

*Ken*  
*Jan 25 1962*  
*dg*

Mr. A. G. Stermac,  
Prin. Foundation Eng;  
Lab. Bldg., Room 107,  
Downsview:

Bridge Division,  
January 24, 1962.

MEMORANDUM TO:

Mr. A. Gray,  
Grades Supervisor,  
Department of Highways,  
Road Design Section,  
Administration Building,  
Downsview, Ontario.

RD: W.P. 916-59  
Kent Street Bridge #20  
Hwy. O.Q. District #9

Attached please find one print of  
Preliminary drawing D 5002-P-1 of the above  
mentioned structure for your use.

Two additional prints are being  
mailed to the Principal Foundation Engineer for  
soils confirmation.



R. Fitzgibbon,  
Bridge Engineering Expediter.

RF/zf

c.c. A. G. Stermac

OFFICE LOCATION -  
DOWNSVIEW AVE.,  
KEELE ST. - HIGHWAY 401  
TORONTO, ONTARIO.



ONTARIO  
DEPARTMENT OF HIGHWAYS

POSTAL ADDRESS -  
DEPARTMENT OF HIGHWAYS  
PARLIAMENT BUILDINGS,  
TORONTO 2, ONTARIO.

Bridge Division,  
January 18, 1961.

MEMORANDUM TO:

Mr. L. Soderman,  
Principal Soils &  
Foundations Engr.,  
Department of Highways,  
Room 107, Lab. Bldg.,  
Downsview, Ontario.

RE: W.P. 946-59,  
Ottawa Queensway Br. #20,  
Kent St. Overpass, Dist. #9.

Herewith one copy of the preliminary foundation  
report BA 1133, for the above structure.

A handwritten signature in cursive script, appearing to read "F. I. Hewson".

FJW:go

F. I. Hewson,  
Consultant Liaison Engineer.

BA 113

# MCROSTIE & ASSOCIATES LTD.

CONSULTING ENGINEERS

OTTAWA 1

CANADA

G. C. MCROSTIE, B.A.S.C., O.L.S., P. ENG., M.E.I.C.  
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393 BELL STREET  
TELEPHONE CE. 2-5334

## KENT STREET & QUEENSWAY

### 1. TERMS OF REFERENCE

We were requested by the Ottawa Office of De Leuw, Cather & Company of Canada Limited to carry out a first stage of a foundation investigation at the site of a structure which would carry the Queensway over Kent Street. A preliminary borehole was to be made so that alternatives of structure type as well as foundation type could be studied.

### 2. CONCLUSIONS & RECOMMENDATIONS

#### 2.1 Foundation Type

At a meeting on August 24th, 1960, attended by Mr. Davis, Mr. Soderman and Mr. Marshall, it was decided that one more borehole should be made at the site. A decision on type of structure had not been finalized at that time but the type of foundation for each of the possible structure alternatives was considered. It was felt that the total and differential settlements induced in the 20 feet of clay beneath the structure and embankment could likely be tolerated. The shear strength of the clay at points of maximum stress such as at the toe of retaining walls or rigid abutment bases would however determine the feasibility of footing type foundations.

If footing foundations were found impracticable<sup>al</sup>, then end bearing piles resting on the rock or dense till at

depths of about 40 feet would have to be used. Additional site investigation work plus studies of a super structure type will provide the material necessary for a decision on foundation type.

### Soil Strengths

- 2.2 For preliminary design purposes a bearing capacity of 3,000 pounds per square foot is recommended for the clays at about elevation 21<sup>4</sup>. It is hoped that by using field vane tests and laboratory triaxial tests, higher shear strength values may be confirmed in the remainder of the investigation but until such confirmation is obtained, the higher values should not be finalized.

### 2.3 Soil Compressibility

A laboratory consolidation test was performed on sample No. 1-4 at a depth of 21 feet and a probably preconsolidation pressure of 5.8 kips per square foot is indicated from the results of the test when performed using routine loading schedules. The loads added to the clay layer by the 15 foot embankment and the new structure will therefore be within the precompressed range. Total settlements of the order of some tenths of a foot and differential settlements of the order of a few tenths of a foot can be roughly estimated by comparison with the settlement studies of adjacent structures such as Metcalfe at Queensway. The pattern of distribution of these settlements can be set out in the next stage of this report and they should be re-examined to see that they are still realistic when the final super-structure type and embankment

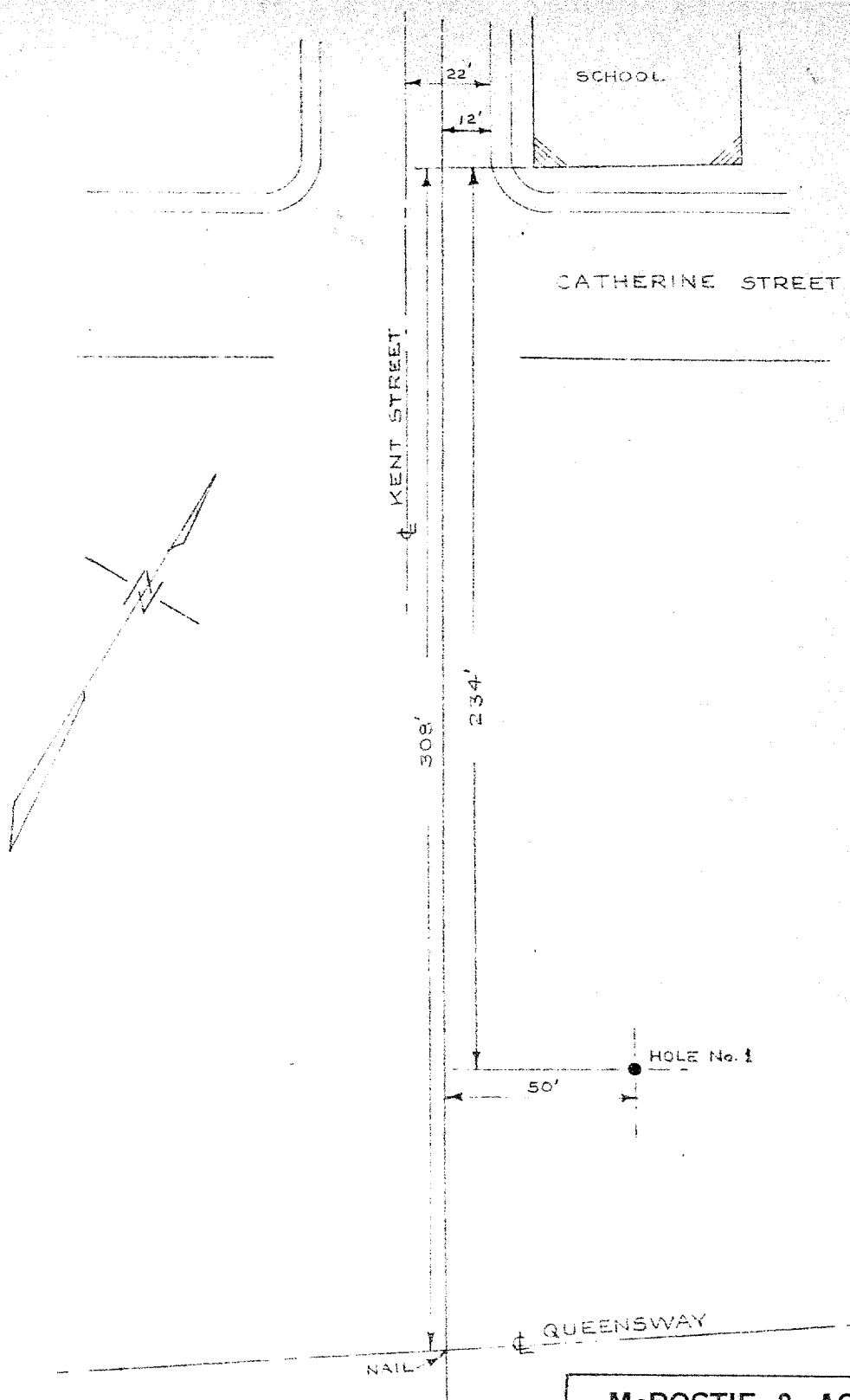
arrangement is known.

### 3. SITE INVESTIGATION

One preliminary borehole was made at the northeast corner of the proposed easterly abutment using our test drilling equipment. Two-inch split barrel samples were taken and the standard penetration test performed in the boreholes through the non-cohesive soil layers and two-inch thin wall samples were taken from the cohesive soil layers. Rock beneath the site was diamond drilled and cores recovered for inspection and logging, during the drilling a careful watch was kept for drops of the drill rods and discontinuities in the drilling so that the structure of the rock might be evaluated.

Laboratory testing included classification tests on representative samples plus one consolidation test at about mid height of the compressible clay layer. Unconfined compression tests and small scale penetrometer tests on the clay samples were also carried out. Our comments on the usefulness of the unconfined test and the penetrometer test on Page No.<sup>10</sup> of our Report No. SF-487, Metcalfe & Queensway are also applicable to testing at this site.

Soil and rock conditions are detailed on Plate 2 attached but can be generalized as consisting of about 7 feet of fill, 3 feet of natural sand, then medium soft to stiff clay to a depth of about 34 feet. Beneath the clay is dense silt and till to rock at about 42 feet. The rock is a shaley limestone of the Eastview Formation. Groundwater was observed in the sand layer about 7 feet below ground surface and while this would usually be considered to be the seasonal low level, the depth of sewers on adjacent municipal streets tend to control the groundwater levels and little fluctuation is to be expected.



**McROSTIE & ASSOCIATES**  
**CONSULTING ENGINEERS**

**BOREHOLE LOCATIONS**  
**KENT ST. AT QUEENSWAY**

**SCALE** 1" = 40'

**PLATE . 1**





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CONSULTING ENGINEERS  
OTTAWA 1

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23-62-124-2  
393 BELL STREET  
TELEPHONE CE. 2-8334

## REPORT ON THE SUBSURFACE INVESTIGATION FOR THE PROPOSED STRUCTURE AT EAST STREET AND THE OTTAWA QUEENSWAY

WP. 946-59

### 1. INTRODUCTION

We were requested by the Ottawa office of Be Lemw Cather & Company of Canada Limited to carry out a subsurface investigation to determine the condition of the subsoil at the site of a proposed structure intended to carry the Queensway over East Street. Design recommendations on the structure, wing walls and retaining walls running east and west were to be included in the final report. A preliminary report on foundation conditions based on a pilot borehole was submitted in October 1960. This report includes final recommendations pertaining to the structure and retaining walls at this site. Many analyses contained in this report are similar to the ones contained in our draft report No. SF-582 A, Bank Street and Queensway; much of the data has been reproduced in this report for completeness.

### 2. CONCLUSIONS AND RECOMMENDATIONS

#### 2.1 Structure

For the proposed structure at this site we recommend an end bearing pile foundation bearing on the cherty limestone rock at about elevation 182. H-piles would be a practical type in view of the boulder content in

the glacial till overlying bedrock at this location. End bearing H-piles are also recommended for the support of the wing walls of the structure.

Since preloading of the underlying cohesive strata is not feasible at this site, consolidation of the compressible layer under the adjacent embankment weight will cause an additional load on the piles by vertical friction. This load will be distributed through the thickness of the compressible layer along the piles. The maximum value of the load due to an estimated cohesion of 1000 p.s.f. is about 15 tons per 12" H-pile and this load would be acting at the base of the pile.

A footing type of foundation was considered and rejected for the structure at this site. Since the level of the future Kent Street is proposed at elevation 112, footings would have had to rest on the softer clay soils near elevation 207 for frost protection. The subsurface investigation revealed that an allowable bearing capacity of only 3000 p.s.f. could be recommended at about elevation 207 whereas 4000 p.s.f. could have been used at some higher elevation. This fact would cause an appreciable increase in the footing width and higher costs would result. However, because elevation 207 was also required for frost protection for pile caps, the length of end bearing piles to bedrock was appreciably reduced. A cost comparison including only the extra structural materials and extra excavation showed a difference of a few thousand dollars in favor of piles. Considering also the added structural difficulties and construction problems with a footing type of foundation because of a considerable slope in the structure at this site, a pile foundation is felt to be more economical.

