BA1134

McROSTIE & ASSOCIATES LTD.

CONSULTING ENGINEERS
OTTAWA 1
CANADA

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393 BELL STREET
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PRELIMINARY STAGE OF THE REPORT ON THE FOUNDATION INVESTIGATION FOR THE PROPOSED STRUCTURE AT METCALFE STREET & THE QUEENSWAY

1. TERMS OF REFERENCE

We were requested by the Ottawa Office of De Leuw, Cather & Company of Canada to carry out a preliminary stage investigation of the subsoil at the site of a proposed structure to carry the Queensway over Metcalfe Street. A preliminary stage of the report on foundation conditions at this site, based on a pilot hole study, was to include analysis of foundation types and recommendations on the type of foundation most suitable.

2. RECOMMENDATIONS

2.1 Structure

2.1.1 Foundation Type

A footing type of foundation would appear suitable for the structure at this site if additional boreholes confirm the subsoil revealed by the pilot hole. The structure would need to resist or allow differential movements not exceeding 0.3 feet at right angles to the Queensway centreline. Footings could then lie at approximately elevation 214 on the stiff highly plastic brownish gray clay determined in the test boring. A pile foundation could be designed to lessen the structure settlements as such; however, settlements caused by neighbouring embankment loads would not be relieved by a pile foundation under the structure and consequently larger differential settlements would be created between the embankments and the structure.

Closed abutments appear to be preferable for the structure on this site since preliminary calculations of predicted settlements indicate that a closed end type of structure would result in smaller differential settlements than an open end structure.

2.1.2 Soil Strength

From the pilot hole study a bearing capacity of 4000 pounds per square foot may be assumed, for preliminary design purposes, for the clay soils at approximately elevation 214. However, it should be emphasized that the bearing capacity stated above can be expected to vary due to the extensive length of the footings and considering the limitations of one borehole. Additional borings will necessarily be required to ascertain the properties of the subsoil before such assumed bearing capacity can be generalized.

2.1.3 Soil Compressibilities

A settlement study of the clay strata between elevation 213.7 and elevation 183.7, determined in the pilot hole revealed that smaller differential settlements could be expected for a closed end type of structure and that larger differential settlements are predicted for an open end structure. Plate No. 3 indicates the actual values of settlement calculated at various points of the structure. All these settlement values are known to be greater than those really expected but they conform with present day standard calculation methods.

Because the settlements calculated by standard methods are known to be excessive, we have attempted another method of calculation used by some authors substituting for the actual initial void ratio the void ratio at the present overburden. The settlement values obtained are considerably smaller, as can be seen on Plate No. 3. We therefore recommend that these lower values be used as a guide in preliminary design.

We are presently studying the compressibility characteristics of the silt and clay stratum between elevation 183.7 and elevation 163.7 and additional settlements can possibly be expected from this layer. This would increase somewhat the reported values of settlement on Plate No. 3.

An analysis determining the inter effects of easterly and westerly abutments and embankments on each other down to a depth of 40 feet below ground surface, was made for a closed end type of structure. The results indicated that no appreciable stress concentration occurred within this depth and consequently settlement values shown on Plate No. 3 would not be significantly altered by this interaction.

2.1.4 Stability of Foundation Soils Adjacent to Abutments.

The stability of the foundation soils adjacent to the abutments in a closed end type of structure was studied following the critical slip circle method. The results of this preliminary study indicate that the abutments and backfill approaches should be stable. Therefore, no slip is anticipated in the foundation soils, due to the abutment and backfill loads, if additional boreholes confirm the nature of the subsoil revealed in the pilot hole to extend throughout the structure area.

The effects of creep movements in the foundation soils were considered but were not felt to have a significant effect on footing type foundations at this site.

2.2 Embankment

2.2.1 Foundation Strength

The pilot hole revealed a five foot layer of fill at the site. A stability analysis indicated that a critical circle exists through the existing fill. Factors of safety obtained, smaller than the accepted 1.5, show that a possible failure can occur if the embankment is laid on the fill.

Various assumptions were made as to the properties of the embankment in an effort to increase the stability of the foundation. However, adequate factors of safety could not be obtained regardless of the anticipated specifications for the embankment with side slopes of 1 vertical to 2 horizontal placed on the existing ground. Consequently, removal of the existing fill appears to be required if slope flattening is not found to be feasible or economical. Additional boreholes would first be necessary to determine the extent of the existing fill layer.

