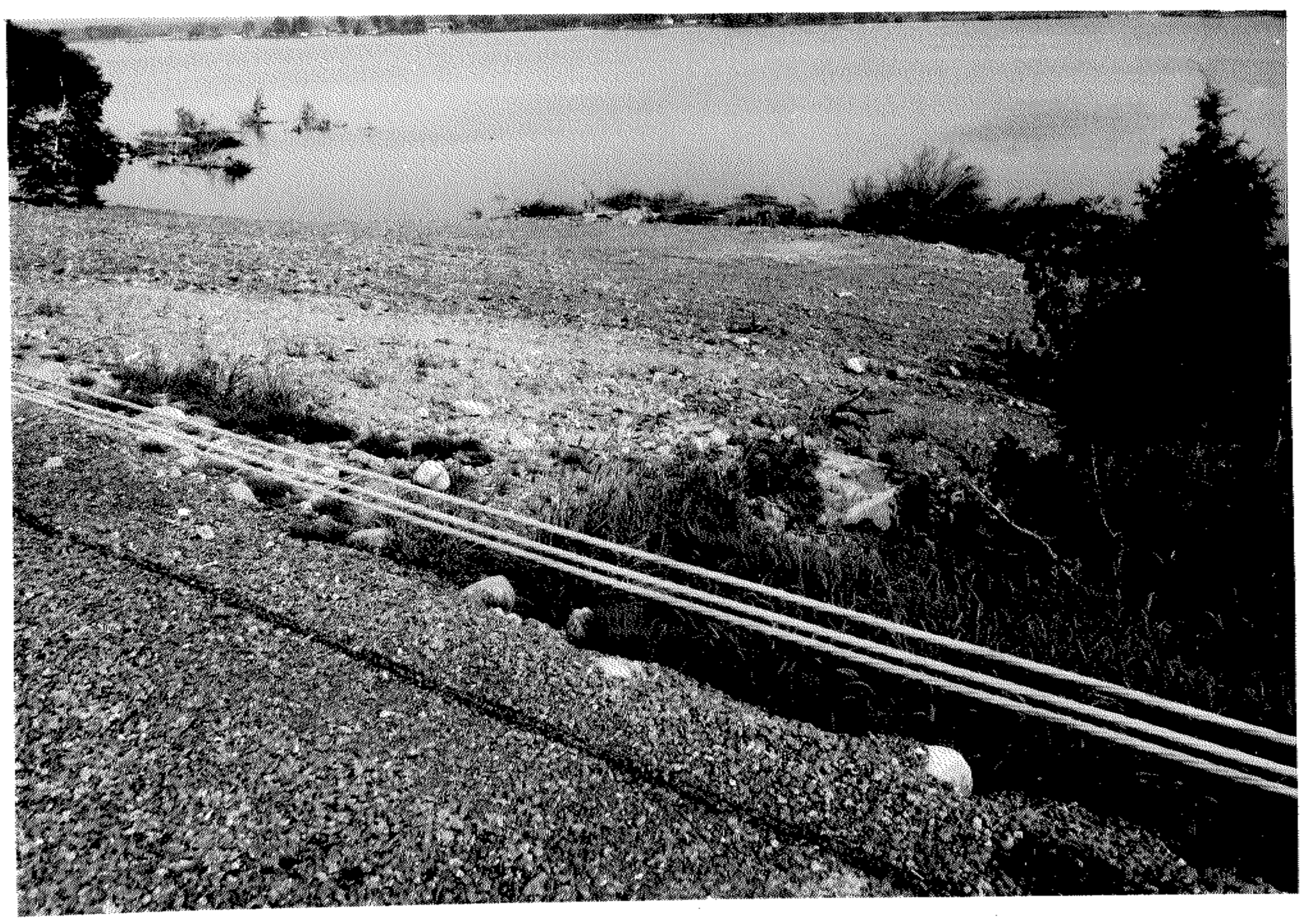
W.O. 73-11053 X

W.O. 73-11053X

W.O. 73-11053X



W.O. 73-11053X

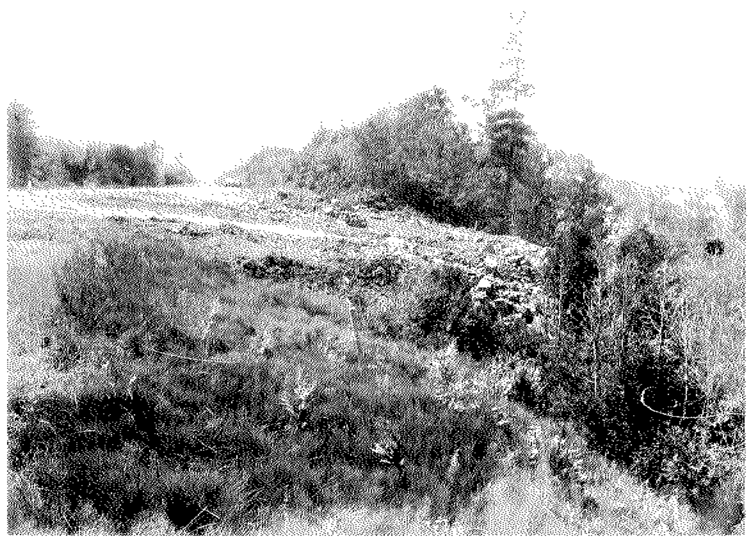


W.O. 73-11053X

W.O. 73-11053X

W.O. 73-11053X

W.O. 73-11053X



W.O. 73-11053X

W.O. 73-11053X

W.O. 73-11053X

W.O. 73-11053X

W.O. 73-11053X

W.O. 73-11053X

W.O. 73-11053X

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31G-165