The stability of the four wing walls adjacent to the structure could be difficult to achieve with a footing type of foundation since the wing walls would be about 30 feet high (footings at el. 207 and finish grade at el. 237) at the abutments. Consequently the design and construction difficulties of such a high wall would render a footing type of foundation uneconomical at this site. Furthermore, in view of the pile type of foundation recommended for the structure at this location, it would be practical to continue the pile support at least under the adjacent wing walls.

## 2.2 Foundation of retaining walls lengthening the structure wing walls

A pile type of foundation is recommended for the sections of retaining walls contiguous with the north and south wing walls of the eastern abutment. Because a pile foundation under the wing walls would be stepped upwards (to minimum frost cover requirement) in order to reduce the height of walls, sufficient lateral support could not be developed, for an adjoining footing scheme, below the wing wall extensions. Consequently, in view of the complexity of the pile support under the eastern abutment and wing walls and because the addition to the wing walls is short, the pile foundation should be continued through, under the north and south retaining walls of the eastern abutment.

A footing type of foundation is recommended for the retaining wall adjoining to the north wing wall of the western abutment. Since the finish grade at the base of the wall will be at approximately elevation 225, footings could bear on the medium dense silty sand near elevation 220. A net allowable bearing value of 3,000 p.s.f. is

recommended for the design of retaining wall footings at this location. With this choice of bearing value, immediate settlements through the granular stratum should be insignificant since the amount of permissible settlement is the basis for allowable bearing values in granular soils. The effect of the present overburden, influencing the number of blows per foot in the standard penetration test, has been considered following Gibbs and Holtz theory. Consequently the density and hence the strength value of the silty sand stratum recommended as support for the retaining wall in this area has been substantially increased from the values derived from the usual interpretation of the number of blows per foot determined with the standard penetration resistance test.

The retaining wall contiguous with the south wing wall of the western abutment should be designed on a pile type of foundation. This foundation type is recommended since the wall will be appreciably high throughout its entire length mainly due to the grading scheme resulting from the necessary low elevation (212) of the future Kent Street at this location. Since footings would have to be near elevation 207 for minimum frost protection and because the Queensway finish grade is proposed at elevation 237 at this site the retaining wall would be 30 feet in height. At elevation 207 a bearing capacity of only 3,000 p.s.f. could be recommended due to the existence of a softer clay deposit at this depth (17 feet below existing ground level) underlying the upper stiff clay crust. Difficulties in construction and design introduced because of the aforementioned reasons would render a footing type of foundation uneconomical at this site.

### 2.3 Stability of retaining wall's considering a deep-seated Shear Failure

A stability analysis following the critical circle method was made for a section through the south west corner of the structure near borehole No.2. The results of this study show that a deep-seated shear failure should not occur and that the wall and embankment should be stable. Adequate factors of safety against a foundation failure can be developed by the shear strength of the cohesive soils at this site.

A further check was made in the stability analysis on the factors of safety considering a 25% reduction in undrained shear strength. This feature may or may not be applicable depending on timing and procedure of construction. However, with such a reduction in undrained shear strength, factors of safety in excess of the accepted 1.5 were determined and consequently the wall and embankment should remain stable under these conditions. Furthermore, considering the possibility of zero shear strength through the granular embankment due to possible cracks, adequate factors of safety against a deep-seated shear failure were again determined. Under these severe assumptions the wall and embankment remained stable. The results of the stability analysis are shown on Plate No. 18.

### 2.4 Soil Compressibility

The total settlement of the embankment at the structure will be a differential settlement between the embankment and the structure. The total long term settlement of the 10-foot embankment due to the underlying compressible

cohesive layer was calculated at this site and the result, reduced according to our experience of actual settlement observations of embankments overlying similar compressible subsoil, shows insignificant settlement. This conclusion is confirmed by the results obtained from a compressibility analysis made for a neighbouring structure (Metcalf Street and Queensway - Report No. SP-487) where a 20-foot embankment surcharge was placed over similar preconsolidated clay soils.

The immediate settlement through the bearing silty sand stratum used as support for the north-west retaining wall at this site should be insignificant as discussed in paragraph 2.2 above. Furthermore, consolidation settlement of this retaining wall through the underlying compressible clay layer was calculated and the result, reduced according to our experience, indicates that a differential settlement along the wall will be insignificant. However, the total long term consolidation settlement, evident only at the joint between the structure and the retaining wall, is estimated to be of the order of 1 inch. This fact should be incorporated in the detailing of this joint.

## 2.5 Construction Precautions

Groundwater levels were observed to be near the recommended bearing elevation of the north-west retaining wall and therefore lowering of the groundwater table is likely to be necessary during construction of the wall footings. Because the bearing stratum is a silty sand in a medium dense condition, the preservation of the in-situ density during construction is of prime importance. Groundwater should not be permitted to

- 7 -

percolate through the bearing layer; a trench or sump, excavated to below the footing level and surrounding the footing area, could be used to lower the groundwater below the bearing elevation. A preferable outlet for the dewatering trench is any available storm sewer but if a piped outlet is not available pumps operating from the sump 24 hours per day could control the groundwater level. The sump trench should be excavated alongside the footing area at a slope no greater than 2 horizontal to 1 vertical in order to preserve sufficient lateral resistance in the soils adjacent to the wall footings. Care should also be exercised to keep construction traffic to a minimum on the footing area.

Any variation in subsail conditions between boreholes detected during construction should be brought to the immediate attention of the supervising authority for appropriate action.

### 3. SITE INVESTIGATION

#### 3.1 Field Work

Nine boreholes were made at the site with our test drilling equipment in the locations shown on Plate No. 1. Fifty-two two-inch split barrel samples were taken and standard penetration tests performed in the boreholes through granular soil layers encountered; twenty eight thin wall tube samples were retrieved from cohesive soil layers. Five thin wall tube samples were taken with a stationary piston sampler in an effort to recover less disturbed samples for triaxial testing. The sampling was done at 5-foot intervals through the cohesive soil strata and generally at 2½-foot intervals in the cohesionless soil layers from about 2½ feet below existing ground surface down to bedrock. Rock beneath the site was diamond drilled and cores were recovered for inspection and logging. A careful watch was kept for drops of drill rods and discontinuities in the drilling so that the structural strength of the rock might be evaluated. All samples were brought to our laboratory to be examined.

Borehole vane tests were made in five borings to evaluate the in-situ shear strength of the cohesive soils. These tests were made at 5 foot intervals alternating with the thin wall tube sampling throughout the cohesive soil strata. Overnight groundwater levels were observed and recorded during the borings.

#### 3.2 Laboratory Testing

A consolidation test was made on sample 1-4 retrieved at a depth representing the mid height of the compressible layer. Unconfined compression tests were made on samples

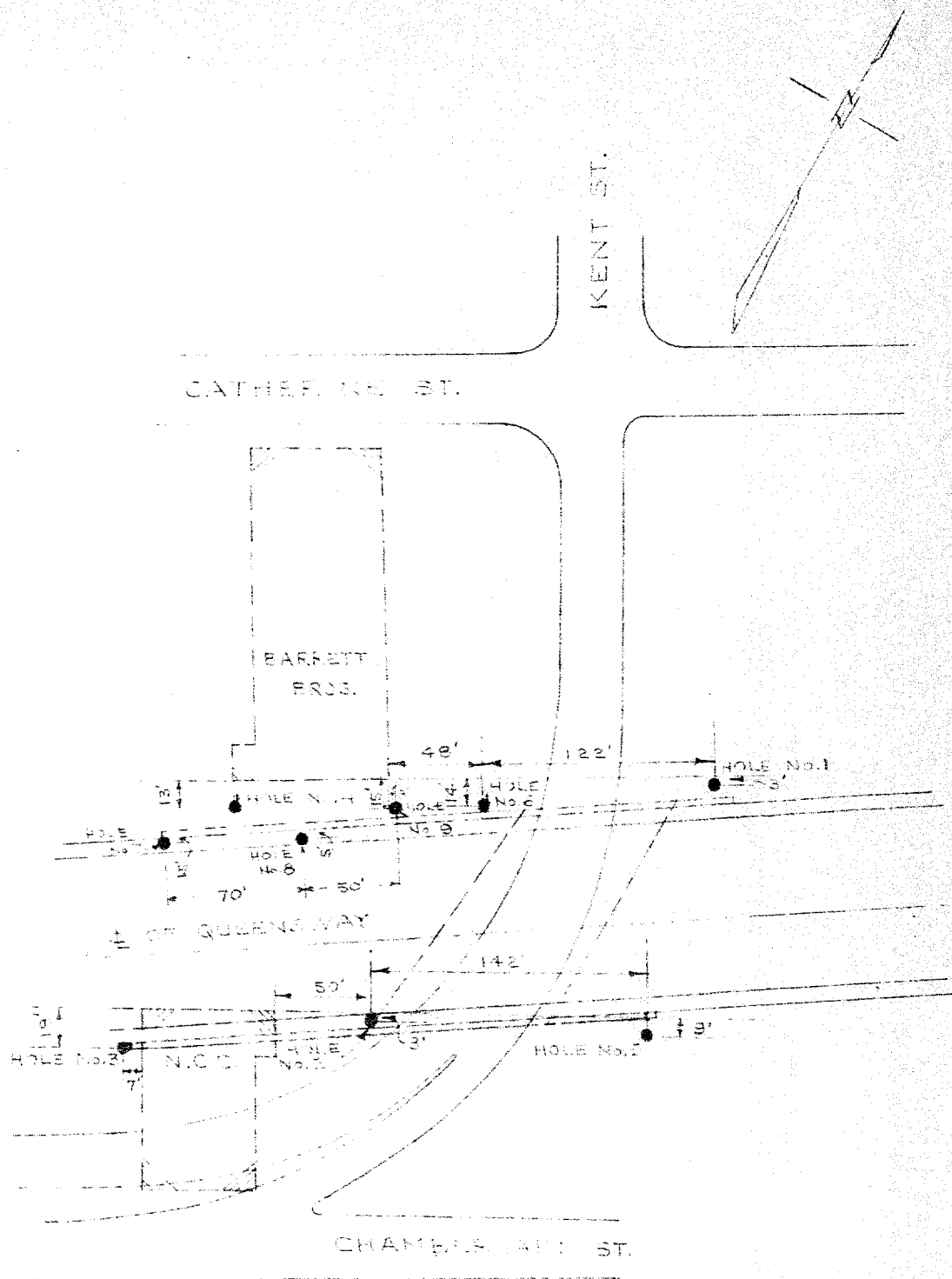


recovered in the pilot borehole No. 1 as part of a preliminary strength determination program. Small scale penetrometer tests (soiltest type) were made at six-inch intervals in each tube to determine the shear strength variation in a vertical direction and to verify results from triaxial tests. Our comments on the usefulness of penetrometer tests on page 10 of our report no. SP-487, Metcalfe Street and Queensway are also applicable to testing at this site.

Thirty-seven unconsolidated undrained triaxial tests were performed on samples retrieved from the cohesive deposit in an effort to evaluate the shear strength of the soils within the critical depth and to confirm the shear strength values determined by other laboratory and field tests. Classification tests were also made on all samples.

### 3.3 Observations

The geotechnical profile of the subsoil and bedrock formations as revealed by each boring is shown on the accompanying Plates No. 2 to No. 70. The subsurface profile can be generalized as consisting of fill varying in thickness from about 2.5 to 7 feet, underlain by a layer of sand and silty sand about 3 to 5 feet thick overlying a medium soft to stiff clay layer between 20 to 34 feet thick. Beneath the clay are loose to dense layers of sand silt mixtures and till to rock at a depth varying between 32.0 to 42.0 feet. The rock is a shaley limestone of the Eastview formation. Groundwater levels observed in most boreholes varied between 4 to 11 feet below ground surface and while this would usually be considered to be seasonal low level, the depth of sewer trenches on adjacent municipal streets tend to control the groundwater levels and little fluctuation is to be expected.



