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ONTARIO
DEPARTMENT OF HIGHWAYS

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

61 F 234C Bridge Division,
January 11, 1961.

MEMORANDUM TO:

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

RE: W.P. 947-59,
Bridge #21 At Bank St., and
W.P. 948-59,
Bridge #22 At O'Connor St., and
Hawthorne Avenue Embankment,
Ottawa Queensway - Dist. #9.

We enclose soils reports BA 1167, BA 1168 and
BA 1169 for the above structures. These are numbered
in the same order as shown above.

PMcW:go

C. W. J. H.
1/1. F. I. Hewson,
Consultant Liaison Engineer.

DE LEUW, CATHER & COMPANY
OF CANADA LIMITED
CONSULTING ENGINEERS
TORONTO OTTAWA ST. JOHN'S

2277 RIVERSIDE DRIVE
OTTAWA 8, ONTARIO
REGENT 3-4160

Our Ref. 4069-Q-3a

July 10th, 1962

Mr. Loi,
Soils Division,
Department of Highways of Ontario,
Parliament Buildings,
Toronto, Ontario.

Dear Sir:

Re: Soils Instrumentation - Ottawa Queensway
O'Connor to Main Street

Further to our telephone conversation this morning, we enclose one copy each of the following plans covering the above section of the Queensway:

Plan and Profile	Station 393 + 00 to Station 405 + 00
Plan and Profile	Station 405 + 00 to Station 417 + 00
Plan and Profile	Station 417 + 00 to Station 430 + 00
Preliminary Drawing	D5067-P1 - Bridge No. 37
Preliminary Drawing	D5068-P1 - Bridge No. 23
Preliminary Drawing	D5070-P1 - Bridge No. 25

Yours truly,

DE LEUW, CATHER & CO. OF CANADA LIMITED



Leon J. Marshall, P. Eng.,
Chief Bridge Engineer.

GSS:cd
Encl.

MCROSTIE & ASSOCIATES LTD.

CONSULTING ENGINEERS
OTTAWA 1

CANADA

G. C. MCROSTIE, B.A.Sc., O.L.S., P. ENG., M.E.I.C.
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W. J. MACLEAN, B.A., O.L.S., O.L.S.

393 BELL STREET
TELEPHONE CE. 2-5334

PRELIMINARY STAGE OF THE REPORT ON THE FOUNDATION INVESTIGATION FOR THE PROPOSED STRUCTURE AT BANK STREET AND THE QUEENSWAY

1. TERMS OF REFERENCE

We were requested by the Ottawa Office of De Leuw, Cather & Company of Canada Ltd. to carry out a preliminary stage investigation of the subsurface conditions at the site of a proposed structure to carry the Queensway over Bank Street. A preliminary stage of the report on foundation conditions at this site, based on a pilot hole study, was to include settlement predictions of a neighbouring building on the northeast side of the proposed structure.

2. CONCLUSIONS AND RECOMMENDATIONS

2.1 Effects of Embankment on Adjacent Building

Plate No. 3 indicates the geometry, available to date, used in computations to predict settlements under two points, A and B of the existing building on the northeast side of the proposed structure. On the basis of the subsoil formation revealed by the pilot hole and a test pit alongside the south wall of the building, a settlement analysis confined to the clay strata between elevations 222 and 182 was made.

The settlements calculated by standard methods were found to be negligible under points A and B, if the assumed geometry is correct. These results are different from an earlier prediction made in a letter dated June 26th, 1959, because a 25 foot embankment was assumed previously, whereas we are now presently considering the effects of a 10 foot surcharge.

The determination of the type of foundation under the existing railway overpass abutments is required along with an additional borehole on the west side of the overpass. The results of this investigation may affect the geometry and type of foundation best suited for the new structure. This in turn may promote appreciable settlements under the southwest corner of the existing adjacent building. Consequently, additional settlement computations may be required.

2.2 Soil Strengths

The pilot hole study indicated that an allowable bearing capacity of 4,000 pounds per square foot can be assumed on "in-situ" clay soils between elevations 219 and 205. On lower "in-situ" soils a bearing capacity of 3,000 pounds per square foot may be assumed down to elevation 182. The type of foundation under the existing overpass may alter the proposed structure span length, footing elevations or even foundation type. On that basis and considering the limitations of one borehole it should be emphasized that bearing capacity values stated above can be expected to vary.

2.3 Soil Compressibilities

Consolidation studies indicated that excessive settlements of the bridge structure should not occur if total stresses on the clay soils do not exceed preconsolidation loads. Bearing pressures in the range stated in paragraph 2.2 above would not create excessive total settlements. However, more details of the geometry of the proposed structure will allow an analysis determining the range of differential settlements that could be expected.

3. SITE INVESTIGATION

3.1 Field Work

A pilot hole was made at the site with our test drilling rig in the location shown on Plate No. 1. Seven tube soil samples were recovered from cohesive soil strata at 5 foot intervals from 15 feet below ground surface down to 47 feet. Four standard split barrel samples were recovered in granular soils in conjunction with standard penetration resistance tests at 5 foot intervals. All samples were examined and classified in our laboratory. The results of standard penetration tests performed gave an indication of the relative densities of the granular strata encountered during drilling. An overnight groundwater level was observed and recorded during the boring.

Bedrock encountered at elevation 169 was diamond drilled and cores recovered for inspection and logging. The evaluation of the structural properties of the rock was assisted by calculating the percentages of core recovery. The presence of seams in the rock formation would have been detected because of a careful watch for drops of drill rods and loss of drill water exercised during the rock drilling operations.

The hand dug test pit No. 2, in the location shown on Plate No. 1, was made in order to reveal the underside of the foundation wall footing of the existing building. Thus the elevation of the footing was established at 222 and used in settlement computations under points A and B. Test pits Nos. 3 and 4 were made in an effort to determine the extent of the fill and sand strata in that area.

3.2 Laboratory Testing

Three consolidation tests were made on samples retrieved at depths representing the mid height of increments of the compressible layer. A group of unconfined

compression tests along with small scale penetrometer tests (Soiltest Pocket Penetrometer) were made to provide preliminary strength values of soil strata encountered. Triaxial tests and field vane tests are planned to confirm these strength results. Classification tests were also made on most samples.

3.2 Observations

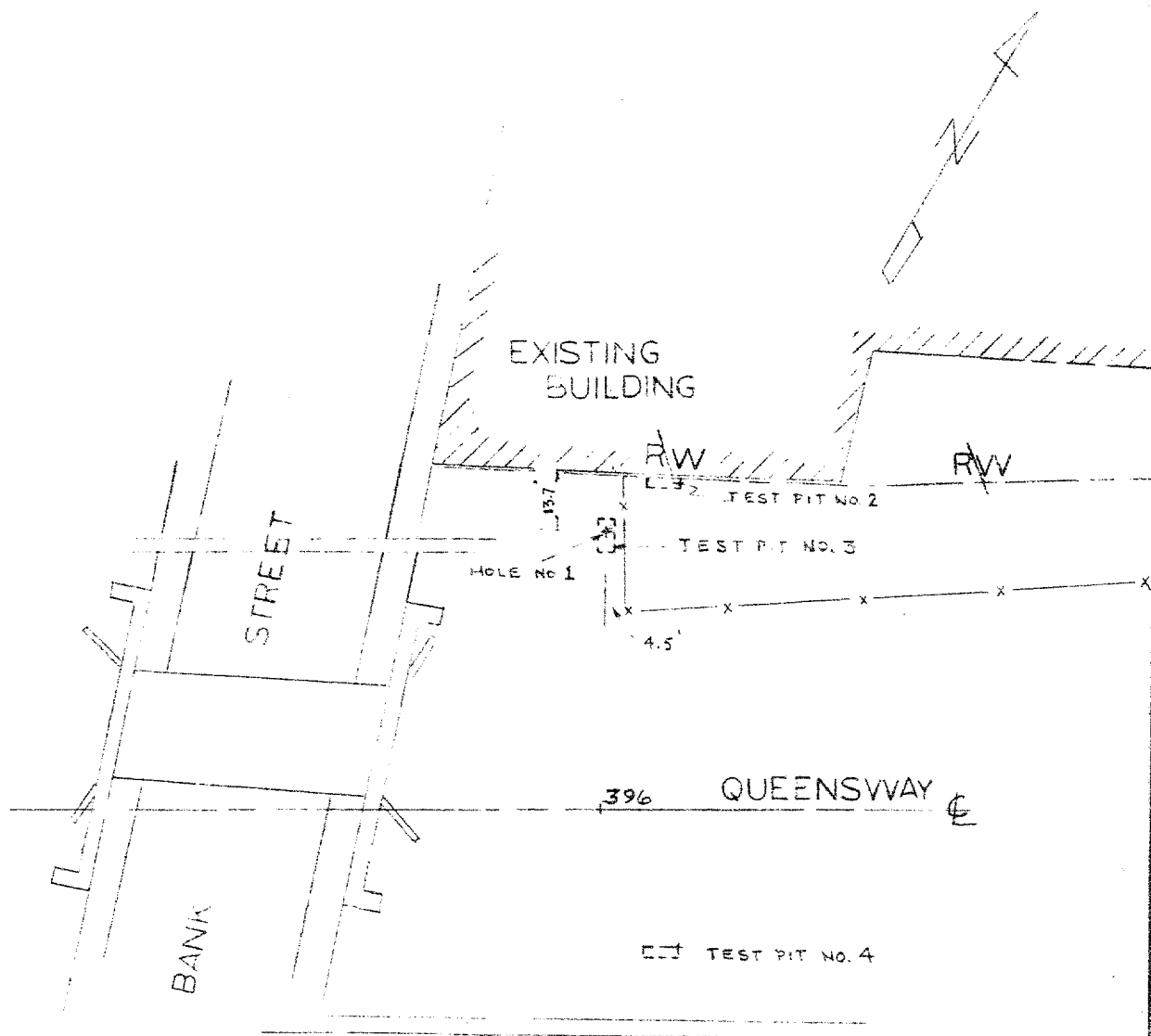
The subsurface profile as revealed by the pilot hole is shown on the accompanying plate No. 2. It can be generalized as consisting of about 2 feet of fill underlain by about 6 feet of medium dense fine sand overlying approximately 38 feet of gray silt and clay mixture and clay generally stiff throughout. About 13 feet of till deposit underlies the latter stratum with increasing density with depth. Rock encountered below the till strata at elevation 169 consists essentially of shaley limestone in a sound condition.

The hand dug pit No. 2 alongside the foundation wall indicated that the wall footing was seated on the stiff clay, revealed in the pilot hole No. 1, at elevation 222. The medium dense fine sand layer down to elevation 219.5 as shown in the borehole reduces in thickness towards the foundation wall of the building and therefore the footing rests on the clay stratum.

Pest Pit No. 3 showed that 3 to 5 feet of broken rock fill was overlying two feet of sand underlain by about $\frac{1}{2}$ foot of gravel overlying the clay stratum. Pit No. 4 revealed $1\frac{1}{2}$ feet of cinder fill and topsoil underlain by 3 feet of fine sand containing 1 inch layers of cohesive soil at about $\frac{1}{2}$ foot intervals underlain by $1\frac{1}{2}$ feet of silt sand gravel mixture overlying the clay layer.

An overnight groundwater level reading in the pilot hole showed the groundwater to be at about 14 feet below ground surface. This level can be

expected to rise somewhat during wetter months, possibly to the bottom of the sand layer. An additional borehole revealing further the extent of the fill and sand strata, should assist in establishing groundwater levels for final design.



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BOREHOLE LOCATIONS
BANK & QUEENSWAY

SCALE 1" = 40'

PLATE 1

McROSTIE & ASSOCIATES

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OTTAWA CANADA

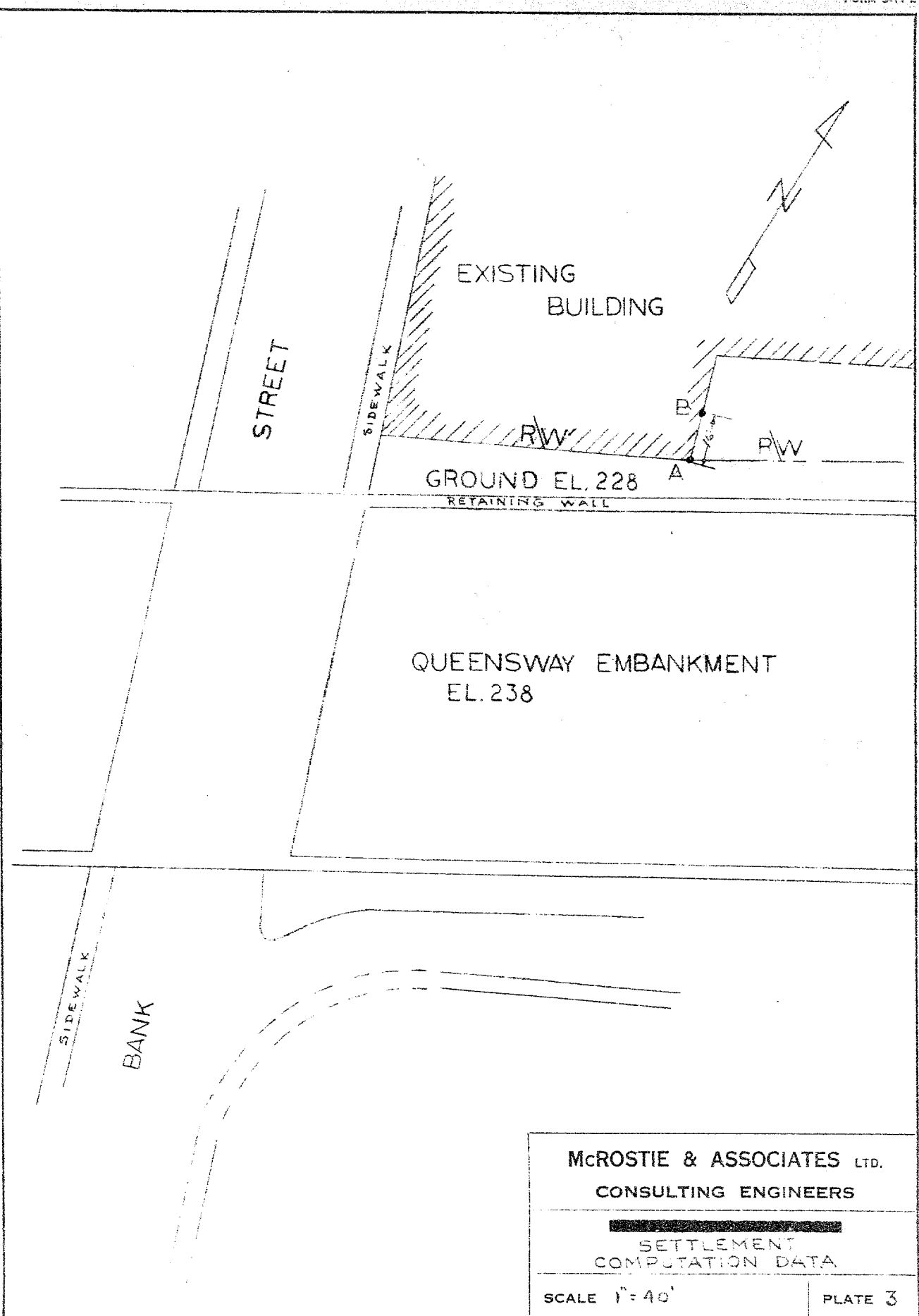
SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

BANK & QUEENSWAY

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 228.0' DATE APRIL 13, 1960
REMARKS B.M. EL. 219.51' CITY B.M. SOUTH WEST CORNER OF PRETORIA
AND DRIVEWAY

HOLE NO. 1

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT. ²	SMALL SCALE PENETROMETER KIPS/FT. ²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PROBING OR VANE TEST				
						LB. HAMMER		NO CASING		
						INCH DROP	INCH DIA. ROD		
							P WPS PER FOOT OR SHEAR STRENGTH IN KIPS PER FT. ²				
				GROUND SURFACE							
				FILL	0'	228.0'					
					2'	226.0'					
		15	1-1	FINE SAND MEDIUM DENSE							
					8.5'	219.5'					
	2.2, 2.4, 2.4	6	1-2	CLAY BROWNISH GRAY HIGH PLASTICITY, STIFF (CH)	11.5'						
2.1	2.6, 2.6, 2.6 4.2, 3.4, 3.0 4.4, 4.4, 4.4 2.0, 2.4		1-3	CLAY REDDISH GRAY HIGH PLASTICITY STIFF TO VERY STIFF							
				(CH)							
2.3	2.2, 2.2, 2.2 3.0, 3.0, 3.0 3.0, 3.0, 3.0 3.2, 3.2, 3.4		1-4		22.5'	205.5'					
2.5	2.2, 2.2, 2.2 3.0, 3.0, 3.0 3.0, 3.0, 3.0 2.0, 2.0, 2.0		1-5	SILTY CLAY, GRAY MEDIUM PLASTICITY							
				STIFF (CL)							
2.5	2.2, 2.2, 2.2 2.2, 2.2, 2.2 2.2, 2.2, 2.2 2.2, 2.2, 2.2		1-6								
2.9	2.2, 2.2, 2.2 2.2, 2.2, 2.2 2.2, 2.2, 2.2 2.2, 2.2, 2.2		1-7	CLAYEY SILT, GRAY LOW PLASTICITY, STIFF (ML)	35'	193.0'					
3.2	2.2, 2.2, 2.2 2.2, 2.2, 2.2 2.2, 2.2, 2.2 2.2, 2.2, 2.2		1-8	SILTY CLAY, GRAY, LOW PLASTICITY, STIFF (CL-ML)	40'	188.0'					
				CLAY GRAY, MEDIUM PLASTICITY, STIFF (CL)	41.5'						
0.9	3.4, 3.4, 3.4 3.4, 3.4, 3.4 3.4, 3.4, 3.4 3.4, 3.4, 3.4		1-9	SILT CLAY IN LAYERS WITH A FEW PEBBLES, GRAY, MEDIUM SOFT	46'						
				SILTY FINE SAND	48'	180.0'					
		2	1-10	SILTY CLAY WITH A LITTLE SAND & GRAVEL LOW PLASTICITY VERY LOOSE (CL-ML)	49.3'	178.7'					
				BOULDERS IN GRAVELLY SAND WITH SOME SILT & A TRACE OF CLAY (TILL)	53'	175.0'					
				DENSE (SM)							
		214	1-11	WEATHERED SHALE	59'	169.0'					
				SHALEY LIMESTONE	60.2'						
				CORE RECOVERY - 88%	65.9'						
				CORE RECOVERY - 76%	71'	157.0'					
				BOTTOM OF HOLE							
R - REMOULDED							0 20 40 60 80 100 % WATER CONTENT NATURAL ○ LIQUID LIMIT □ PLASTIC LIMIT △ PLATE 2				



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SETTLEMENT
COMPUTATION DATA

SCALE 1" = 40'

PLATE 3

MCROSTIE & ASSOCIATES LTD.

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OTTAWA 1

CANADA

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

1. INTRODUCTION

We were requested by the Ottawa Office of De Leuw Cather and Company of Canada Limited to carry out a subsurface investigation determining the subsurface conditions at the site of a proposed structure to carry the Queensway over Bank Street. The report was to include design recommendations on the structure and retaining walls running east along with an examination of the effects of the Queensway embankment at that location on an adjacent existing building. A preliminary report on foundation conditions, based on a pilot borehole and test pit study was submitted in December, 1960. This report contains final recommendations pertaining to the structure and retaining walls at this site. It includes also pertinent data and results of analyses submitted in the preliminary report.

2. CONCLUSIONS AND RECOMMENDATIONS

2.1 Structure

A pile type of foundation is recommended for the proposed structure at this site. End bearing piles could rest on the shaley limestone rock determined in hole 1 and 2 at elevation 168 and 174 respectively. H-piles would be a practical and economical type of pile for this site. A tube or shell type of pile is less desirable because of the boulder content in the glacial till deposit overlying

bedrock. End bearing H-piles are also recommended for the support of the wing walls of the structure. The removal of the existing railway bridge abutments and footings would be required to allow the pile driving operations. In addition to the design loads on the piles, it should be recognized that because preloading of the underlying cohesive layer is not feasible at this site, consolidation of the compressible stratum under the weight of the adjacent embankment will produce an additional load on the piles by negative friction. This load will be distributed through the thickness of the cohesive layer along the piles. The maximum value of the load due to an estimated adhesion of 1,000 p.s.f. is about 30 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. Footings would have had to rest on in-situ clay soils near elevation 203 since the underside of the existing railway bridge footings were determined to be presently bearing at that elevation. This would cause a large portion of the abutment to be underground and a substantial increase in abutment cost, in view of the proposed elevation of 217 for the future Bank Street at this location. Furthermore, the subsurface investigation revealed that an allowable bearing capacity of 4000 p.s.f. was available above elevation 205; below this level a bearing capacity of only 3000 p.s.f. could be recommended. This fact alone would cause an appreciable increase in the footing width and higher costs would result. A cost comparison including only the extra structural materials and extra excavation showed a difference of a few thousand dollars in favor of footings. However, considering the added structural difficulties and construction problems, such as shoring

and dewatering a deep excavation at this site, that would be encountered with a footing type of foundation, a pile foundation is felt to be more economical.

The stability of the northern wing wall of the east abutment could be difficult to achieve economically with a footing type of foundation since the wall would be about 31 feet high (footings at el. 203 and finish grade at el. 234) and would have a toe space considerably restricted by an adjacent existing building. With this height of wall supported on a footing foundation an analysis was commenced to determine the possible increase in overturning moment due to consolidation of the compressible subsoil. The study was abandoned when other factors indicated that the wing wall should be supported on piles. The design and construction of the southern wing wall of the east abutment on footings is not possible because of an existing adjacent sewer and 48-inch water main, located at the toe of the wing wall, that cannot be economically relocated.

A footing foundation for the northern wing of the west abutment would also have to rest on in-situ clay soils, of lower bearing capacity, at about elevation 203 due to the existing railway bridge foundation. This would cause the wing wall to be 31 feet high and consequently the design and construction difficulties of such a high wall would render a footing type of foundation uneconomical at this site. In addition, since the southern wing wall of the west abutment is not extensive and since a pile type of foundation is recommended for the entire structure and other wing walls, it would be practical to continue the pile support for this section.