W.P. No. 159-73-01

CONT. No. 74-121

W. O. No. 73-11053 (x)

STR. SITE No. N/A

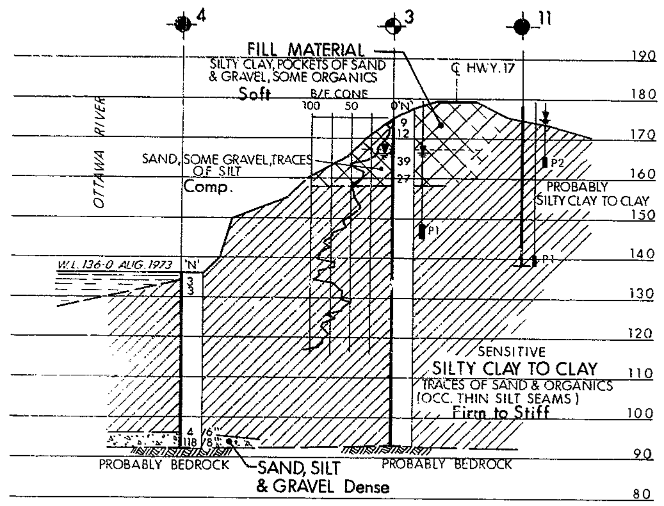
HWY. No. 17 DIST. 9

LOCATION SLOPE IMPROVEMENT 1.5 mi. W. to 2.0 mi. E. of CUMBERLAND 3.5 mi. VARIOUS locations, incl., a location at ROCKLAND W. Lts., 0.1 mi.

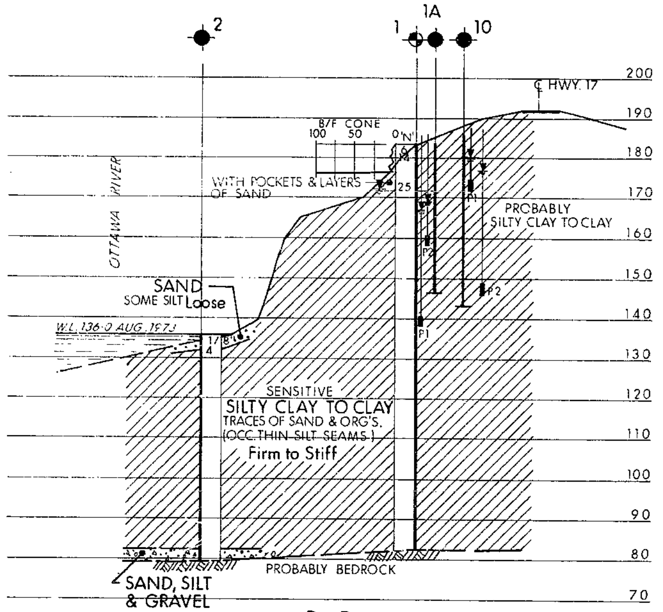
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. 2