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BOREHOLE LOCATIONS  
KENT & QUEENSWAY

SCALE 1" = 80'

PLATE 1

KENT AT QUEENSWAY

HOLE No.

1

R - REMOULDED

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## CONSULTING ENGINEERS

### OTTAWA CANADA

# SOIL PROFILE AND SUMMARY

## OF FIELD AND LABORATORY TESTS

KENT &amp; QUEENSWAY

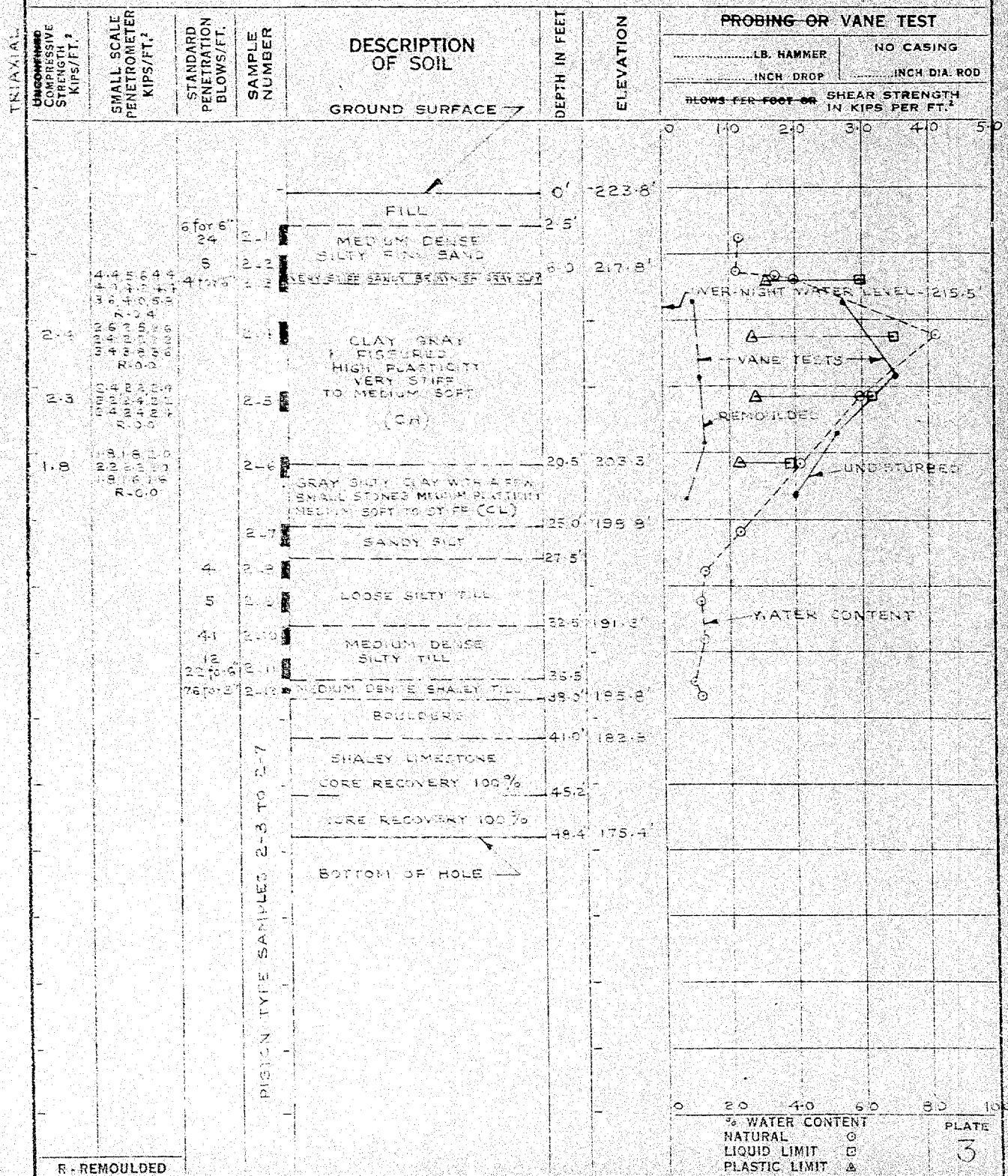
ELEVATION OF GROUND SURFACE (ZERO DEPTH) 223.5

DATE NOV. 27, 1961

HOLE NO.

REMARKS SEE PLATE No. 2

2







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### OTTAWA CANADA

# SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

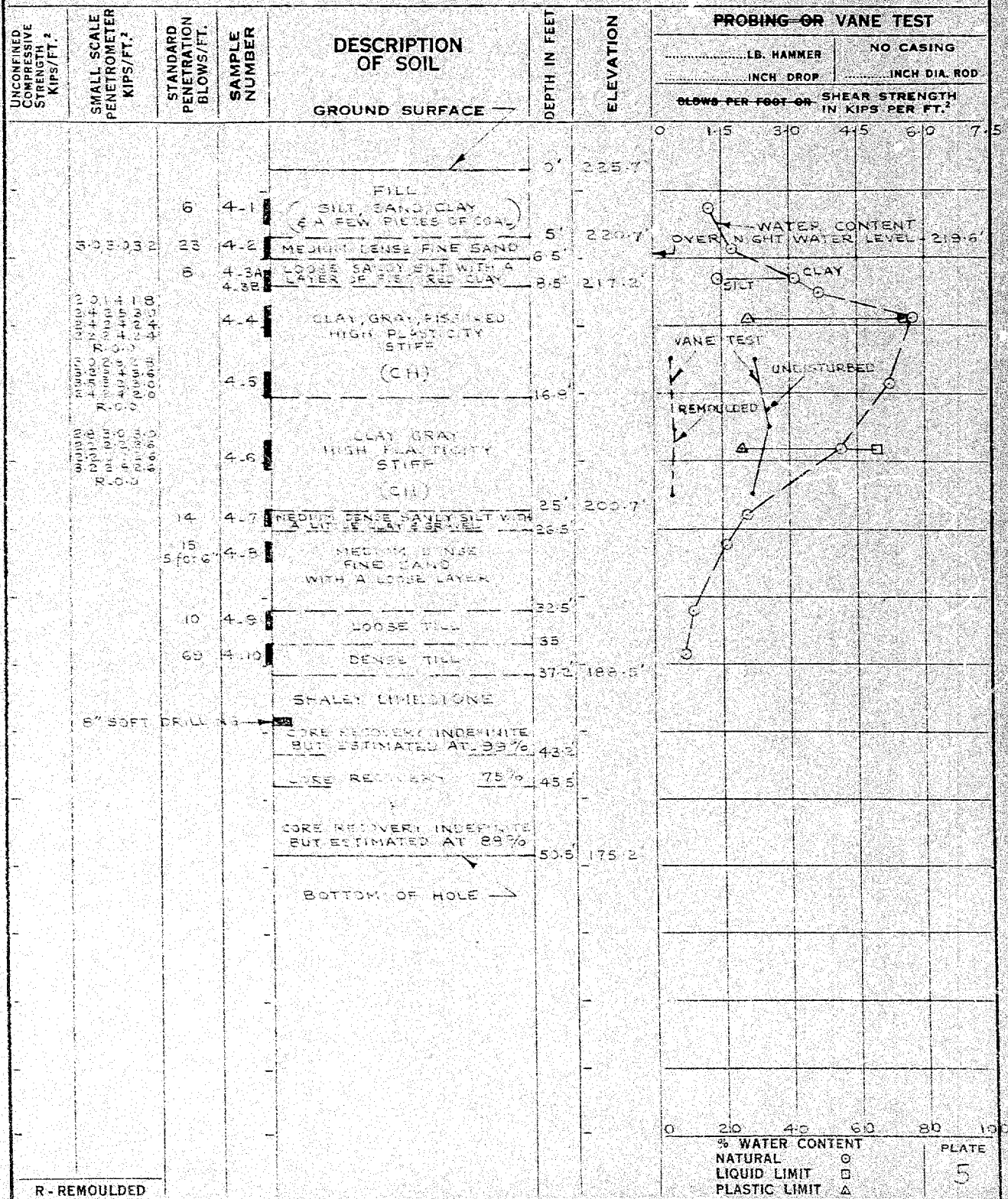
KENT &amp; QUEENSWAY

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 225.7' DATE JAN. 12, 1962

HOLE NO.

REMARKS SEE PLATE No. 2

4



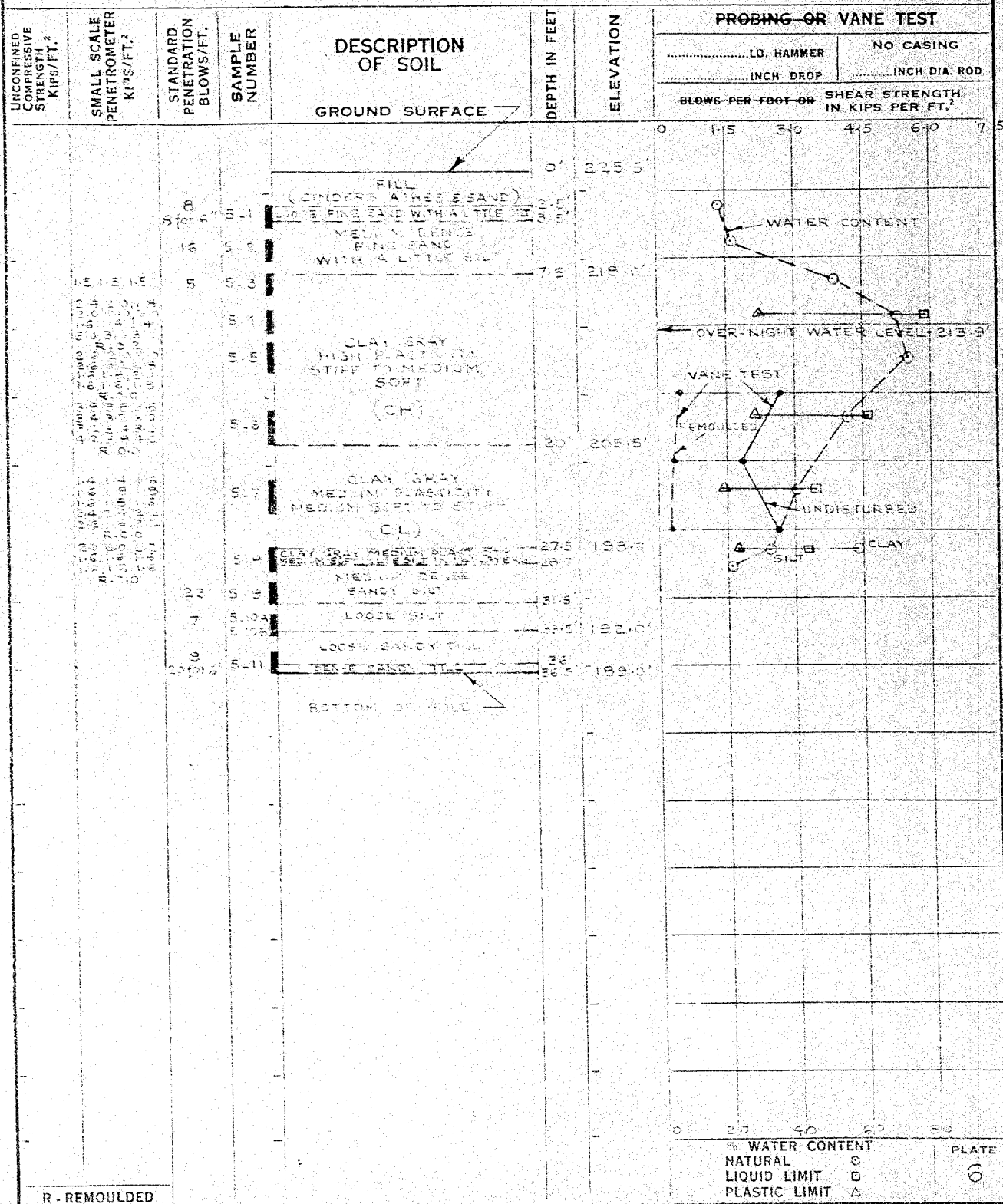
# McROSTIE & ASSOCIATES LTD.