2.2 Effects of Embankment on adjacent building.

Plate No. 18 indicates the revised geometry used in computation of settlements under points A and B of a building adjacent to the Queensway embankment. A settlement analysis

by standard methods, confined to the clay strata between elevations 222 and 182, showed that only negligible settlements would be induced by the Queensway embankment under points A and B of the existing building. Although the Queensway embankment has been moved to the North to within 5 feet of point A of the building, these results were obtained because the effects of only a 6-foot surcharge were considered (present ground el. 228 and future embankment el. 234). In a preliminary analysis negligible settlements were also predicted under points A and B with a 10-foot surcharge but the Queensway embankment was then at 11 feet from point A.

2.3 Foundation c. Retaining Walls East of Structure

A footing type of foundation is recommended for the north retaining wall east of the Bank Street structure between stations 396+13 and 401+13. Footings could bear on the stiff clay crust at about elevation 220 and a net allowable bearing capacity of 4000 p.s.f. is recommended for this layer. At this bearing level footings would have adequate frost protection since the proposed finish grade at the base of the wall will be near elevation 226. A 6-inch layer of crushed stone may be placed at the bearing level to minimize remoulding of the clay and to develop adequate sliding resistance at the contact surface. A value of 1000 p.s.f. may be assumed in the design of the retaining wall for adhesion between the crushed stone cushion and the bearing clay. The joint at the bearing level between the pile supported north-east wing wall of the structure (which will be stepped upwards to minimum frost cover requirement) and the footing supported north retaining wall should be given special attention during construction. Care should be exercised during the pile cap excavation in order not to prolong the excavation to below the adjoining retaining wall footing since the allowable bearing capacity stated above is for undisturbed soil.

The south retaining wall east of the Bank Street structure cannot be supported on footings throughout its entire length because of an existing 48-inch diameter watermain located at the toe of the proposed wall. A pile type of foundation is recommended for the section of retaining wall between the south-east wing wall of the structure and station 398+00. It would be practical to continue the type of end bearing H-piles recommended for the support of the structure abutments and wing walls under this section of the retaining wall. A footing type of foundation is recommended for the remainder of the wall. The section of retaining wall between stations 398+00 and 398+65 could bear on footings at elevation 213 while the section of wall between stations 398+65 and 400+00 could bear on footings at elevation 212. These bearing levels are imposed in view of the existing adjacent watermain invert elevation. It is felt that at the recommended bearing elevations sufficient lateral support will be available in the soils below the footings. A net allowable bearing capacity of 3000 p.s.f. is recommended for both bearing elevations. A value of 800 p.s.f. is estimated for adhesion between a crushed stone cushion under the footings and the bearing clay surface; this value may be used in the design of the retaining wall to resist sliding. Our comments concerning the joint between the pile supported and footing supported walls at station 396+13 in the north retaining wall are also applicable to the joint at station 398+00 in the south retaining wall.

A timber pile scheme was studied, in lieu of a footing type of foundation, for the section of wall between stations 398+00 and 400+00 in an effort to reduce design and construction difficulties alongside the existing watermain. The friction pile scheme was rejected since

it would not provide any significant increase in allowable capacity and because an appreciable time lag would be necessary before the piles could be loaded. End bearing H-piles were also rejected in view of the small loads imposed and because of the extensive depth to an adequate bearing level.

2.4 Foundation of retaining wall west of structure

A footing foundation is recommended for the north retaining wall between the north west wing wall of the structure and station 392+53. Footings could bear on the medium dense silty sand fill at about elevation 222, and with the proposed ground surface at elevation 228 at the base of the wall, sufficient frost protection will be provided for the foundation. A net allowable bearing value of 3000 p.s.f. is recommended for this layer. Immediate settlements through the granular stratum should be insignificant since the bearing value recommended is based on permissible settlement. The density of the bearing stratum has been determined following Gibbs and Holtz theory which recognizes the effect of the present overburden influencing the number of blows per foot in the standard penetration resistance test. Thus the density and strength value of the recommended bearing level have been substantially increased from the values normally derived with the standard penetration test.

Variation between boreholes indicate that loose fill may be found in some areas along the retaining wall location at this site. If such a subsoil condition is encountered during the footing excavation, the loose fill should be sub-excavated down to the underlying stiff clay crust. These areas may then be backfilled up to the bearing level with granular material compacted to 95% of the standard Proctor density. This technique will be more economical than lowering the wall footing to the elevation of the underlying stiff clay and also more economical than any end bearing pile foundation.

2.5 Stability of retaining walls considering a deep-seated shear failure

An examination of a section of wall, 20 feet high, at the south east corner of the Bank Street structure revealed that a deep-seated shear failure should not occur and that the wall and embankment should be stable. This conclusion is based on the results obtained from analyses made for a geometrically similar section of wall, 20 feet high, at the south west corner of the neighbouring Kent Street structure (report No. SF-583). Both walls are underlain by cohesive subsoil of relatively equal strength and thickness and both are pile supported. Consequently it is felt that adequate factors of safety against shear failure obtained in the stability analysis of the wall at Kent Street could be reproduced for the wall at the Bank Street structure. Furthermore, with a 25% reduction in undrained shear strength factors of safety in excess of the accepted 1.5 were determined in the Kent Street stability analysis and these results could be reproduced for the section of wall examined at Bank Street. Adequate factors of safety against a deep-seated shear failure could also be obtained considering zero shear strength through the granular embankment at this site, as it was determined in the study made at Kent Street.

2.6 Soil Compressibility

The total settlement of the embankment at the structure will be a differential settlement between the embankment and the pile supported structure. The long term total settlement of the embankment (about 18 feet high) at the south east wing wall due to the underlying compressible clay strata was calculated and the result, reduced according to our experience of actual settlement observations of embankments overlying similar compressible subsoil, show insignificant settlement. Similar results obtained

from compressibility analyses made for the Queensway structures at Kent and Metcalfe Streets. (reports No. SF-583 and SF-487 respectively) confirm this conclusion.

A typical section of retaining wall supported on footings bearing on clay with a 10-foot embankment was examined and consolidation settlement of the wall through the underlying compressible clay strata was calculated. The result, reduced according to our experience, indicates insignificant differential settlement along the wall. However, the total long term consolidation settlement, evident only at the joint between the end bearing pile supported section of retaining wall or wing wall and the footing supported retaining wall, is estimated to be of the order of 1 inch. This fact should be incorporated in the detailing of such joints.

2.7 Construction Precautions

The excavation for the north east retaining wall adjacent to the Army supply depot building should not be enlarged northerly below the level of the existing wall footings in order to preserve adequate lateral support in the soils within the stressed zone. Underpinning of the adjacent building footing or lateral support of the soils beneath the adjacent building footing will be required only if the retaining wall excavation is unnecessarily enlarged.

A careful watch should be made, during the excavation for the south east retaining wall footing at this site to detect any local enlargement or deepening in the existing watermain excavation. If any variations in the soils are found at the recommended bearing levels they should be brought to the immediate attention of the supervising authority for appropriate action. Of course care should also be exercised during this excavation to protect the existing adjacent watermain.

3. SITE INVESTIGATION

3.1 Field Work

Sixteen boreholes were made at the site at the locations shown on Plate No. 1. Twelve were made with our test drilling rig and four by a drilling contractor. Seventy-seven two-inch split barrel samples were retrieved, and standard penetration tests were performed in the boreholes, through granular soil layers encountered; forty thin wall tube samples were retrieved from cohesive soil layers. Thirty-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 twelve 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.

Four hand dug test pits were made in the locations shown on Plate No. 1 as a supplement to the pilot borehole No. 1 in a preliminary investigation. Test pit No. 2 was made in order to reveal the elevation of the underside of the foundation wall footing of an existing building adjacent to the proposed north east wing wall of the structure.

Thus the elevation of the footing was established and used in settlement computations for points A and B. Test pits Nos. 3 and 4 were made to verify the extent of the fill and sand strata in the area. Test pit No. 5 was made in an effort to determine, by horizontal probings, whether the existing railroad bridge is supported on a pile or a footing foundation.

3.2 Laboratory Work

Three consolidation tests were made on samples 1-3, 1-5, 1-8 retrieved at depths representing the mid height of increments of the compressible layer underlying the structure at this site. 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. SF-487, Metcalfe Street and Queensway are also applicable to testing at this site.

Fifty unconsolidated undrained triaxial tests were performed on samples retrieved from the cohesive deposit to estimate the shear strength of the soils within the critical depth and to verify the shear strength values obtained by other laboratory and field tests. Classification tests were also made on most samples.

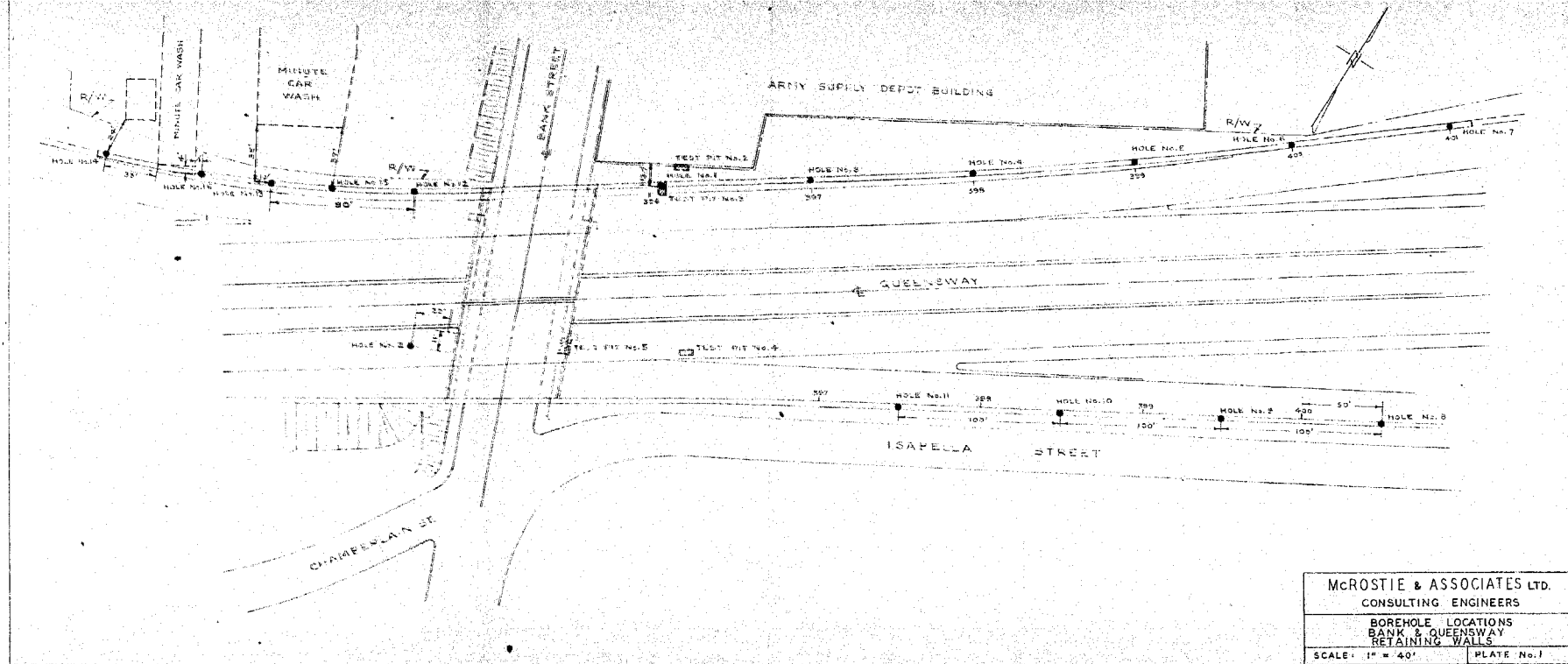
3.3 Observations

The geotechnical profile of the subsoil and rock formations as revealed by the borings is shown on the accompanying Plates No. 2 to No. 17. The subsurface profile can be generalized as consisting of fill varying in thickness from 2.0 to 12.5 feet, underlain by a layer of sand and silty sand about 1.5 to 6.5 feet thick overlying a clay

layer varying from stiff to medium soft with depth between 28 and 40 feet thick. Beneath the clay are loose to dense layers of sand silt mixtures and till to rock at a depth varying between 45 and 78 feet. The rock encountered east of borehole No. 11 is a shale of the Billings formation while the rock west of borehole No. 11 is shaley limestone of the Eastview formation.

The hand dug test pit No. 2 alongside the foundation wall indicated that the footing was bearing on the stiff clay, revealed in the pilot hole No. 1 at elevation 222. The medium dense fine sand layer down to elevation 219.5 as shown in the borehole reduces in thickness towards the foundation wall of the building and therefore the footing rests on the clay stratum. Test pit No. 3 showed that 3 to 5 feet of broken rock fill was overlying two feet of sand underlain by about $\frac{1}{2}$ foot of gravel overlying the clay stratum. Pit No. 4 revealed $1\frac{1}{2}$ feet of cinder fill and topsoil underlain by 3 feet of fine sand containing 1 inch layers of cohesive soil at about $\frac{1}{2}$ foot intervals underlain by $1\frac{1}{2}$ feet of silt sand gravel mixture overlying the clay layer. The horizontal probings made in test pit No. 5 did not reveal the presence of piles below the south east corner of the bridge abutment in the area investigated. The underside of the abutment footing was found to be bearing on clay at elevation 203.

Groundwater levels observed in most boreholes varied between 7 and 22 feet below ground surface. However, the depth of sewer trenches on adjacent municipal streets should tend to control the groundwater levels in that area.



McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

BANK & QUEENSWAY

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 228.0' DATE APRIL 13, 1960
REMARKS B.M. EL. 219.51' CITY B.M. SOUTH WEST CORNER OF PRETORIA
AND DRIVEWAY

HOLE NO. 1

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT. ²	SMALL SCALE PENETROMETER KIPS/FT. ¹	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PROBING OR VANE TEST	
						LB. HAMMER	NO CASING
						INCH DROPINCH DIA. ROD
GROUND SURFACE							BLOWS PER FOOT OR	SHEAR STRENGTH IN KIPS PER FT. ²
				FILL	0'	228.0'		
				FINE SAND	2'	226.0'		
		15	1-1	MEDIUM DENSE				
					8.5'	219.5'		
	22,24,24	6	1-2	CLAY BROWNISH GRAY HIGH PLASTICITY, STIFF (CH)	11.5'			
2.1	2.6 4.5 4.5 7.0		1-3	CLAY BROWNISH GRAY HIGH PLASTICITY STIFF TO VERY STIFF (CH)				
2.3	2.0 3.0 3.0 3.0		1-4		22.5'	205.5'		
2.5	2.0 2.0 2.0 2.0		1-5	SILTY CLAY, GRAY MEDIUM PLASTICITY STIFF (CL)				
2.5	2.0 2.0 2.0 2.0		1-6					
2.9	2.0 2.0 2.0 2.0		1-7	CLAYEY SILT, GRAY LOW PLASTICITY, STIFF (ML)	35'	193.0'		
3.2	2.0 2.0 2.0 2.0		1-8	SILTY CLAY, GRAY LOW PLASTICITY, STIFF (CL-ML)	40'	183.0'		
0.9	2.0 2.0 2.0 2.0		1-9	CLAY GRAY, MEDIUM PLASTICITY, STIFF (CL)	41.5'			
	2.0 2.0 2.0 2.0		1-10	SILTY CLAY IN LAYERS WITH A FEW PEBBLES GRAY MEDIUM SOFT SILTY FINE SAND	46'			
	2.0 2.0 2.0 2.0	2	1-11	SILTY CLAY WITH A LITTLE SAND & GRAVEL LOW PLASTICITY VERY LOOSE (CL-ML)	48.3'	180.0'		
				BOULDERS IN GRAVELLY SAND WITH SOME SILT & A TRACE OF CLAY (TILL)	49.3'	178.7'		
				DENSE (SM)	53'	175.0'		
		214	1-12	WEATHERED SHALE	59'	169.0'		
				SHALEY LIMESTONE	60.2'			
				CORE RECOVERY - 88%	65.9'			
				CORE RECOVERY - 76%	71'	157.0'		
				BOTTOM OF HOLE				

OVER-NIGHT WATER LEVEL - 214.2'

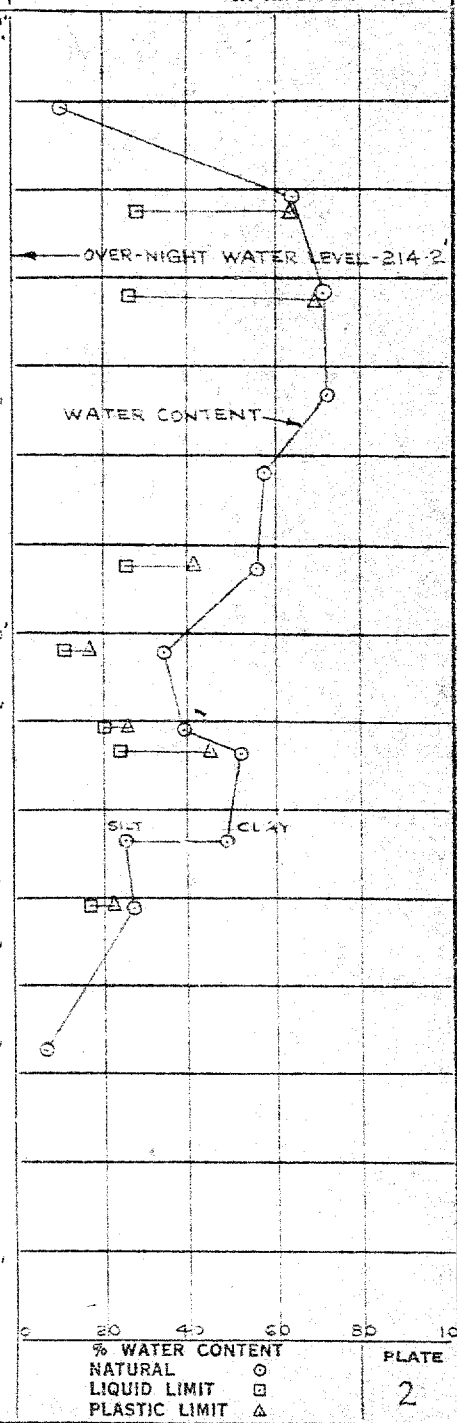
WATER CONTENT

SILT

CLAY

% WATER CONTENT	20	40	60	80	100
NATURAL					
LIQUID LIMIT					
PLASTIC LIMIT					

PLATE 2



R - REMOULDED

McROSTIE & ASSOCIATES LTD.

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

BANK & QUEENSWAY

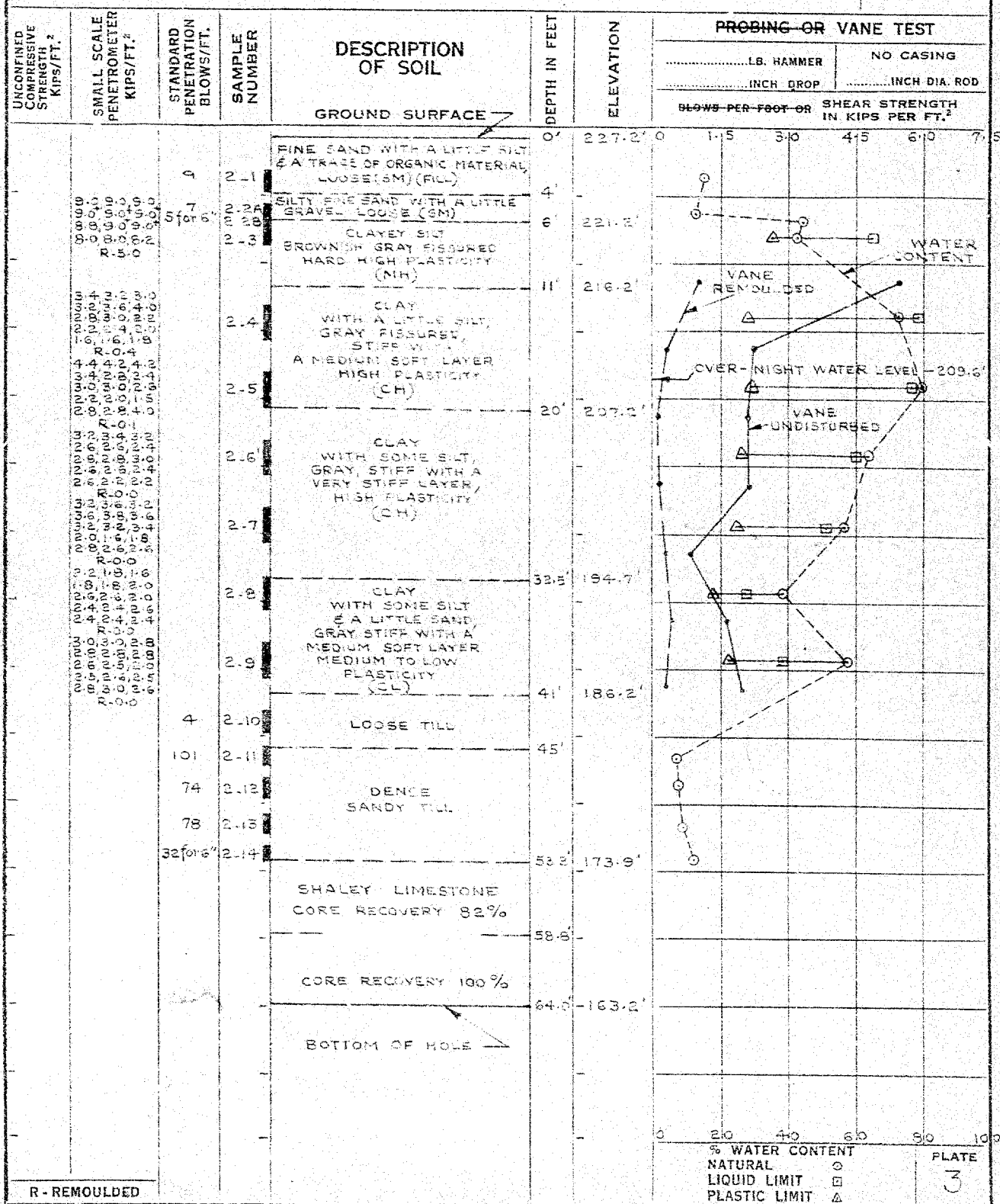
ELEVATION OF GROUND SURFACE (ZERO DEPTH) 227.2'

DATE MAY 10, 1961

HOLE NO.

REMARKS SEE PLATE No. 2

2



R - REMOULDED

McROSTIE & ASSOCIATES LTD.