REMARKS:

G.I.-30 SEPT. 1976



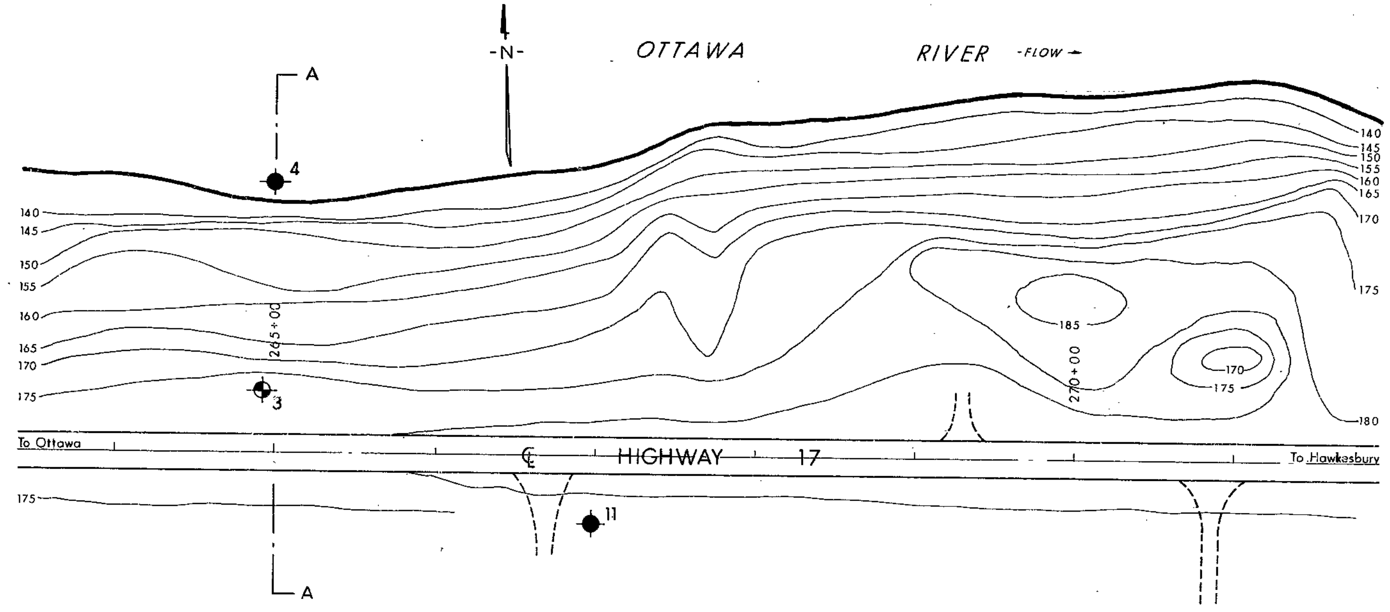
A-A



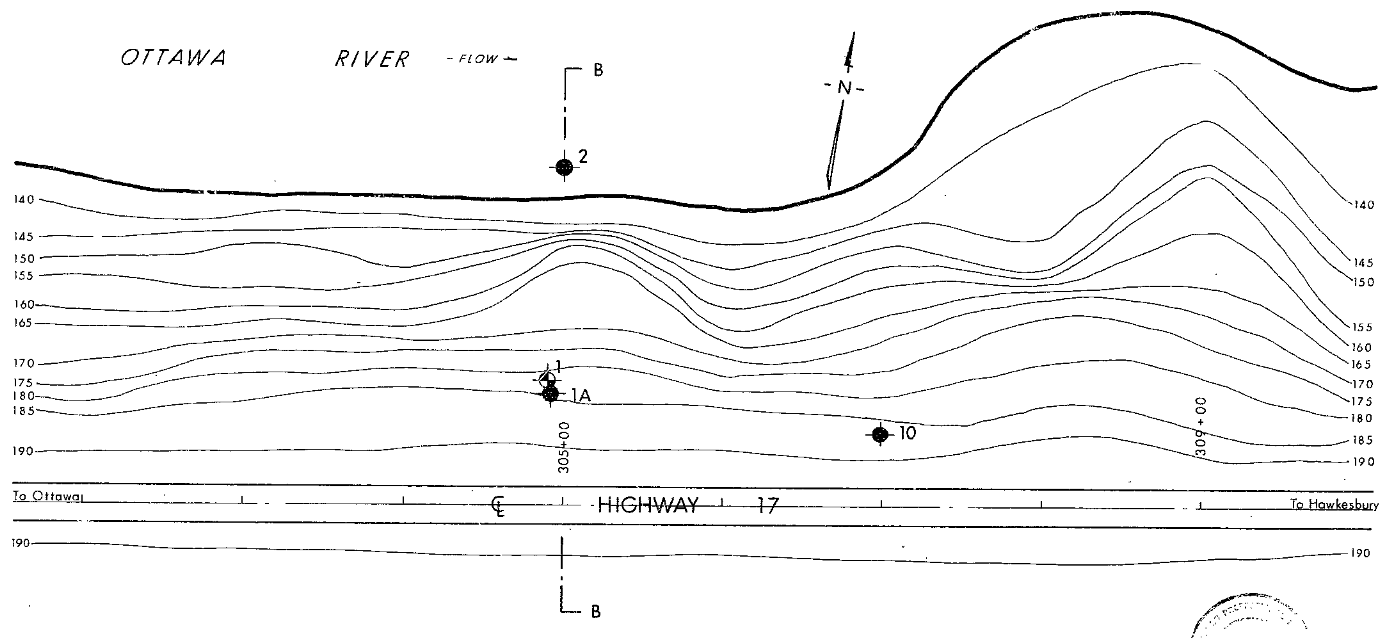
B-B

SECTIONS

VERT. 20 10 0 SCALE 20 40 FT.
HORIZ. 50 25 0 50 100



SITE 'A'

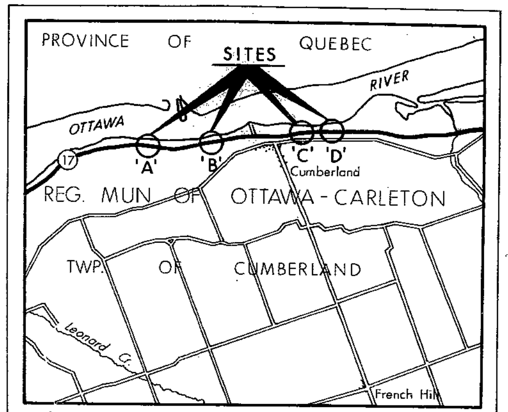


SITE 'B'

PLANS

50 25 0 SCALE 50 100 FT.

NOTE:
The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office, Downsview, and at the OTTAWA District Office.



KEY PLAN
SCALE IN MILES
1 0.5 0 1 2 3 4

LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation. AUG. 1973

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5	176·7	363+78	31' LT.
6	136·0	364+00	155' LT.
7	171·9	376+91	35' LT.
8	136·0	377+00	150' LT.
9	136·0	373+00	166' LT.
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REVISIONS	DATE	BY	DESCRIPTION