## CONSULTING ENGINEERS

### OTTAWA CANADA

# SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

KEIT &amp; QUEENSWAY

 ELEVATION OF GROUND SURFACE (ZERO DEPTH) 225.5 DATE JAN. 4, 1962  
 REMARKS SEE PLATE No. 2
HOLE NO.  
5

# McROSTIE & ASSOCIATES LTD.

## CONSULTING ENGINEERS

### OTTAWA CANADA

# SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TEST'S

KENT &amp; QUEENSWAY

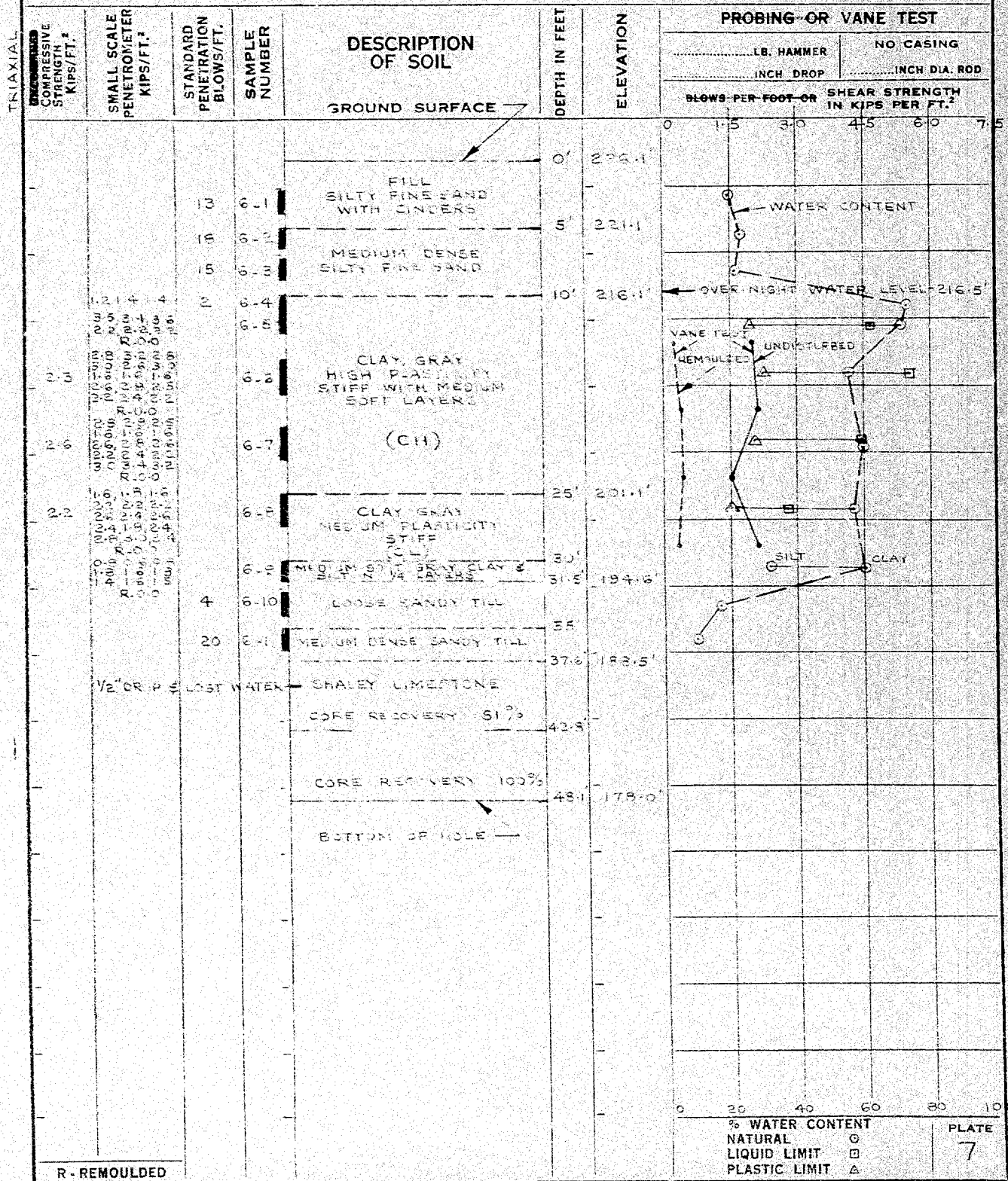
ELEVATION OF GROUND SURFACE (ZERO DEPTH) 226.1'

DATE JAN. 9, 1962

HOLE NO.

REMARKS SEE PLATE No. 2

6



R - REMOULDED





KENT & QUEENSWAY

8

**R - REMOULDED**

% WATER CONTENT	
NATURAL	⊙
LIQUID LIMIT	□
PLASTIC LIMIT	△

**PLATE**

7B



# TRIAxIAL COMPRESSION TEST

## TEST SERIES SUMMARY SHEET

PROJECT: E-439

SITE: KENT & QUEENSWAY

DATE: DEC 14/61

HOLE NO. 2

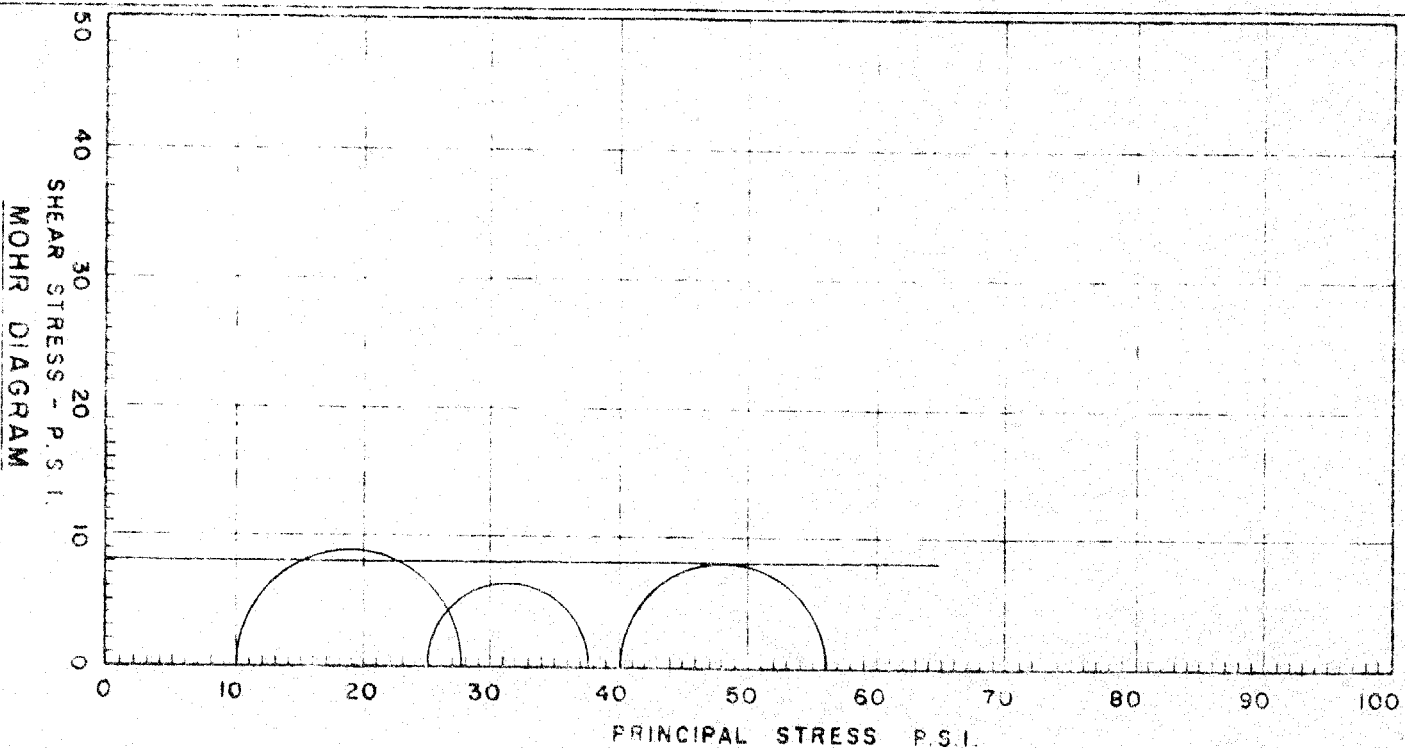
LOCATION:

DEPTH:

TEST NO.	2-4	2-4	2-4					AVERAGE
DEPTH								
LATERAL PRESSURE (PSI)	10	25	40					
COMPRESSIVE STRESS (PSI) - $q_c$	17.7	12.64	16.10					
WATER CONTENT - W %								
WET DENSITY - $\gamma_m$								

$C = 8.1 \text{ P.S.I.}$

$\phi = 0^\circ$



TESTED:

DATE:

COMPUTED:

DATE:

PLOTTED:

DATE:

CHECKED:

DATE:

MCROSTIE & ASSOCIATES  
CONSULTING ENGINEERS

MAR 14 1962

BR 822

PLATE NO. 5

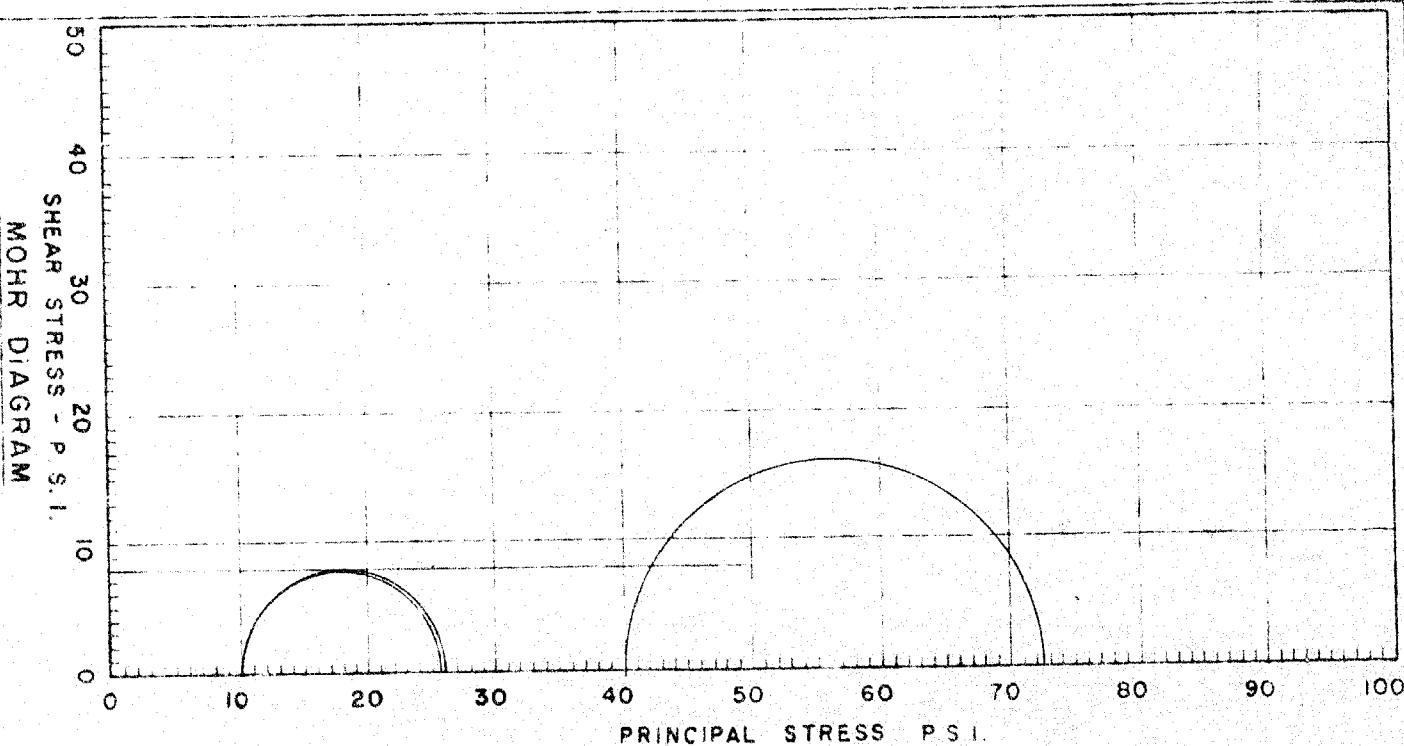
# TRIAxIAL COMPRESSION TEST

## TEST SERIES SUMMARY SHEET

PROJECT: E-438 SITE: KENT & QUEENSWAY DATE: DEC 14/61  
 HOLE NO. 2 LOCATION: DEPTH:

TEST NO.	2-5	2-5	2-5	AVERAGE
DEPTH				
LATERAL PRESSURE (PSI)	10	10	40	
COMPRESSIVE STRESS (PSI) - qc	15.6	15.9	32.4	
WATER CONTENT - W %				
WET DENSITY - $\gamma_w$				

$C = 8 \text{ P.S.I.}$   $\phi = 0^\circ$



HIGHLY PLASTIC CLAY (CH)

TESTED: DATE: \_\_\_\_\_  
 COMPUTED: DATE: \_\_\_\_\_  
 PLOTTED: D.M. DATE: 14-12-61  
 CHECKED: S.B. DATE: 14-12-61

MCROSTIE & ASSOCIATES  
 CONSULTING ENGINEERS

MAR 14/65

SR 822

PLATE NO. 3

# TRIAxIAL COMPRESSION TEST

## TEST SERIES SUMMARY SHEET

PROJECT: E-439

SITE: KENT & QUEENSWAY

DATE: DEC 14/61

HOLE NO. 2

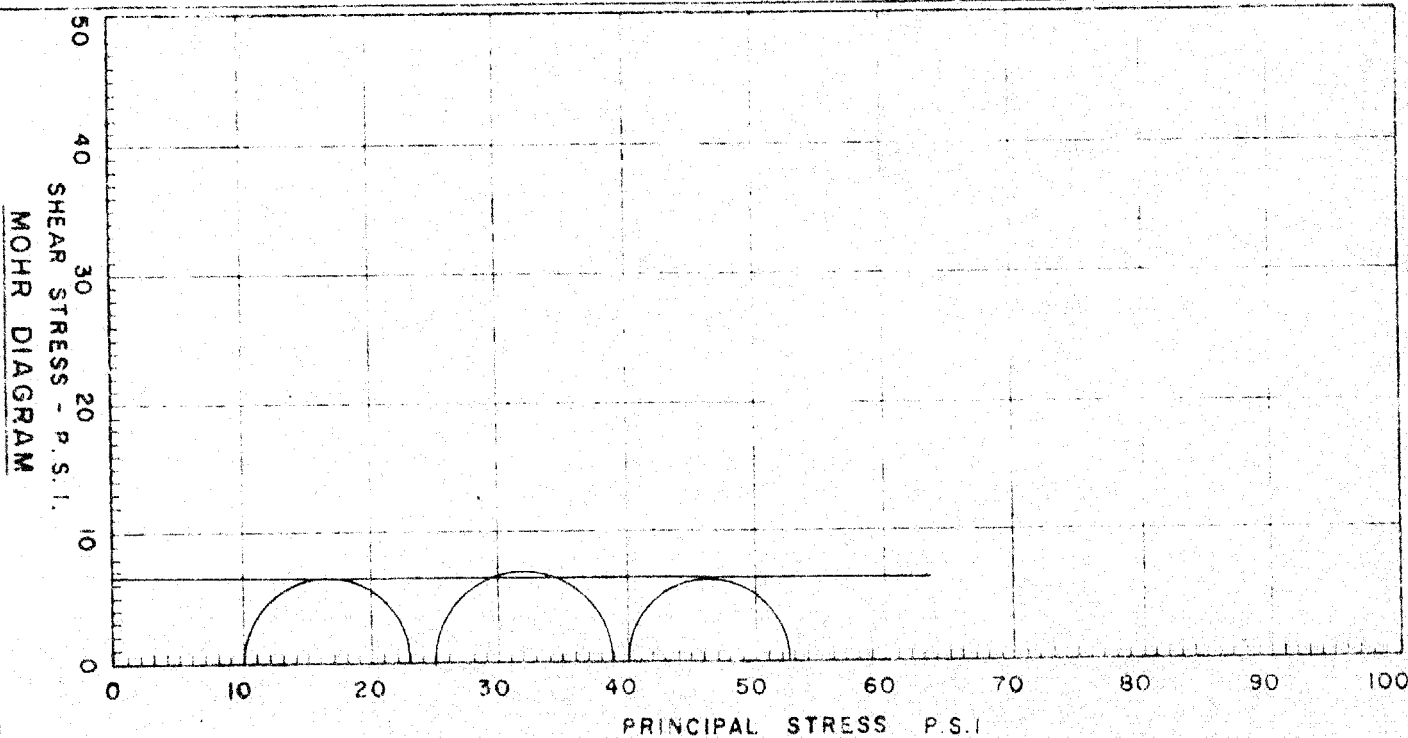
LOCATION

DEPTH:

TEST NO.	2-6	2-6	2-6						AVERAGE
DEPTH									
LATERAL PRESSURE (PSI)	10	25	40						
COMPRESSIVE STRESS (PSI) - $q_c$	12.9	14.0	12.6						
WATER CONTENT - W%									
WET DENSITY - $\gamma_m$									

$C = 6.4 \text{ P.S.I.}$

$\phi = 0.0^\circ$



MEDIUM PLASTIC SILTY CLAY (CL)

TESTED: \_\_\_\_\_ DATE: \_\_\_\_\_  
 COMPUTED: \_\_\_\_\_ DATE: \_\_\_\_\_  
 PLOTTED: D.M. DATE: 14-12-61  
 CHECKED: G.B. DATE: 14-12-61

MCROSTIE & ASSOCIATES  
 CONSULTING ENGINEERS

MAK 14/55

BR 822

# TRIAXIAL COMPRESSION TEST

## TEST SERIES SUMMARY SHEET

PROJECT: E-439		HOLE NO. 3		LOCATION:		SITE: KENT & QUEENSWAY		DATE: 20-1-62	
TESTED:		COMPUTED:		PLOTTED:		CHECKED:		DATE:	
TEST NO.		3-3A		3-3B		3-3C		3-3D	
DEPTH									
LATERAL PRESSURE (PSI)		40		10		40		10	
COMPRESSIVE STRESS (PSI) - qc		11.4		15.6		9.06		8.42	
WATER CONTENT - W %									
WET DENSITY - $\gamma_m$									
$C = 4.5 \text{ PSI.}$ $\phi = 0^\circ$									
<p>MOHR DIAGRAM</p> <p>PRINCIPAL STRESS P.S.I.</p> <p>SHEAR STRESS - P.S.I.</p> <p>HIGHLY PLASTIC CLAY (CH)</p>									

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

# TRIAxIAL COMPRESSION TEST

## TEST SERIES SUMMARY SHEET

PROJECT: E-439

SITE: KENT & QUEENSWAY

DATE: 20-1-62

HOLE NO. 3

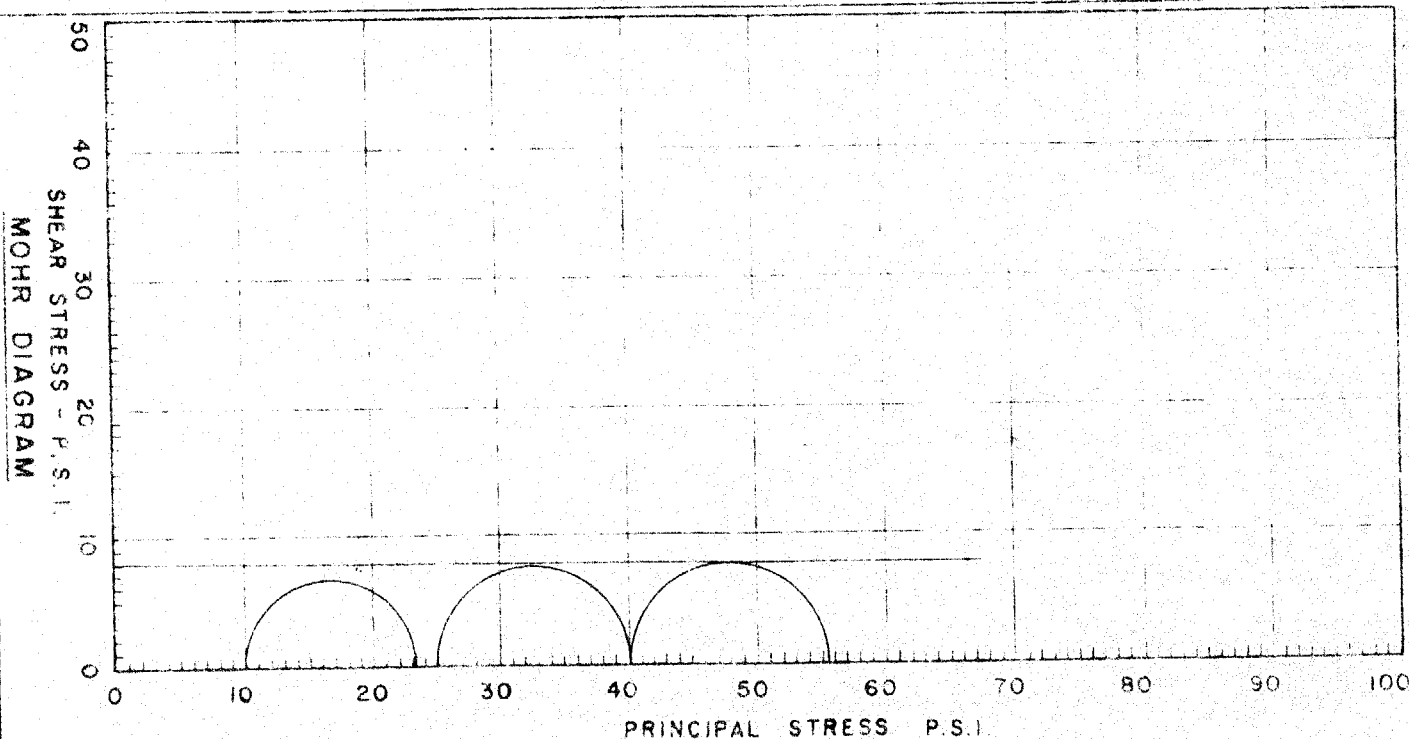
LOCATION

DEPTH:

TEST NO.	3-4A	3-4B	3-4C							AVERAGE
DEPTH										
LATERAL PRESSURE (P.S.I.)	10	40	25							
COMPRESSIVE STRESS (P.S.I.) - qc	13.16	15.5	15.1							
WATER CONTENT - W%										
WET DENSITY - $\gamma_m$										

C = 8 P.S.I.