If the 5 foot layer of fill is removed then the embankment could be laid on the very stiff clayey silt crust as revealed by the pilot hole. With controlled compaction the embankment should be built with characteristics of density equal to 120 p.c.f. and angle of internal friction equal to 32 degrees. These properties would assure a factor of safety of 1.5 for the critical circle passing through the embankment and foundations if the nature of the subsoil is confirmed to be similar to that revealed in the pilot hole.

The stability analysis was also carried out through the underlying clayey silt and clay strata. The results of this study indicate that adequate factors of safety against foundation failure can be developed by the shear strength of the cohesive soils and embankments, on the basis of the pilot borehole. Plate No. 4 shows the results of these analyses. Confirmation of the shear strength of the underlying cohesive soils should be obtained by further borings before generalizing the adequacy of the subsoil against shear failure.

The stability analyses were checked by a limited amount of investigation using the Bendix G-15D electronic computer. A commercially available programme was used which considers sufficient variables for the study of highway embankments.

The above analyses assumed that all the shear strength of the embankment was developed. This assumption appeared satisfactory for preliminary analysis. Further studies should include the possibility of zero shear strength due to possible cracks through the embankment and factors of safety determined on that basis.

For a complete stability analysis a relaxation mechanism, working in terms of total stresses should be considered since it may cause a loss in shear strength with time. A study along this school of thought was carried out a short time ago on similar local clays and it was deduced that a 25% decrease in shear strength could occur with time. This is recognized to be a maximum reduction since it would be counteracted by the effects of consolidation and drainage during and immediately after construction.

In the stability analysis a further check was made on the factors of safety considering the 25% loss in shear strength. Slips passing through the cohesive strata were studied; in each case, factors of safety determined were in excess of 1.5

(see Plate No. 4). Consequently, the embankment and foundation should remain stable even with a 25% reduction in shear strength of the bearing clay strata with time; this, of course, providing that additional boreholes confirm the nature of the subsoil as revealed in the pilot hole.

2.2.2 Consolidation

The predicted settlement of a few points on the embankment centreline can be seen on Plate No. 3. This settlement analysis was also confined to the clay strata between elevations 213.7 and 183.7 and foundation sizes were chosen for a slightly lower bearing capacity than the 4000 psf now suggested. Therefore an adjustment in the reported values can be expected. For the reasons stated in 2.1.3 above, we recommend, for the present, that the lower values of settlement reported be used as a guide in preliminary design. Confirmation as to the nature of the subsoil below the entire structure area will be required before final estimates of settlement are made.

As can be seen on Plate No. 3, the settlement values predicted for various points of the structure and embankments are large. Field observations of settlements on structures built on similar subsoil (Highway 17 at Greens Creek & Kars Bridge) substantiate the fact that predicted settlements are greater than those actually observed. The reason for this difference is probably chiefly due to the difficulty in determining the maximum load imposed on the cohesive strata in the course of their geological history. The laboratory void ratio vs log of pressure curve for an "undisturbed" soil sample is known to have lower coordinates than the actual field curve. The laboratory curve yields a preconsolidation load which corresponds to an appreciable amount of settlement. Consequently, even if the structure and embankment loads combined fall near the estimated preconsolidation pressure on the laboratory curve, predicted settlements remain considerable. However, if the field curve of the soil could be determined, the preconsolidation load would be somewhat higher and the change in void ratio would be correspondingly smaller; therefore, lower settlement values would be predicted.

Another reason for disagreement between calculated and measured settlements may be that the increasesin vertical stress are usually calculated on the basis of Boussinesq's theory which assumes a homogeneous soil. In all the local measured settlements a crust of stiffer soil exists as an upper layer and a stress distribution theory which recognizes this, such as Westergard, is known to give significant differences in the calculated vertical stress increases.

Any underground pipes or conduits which pass beneath or adjacent to the embankment should be designed to allow for the appreciable differential and total settlements. Predicted settlements at points A, H and D shown on Plate No. 3 seem to indicate that relatively small settlements can be expected to extend to adjacent structures. However, the amount of settlement extending towards a particular neighbouring structure would have to be studied individually.