GEOCRES № 31G-165

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
DESIGN SERVICES BRANCH – FOUNDATIONS OFFICE

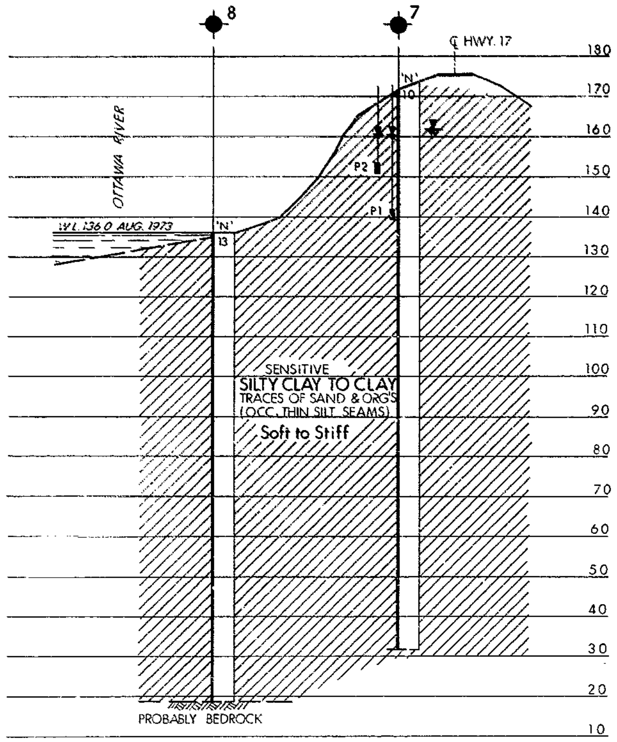
SLOPE STABILITY STUDIES
ALONG OTTAWA RIVER BANK
STA. 265+00 TO STA. 309+00

HIGHWAY NO. 17 DIST. NO. 9
~~CO.~~ REG. MUN. OF OTTAWA - CARLETON
TWP. CUMBERLAND LOT CON.

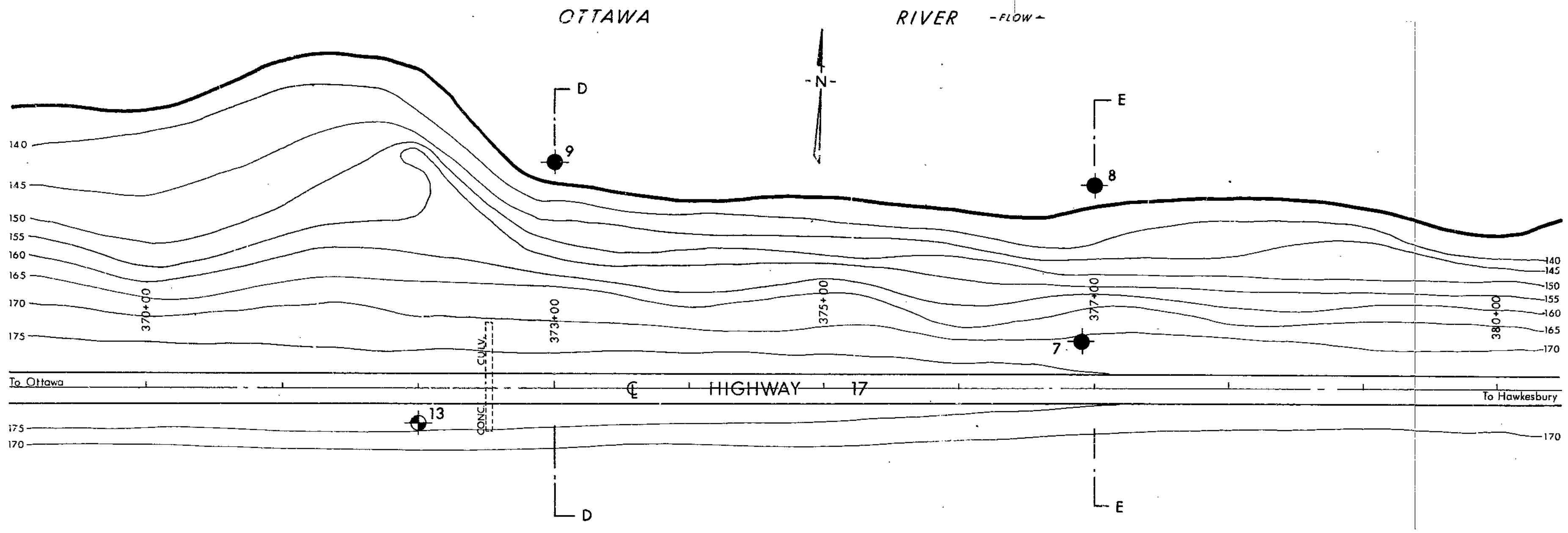
BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. C.P.	CHECKED	W.P. NO. —	DRAWING NO. 73-11053 A
DRAWN S.O.	CHECKED	W.O. NO. 73-11053X	
DATE 6 NOV. 1973		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