$\phi = 0^\circ$



HIGHLY PLASTIC CLAY (CH)

MOHR DIAGRAM

TESTED: DATE: 20-1-62  
 COMPUTED: G.B. DATE: 20-1-62  
 PLOTTED: G.B. DATE: 20-1-62  
 CHECKED: D.M. DATE: 20-1-62

MCROSTIE & ASSOCIATES  
 CONSULTING ENGINEERS

YEAR 4/55

BR 922

PLATE No. 12



# TRIAxIAL COMPRESSION TEST

## TEST SERIES SUMMARY SHEET

PROJECT: E-439		SITE: KENT & QUEENSWAY		DATE: 20-1-62	
HOLE NO. 3		LOCATION		DEPTH:	
TEST NO.	3-5A	3-5B	3-5C	3-5D	AVERAGE
DEPTH					
LATERAL PRESSURE (P.S.I.)	10	40	25	40	
COMPRESSIVE STRESS (P.S.I.) - qc	19.49	20.8	21.8	21.4	
WATER CONTENT - W%					
WET DENSITY - $\gamma_m$					
$C = 11 \text{ P.S.I.}$ $\phi = 0^\circ$					
<p style="text-align: center;">PRINCIPAL STRESS P.S.I.</p> <p style="text-align: center;">SHEAR STRESS - P.S.I.</p> <p style="text-align: center;">MOHR DIAGRAM</p> <p style="text-align: center;">HIGHLY PLASTIC CLAY (CH)</p>					

TESTED: \_\_\_\_\_  
 COMPUTED: G.B.  
 PLOTTED: G.S.  
 CHECKED: D.M.

DATE: 20-1-62  
 DATE: 20-1-62  
 DATE: 20-1-62

McROSTIE & ASSOCIATES  
 CONSULTING ENGINEERS

MAR 14/55

BR 822

# TRIAXIAL COMPRESSION TEST

## TEST SERIES SUMMARY SHEET

PROJECT: E-439		SITE: KENT & QUEENSWAY		DATE: 5-1-62	
HOLE NO. 6		LOCATION:		DEPTH:	
TEST NO.	6-5(A)	6-5(B)			AVERAGE
DEPTH					
LATERAL PRESSURE (PSI)	40	10			
COMPRESSIVE STRESS (PSI) - $q_0$	15.6	20.2			
WATER CONTENT - W %					
WET DENSITY - $\gamma_m$					
$C =$ $\phi =$					
<p>MOHR DIAGRAM</p> <p>PRINCIPAL STRESS P.S.I.</p> <p>SHEAR STRESS - P.S.I.</p> <p>HIGHLY PLASTIC CLAY (CH)</p>					
TESTED:	DATE:		DATE:		
COMPUTED:	DATE:		DATE:		
PLOTTED:	DATE:		DATE:		
CHECKED:	DATE:		DATE:		
<p>MCROSTIE &amp; ASSOCIATES</p> <p>CONSULTING ENGINEERS</p>					

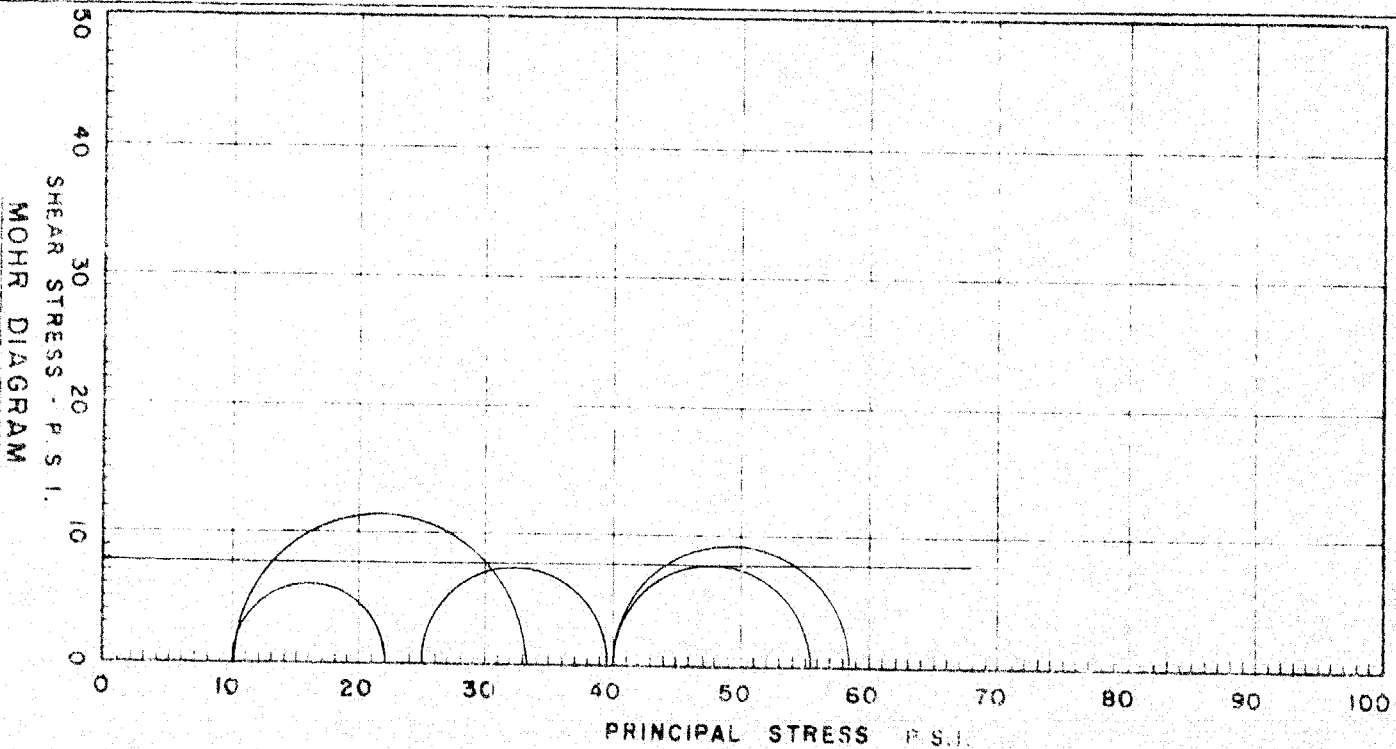
# TRIAXIAL COMPRESSION TEST TEST SERIES SUMMARY SHEET

PROJECT: E-439 SITE: KENT & QUEENSWAY DATE: 5-1-62  
HOLE NO. 6 LOCATION: DEPTH:

TEST NO.	6-6(A)	6-6(B)	6-6(C)	6-6(D)	6-6(E)				AVERAGE
DEPTH									
LATERAL PRESSURE (PSI)	40	10	40	25	10				
COMPRESSIVE STRESS (PSI) - qc	15.4	23.2	18.4	14.5	12.0				
WATER CONTENT - W %									
WET DENSITY - $\gamma_m$									

C = 7.9 P.S.I.

$\phi = 0^\circ$



HIGHLY PLASTIC CLAY (CH)

TESTED: DATE: 5-1-62  
COMPUTED: G.B. DATE: 5-1-62  
PLOTTED: G.B. DATE: 5-1-62  
CHECKED: DM DATE: 5-1-62

MCROSTIE & ASSOCIATES  
CONSULTING ENGINEERS  
OTTAWA CANADA

# TRIAxIAL COMPRESSION TEST

## TEST SERIES SUMMARY SHEET

PROJECT: E-439

SITE: KENT & QUEENSWAY

DATE: 5-1-62

HOLE NO. 6

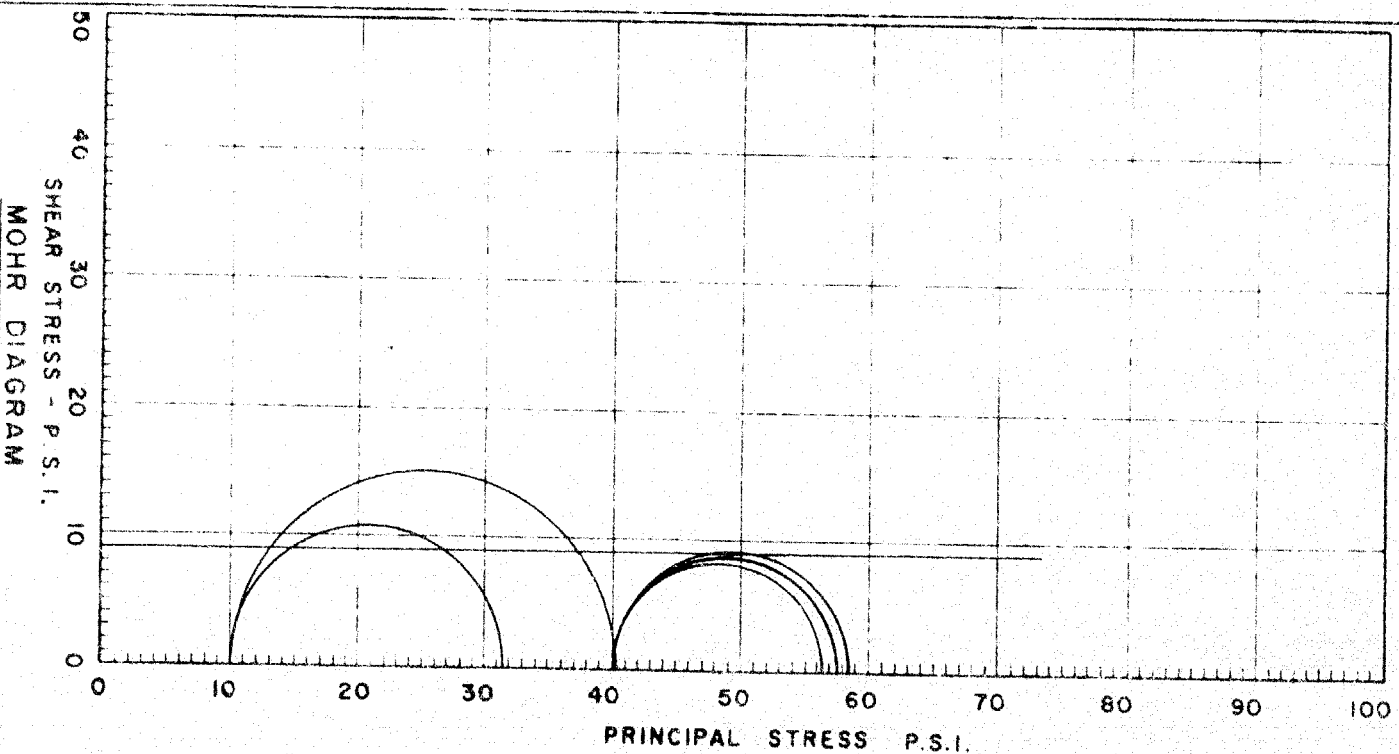
LOCATION:

DEPTH:

TEST NO.	6-7(A)	6-7(B)	6-7(C)	6-7(D)	6-7(E)					AVERAGE
DEPTH										
LATERAL PRESSURE (PSI)	40	10	40	10	40					
COMPRESSIVE STRESS (PSI) - $q_c$	17.7	30.4	16.5	21.5	18.6					
WATER CONTENT - W %										
WET DENSITY - $\gamma_m$										

$C = 9 \text{ PSI}$

$\phi = 0^\circ$



TESTED: \_\_\_\_\_  
 COMPUTED: D.M.  
 PLOTTED: D.M.  
 CHECKED: G.B.