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OTTAWA CANADA

SOIL PROFILE AND SUMMARY

OF FIELD AND LABORATORY TESTS

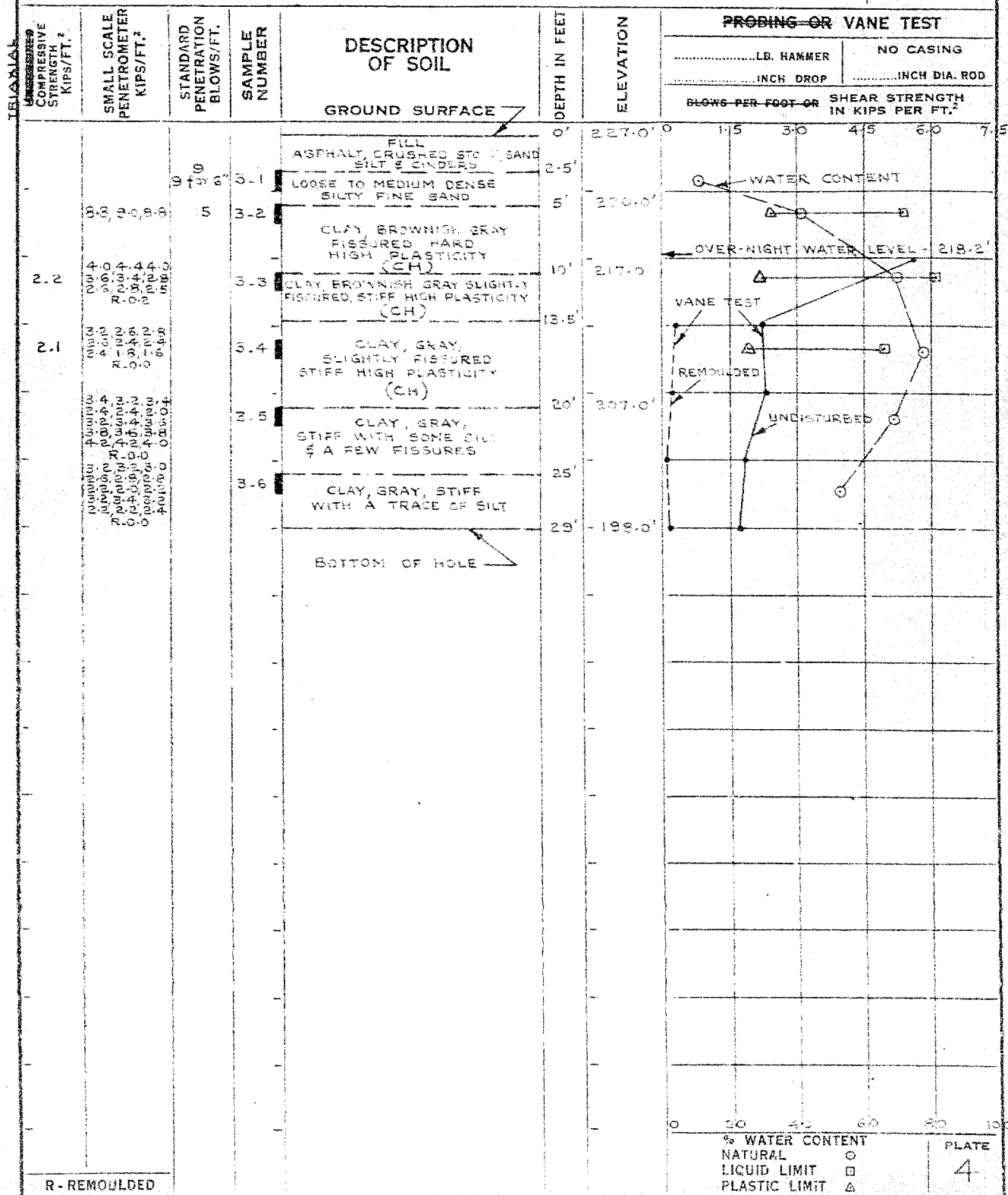
BANK & QUEENSWAY
RETAINING WALL

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 227.0' DATE JAN. 18, 1962

REMARKS SEE PLATE No. 2

HOLE NO.

3



R - REMOULDED

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY
OF FIELD AND LABORATORY TESTS

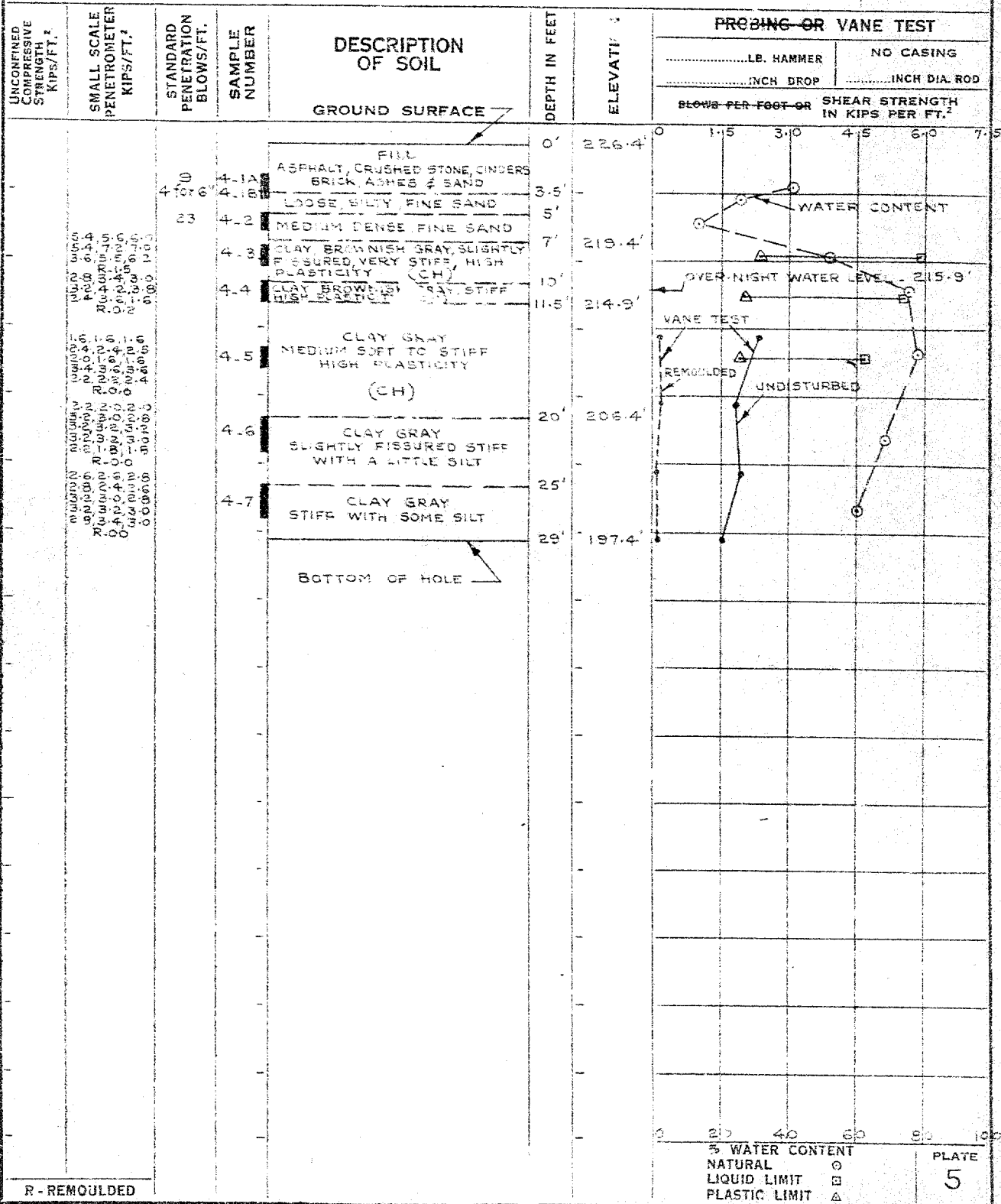
BANK & QUEENSWAY
RETAINING WALL

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 226.4'

DATE JAN. 13, 1962

HOLE NO. 4

REMARKS SEE PLATE No. 2



R - REMOULDED

PLATE
5

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY
OF FIELD AND LABORATORY TESTS

BANK & QUEENSWAY
RETAINING WALL

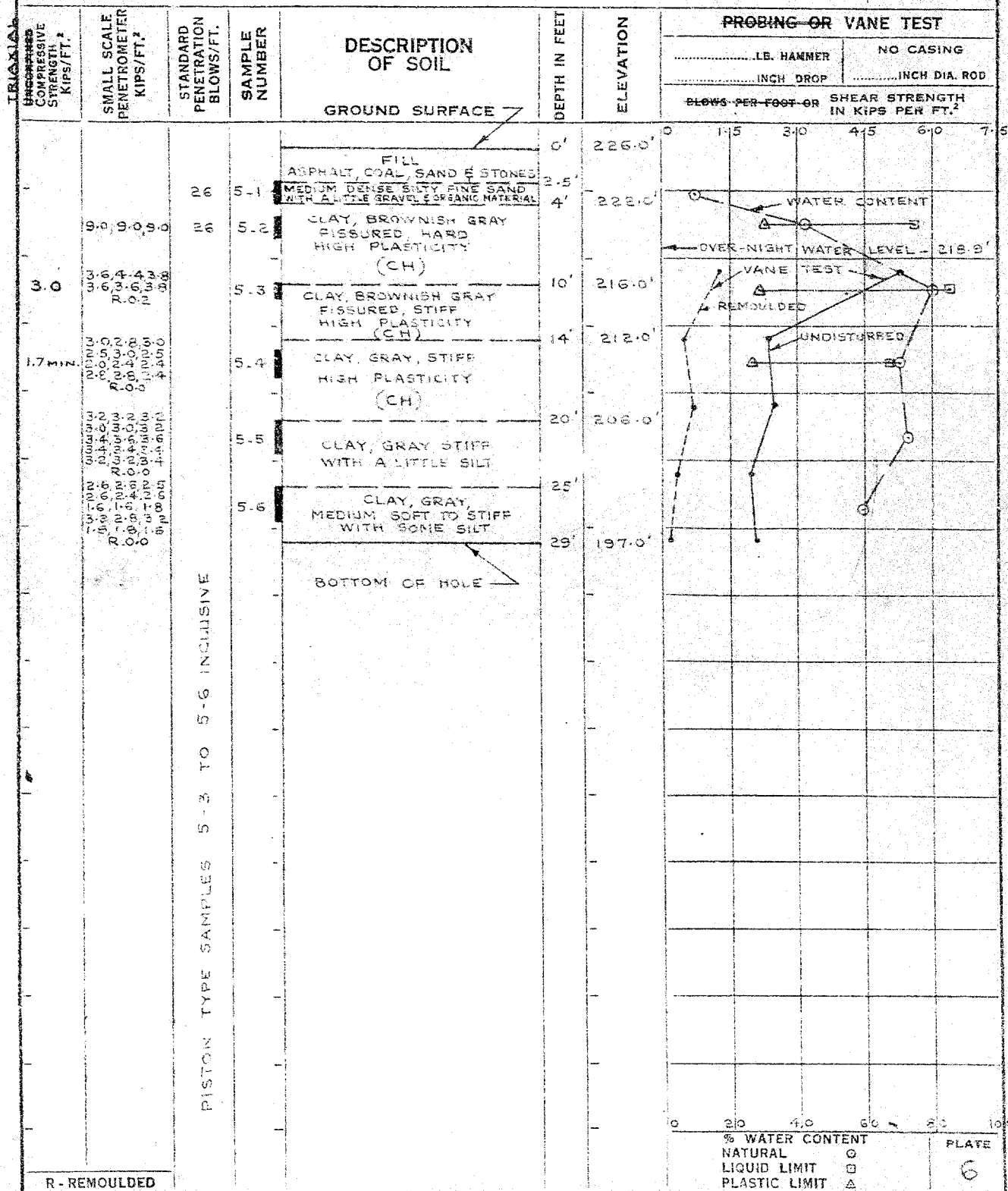
ELEVATION OF GROUND SURFACE (ZERO DEPTH) 226.0'

DATE JAN. 21, 1962

HOLE NO.

REMARKS SEE PLATE No. 2

5



McROSTIE & ASSOCIATES LTD.

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY

OF FIELD AND LABORATORY TESTS

BANK & QUEENSWAY
RETAINING WALL

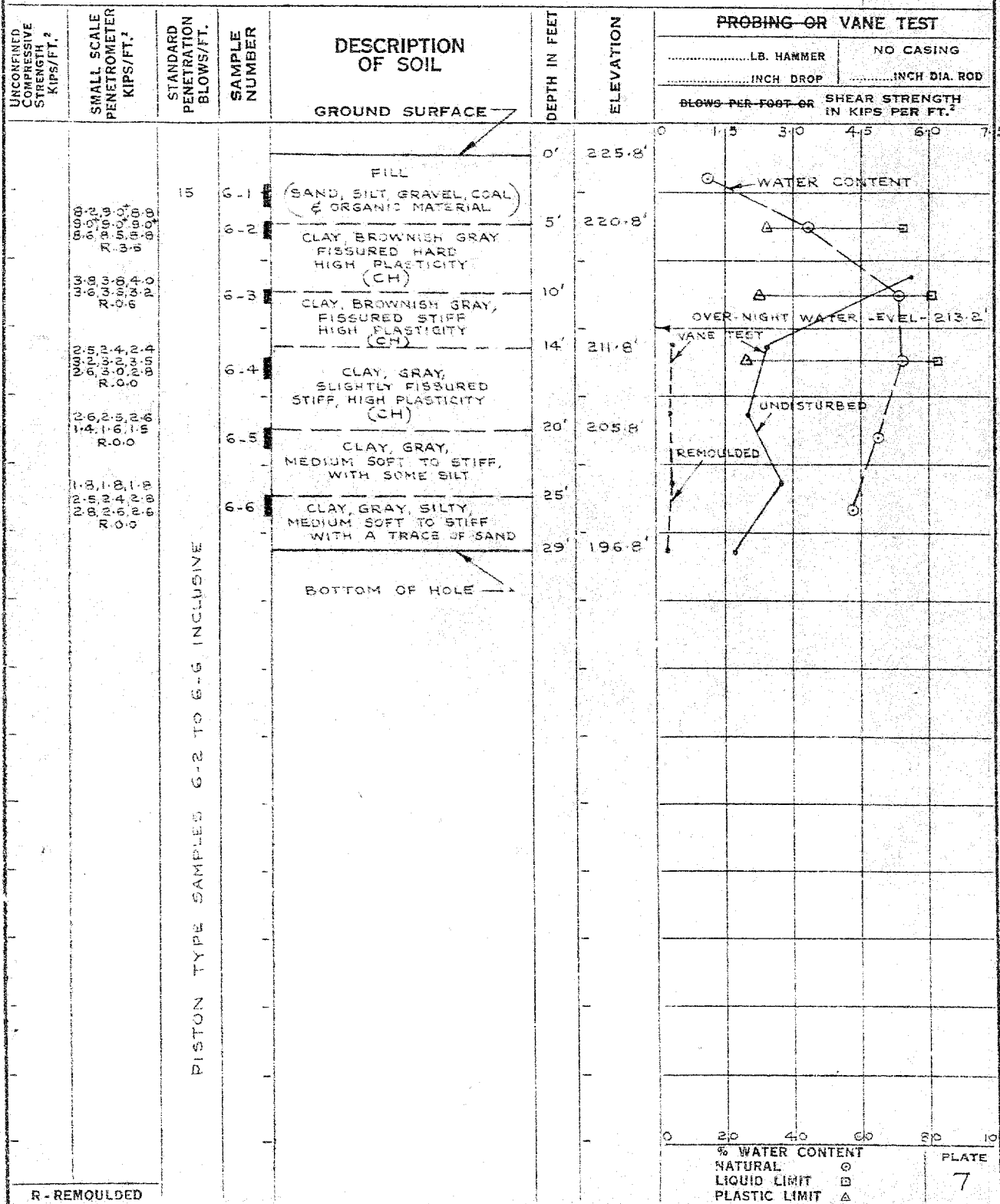
ELEVATION OF GROUND SURFACE (ZERO DEPTH) 225.8'

DATE JAN. 21, 1962

HOLE NO.

REMARKS SEE PLATE No. 2

6



R - REMOULDED

McROSTIE & ASSOCIATES LTD.

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

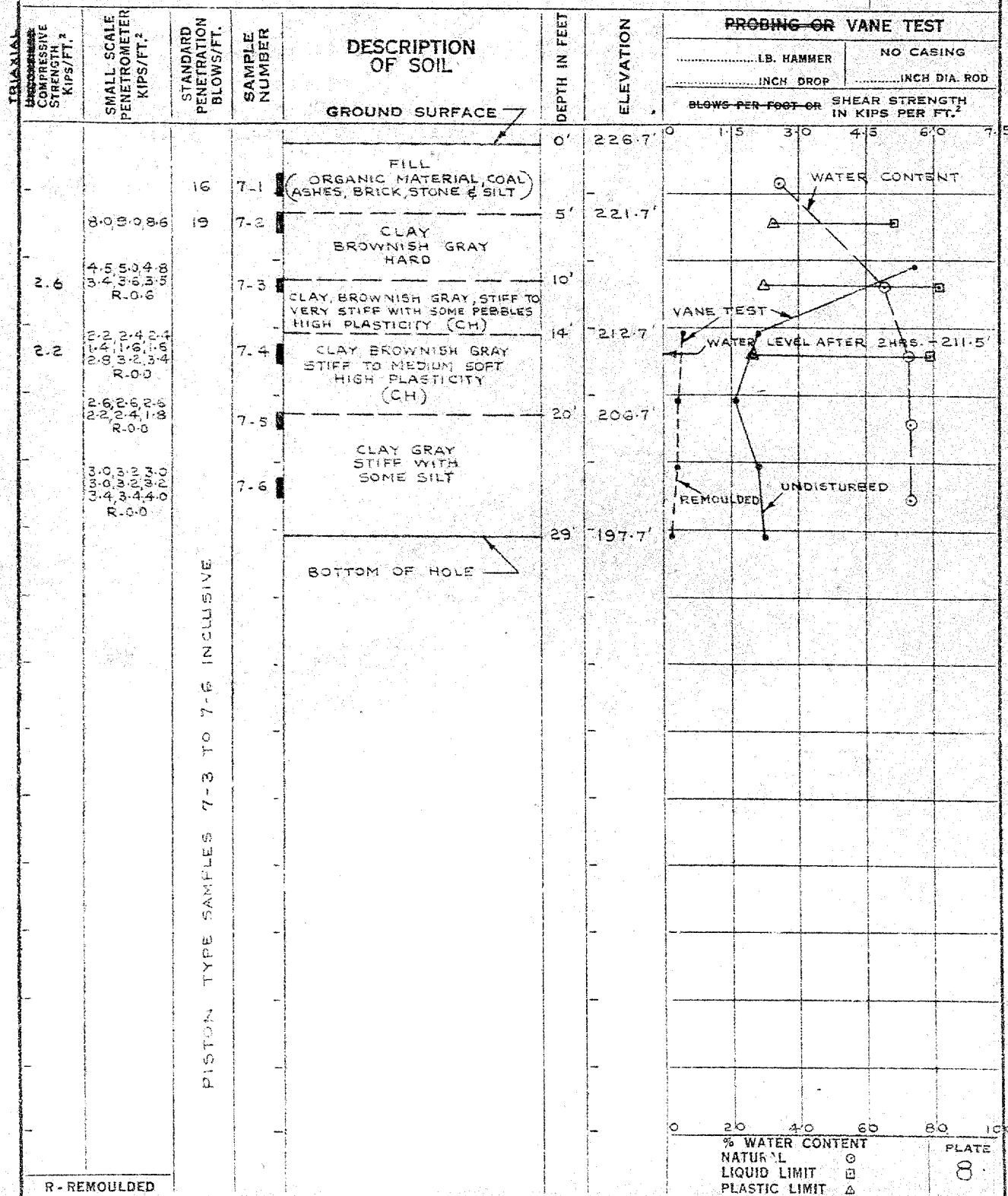
BANK & QUEENSWAY RETAINING WALL

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 226.7'DATE JAN. 24, 1962

HOLE NO.

REMARKS SEE PLATE No. 2

7



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OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

[illegible]ELEVATION OF GROUND SURFACE (ZERO DEPTH) 22.00

DATE 7-2-83

HOLE NO.