An analysis of the timing of consolidation settlement was made for the cohesive strata between elevations 213.7 and 183.7 assuming two-way drainage. The coefficient of consolidation more representative of these layers, from the pilot hole study, was established between 0.42 and 0.57 square inches perminute from sample No. 1-5. On the basis of these coefficients an average predicted time of settlement for 50% consolidation was found to be approximately one-quarter year, and the average predict d time of settlement for 90% consolidation was determined at approximately one year. These are shorter times than those that have been calculated from other similar sites and detailed confirmation of the coefficient of consolidation would be required by additional sampling and testing over the structure area before a final prediction of time of settlement could be made at this site, should it be of significance.

Preliminary discussions indicated that a pre-loading program (construction of the embankment one season before construction of structure) was not feasible for administrative reasons. For closed-end abutment structures, pre-loading presents additional difficulties.

On the basis of the pilot hole study it can be seen that settlement observation devices on foundation piers when built or in the embankments would help to improve the state of knowledge in the profession and our ability to forecast settlements.

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 [illegible] No. 1. Eight Shelby tube soil samples were recovered from cohesive soil strata at 5 foot intervals down to a depth of 30 feet below ground surface and at 10 foot intervals, below 30 feet, down to more granular soil layers. Five two-inch split barrel samples were recovered in granular soil strata in conjunction with standard penetration resistance tests at 10 foot intervals. All the samples were brought to our laboratory for examination and classification. An indication of the relative densities of the granular layers encountered was supplied by the results of the standard penetration tests performed. During the boring operations groundwater level was observed and recorded.

Bedrock encountered at 104.3 feet below ground surface was diamond drilled. The cores were recovered for inspection and logging. Core recovery percentages were determined in an effort to evaluate the structural properties of the rock. The presence of seams in the bedrock formation was detected by careful watch for drops of drill rods and loss of drill water during the rock drilling operations.

3.2 Laboratory Testing

Classification tests were made on most samples, two consolidation tests were made at depths representing the mid height of increments of the compressible layer, and preliminary strength determinations were made, a group of unconfined compression tests were made to provide correlation with those tests in other areas, but it is recognized that in the fissured zones the results are lower shear strength values than may be confirmed with other techniques.