DATE: 5-1-62  
 DATE: 5-1-62  
 DATE: 5-1-62

MCROSTIE & ASSOCIATES  
 CONSULTING ENGINEERS  
 OTTAWA CANADA

# TRIAXIAL COMPRESSION TEST

## TEST SERIES SUMMARY SHEET

PROJECT: E-439

SITE: KENT & QUEENSWAY

DATE: 6-1-62

HOLE NO. 6

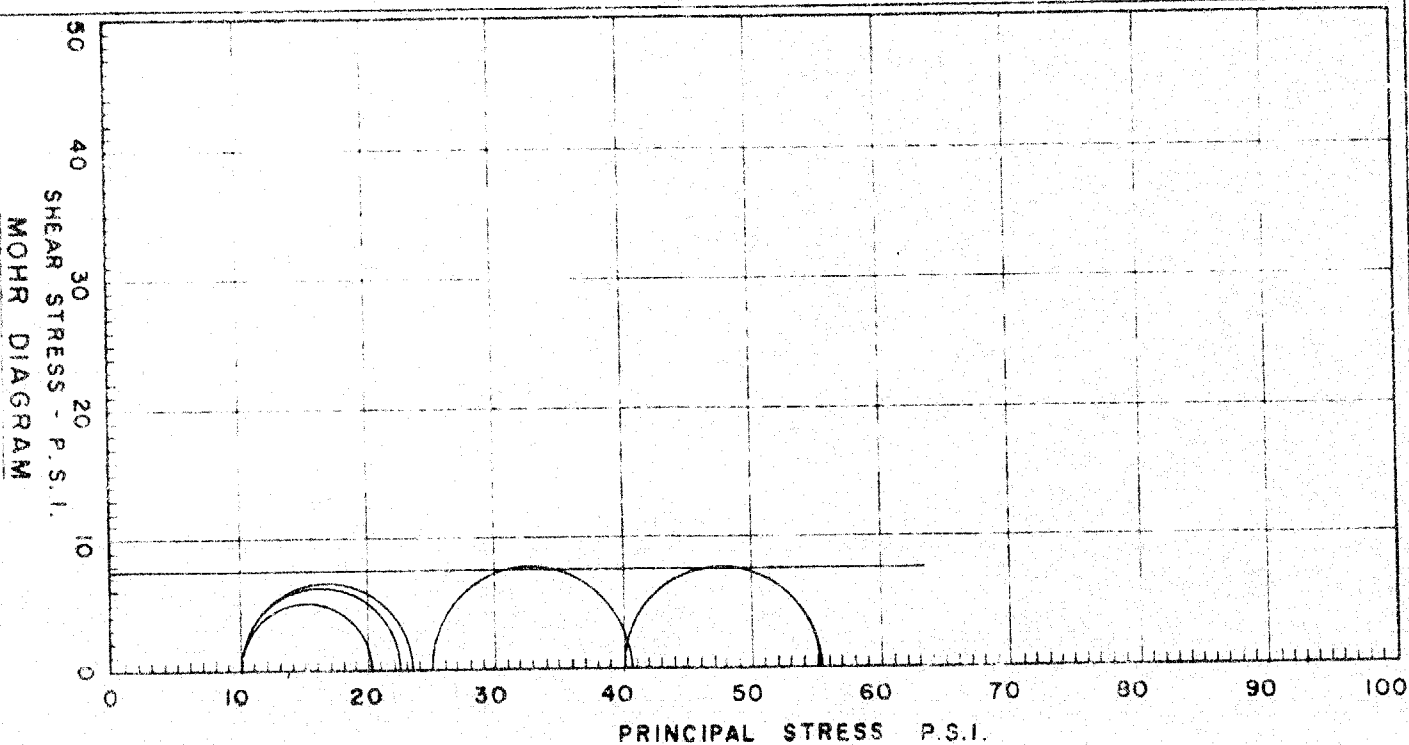
LOCATION:

DEPTH:

TEST NO.	6-8(A)	6-8(B)	6-8(C)	6-8(D)	6-8(E)					AVERAGE
DEPTH										
LATERAL PRESSURE (PSI)	40	10	25	10	10					
COMPRESSIVE STRESS (PSI) - $q_c$	15.2	13.4	15.6	10.2	12.5					
WATER CONTENT - W%										
WET DENSITY - $\gamma_m$										

$C = 7.5 \text{ P.S.I.}$

$\phi = 0^\circ$



MEDIUM PLASTIC CLAY (CL)

MOHR DIAGRAM

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

DATE:

DATE: 6-1-62

TESTED:

COMPUTED:

DM

DATE:

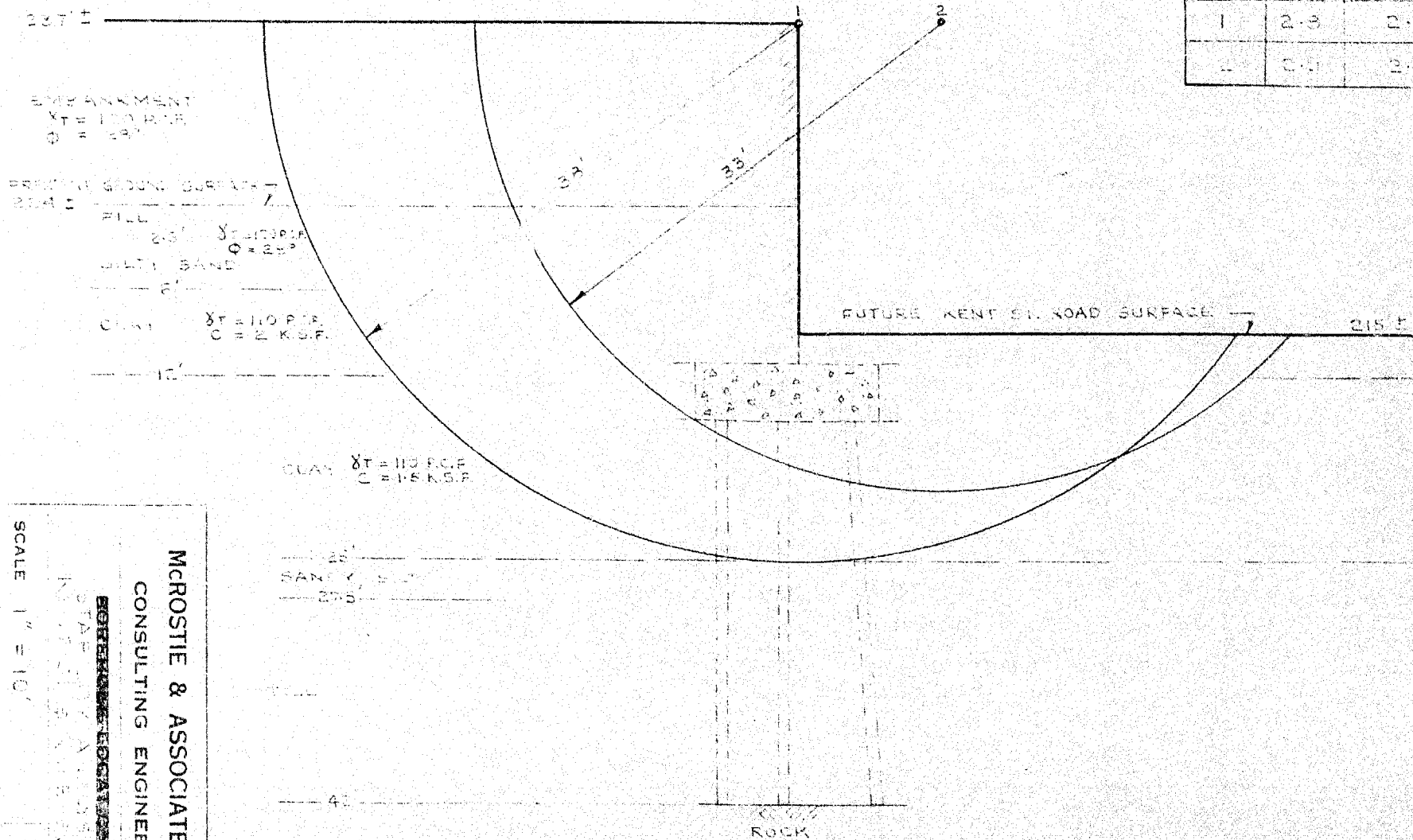
GB

DATE:

6-1-62

CHECKED:

SLIP No.	F.R.	ES. 65% LOSS OF STRENGTH
1	2.3	2.1
2	0.1	2.2



SOUTH WEST CORNER OF WEST ABUTMENT

MCROSTIE & ASSOCIATES LTD.  
CONSULTING ENGINEERS

~~CONFIDENTIAL~~ LOCATION

STATE OF NEW YORK  
COUNTY OF ALBANY

SCALE 1" = 10'

PLATE 12

VOID RATIO

1.8  
1.7  
1.6  
1.5  
1.4  
1.3  
1.2

PRESSURE VOID RATIO CURVE  
E-439 KENT & QWY  
SAMPLE No 1-4  
DEPTH 210'

$P_N(\text{MIN}) = 2.60 \text{ TONS/ft}^2$   
 $P_N(\text{PROB.}) = 2.96 \text{ TONS/ft}^2$   
 $P_N(\text{MAX}) = 3.08 \text{ TONS/ft}^2$   
INITIAL  $e = 1.816$   
 $C_c = 2.030 - .204 = 1.826$

Tested by D.M. & A.G.  
Calculated by A.G.  
Checked by D.M.