REMARKS: 2500 0000 0000 0000

[illegible]

McROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY
OF FIELD AND LABORATORY TESTS

BANK & QUEEN STS.
 OTTAWA, CANADA

FORM S-10-1

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 124.3 DATE FEB. 15, 1961 HOLE NO. 1
 REMARKS SEE FIELD NO. 2

SMALL SCALE PENETROMETER KIP/FT.	STANDARD PENETRATION BLOW/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PROBING OR VANE TEST	
						LB. HAMMER DOWN DEEP	NO CASING DOWN DEEP
						KNOWS PER FOOT OR IN KIPS PER FT.	SHEAR STRENGTH IN KIPS PER FT.
			GROUND SURFACE	0.0	124.3		
				0.5	123.8		
				1.0	123.3		
				1.5	122.8		
				2.0	122.3		
				2.5	121.8		
				3.0	121.3		
				3.5	120.8		
				4.0	120.3		
				4.5	119.8		
				5.0	119.3		
				5.5	118.8		
				6.0	118.3		
				6.5	117.8		
				7.0	117.3		
				7.5	116.8		
				8.0	116.3		
				8.5	115.8		
				9.0	115.3		
				9.5	114.8		
				10.0	114.3		
				10.5	113.8		
				11.0	113.3		
				11.5	112.8		
				12.0	112.3		
				12.5	111.8		
				13.0	111.3		
				13.5	110.8		
				14.0	110.3		
				14.5	109.8		
				15.0	109.3		
				15.5	108.8		
				16.0	108.3		
				16.5	107.8		
				17.0	107.3		
				17.5	106.8		
				18.0	106.3		
				18.5	105.8		
				19.0	105.3		
				19.5	104.8		
				20.0	104.3		
				20.5	103.8		
				21.0	103.3		
				21.5	102.8		
				22.0	102.3		
				22.5	101.8		
				23.0	101.3		
				23.5	100.8		
				24.0	100.3		
				24.5	99.8		
				25.0	99.3		
				25.5	98.8		
				26.0	98.3		
				26.5	97.8		
				27.0	97.3		
				27.5	96.8		
				28.0	96.3		
				28.5	95.8		
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				34.0	90.3		
				34.5	89.8		
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				36.0	88.3		
				36.5	87.8		
				37.0	87.3		
				37.5	86.8		
				38.0	86.3		
				38.5	85.8		
				39.0	85.3		
				39.5	84.8		
				40.0	84.3		
				40.5	83.8		
				41.0	83.3		
				41.5	82.8		
				42.0	82.3		
				42.5	81.8		
				43.0	81.3		
				43.5	80.8		
				44.0	80.3		
				44.5	79.8		
				45.0	79.3		
				45.5	78.8		
				46.0	78.3		
				46.5	77.8		
				47.0	77.3		
				47.5	76.8		
				48.0	76.3		
				48.5	75.8		
				49.0	75.3		
				49.5	74.8		
				50.0	74.3		
				50.5	73.8		
				51.0	73.3		
				51.5	72.8		
				52.0	72.3		
				52.5	71.8		
				53.0	71.3		
				53.5	70.8		
				54.0	70.3		
				54.5	69.8		
				55.0	69.3		
				55.5	68.8		
				56.0	68.3		
				56.5	67.8		
				57.0	67.3		
				57.5	66.8		
				58.0	66.3		
				58.5	65.8		
				59.0	65.3		
				59.5	64.8		
				60.0	64.3		
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				62.5	61.8		
				63.0	61.3		
				63.5	60.8		
				64.0	60.3		
				64.5	59.8		
				65.0	59.3		
				65.5	58.8		
				66.0	58.3		
				66.5	57.8		
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				69.5	54.8		
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				95.0	29.3		
				95.5	28.8		
				96.0	28.3		
				96.5	27.8		
				97.0	27.3		
				97.5	26.8		
				98.0	26.3		
				98.5	25.8		
				99.0	25.3		
				99.5	24.8		
				100.0	24.3		

WATER CONTENT
 NATURAL
 LIQUID LIMIT
 PLASTIC LIMIT

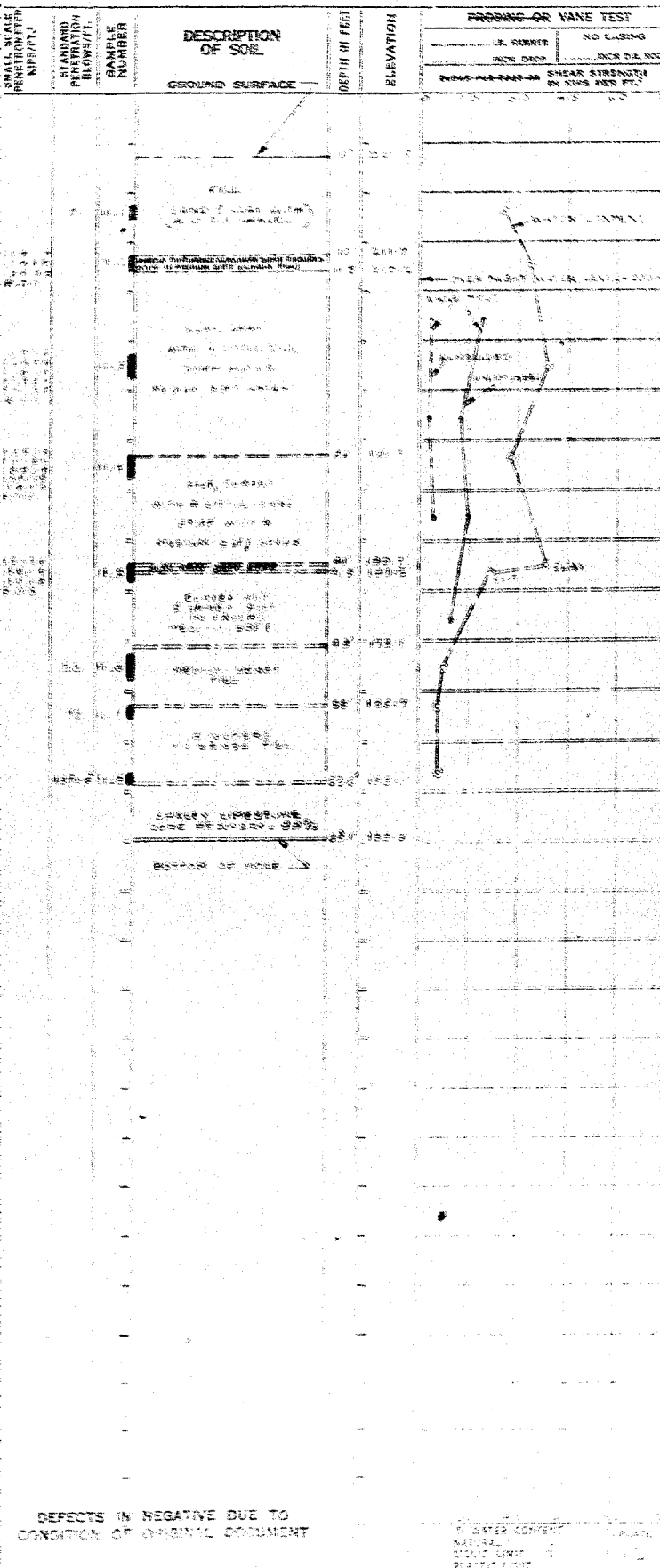
ROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY
OF FIELD AND LABORATORY TESTS

BATA & ROULETWAY
 RETAIL STORE

LOCATION OF GROUND SURFACE (ZERO DEPTH) 2200 DATE 1964 HOLE NO. 1

KS 100



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CONSULTING ENGINEERS

OTTAWA CANADA

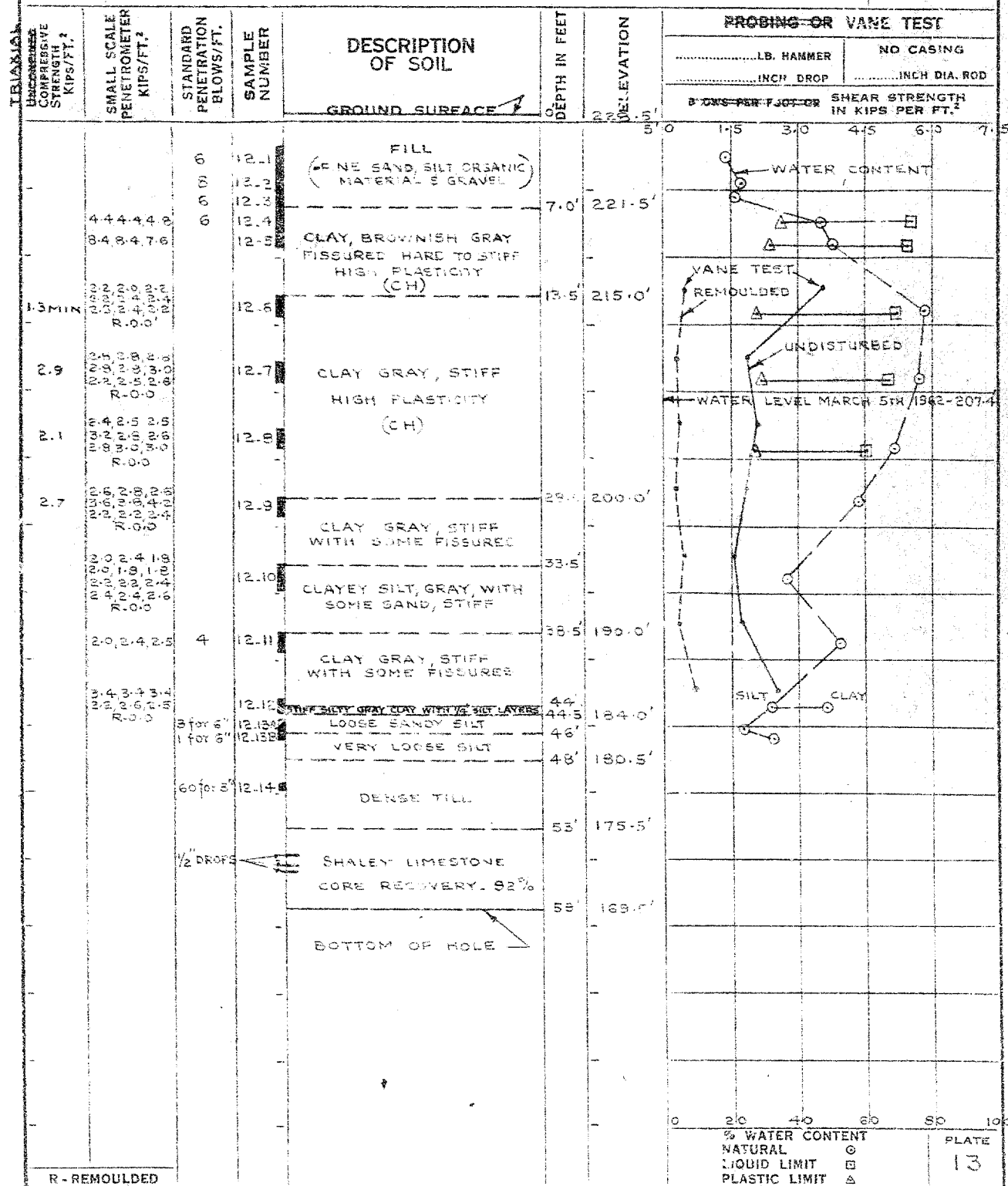
SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

BANK & QUEENSWAY
RETAINING WALLS

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 228.5' DATE MARCH 1, 1962
REMARKS SEE PLATE No. 2

HOLE NO.

12



R - REMOULDED

PLATE

13

McROSTIE & ASSOCIATES LTD.

CONSULTING ENGINEERS

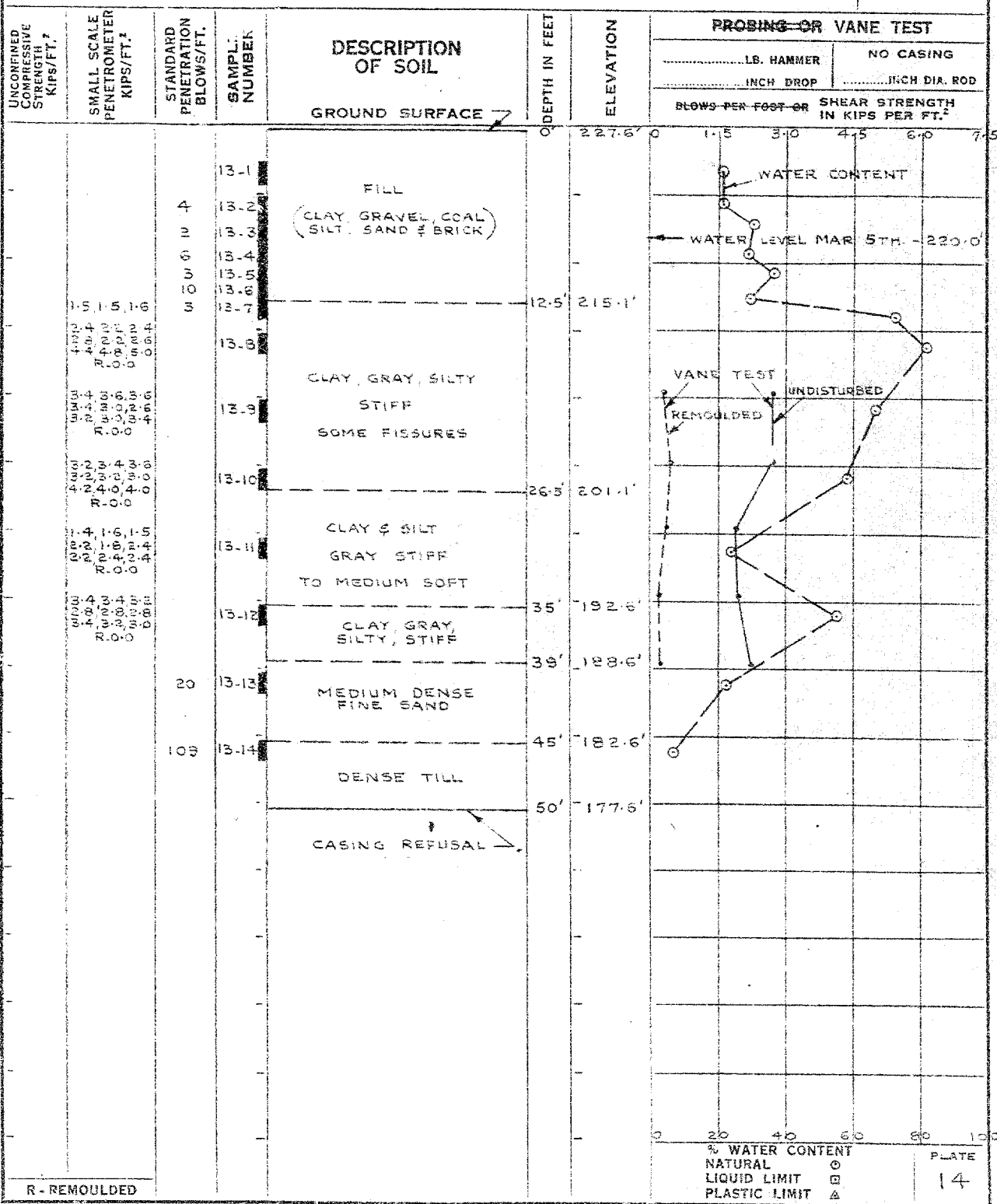
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

BANK & QUEENSWAY
RETAINING WALLS

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 227.6' DATE MAR. 2, 1962
REMARKS SEE PLATE No. 2

HOLE NO.
13



McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

**SOIL PROFILE AND SUMMARY
OF FIELD AND LABORATORY TESTS**

BANK & QUEENSWAY
RETAINING WALLS

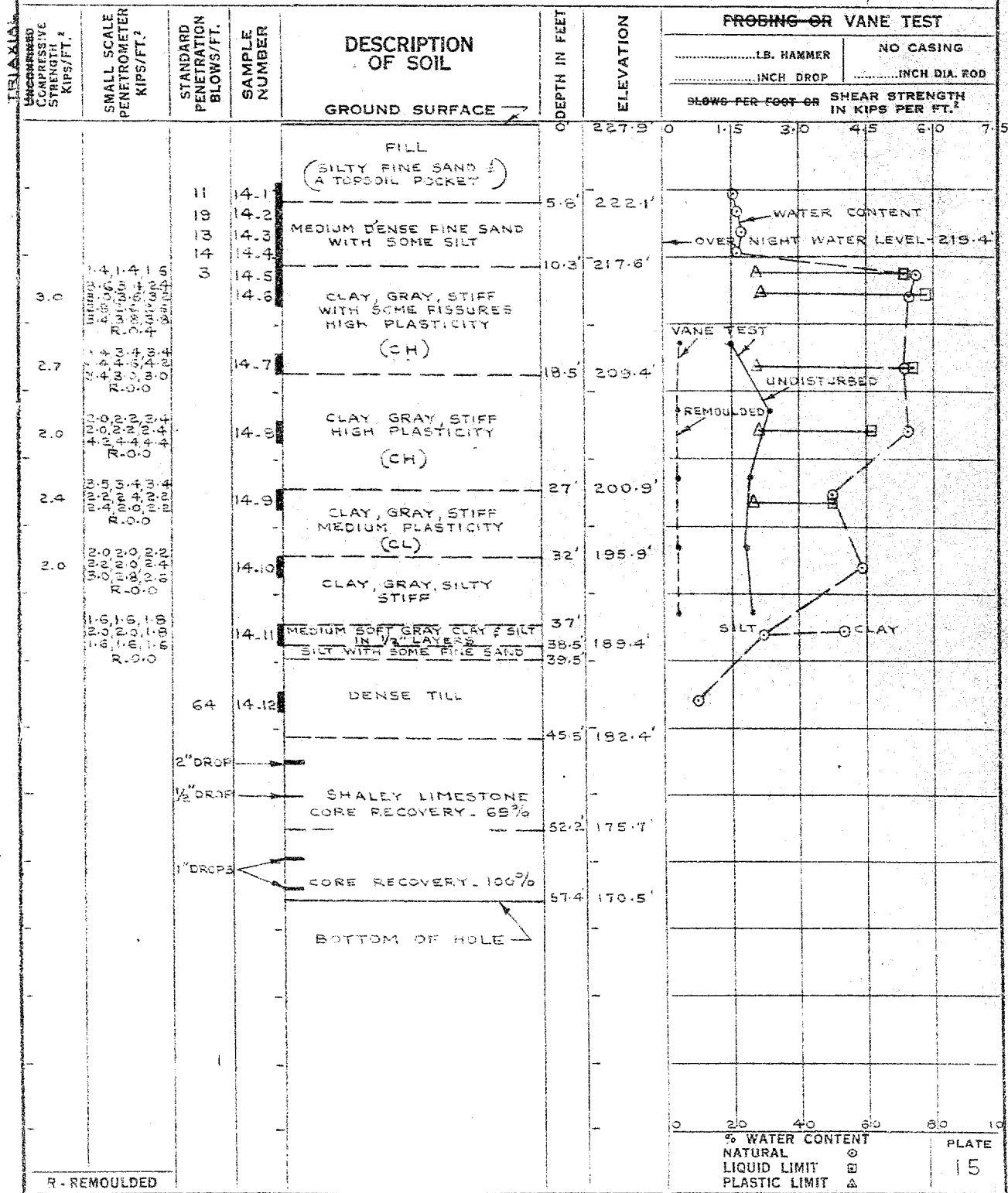
ELEVATION OF GROUND SURFACE (ZERO DEPTH) 227.9'

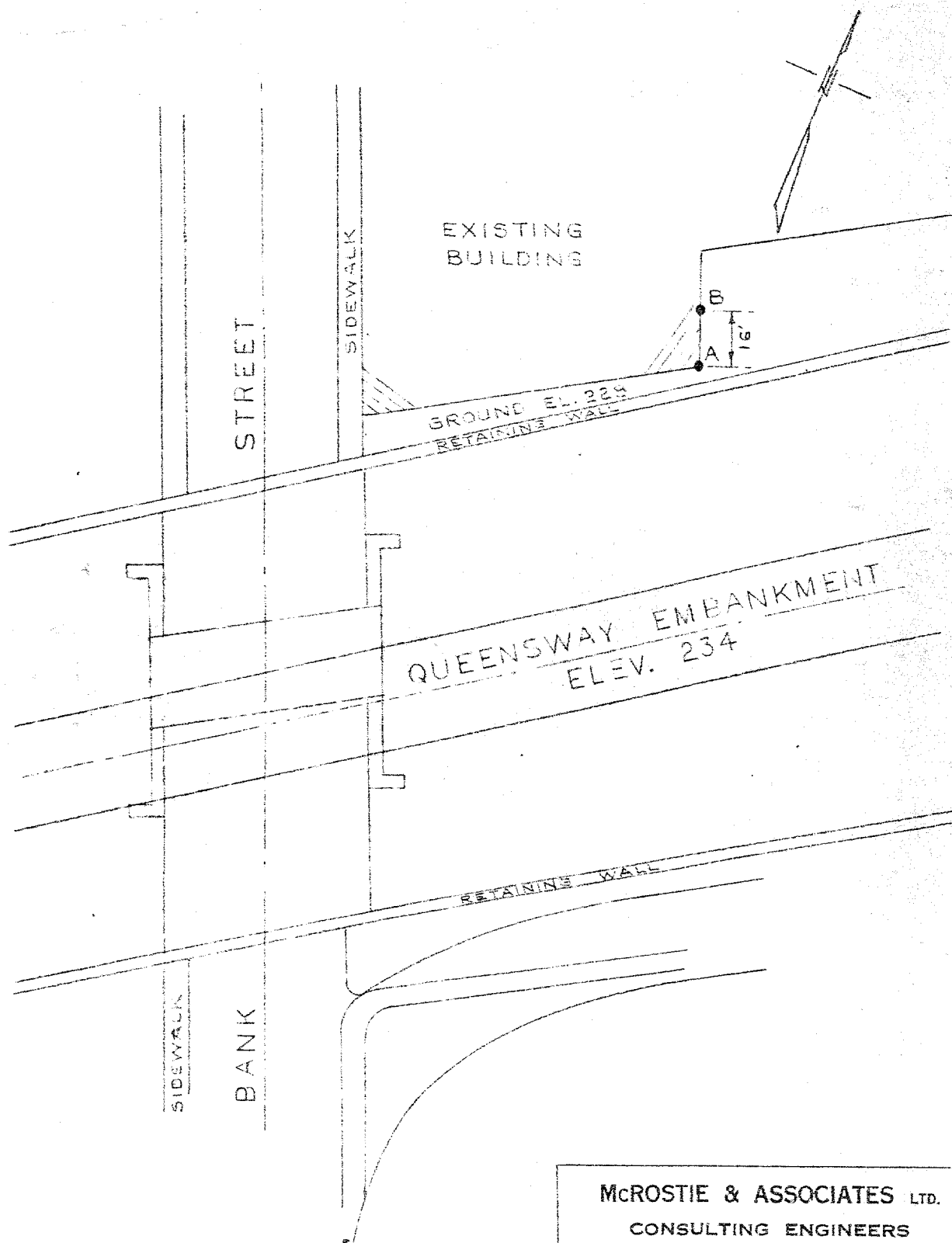
DATE FEB. 24, 1962

HOLE No.

REMARKS SEE PLATE No. 2

14





REVISED AS PER
DE LEON CATHY & CO. OF CANADA LTD.
DWG. No. C44 W-5 DATED DEC. 12, 1961.