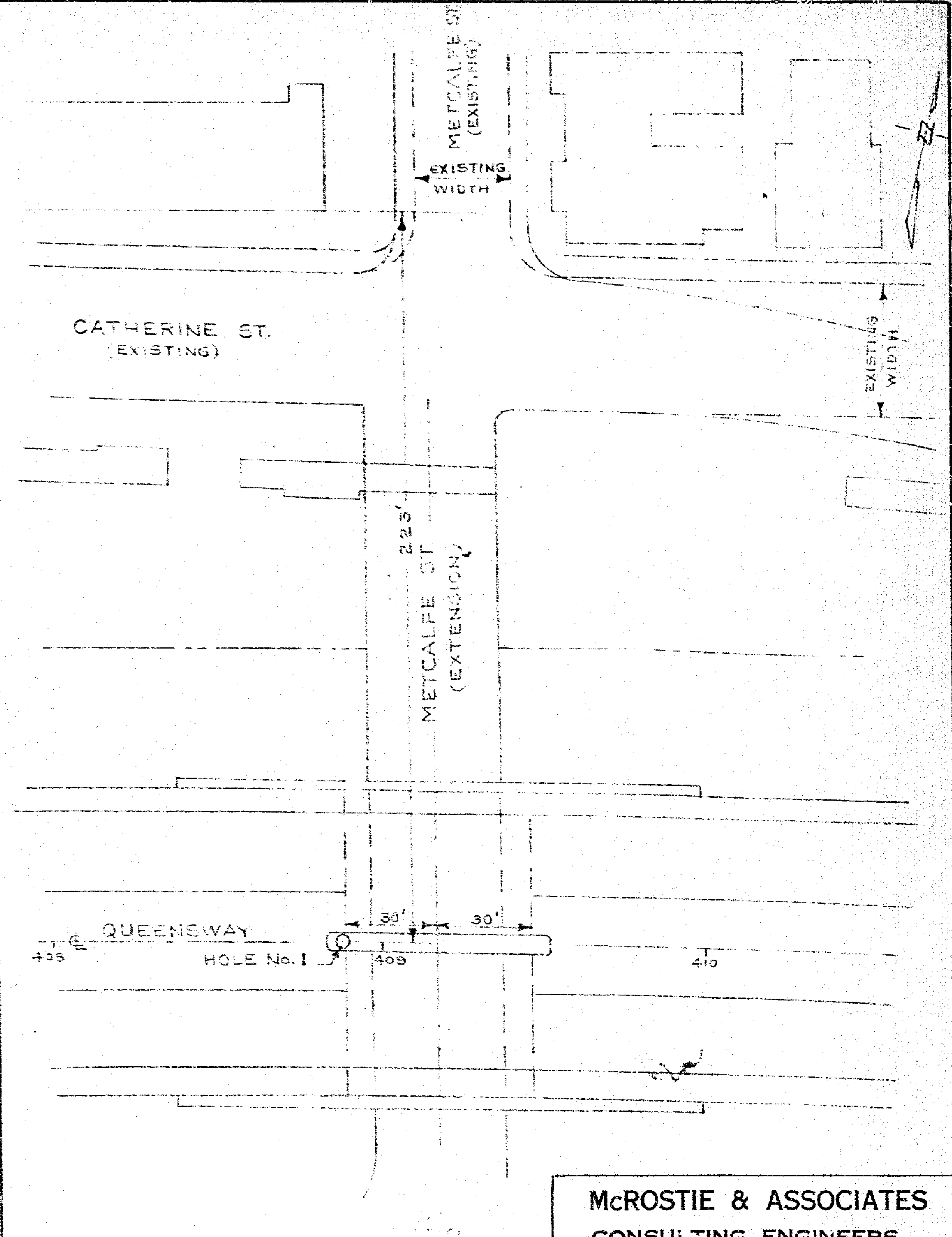
Small scale penetrometer tests (Soiltest Pocket Penetrometer) were made at each six inch interval in the soil before it was extruded from the tube. This simple procedure provides useful information on the variations in strength in a vertical direction. In fissured clays we feel that the penetrometer may provide a more representative answer than the unconfined test since the soil is confined during the penetrometer reading by the lateral pressure of the tube. Penetrometer tests on remoulded samples were attempted at each six inch interval to check that the remoulded strength had not changed. A zero reading on the penetrometer means that the remoulded strength was less than o.l kips per square foot.

3.3 <u>Observations</u>

The geotechnical profile of the subsoil and bedrock formations as revealed by the pilot hole is shown on the accompanying Plate No. 2. It can be generalised as consisting of approximately 5 feet of fill, mostly fine sand, underlain by 63 feet of gray silt and clay mixture and clay, decreasing in consistency from very stiff to medium soft with depth. Underlying these strata are approximately 31 feet of sand and silt mixtures which overlie about 5 feet of loose to medium dense glacial till underlain by bedrock. This rock was encountered at elevation 119.4 feet and consists of black shale with the upper layers possibly fractured. High percentages of core recovery from the lower strata would indicate bedrock to be structurally sound.

An overnight groundwater level reading in the pilot hole showed that the groundwater was about 21 feet below ground surface. This level can be expected to rise appreciably during wetter seasons. Additional boreholes revealing further the extent of the fill and silt strata, should assist in establishing groundwater levels for final design.

A comprehensive preliminary study has been made on the strength of one borehole. Consequently before applying these recommendations to the final design of the structure and embankments at this site, confirmation of the nature of the subsoil under the entire construction area should be obtained by additional investigation.



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BOREHOLE LOCATIONS
METCALFE ST. & QUEENSWAY

SCALE 1" = 40'	**PLATE** 1

FORM S-10-4

McROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

METCALFE & QUEENSWAY

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 223.7' DATE DEC. 1 TO 7, 1955