McROSTIE & ASSOCIATES, LTD.  
CONSULTING ENGINEERS  
OTTAWA

PRESSURE TONS/ft<sup>2</sup>

#60-F-257C

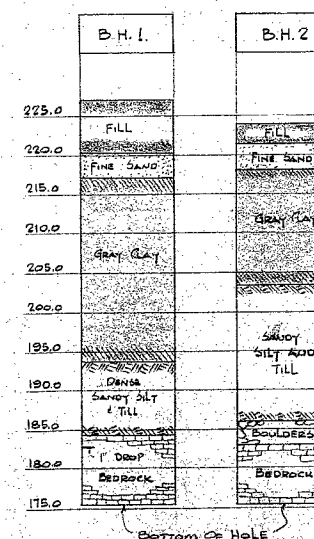
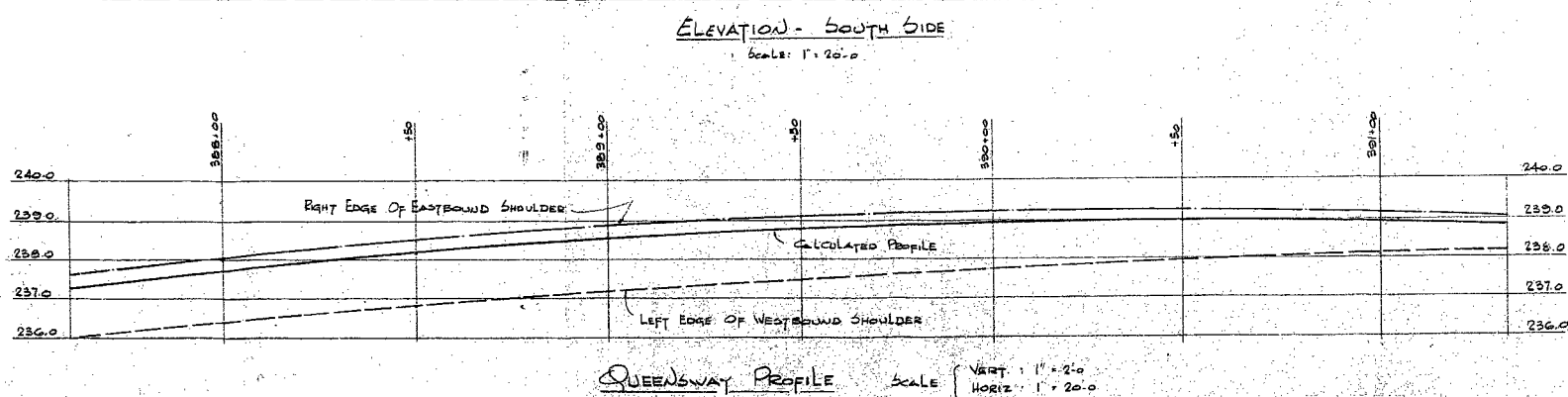
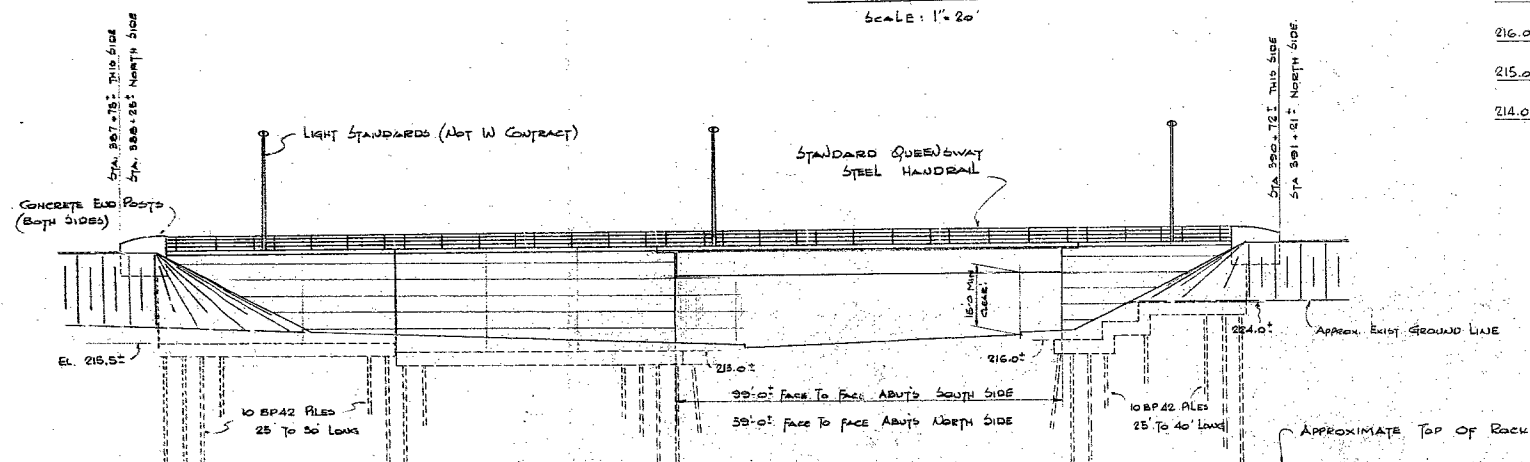
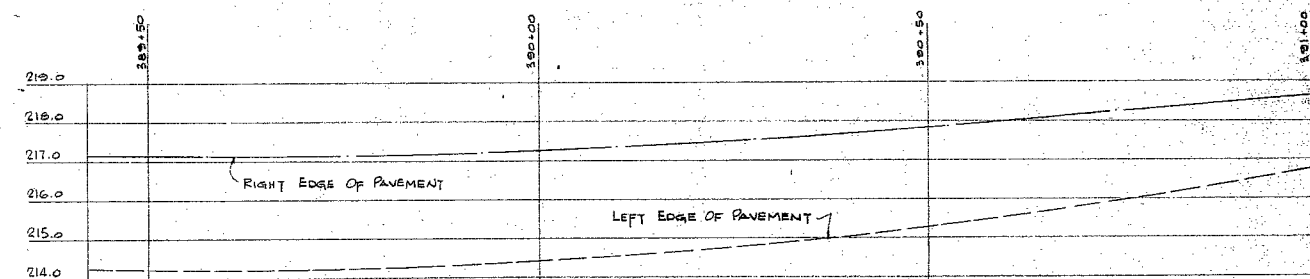
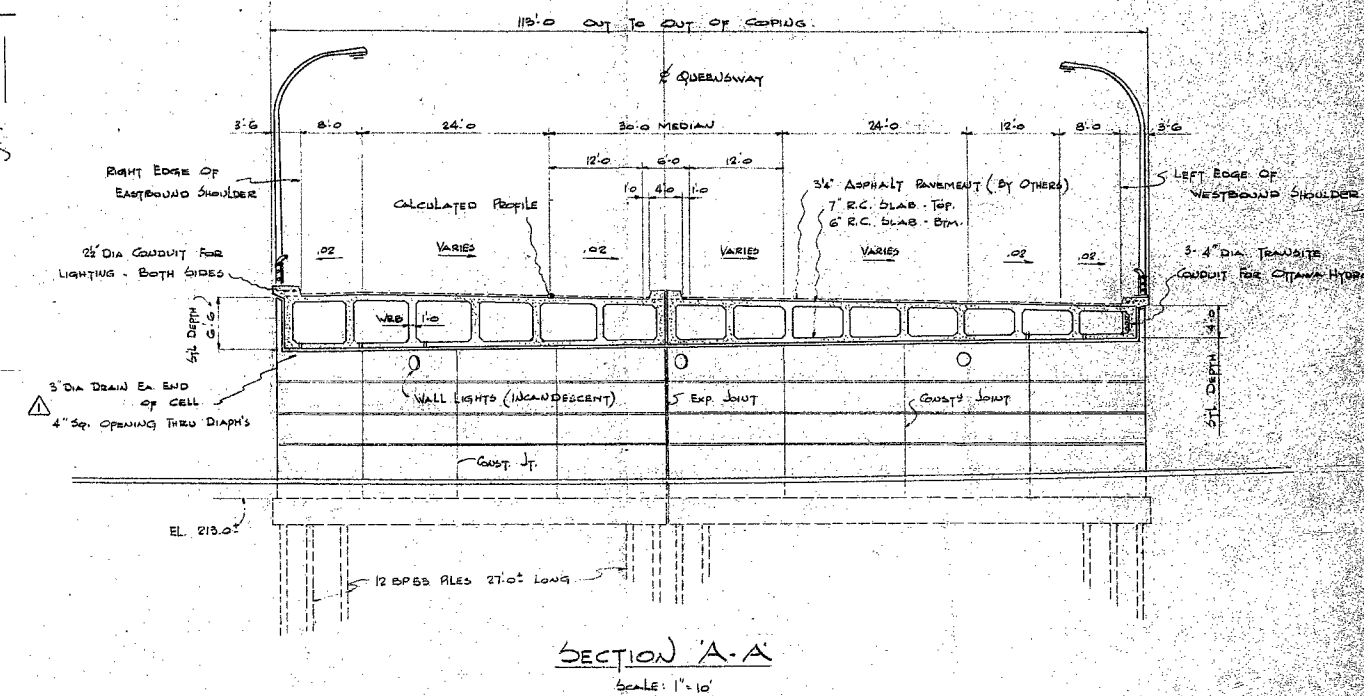
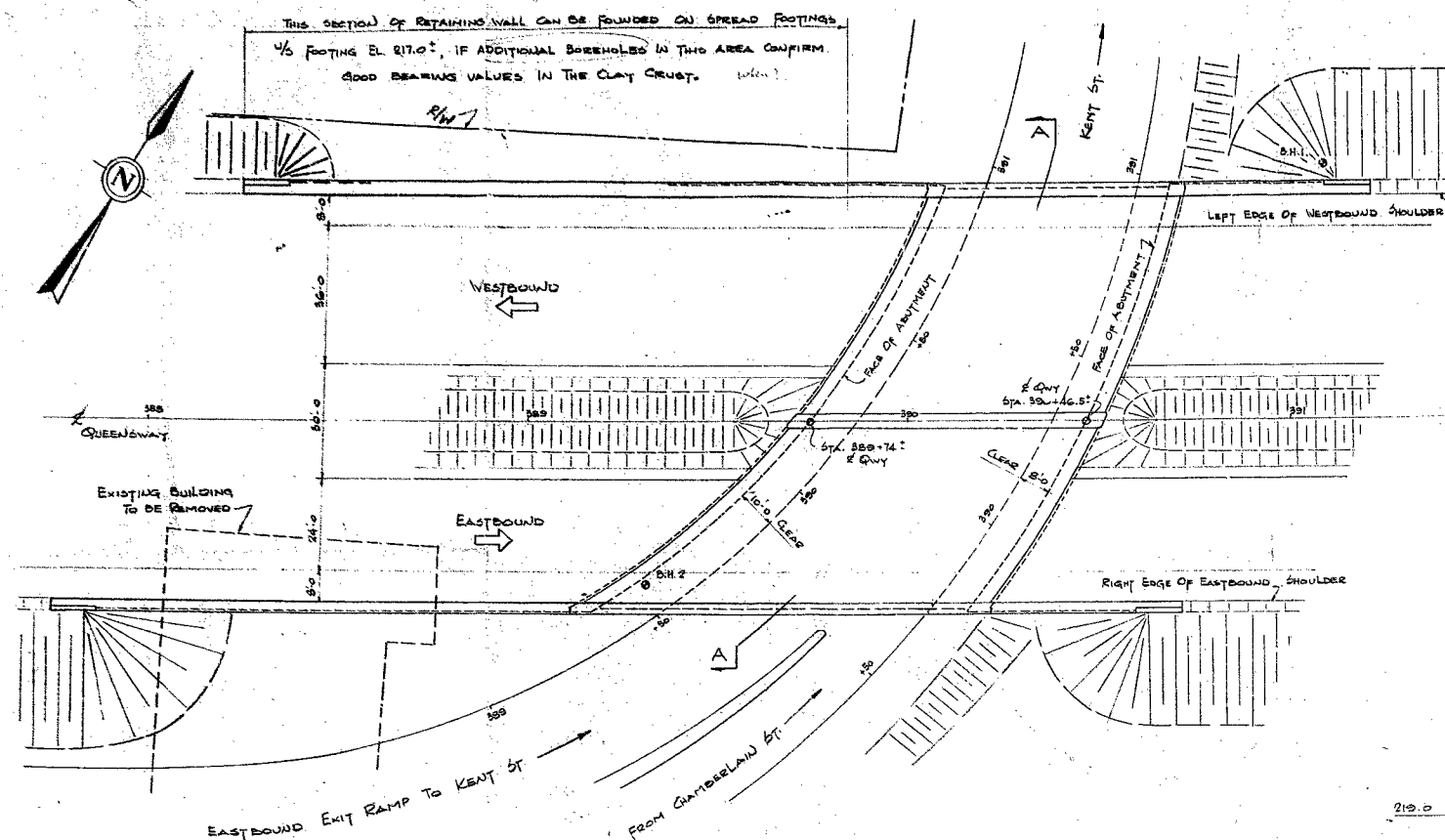
W.P. #946-59

KENT ST. (OTTAWA)

BRIDGE #20


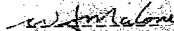
QUEENSWAY





NOTES:

DESIGN SPECIFICATIONS: A.A.S.H.O. SPECIFICATIONS FOR HIGHWAY BRIDGES.  
LIVE LOAD: H20-S16.  
SUPERSTRUCTURE: HOLLOW BOX GIRDER.  
SUBSTRUCTURE: ABUTMENTS & RETAINING WALL ON PILES TO ROCK, EXCEPT AS NOTED.  
CONCRETE STRENGTH: 3000 P.S.I. THROUGHOUT.  
FOUNDATIONS SHOWN ARE BASED ON PRELIMINARY SOILS REPORT BA1123  
AND DRAFT OF FINAL REPORT OF-582A  
PROVISION FOR TRAFFIC DURING CONSTRUCTION NOT REQUIRED.

	REVISED TO APPROVAL	15/06/01
No	Revisions	By Date
DEPARTMENT OF HIGHWAYS OF ONTARIO		
OTTAWA QUEENSWAY LIMITED-ACCESS HIGHWAY OTTAWA CANADA		
BRIDGE N° 20		NT. ST.
PRELIMINARY		N
DE LEUW CATHER & CO. OF CANADA LIMITED Consulting Engineers  	DEPT. OF HIGHWAYS OF ONTARIO JAN 22 1962  Director of Planning & Design	
Designed by: G.S.S. Drawn by: R.T. Checked by:	Date: JAN. 1962  Scale: AS SHOWN	DWG. No. D 5002-PI
Sheet		of

DISTRICT № 9  
W.P. № 946-59

640-1