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS

~~BOREHOLE LOGS~~
SETTLEMENT
COMPUTATION DATA

SCALE 1" = 40'

PLATE 8

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 23-1-62	
HOLE NO. 3		LOCATION:		DEPTH: 10'6"-11'6"	
TEST NO.	3-3A	3-3B			AVERAGE
DEPTH	11'0" 11'6"	10'6" 11'0"			
LATERAL PRESSURE (PSI)	10	40			
COMPRESSIVE STRESS (PSI) - qc	14.2	18.4			
WATER CONTENT - W%	73.9	64.2			
WET DENSITY - γ_m					
$C = 8 \text{ PSI.}$ $\phi = 0^\circ$					
<p style="text-align: center;">(CH) HIGHLY PLASTIC CLAY</p>					

TESTED: D.M. DATE: 23-1-62

COMPUTED: D.M. DATE: 23-1-62

PLOTTED: D.M. DATE: 23-1-62

CHECKED: G.B. DATE: 24-1-62

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 23-1-62	
HOLE NO. 3		LOCATION:		DEPTH: 15'0"-16'6"	
TEST NO.	3-4A	3-4B	3-4C		
DEPTH	16'0" 16'6"	15'6" 16'0"	15'0" 15'6"		
LATERAL PRESSURE (P.S.I.)	10	40	40		
COMPRESSIVE STRESS (P.S.I.) - q_c	16.8	9.2	13.2		
WATER CONTENT - W%	71.7	64.6	75.2		
WET DENSITY - γ_m					
$C = 7.5 \text{ P.S.I.}$ $\phi = 0^\circ$					
<p style="text-align: center;">PRINCIPAL STRESS P.S.I.</p> <p style="transform: rotate(-90deg); position: absolute; left: 150px; top: 550px;">SHEAR STRESS - P.S.I.</p> <p style="position: absolute; left: 130px; top: 570px;">MOHR DIAGRAM</p>					
(CH) HIGHLY PLASTIC CLAY					

TESTED: O.M. DATE: 23-1-62

COMPUTED: O.M. DATE: 23-1-62

PLOTTED: O.M. DATE: 23-1-62

CHECKED: G.B. DATE: 24-1-62

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

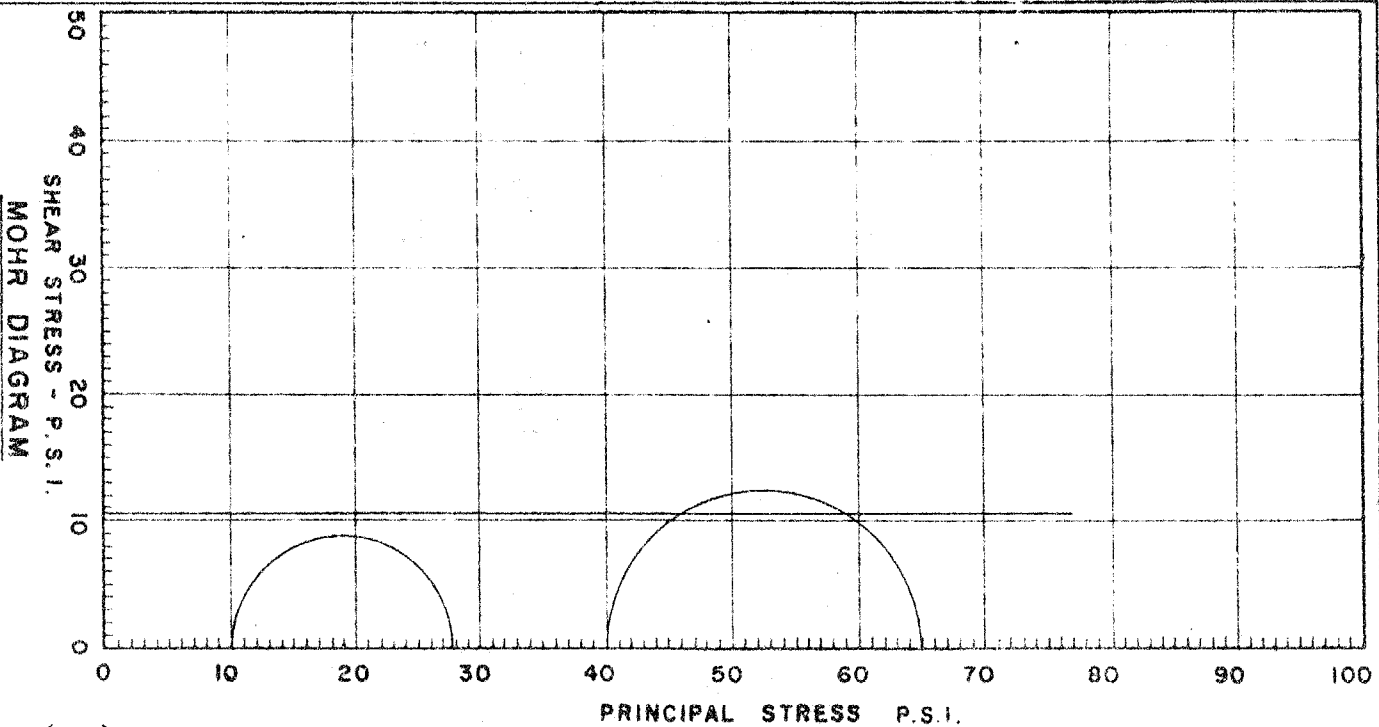
TEST SERIES SUMMARY SHEET

PROJECT: E-440 SITE: BANK & QUEENSWAY DATE: 25-1-62
 HOLE NO. 5 LOCATION: DEPTH: 10'0" - 11'0"

TEST NO.	5-3A	5-3B							AVERAGE
DEPTH	10' 6" 11' 0"	10' 0" 10' 6"							
LATERAL PRESSURE (PSI)	10	40							
COMPRESSIVE STRESS (PSI) - qc	17.9	25.0							
WATER CONTENT - W %	73.5	72.3							
WET DENSITY - γ_m									

C = 10.5 P.S.I.

$\phi = 0^\circ$



(CH) HIGHLY PLASTIC CLAY

TESTED: D.M.
 COMPUTED: D.M.
 PLOTTED: D.M.
 CHECKED: D.M.

DATE: 25-1-62

MCROSTIE & ASSOCIATES
 CONSULTING ENGINEERS
 OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 25-1-62	
HOLE NO. 5		LOCATION:		DEPTH: 15'6"-17'0"	
TEST NO.	5-4A	5-4B	5-4C		AVERAGE
DEPTH	16'6" 17'0"	16'0" 16'6"	15'6" 16'0"		
LATERAL PRESSURE (PSI)	10	40	10		
COMPRESSIVE STRESS (P.S.I.) - qc	12.2	18.4	11.0		
WATER CONTENT - W%	65.0	74.5	65.0		
WET DENSITY - γ_m					
$C = 6 \text{ P.S.I. MIN.}$ $\phi = 0^\circ$					
<p style="text-align: center;">(CH) HIGHLY PLASTIC CLAY</p>					

TESTED: _____ DATE: _____

COMPUTED: _____ DATE: _____

PLOTTED: _____ DATE: 25-1-62

CHECKED: _____ DATE: _____

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 25-1-62	
HOLE NO. 7		LOCATION:		DEPTH: 10'0" - 11'0"	
TEST NO.	7-3A	7-3B			AVERAGE
DEPTH	10' 6" 11' 0"	10' 0" 10' 6"			
LATERAL PRESSURE (PSI)	10	40			
COMPRESSIVE STRESS (PSI) - qc	21.8	14.7			
WATER CONTENT - W%	62.9	64.3			
WET DENSITY - γ_m					
$C = 9.3 \text{ PSI}$ $\phi = 0^\circ$					
<p style="text-align: center;">MOHR DIAGRAM</p> <p style="text-align: center;">SHEAR STRESS - P.S.I.</p> <p style="text-align: center;">PRINCIPAL STRESS P.S.I.</p>					
(CH) HIGHLY PLASTIC CLAY					

TESTED: _____ DATE: _____

COMPUTED: _____ DATE: _____

PLOTTED: D.M. DATE: 25-1-62

CHECKED: _____ DATE: _____

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

TRIAxIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		HOLE NO. 7		LOCATION:		SITE: BANK @ QUEENSWAY		DATE: 25-1-62	
TESTED:		7-4A		7-4B		7-4C		AVERAGE	
COMPUTED:		16' 0"		15' 6"		15' 0"			
PLOTTED:		16' 6"		16' 0"		15' 6"			
CHECKED:		10		40		10			
DATE:		14.0		18.05		7.5			
DATE:		73.9		80.0		74.7			
DATE:		WET DENSITY - γ_m							
DATE:		C = 8 P.S.I.		$\phi = 0^\circ$					
DATE:									
DATE:									
DATE:									
DATE:									
DATE:		<p>(CH) HIGHLY PLASTIC CLAY</p>							

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

FORM S-24

TRIAXIAL COMPRESSION TEST

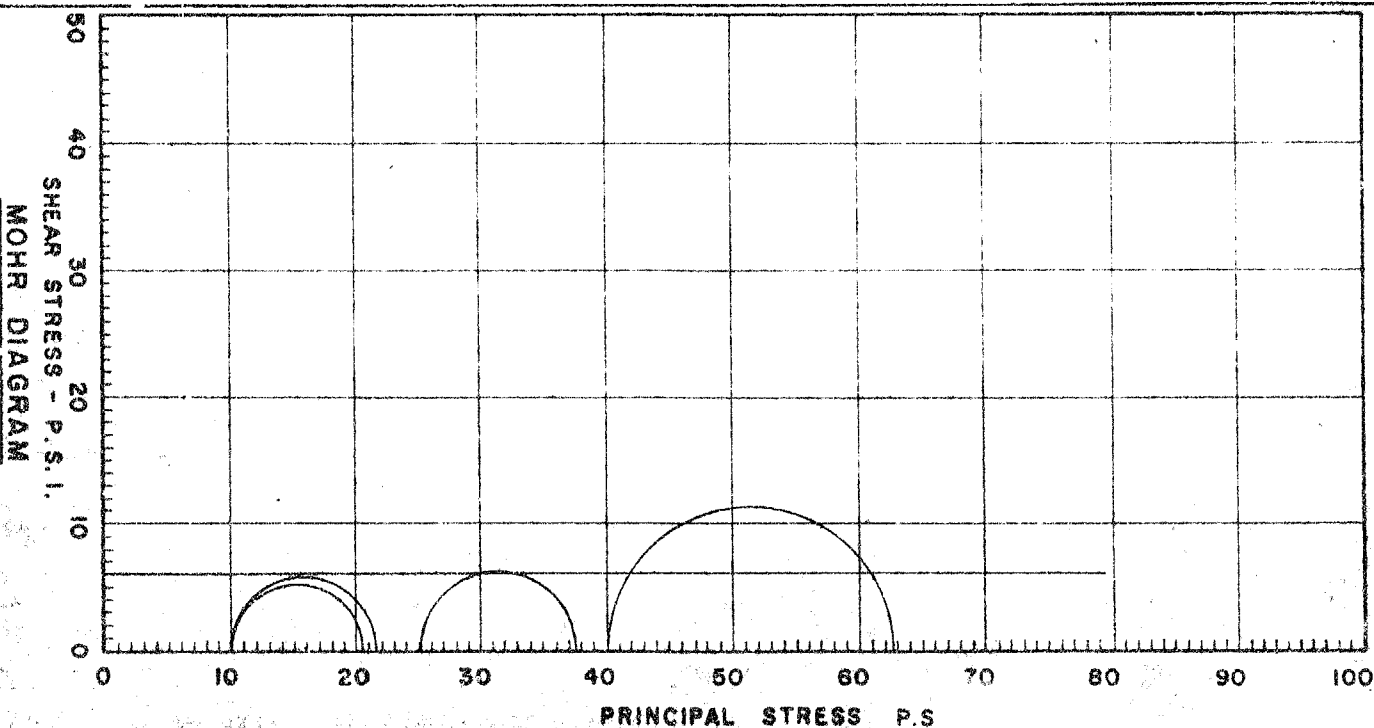
TEST SERIES SUMMARY SHEET

PROJECT: E-4-40 SITE: BANK & QUEENSWAY DATE: 9-2-62
 HOLE NO. 8 LOCATION: DEPTH: 76"-9'0"

TEST NO.	8-3A	8-3B	8-3C	8-3D						AVERAGE
DEPTH	8'6" 9'0"	8'2" 8'6"	7'10" 8'2"	7'6" 7'10"						
LATERAL PRESSURE (P.S.I.)	10	40	10	25						
COMPRESSIVE STRESS (P.S.I.) - qc	11.3	22.8	10.3	12.5						
WATER CONTENT - W %	73.0	76.5	70.6	81.2						
WET DENSITY - γ_m										

C = 6 P.S.I. MIN.

$\phi = 0^\circ$



(CH) HIGHLY PLASTIC CLAY

TESTED: _____
 COMPUTED: _____
 PLOTTED: D.M.
 CHECKED: G.B.
 DATE: _____
 DATE: 9-2-62
 DATE: 9-2-62

MCROSTIE & ASSOCIATES
 CONSULTING ENGINEERS
 OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

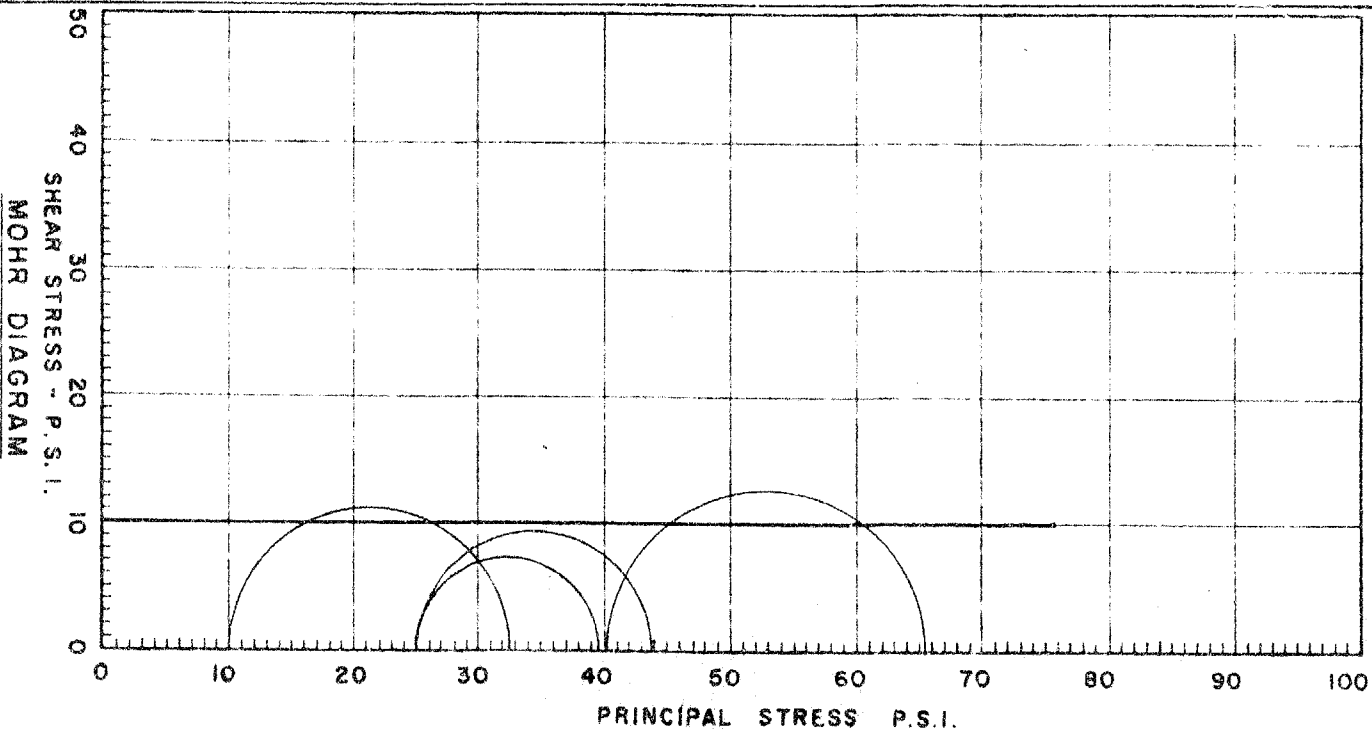
TEST SERIES SUMMARY SHEET

PROJECT: E-440 SITE: BANK & QUEENSWAY DATE: 9-2-62
 HOLE NO. 8 LOCATION: DEPTH: 12'6" - 14'0"

TEST NO.	8-4A	8-4B	8-4c	8-4D					AVERAGE
DEPTH	13' 7" 14' 0"	13' 3" 13' 7"	12' 10" 13' 3"	12' 6" 12' 10"					
LATERAL PRESSURE (PSI)	10	40	25	25					
COMPRESSIVE STRESS (P.S.I.) - qc	22.8	25.8	14.5	18.7					
WATER CONTENT - W%	66.2	68.8	68.2	78.5					
WET DENSITY - γ_m									

C = 10 P.S.I.

$\phi = 0^\circ$



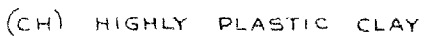
TESTED: _____
 COMPUTED: _____
 PLOTTED: D.M. _____
 CHECKED: G.B. _____
 DATE: _____
 DATE: 9-2-62
 DATE: 9-2-62

MCROSTIE & ASSOCIATES
 CONSULTING ENGINEERS
 OTTAWA CANADA

TEST SERIES SUMMARY SHEET

DATE: 9-2-62

DEPTH: 17'6" - 19'0"

$$\Phi = O^2$$
[illegible]

CONSULTING ENGINEERS

OTTAWA CANADA

TRIAxIAL COMPRESSION TEST

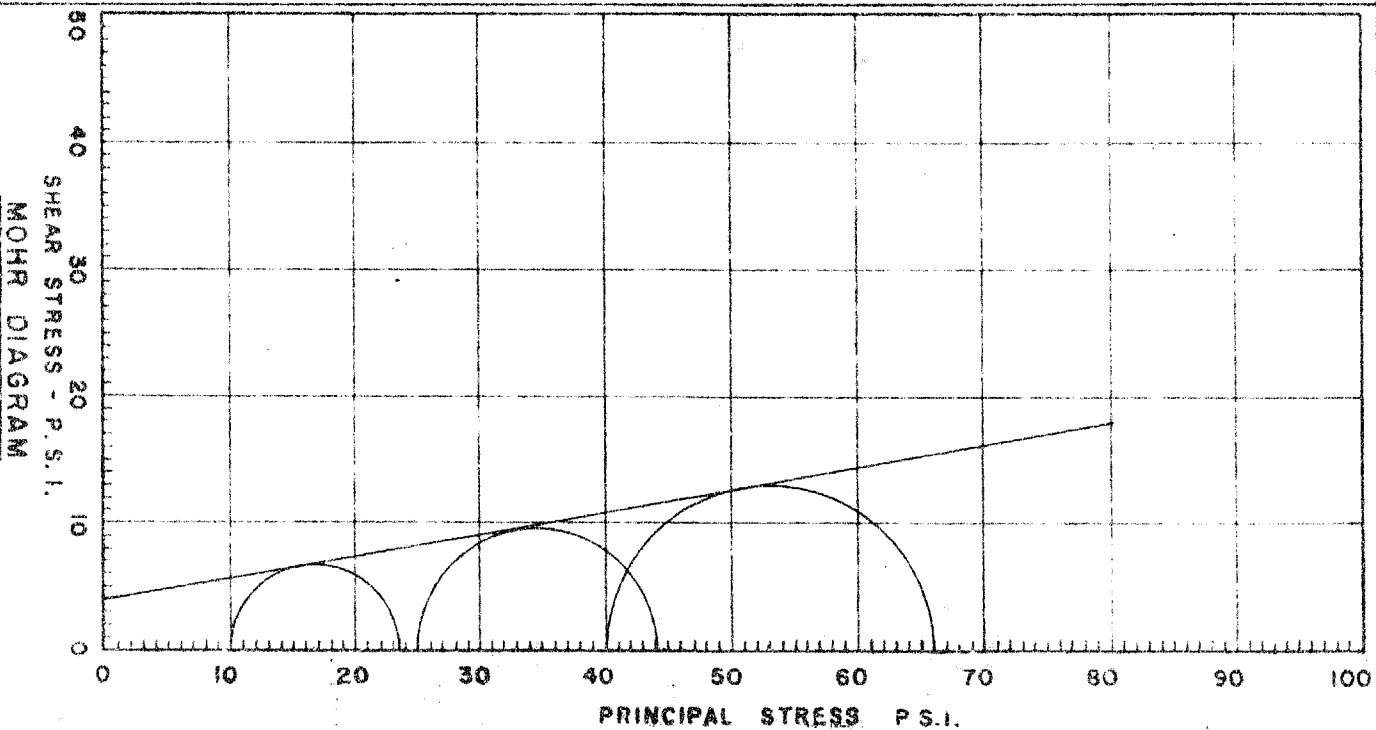
TEST SERIES SUMMARY SHEET

PROJECT: E-440 SITE: BANK & QUEENSWAY DATE: 9-2-62
 HOLE NO. 8 LOCATION: DEPTH: 22'6"-24'0"

TEST NO.	8-6A	8-6B	8-6C	8-6D					AVERAGE
DEPTH	23'6" 24'0"	23'2" 23'6"	22'10" 23'2"	22'6" 22'10"					
LATERAL PRESSURE (PSI)	10	40	25	10					
COMPRESSIVE STRESS (PSI) - qc	13.2	26.0	19.1	13.1					
WATER CONTENT - W%	61.0	59.7	61.4	62.3					
WET DENSITY - γ_m									

$C = 4.0$ P.S.I.

$\phi = 10^\circ$



(CL) MEDIUM PLASTIC CLAY

TESTED: _____
 COMPUTED: _____
 PLOTTED: O.M.
 CHECKED: G.B.
 DATE: 9-2-62
 DATE: 9-2-62

MCROSTIE & ASSOCIATES
 CONSULTING ENGINEERS
 OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 12-2-62	
HOLE NO. 10		LOCATION:		DEPTH: 10'0"-11'6"	
TEST NO.	10-4A	10-4B	10-4c		
DEPTH	11'0" 11'6"	10'6" 11'0"	10'0" 10'6"		
LATERAL PRESSURE (PSI)	15	45	30		
COMPRESSIVE STRESS (PSI) - qc	12.4	15.1	22.9		
WATER CONTENT - W %	62.0	66.5	74.4		
WET DENSITY - γ_m					
$C = 7 \text{ P.S.I.}$ $\phi = 0^\circ$					
<p>MOHR DIAGRAM</p> <p>(CH) HIGHLY PLASTIC CLAY</p>					
TESTED: D.M.	DATE: 12-2-62	MCROSTIE & ASSOCIATES			
COMPUTED: D.M.	DATE: 12-2-62	CONSULTING ENGINEERS			
PLOTTED: G.B.	DATE: 12-2-62	OTTAWA CANADA			
CHECKED: G.B.	DATE: 12-2-62				

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 9-2-62	
HOLE NO. 10		LOCATION:		DEPTH: 15'0"-16'6"	
TEST NO.	10-5A	10-5B	10-5c		
DEPTH	16'0" 16'6"	15'6" 16'0"	15'0" 15'6"		
LATERAL PRESSURE (PSI)	15	45	30		
COMPRESSIVE STRESS (P.S.I.) - qc	18.3	20.7	20.2		
WATER CONTENT - W %	70.4	72.3	74.5		
WET DENSITY - γ_m					
$C = 10.2 \text{ P.S.I.}$ $\phi = 0^\circ$					
<p>MOHR DIAGRAM</p> <p>PRINCIPAL STRESS P.S.I.</p> <p>SHEAR STRESS - P.S.I.</p> <p>(CH) HIGHLY PLASTIC CLAY</p>					

TESTED: D.M. DATE: 9-2-62

COMPUTED: G.B. DATE: 9-2-62

PLOTTED: G.B. DATE: 9-2-62

CHECKED: A.G. DATE: 10-2-62

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 10-2-62	
HOLE NO. 10		LOCATION:		DEPTH: 20'0"-21'6"	
TEST NO.	10-6A	10-6B	10-6c	10-6d	AVERAGE
DEPTH	21'0" 21'6"	20'8" 21'0"	20'4" 20'8"	20'0" 20'4"	
LATERAL PRESSURE (P.S.I.)	15	45	30	15	
COMPRESSIVE STRESS (P.S.I.) - qc	34.3	31.2	17.3	21.9	
WATER CONTENT - W %	62.9	58.0	61.3	59.6	
WET DENSITY - γ_m					
$C = 10 \text{ P.S.I. MIN.}$ $\phi =$					
<p>MOHR DIAGRAM</p> <p>(CH) HIGHLY PLASTIC CLAY</p>					

TESTED: D.M.