REMARKS B.M. (EL 219.5') CITY B.M. AT CORNER OF PRETORIA & DRIVEWAY

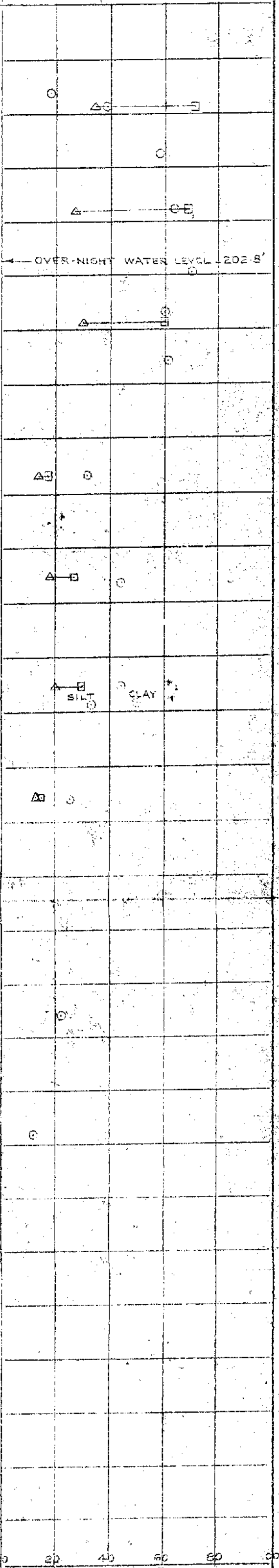
HOLE NO. 1

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL (GROUND SURFACE)	DEPTH IN FEET	ELEVATION
					0'	223.7'
				FILL	5'	
			1-1A	~~FINE SAND, VERY LOOSE~~	5.5'	218.2'
	7.2, 7.6, 7.8	1 for 6" 7	1-1B	CLAYEY SILT BROWNISH GRAY FISSURED, VERY STIFF HIGH PLASTICITY (MH)	10'	213.7'
2.4	4.0, 4.0, 4.0 3.4, 3.2, 3.6 2.4, 3.0, 3.4 R-0.2		1-2	CLAY, BROWNISH GRAY, FISSURED STIFF, HIGH PLASTICITY (CH)	15'	
2.3	1.8, 1.6, 2.0 1.6, 1.6, 1.4 2.8, 3.0, 3.0 2.2, 2.2, 2.2 2.2, 2.0, 2.4 R-0.0		1-3			
2.0	2.4, 2.4, 2.2 3.5, 3.3, 3.6 2.0, 2.0, 2.0 3.4, 3.0, 3.2 2.8, 3.2, 2.8 R-0.0		1-4			
2.9	2.2, 2.5, 2.6 2.6, 2.4, 2.6 2.6, 2.4, 2.6 2.6, 2.6, 2.2 1.4, 1.5, 1.4 R-0.0		1-5	CLAY, GRAY STIFF HIGH PLASTICITY (CH)		
2.5	2.6, 2.8, 2.6 2.6, 2.8, 3.0 3.0, 2.8, 2.8 3.4, 3.0, 3.0 3.2, 3.0, 3.5 R-0.0		1-6			
1.2	1.6, 1.6, 1.8 1.0, 1.0, 1.2 1.2, 1.2, 1.2 1.6, 1.6, 1.6 1.5, 1.5, 1.6 R-0.0		1-7	SILT WITH SOME CLAY & A LITTLE FINE SAND, GRAY, MEDIUM SOFT LOW PLASTICITY (ML)	40'	183.7'
2.1	2.0, 1.8, 1.8 2.2, 2.0, 1.8 1.8, 1.8, 1.8 2.0, 2.0, 1.8 R-0.0		1-8	CLAY, GRAY, MEDIUM SOFT LOW PLASTICITY (CL)	50'	
2.1 0.6	2.0, 2.0, 1.8 1.9, 1.8, 2.0 1.8, 1.8, 2.0 1.2, 1.4, 1.5 R-0.0		1-9		61.3'	162.4'
				SILT & CLAY IN ½" LAYERS GRAY, MEDIUM SOFT (ML & CL IN LAYERS)	68'	
		0	1-10	SANDY SILT WITH SOME CLAY LOW PLASTICITY VERY LOOSE (ML)	80'	143.7'
		21	1-11	SILT WITH SOME FINE SAND & A TRACE OF COARSE SAND MEDIUM DENSE		
		18	1-12		99'	124.7'
		9 7 for 6"	1-13	TILL, LOOSE	101'	
				TILL, MEDIUM DENSE	104.3'	119.4'
				SHALE CORE RECOVERY INDEFINITE BUT GREATER THAN 59%	110'	
				SHALE CORE RECOVERY 90%	113.3'	
				SHALE CORE RECOVERY 93%	118.3'	105.4'
				BOTTOM OF HOLE		