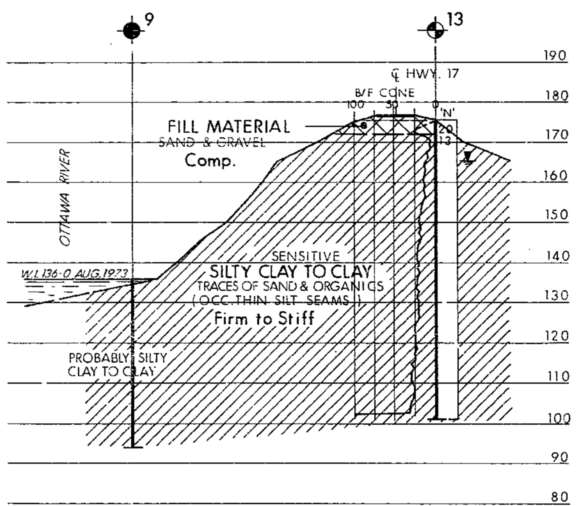
SNO-TRACE 300-8-72



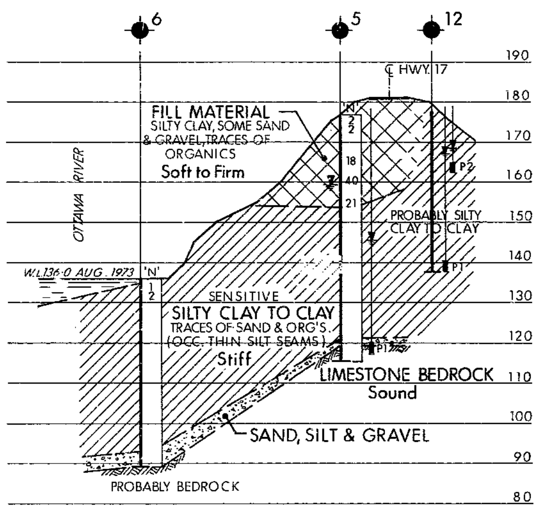
E - E



SITE 'D'



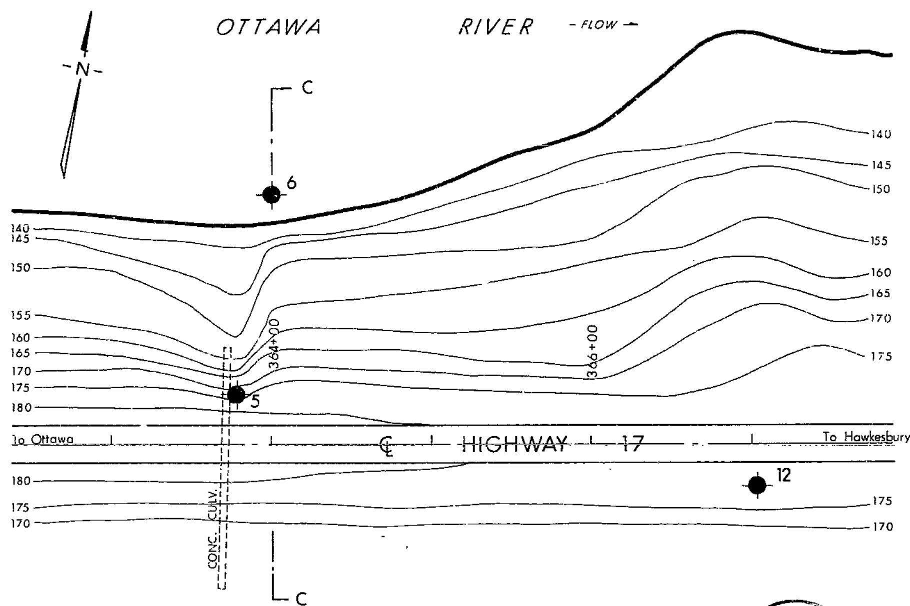
D-D



C-C

SECTIONS

VERT. 20 10 0 SCALE 20 40 FT.
HORIZ. 50 25 0 50 100



SITE 'C'

PLANS

50 25 0 SCALE 50 100 FT.

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REVISIONS		
DATE	BY	DESCRIPTION

GEOCRES Nº 31G-165

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
DESIGN SERVICES BRANCH—FOUNDATIONS OFFICE

SLOPE STABILITY STUDIES
ALONG OTTAWA RIVER BANK
STA. 361+00 TO STA. 380+00

HIGHWAY NO. 17 DIST. NO. 9
~~CO.~~ REG. MUN. OF OTTAWA-CARLETON
TWP. CUMBERLAND LOT ___ CON. ___

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD. C.P.	CHECKED	W.P. NO. —	DRAWING NO. 73-11053 B
DRAWN S.O.	CHECKED	W.O. NO. 73-11053X	
DATE 6 NOV. 1973		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

C.S.L.P.