COMPUTED: G.B.

PLOTTED: G.B.

CHECKED: A.G.

DATE: 9-2-62

DATE: 10-2-62

DATE: 10-2-62

DATE: 10-2-62

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 2-3-62	
HOLE NO. 12		LOCATION:		DEPTH: 13'6"-15'0"	
TEST NO.	12-6A	12-6B	12-6C		AVERAGE
DEPTH	14'6" 15'0"	14'0" 14'6"	13'6" 14'0"		
LATERAL PRESSURE (PSI)	10	40	25		
COMPRESSIVE STRESS (P.S.I.) - qc	8.5	24.0	9.4		
WATER CONTENT - W %	65.8	74.9	67.4		
WET DENSITY - γ_m					
$C = 4.6 \text{ PSI MIN}$ $\phi = 0^\circ$					
<p style="text-align: center;">(CH) HIGHLY PLASTIC CLAY</p>					
TESTED: D.M.	DATE: 5-3-62	<p style="text-align: center;">MCROSTIE & ASSOCIATES CONSULTING ENGINEERS OTTAWA CANADA</p>			
COMPUTED: D.M.	DATE: 5-3-62				
PLOTTED: G.B.	DATE: 5-3-62				
CHECKED: G.B.	DATE: 5-3-62				

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 2-3-62	
HOLE NO. 12		LOCATION:		DEPTH: 18'6" - 20'0"	
TEST NO.	12.7A	12.7B	12.7C		AVERAGE
DEPTH	19'6" 20'0"	19'0" 19'6"	18'6" 19'0"		
LATERAL PRESSURE (PSI)	10	40	40		
COMPRESSIVE STRESS (P.S.I.) - qc	25.4	20.6	19.0		
WATER CONTENT - W%	72.5	69.2	74.8		
WET DENSITY - γ_m					
$C = 10 \text{ P.S.I.}$ $\phi = 0^\circ$					
<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;">(CH) HIGHLY PLASTIC CLAY</p>					
TESTED: O.M.	DATE: 5-3-62				
COMPUTED: D.M.	DATE: 5-3-62				
PLOTTED: G.B.	DATE: 5-3-62				
CHECKED: G.B.	DATE: 5-3-62				
<p style="text-align: center;">MCROSTIE & ASSOCIATES CONSULTING ENGINEERS OTTAWA CANADA</p>					

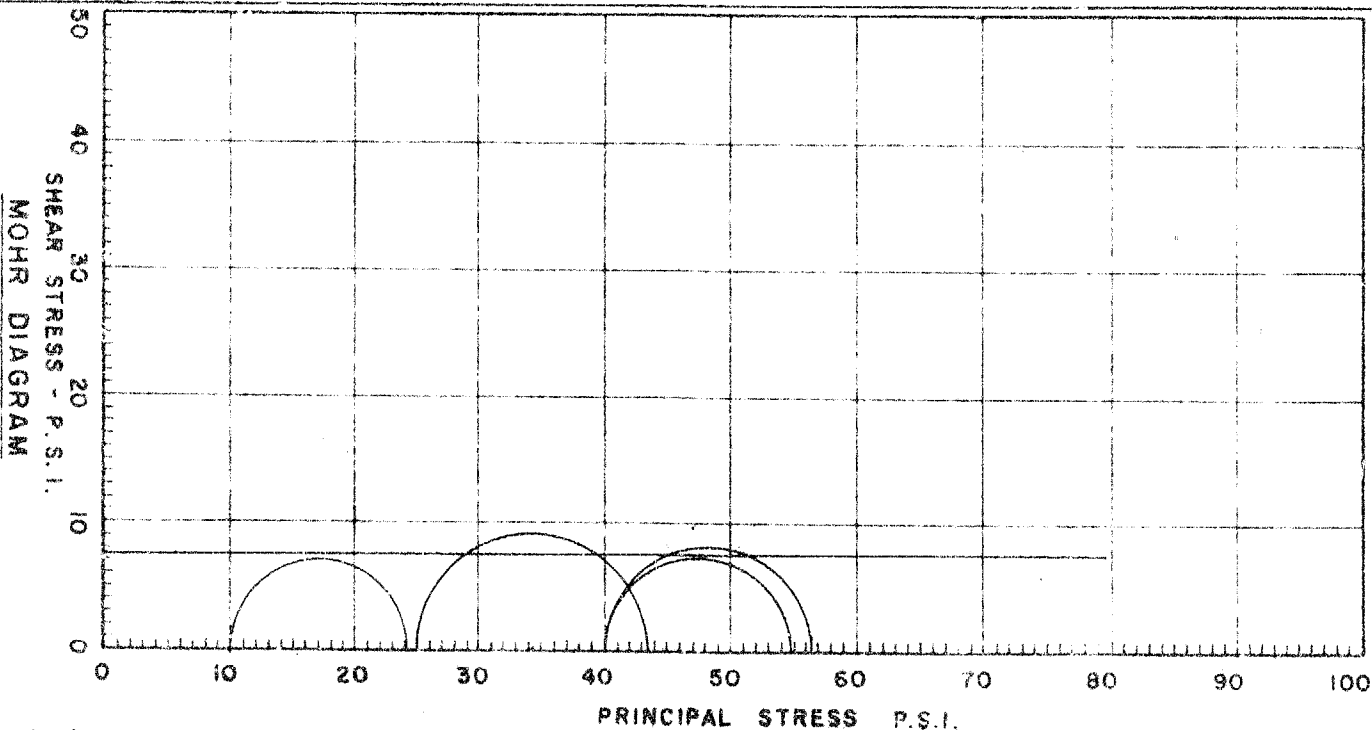
TRIAXIAL COMPRESSION TEST TEST SERIES SUMMARY SHEET

PROJECT: E-440 SITE: BANK & QUEENSWAY DATE: 5-3-62
HOLE NO. 12 LOCATION: DEPTH: 23'6"-25'0"

TEST NO.	12-8A	12-8B	12-8C	12-8D					AVERAGE
DEPTH	24'6" 25'0"	4'0" 24'6"	23'9" 24'0"	23'6" 23'9"					
LATERAL PRESSURE (P.S.I.)	10	40	25	40					
COMPRESSIVE STRESS (P.S.I.) - qc	14.1	16.5	18.6	14.8					
WATER CONTENT - W%	63.9	63.5	71.5	65.9					
WET DENSITY - γ_m									

C = 7.5 P.S.I.

$\phi = 0^\circ$



TESTED: D.M. DATE: 5-3-62
COMPUTED: G.B. DATE: 5-3-62
PLOTTED: G.B. DATE: 5-3-62
CHECKED: D.M. DATE: 5-3-62

MCROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

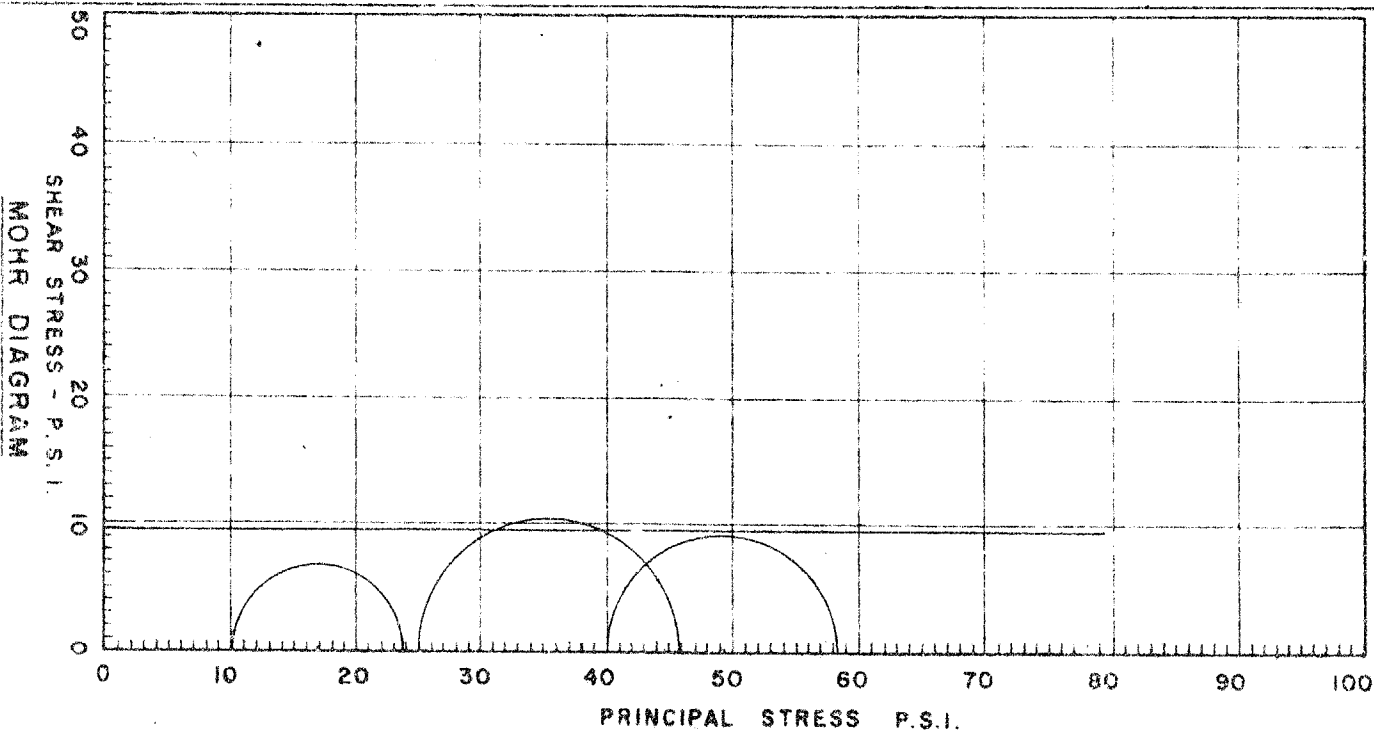
TEST SERIES SUMMARY SHEET

PROJECT: E-440 SITE: BANK & QUEENSWAY DATE: 5-3-62
 HOLE NO. 12 LOCATION: DEPTH: 28'6"-30'0"

TEST NO.	12-9A	12-9B	12-9c							AVERAGE
DEPTH	29'6" 30'0"	29'0" 29'6"	28'6" 29'0"							
LATERAL PRESSURE (PSI)	10	40	25							
COMPRESSIVE STRESS (PSI) - qc	13.9	18.4	20.9							
WATER CONTENT - W%	57.8	60.0	57.4							
WET DENSITY - γ_m										

$C = 9.5 \text{ P.S.I.}$

$\phi = 0^\circ$



MOHR DIAGRAM

SHEAR STRESS - P.S.I.

PRINCIPAL STRESS P.S.I.

TESTED: D.M. DATE: 5-3-62
 COMPUTED: D.M. DATE: 5-3-62
 PLOTTED: G.B. DATE: 5-3-62
 CHECKED: G.B. DATE: 5-3-62

MCROSTIE & ASSOCIATES
 CONSULTING ENGINEERS
 OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 28-2-62	
HOLE NO. 14		LOCATION:		DEPTH: 12'0"-13'4"	
TEST NO.	14-6A	14-6B	14-6C		AVERAGE
DEPTH	13'0" 13'4"	12'6" 13'0"	12'0" 12'6"		
LATERAL PRESSURE (PSI)	10	40	10		
COMPRESSIVE STRESS (P.S.I.) - qc	13.6	10.4	21.9		
WATER CONTENT - W%	76.0	67.6	73.5		
WET DENSITY - γ_m					
$C = 10.5 \text{ P.S.I.}$ $\phi = 0^\circ$					
<p>(CH) HIGHLY PLASTIC CLAY</p>					

TESTED: D.M.

COMPUTED: D.M.

PLOTTED: D.M.

CHECKED: G.B.

DATE: 28-2-62

DATE: 28-2-62

DATE: 28-2-62

DATE: 1-3-62

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

TRIAxIAL COMPRESSION TEST

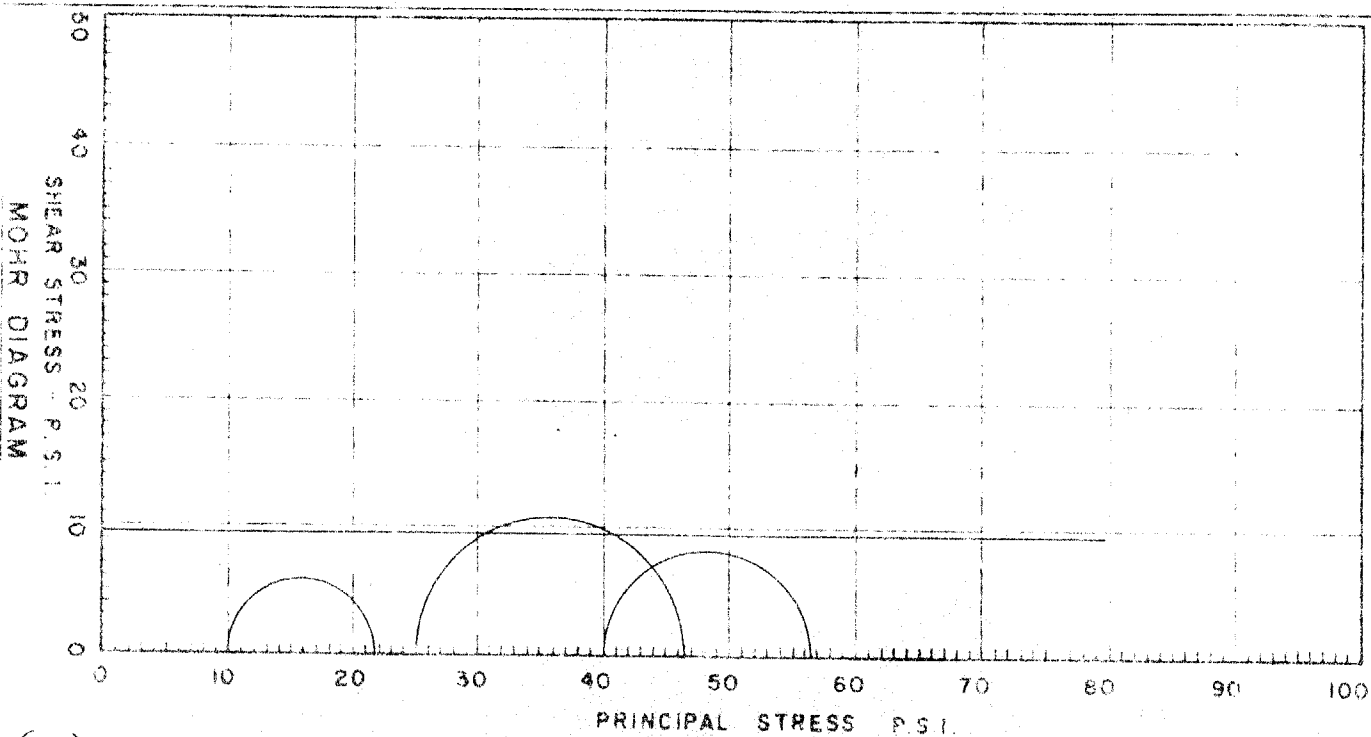
TEST SERIES SUMMARY SHEET

PROJECT: E-440 SITE: BANK & QUEENSWAY DATE: 28-2-62
 HOLE NO. 14 LOCATION: DEPTH: 170"-186"

TEST NO.	14-7A	14-7B	14-7C						AVERAGE
DEPTH	18'0" 18'6"	17'6" 18'0"	17'0" 17'6"						
LATERAL PRESSURE (PSI)	10	40	25						
COMPRESSIVE STRESS (PSI) - qc	11.8	16.7	21.6						
WATER CONTENT - W%	72.8	71.1	69.0						
WET DENSITY - γ_m									

$C = 9.5 \text{ P.S.I.}$

$\phi = 0^\circ$



TESTED: D.M. DATE: 28-2-62
 COMPUTED: D.M. DATE: 28-2-62
 PLATTED: D.M. DATE: 28-2-62
 CHECKED: G.B. DATE: 1-3-62

MOROSIE & ASSOCIATES
 CONSULTING ENGINEERS
 OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 20-2-62	
HOLE NO. 14		LOCATION:		DEPTH: 22'0" - 23'6"	
TEST NO.	14-8A	14-8B	14-8C		
DEPTH	23'0" 23'6"	22'6" 23'0"	22'0" 22'6"		
LATERAL PRESSURE (PSI)	10	40	25		
COMPRESSIVE STRESS (P.S.I.) - q_c	14.3	14.7	12.6		
WATER CONTENT - W %	67.2	63.7	70.9		
WET DENSITY - γ_m					
$C = 7.1 \text{ P.s.i.}$ $\phi = 0^\circ$					
<p>MOHR DIAGRAM</p> <p>(CH) HIGHLY PLASTIC CLAY</p>					

TESTED: _____
 COMPUTED: D.M.
 PLOTTED: _____
 CHECKED: G.B.
 DATE: 20-2-62
 DATE: 1-3-62

MCROSTIE & ASSOCIATES
 CONSULTING ENGINEERS
 OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 28-2-62	
HOLE NO. 14		LOCATION:		DEPTH: 27'0"-28'6"	
TEST NO.	14-9A	14-9B	14-9C		
DEPTH	28'0" 28'6"	27'6" 28'0"	27'0" 27'6"		
LATERAL PRESSURE (P.S.I.)	10	40	25		
COMPRESSIVE STRESS (P.S.I.) - qc	17.8	16.6	11.4		
WATER CONTENT - W %	59.5	61.7	62.0		
WET DENSITY - γ_m					
$C = 8.5 \text{ P.S.I.}$ $\phi = 0^\circ$					
<p>(CL) MEDIUM PLASTIC CLAY</p>					

TESTED: D.M. DATE: 28-2-62

COMPUTED: D.M. DATE: 28-2-62

PLOTTED: D.M. DATE: 1-3-62

CHECKED: G.B. DATE: 1-3-62

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

PROJECT: E-440		SITE: BANK & QUEENSWAY		DATE: 2-3-62	
HOLE NO. 14		LOCATION:		DEPTH: 32'0" - 33'6"	
TEST NO.	14-10A	14-10B	14-10C		
DEPTH	33'0" 33'6"	32'6" 33'0"	32'0" 32'6"		
LATERAL PRESSURE (PSI)	10	25	40		
COMPRESSIVE STRESS (P.S.I.) - qc	24.5	13.1	14.1		
WATER CONTENT - W%	57.3	38.0	39.4		
WET DENSITY - γ_m					
$C = 6.8 \text{ P.S.I.}$ $\phi = 0^\circ$					
<p style="text-align: center;">PRINCIPAL STRESS P.S.I.</p> <p style="transform: rotate(-90deg); position: absolute; left: 100px; top: 450px;">SHEAR STRESS - P.S.I.</p> <p style="position: absolute; left: 100px; top: 550px;">MOHR DIAGRAM</p>					

TESTED: D.M. DATE: 5-3-62

COMPUTED: D.M. DATE: 5-3-62

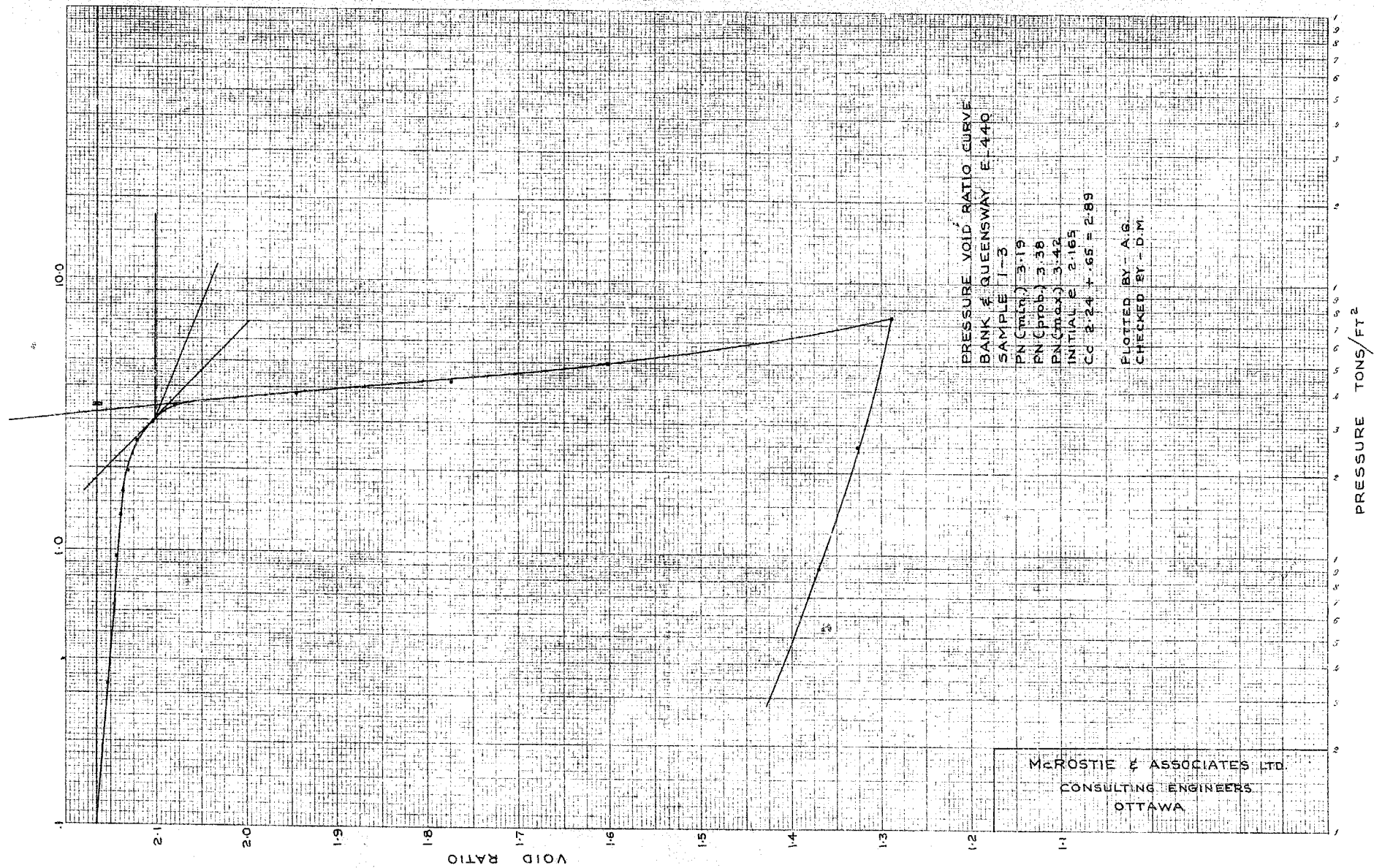
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CHECKED: G.B. DATE: 5-3-62