~~PROBING OR VANE TEST~~

......LB. HAMMER ......INCH DROP | NO CASING ......INCH DIA. ROD

~~BLOWS PER FOOT OR~~ ~~SHEAR STRENGTH IN KIPS PER FT.²~~



% WATER CONTENT
NATURAL ○
LIQUID LIMIT □
PLASTIC LIMIT △

PLATE 2

L.-M. CO.—A33078

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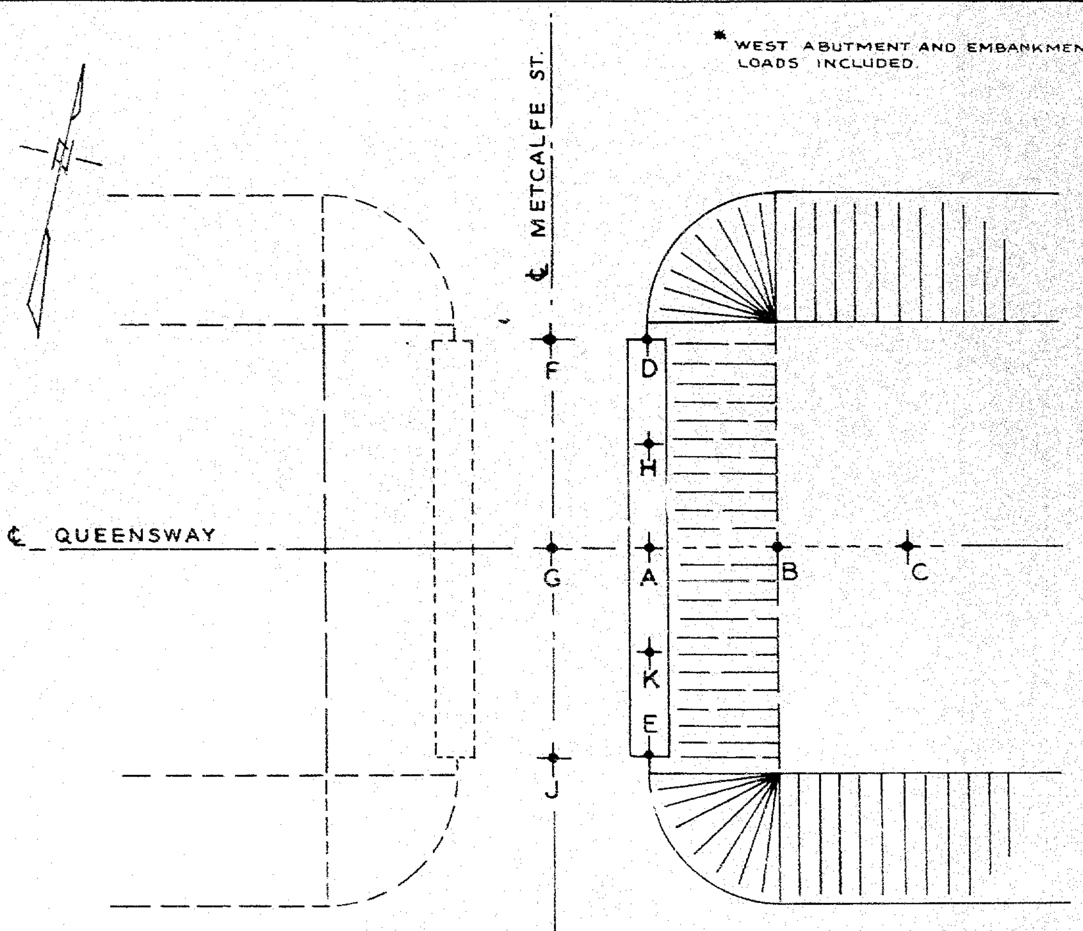
OTTAWA, CANADA

TYPE OF STRUCTURE	FOOTING SIZE	VOID RATIO	SETTLEMENT AT POINT						
			A	B	C	D & E	* F & J	* G	H & K
OPENED END	9.5' x 130'	INITIAL	1.08'	1.29'	1.62'	0.93'			
		PRESENT OVERBURDEN	0.42'	0.66'	0.99'	0.27'			
CLOSED END	12' x 130'	INITIAL	1.41'	1.51'	1.53'	1.08'	0.75'	0.75'	1.41'
		PRESENT OVERBURDEN	0.75'	0.84'	0.87'	0.42'	0.08'	0.11'	0.75'

* WEST ABUTMENT AND EMBANKMENT LOADS INCLUDED.