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

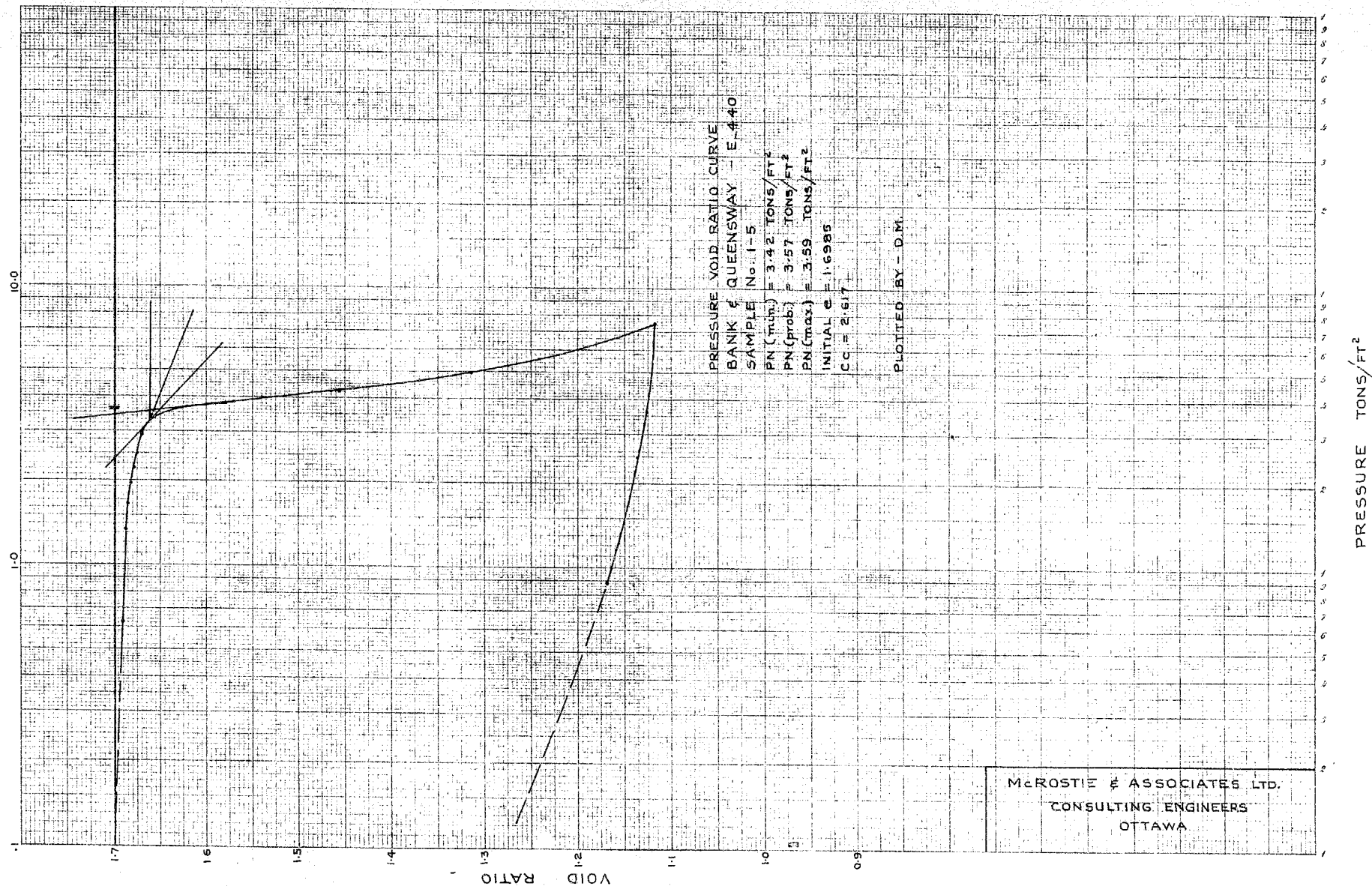
OTTAWA CANADA

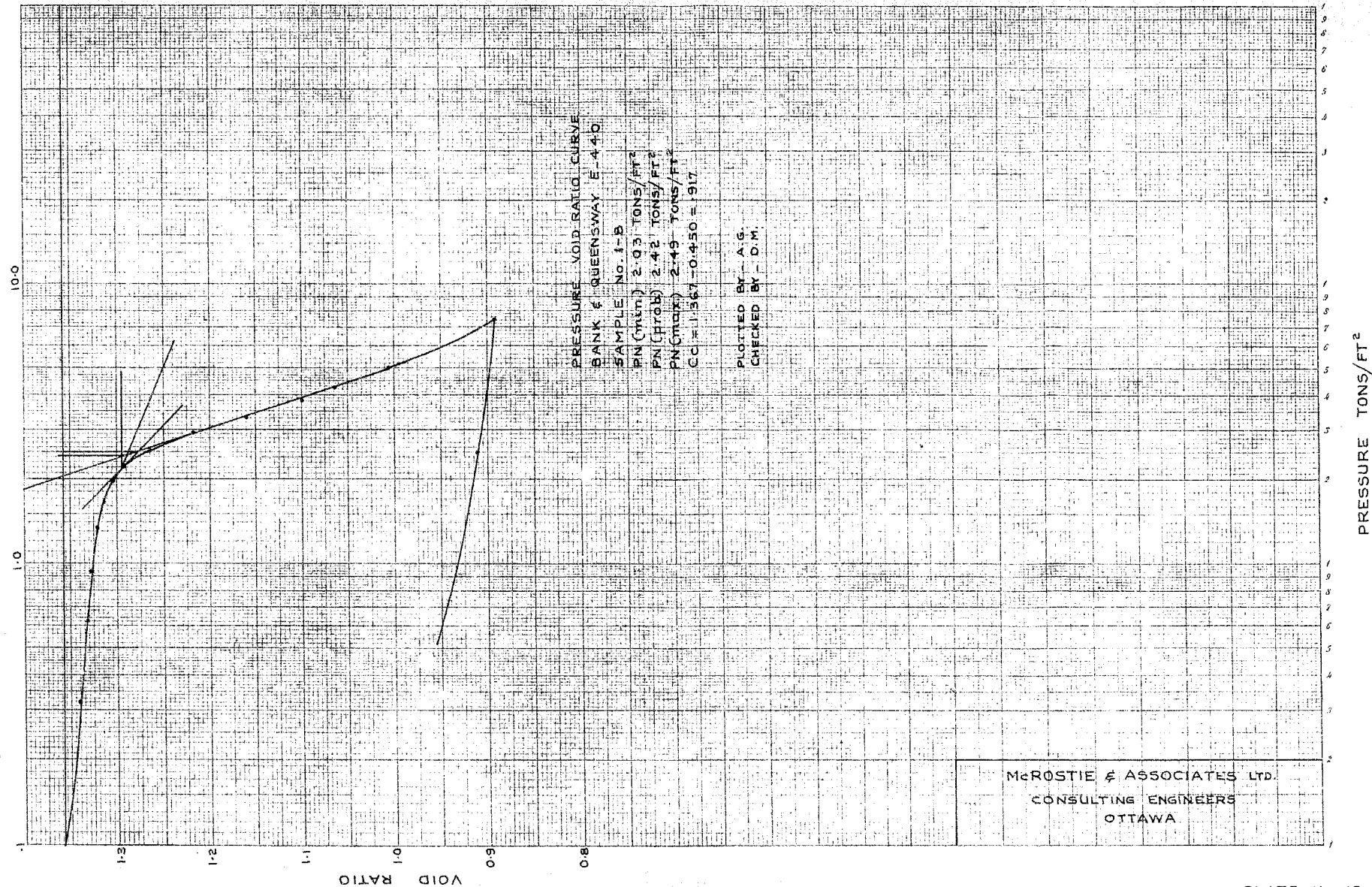


PRESSURE VOID RATIO CURVE
BANK # QUEENSWAY E-440
SAMPLE 1-3
PN (min) 3.19
PN (prob) 3.38
PN (max) 3.42
INITIAL E 2.165
Cc 2.24 + .65 = 2.89

PLOTTED BY - A.G.
CHECKED BY - D.M.

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA

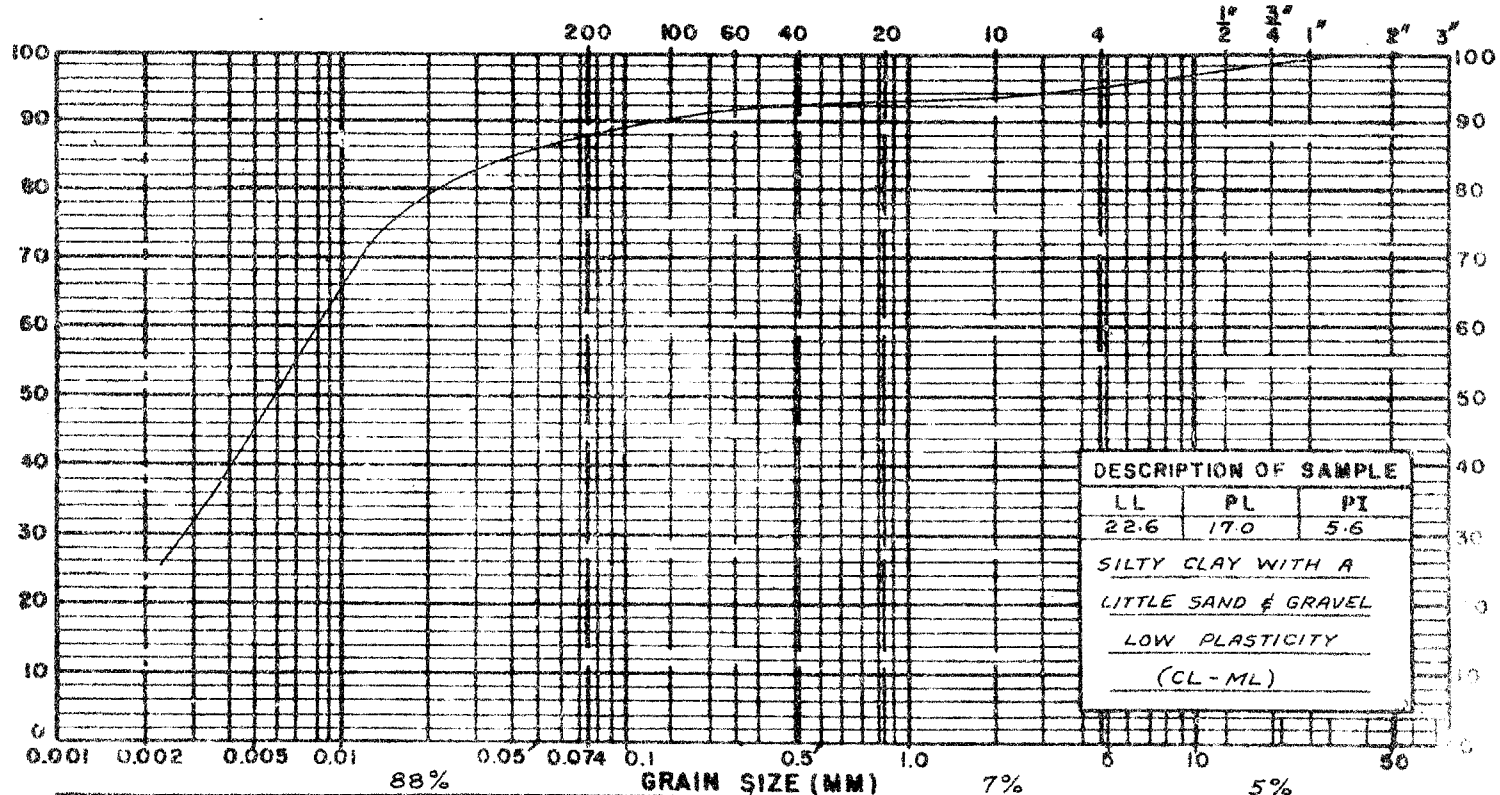




McROSTIE & ASSOCIATES LTD.
 CONSULTING ENGINEERS
 OTTAWA

UNIFIED SOIL CLASSIFICATION
MECHANICAL ANALYSIS OF SOILS
U.S. STANDARD SIEVE SIZE

PERCENT FINER BY WEIGHT



DESCRIPTION OF SAMPLE

LL	PL	PI
22.6	17.0	5.6
SILTY CLAY WITH A LITTLE SAND & GRAVEL LOW PLASTICITY (CL-ML)		

CLAY OR SILT

SAND

GRAVEL

FINE

MEDIUM

COARSE

FINE

COARSE

CRITERIA

SOIL TYPE	Cu	Cc
GW	>4	1-3
SW	>6	1-3

PROJECT BANK & QUEENSWAY E-440

SAMPLE No. 1-10

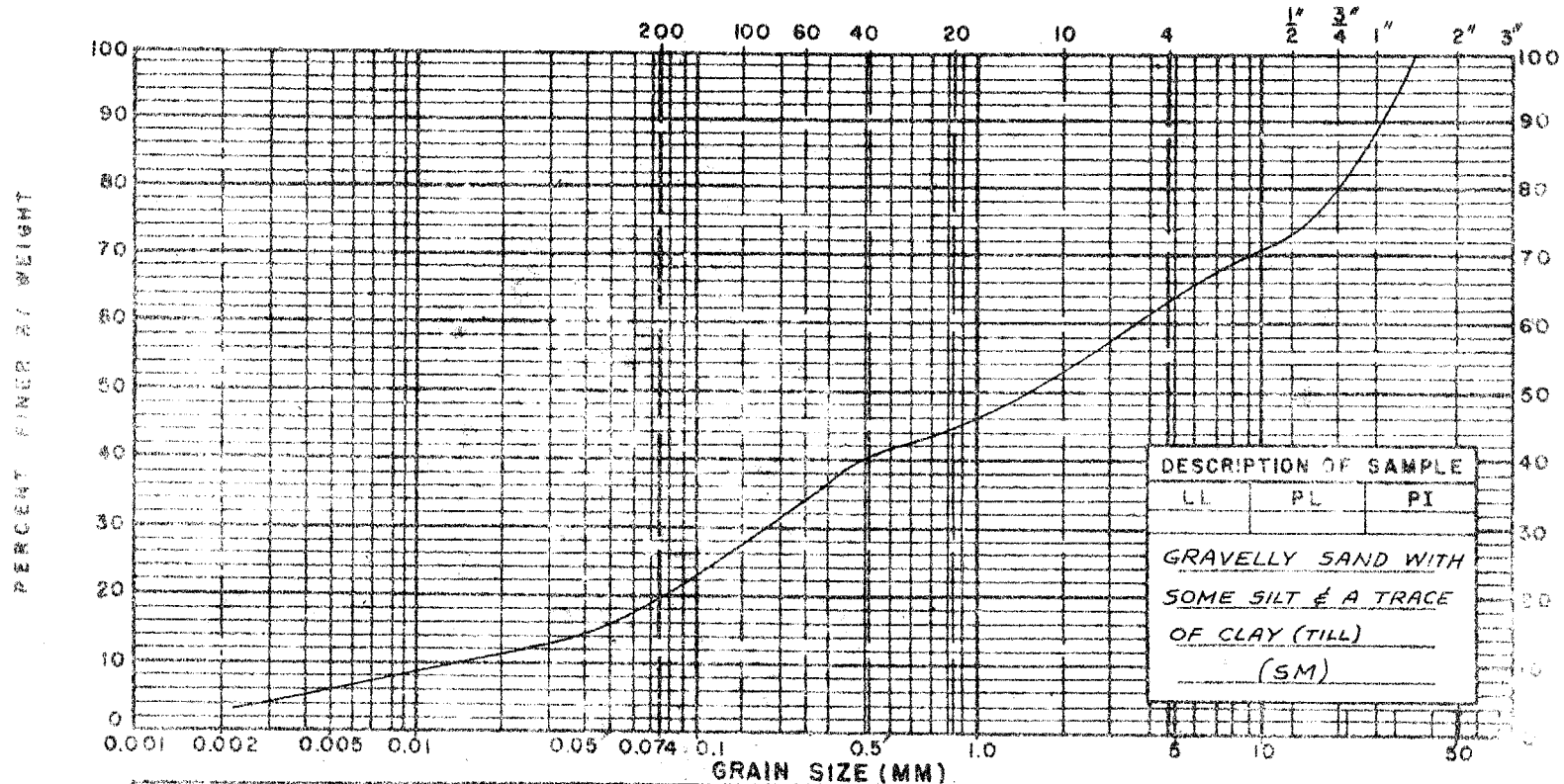
PLOTTED A.G. DATE APRIL 21, 1960

REMARKS DEPTH 50' TO 51'6"

CHECKED G.B. DATE APRIL 21, 1960

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA, CANADA

UNIFIED SOIL CLASSIFICATION
MECHANICAL ANALYSIS OF SOILS
U.S. STANDARD SIEVE SIZE



DESCRIPTION OF SAMPLE

LL	PL	PI
GRAVELLY SAND WITH		
SOME SILT & A TRACE		
OF CLAY (TILL)		
(SM)		

CLAY OR SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE
20%			44%		36%

CRITERIA		
SOIL TYPE	Cu	Cc
GW	> 4	1-3
SW	> 6	1-3

PROJECT BANK & QUEENSWAY E-440

SAMPLE No. 1-11

PLOTTED A.G. DATE APRIL 21, 1960

REMARKS DEPTH: 57'6" TO 58'8"

CHECKED G.B. DATE APRIL 21, 1960

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA, CANADA

#61-F-234C

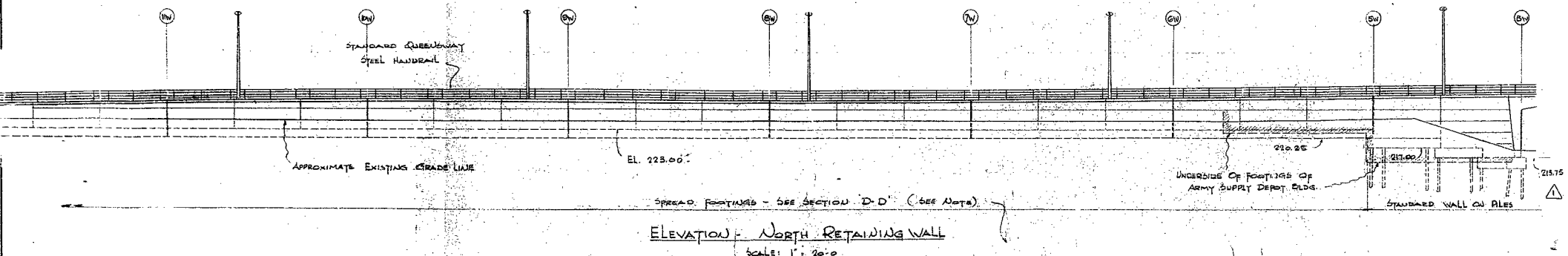
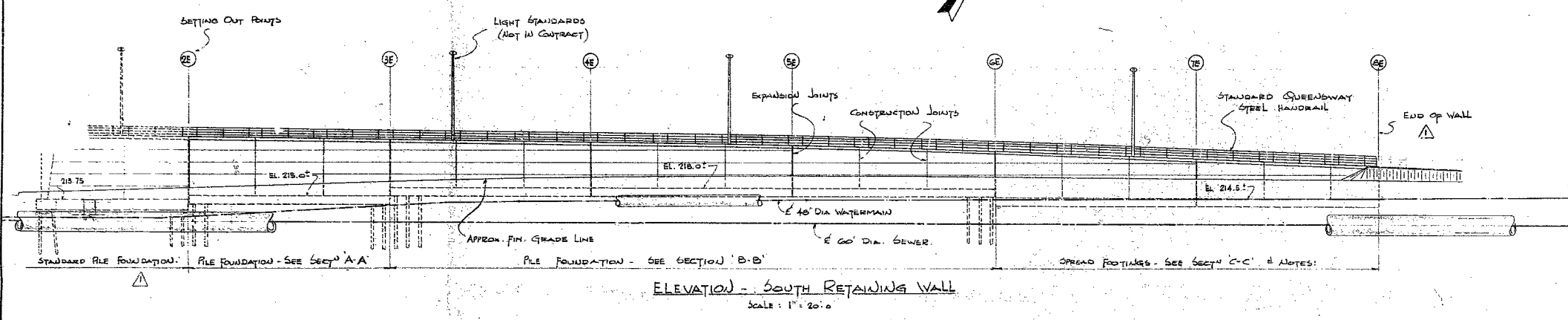
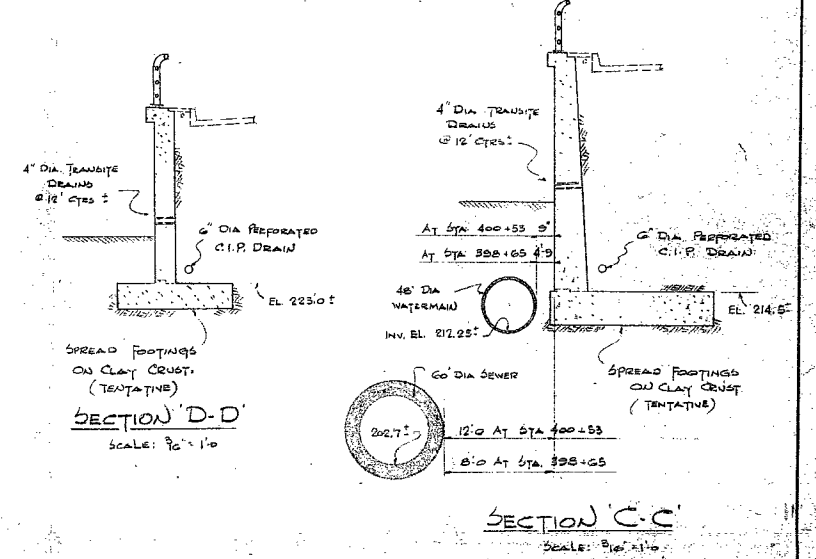
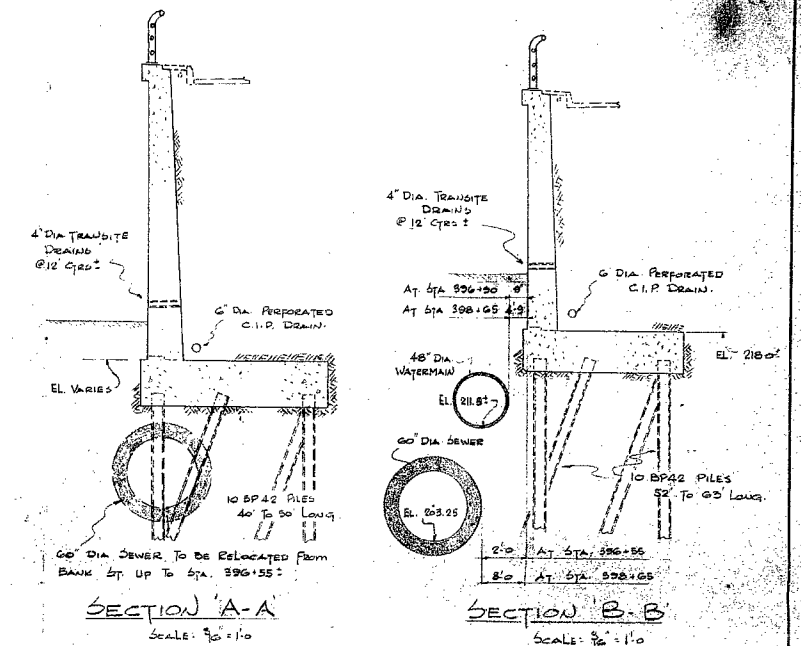
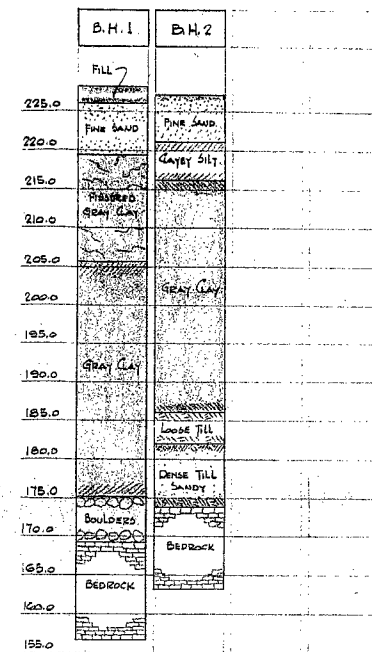
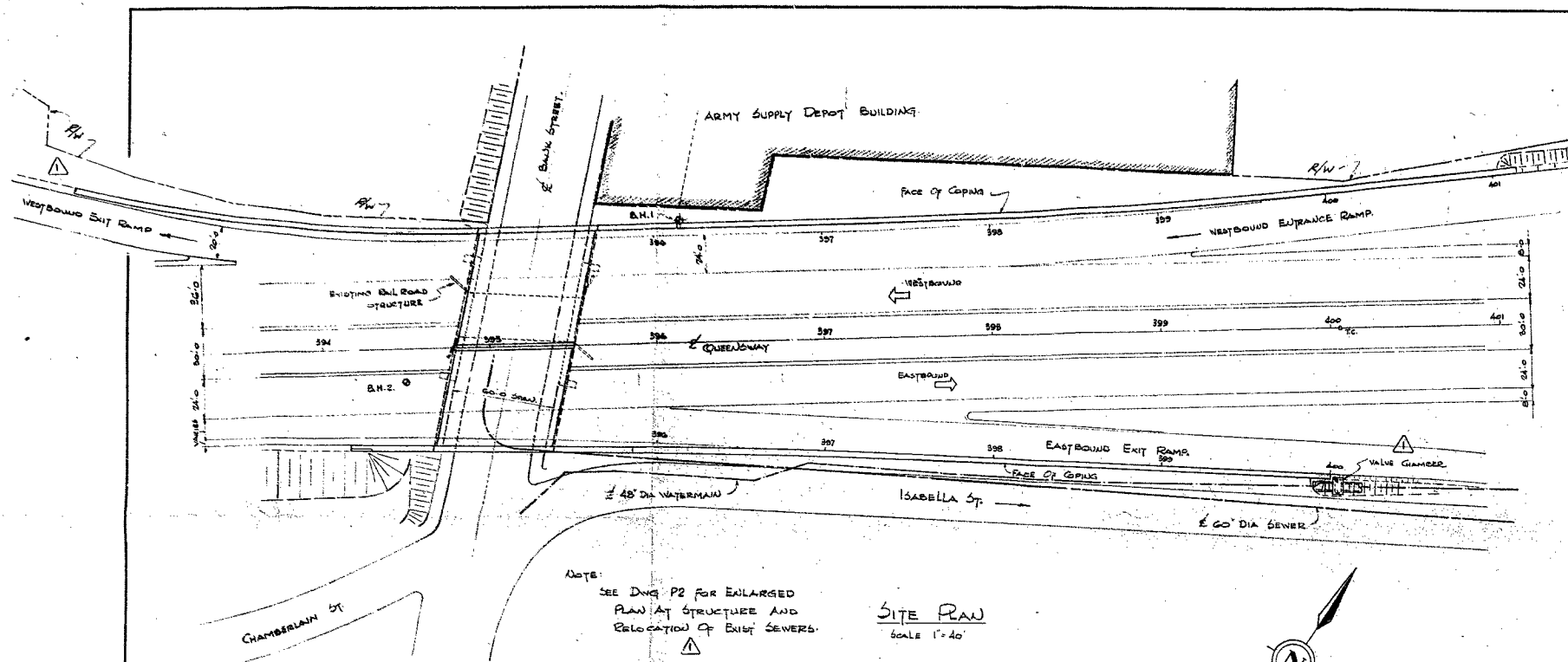
W.P. #947-59 &

#948-59

QUEENSWAY

OTTAWA

BRIDGE #21 &
#22



NOTES:

DESIGN SPECIFICATIONS: A.A.S.H.O. SPECIFICATIONS FOR HIGHWAY BRIDGES.

LINE LOAD: H20-SIG.

SUBSTRUCTURE: R.C. RIGID FRAME.

SPREAD FOOTINGS: SPREAD FOOTINGS OR FOOTINGS ON PILES AS NOTED ON DWGS.

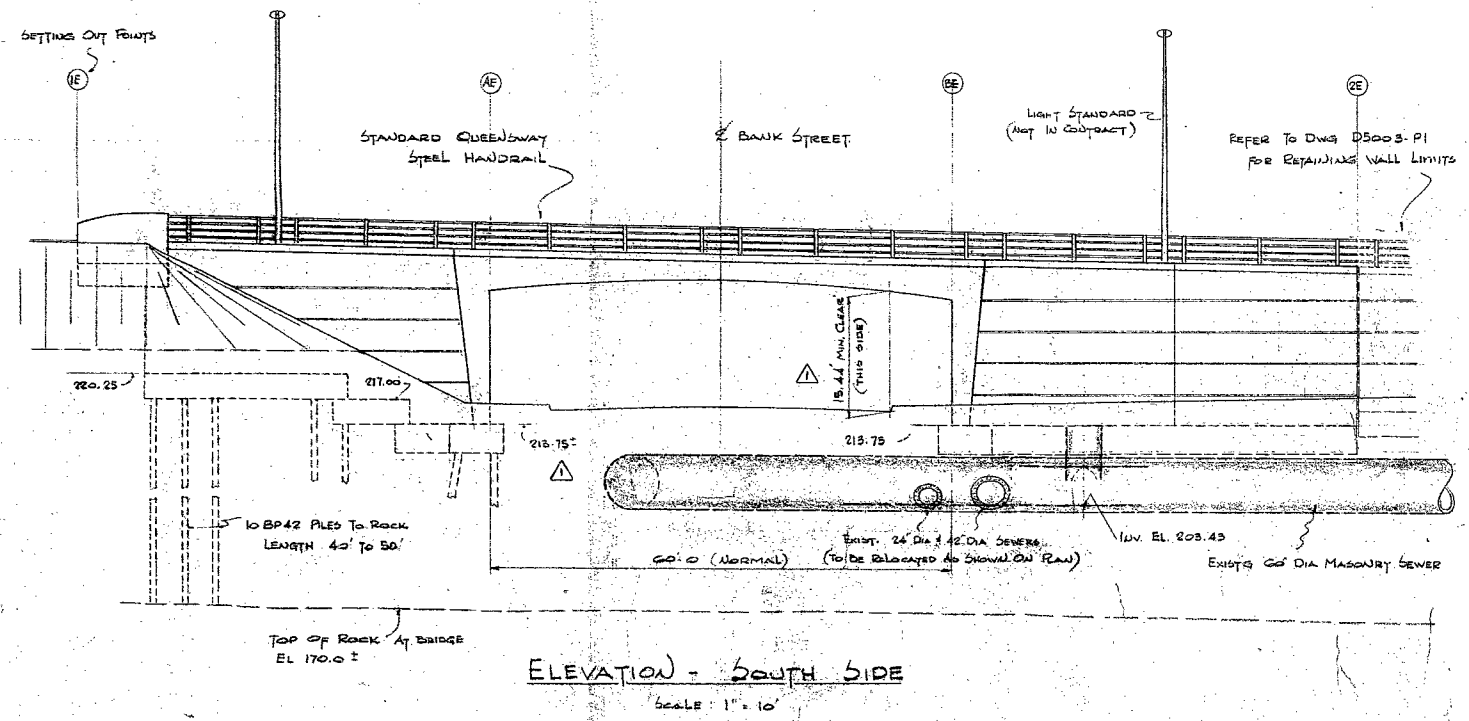
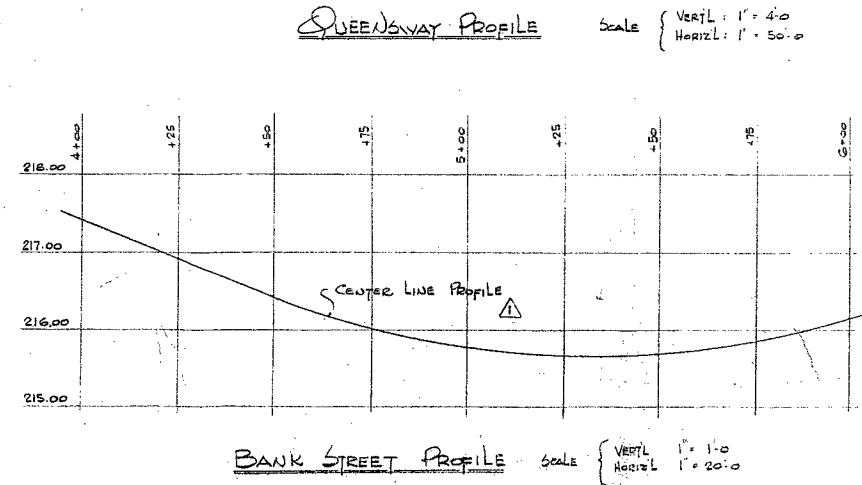
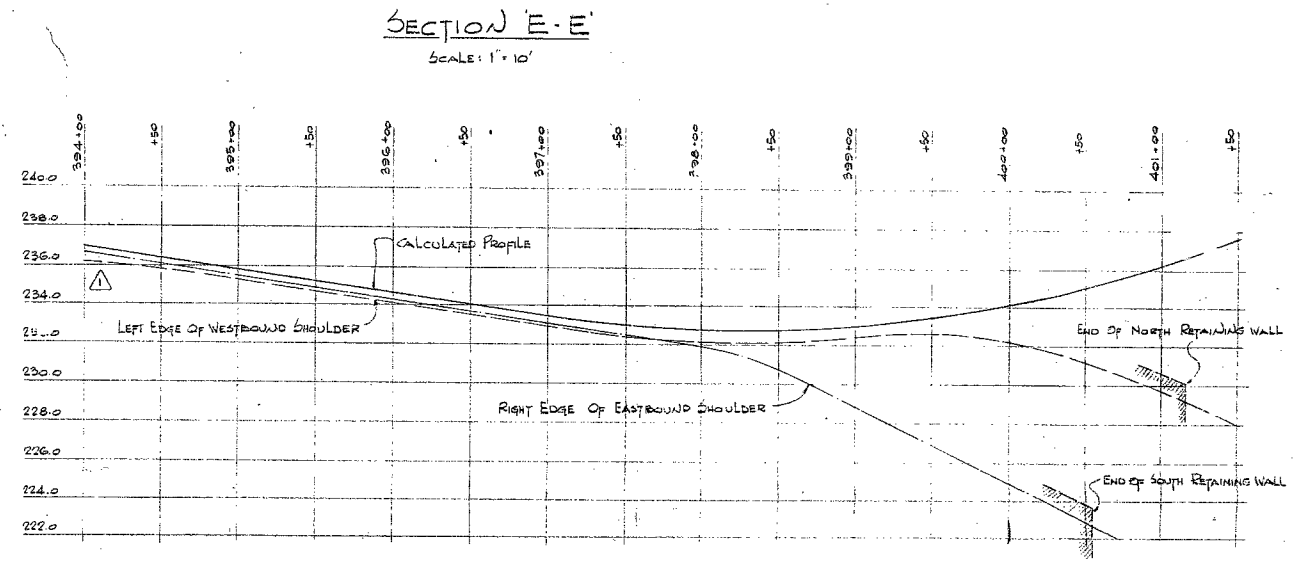
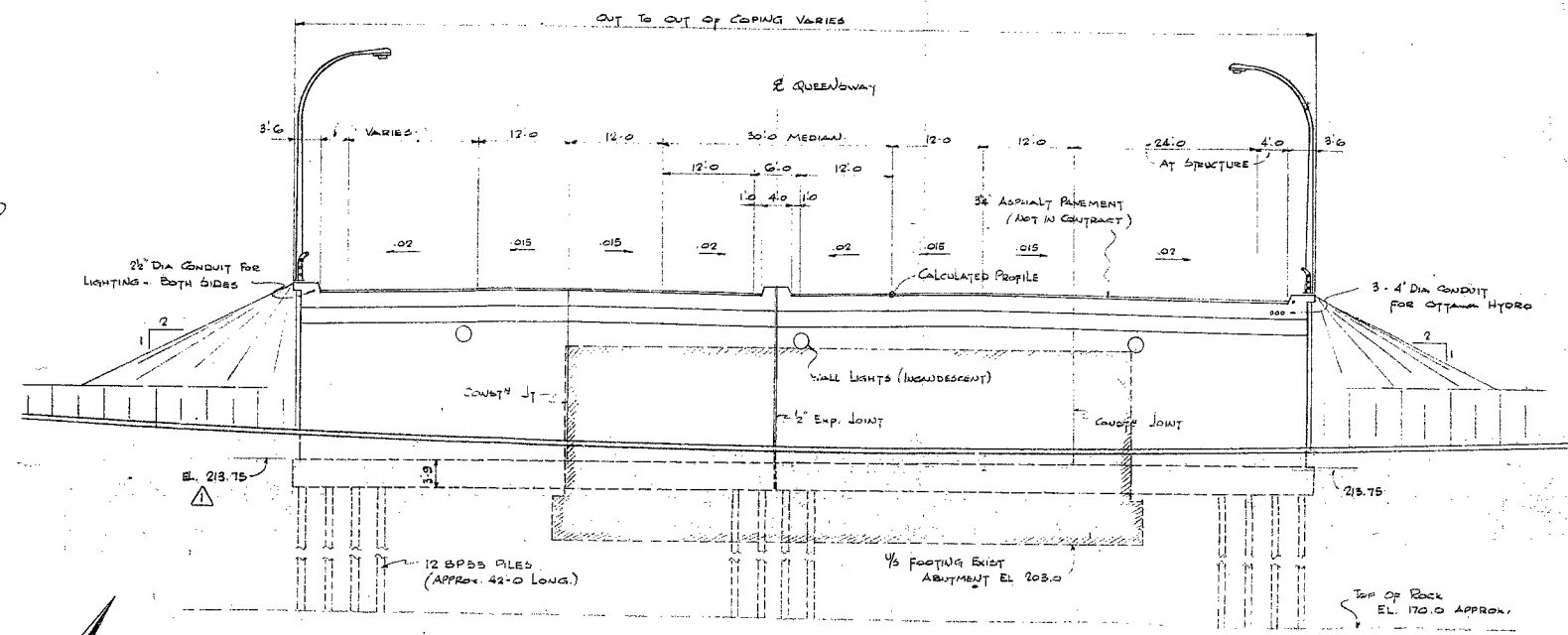
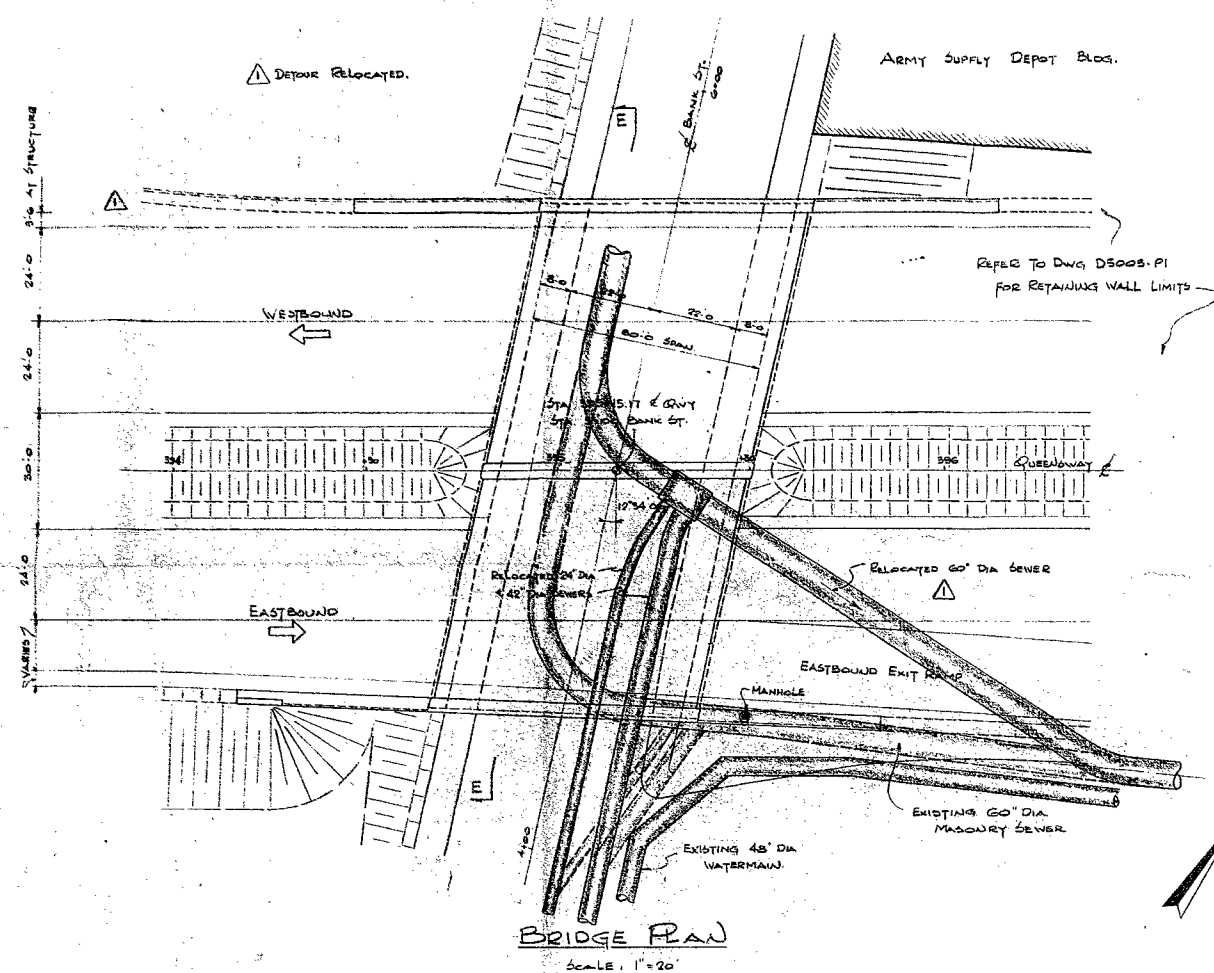
CONCRETE STRENGTH: 3000 P.S.I. THROUGHOUT.


FOUNDATIONS SHOWN ARE BASED ON PRELIMINARY SOILS REPORT BA112 AND DRAFT OF FINAL REPORT SF-582A.

THRUST BLOCKS AT END POINTS IN 48" DIA WATERMAIN TO REMAIN IN PLACE DURING CONSTRUCTION.

GENERAL REVISIONS		By	Date
No.	Revision		
DEPARTMENT OF HIGHWAYS OF ONTARIO			
OTTAWA QUEENSWAY LIMITED-ACCESS HIGHWAY			
OTTAWA CANADA			
BRIDGE #21 AT BANK ST.			
PRELIMINARY SITE PLAN			
DE LEUW CATHY & CO. OF CANADA LIMITED Consulting Engineers		DEPT. OF HIGHWAYS OF ONTARIO FEB 20 1962 Director of Planning & Design	
Designed by B.S.E.	Date JAN. 1962	DWG. No. D5003-P1	
Drawn by P.T.	Scale AS SHOWN	Sheet 1 of 2	
Checked by G.G.S.			

DISTRICT #9
W.P. #947-59



	GENERAL REVISIONS	P.T.	8/2	1962
No	Revisions	By	Date	
DEPARTMENT OF HIGHWAYS OF ONTARIO				
OTTAWA QUEENSWAY LIMITED-ACCESS HIGHWAY OTTAWA CANADA				
BRIDGE NO 21 AT BANK ST. PRELIMINARY PLAN.				
DE LEUW CATHIER & CO. OF CANADA, LIMITED Consulting Engineers		DEPT. OF HIGHWAYS OF ONTARIO FEB 20 1962 Director of Planning & Design		
Designed by: G. S.S.	Date: JAN. 1962	DWG. No. D-5003-P2		
Drawn by: P.T.	Scale: AS SHOWN	Sheet 2 of 2		
Checked by: G.S.S.				

PRINT RECORD		
NO.	FOR	DATE