SETTLEMENTS AT VARIOUS POINTS FOR OPENED AND CLOSED END TYPE OF STRUCTURE.

SCALE: 1" = 50'

PLATE 3

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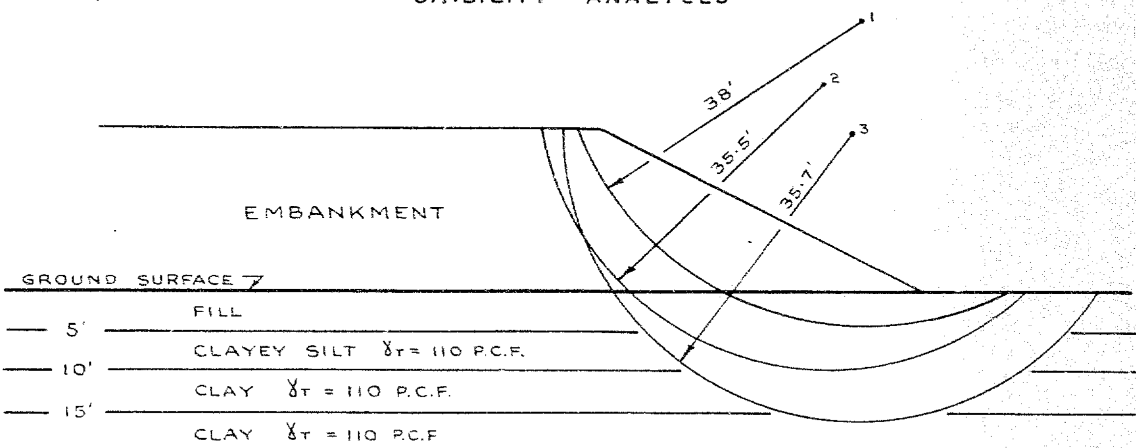
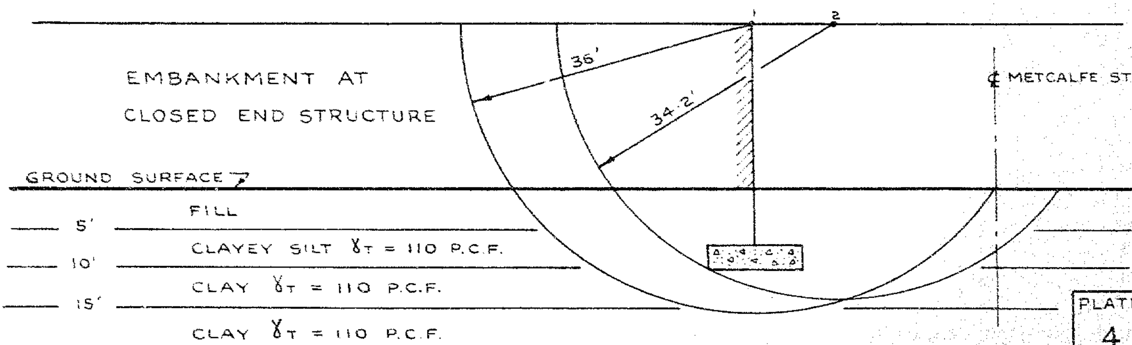
CONSULTING ENGINEERS
OTTAWA, CANADA

STABILITY ANALYSIS OF	SLIP No.	EMBANKMENT		EXISTING FILL		COHESION ALONG ARC K.S.F.	F.S.	F.S. 25% LOSS OF STRENGTH
		$\phi°$	γ_T P.C.F.	$\phi°$	γ_T P.C.F.			
EMBANKMENT	1	28	120	28	100		1·29	
		30	120	28	100		1·32	
		32	120	28	100		1·36	
		30	120	REMOVED AND COMPACTED AS EMBANKMENT			1·44	
		32	120				1·56	
	2	28	120	28	100	3·5	5·3	4·0
		28	120	28	100	1·5	2·5	2·0
	3	28	120	28	100	3·5, 1·5, 1·25	3·8	2·8
STRUCTURE CLOSED END	1	28	120	28	100	3·5, 1·5	3·2	2·5
	2	28	120	28	100	3·5, 1·5	2·8	2·1

STABILITY ANALYSES





SCALES: 1" = 20'

PLATE 4

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. 950-59,
Ottawa Queensway Br. #37,
Metcalfe St. Overpass, Dist. #9.

Herewith one copy of the preliminary foundation report BA 1134, for the above structure.

FJW:go

F. I. Hewson,
Consultant Liaison Engineer.

May 2nd, 1962

OTTAWA QUEENSWAY

BRIDGE NO. 37 AT METCALFE STREET

W.P. NO. 950-59

PRELIMINARY DRAWING NO. D5067-P1

Location

The proposed structure is to carry the Ottawa Queensway over Metcalfe Street between Isabella Street and Catherine Street.

Soils Report and Foundations

A preliminary soils report (BA-11~~5~~34), prepared by ~~H.Q. Golder~~ McRostie and Associates Ltd., indicates bedrock at approximately 104 feet below ground surface. The bedrock is overlain by about 5 feet of loose to dense till, 31 feet of loose sandy silt and clay, 26 feet of silty clay, 25 feet of stiff gray clay, 10 feet of stiff, brownish gray, fissured clay, and 5 feet of fill respectively.

The above report recommends that the structure be founded on spread footings in the stiff brownish gray clay. A differential settlement of 0.3 feet is predicted at right angles to the Queensway centreline. Also, a closed structure is recommended since smaller differential settlements result.

It is expected that the above will be confirmed in the final soils report now in progress.

Structure Type

A reinforced concrete solid deck rigid frame is proposed for the overpass structure. This type of structure has been selected for the site because of its economy, appearance and general usefulness.

The rigid frame structure will be designed transversely to resist the predicted settlement, thus a longitudinal expansion joint has been omitted. If the predicted differential settlements were permitted to occur between the outer edges and centreline of the structure, the deck superelevation and crossfalls would be seriously affected.

Estimated Cost

Estimated Construction Cost = $115,000.

GSS:nm

ONTARIO

DEPARTMENT OF HIGHWAYS

Memo to Mr. A. G. Stermac
Principal Foundation Eng.

Date May 7, 1962

Subject W.P. 953-59 Main St. Br. No. 25
W.P. 950-59 Metcalfe St. Br. 37
W.P. 951-59 Elgin St.Br. No. 23
Ottawa Queensway

From F. I. Hewson

Herewith are prints of the preliminary sketches for these overpasses. They will be discussed at the May 10, 1962 meeting of the Queensway Committee in Ottawa. Apparently instrumentation is being considered for all three. I had thought only Elgin and the Rideau required this attention.

You will note that Metcalfe differs from the others in that it has 1/3 greater design values and much greater predicted settlement.

F. I. Hewson

FIH/et

F. I. Hewson,
Consultant Liaison Engineer.

May 2nd, 1962

OTTAWA QUEENSWAY

BRIDGE NO. 25 AT MAIN STREET

W.P. NO. 953-59

PRELIMINARY DRAWING NO. D5070-P1

Location

The proposed structure is to carry the Ottawa Queensway over Main Street approximately 800 feet east of the Rideau Canal.

Soils Report and Foundations

The soils report for this structure is presently being prepared by H.Q. Golder and Associates Ltd.

Preliminary copies of the boreholes indicate bedrock at approximately 95 feet below ground surface. The bedrock is overlain by about 23 feet of very dense sandy silt, 11 feet of dense silt, 56 feet of stiff clay and 5 feet of loose sand, respectively.

H.Q. Golder and Associates have verbally recommended founding the structure on spread footings in the stiff clay at approximately elevation 213. Settlements of 1½ inches at the centreline and one inch at outer edges of the structure have been predicted.

Structure Type

A reinforced concrete solid deck rigid frame is proposed for the overpass structure. This type of structure has been selected for the site because of its economy, appearance and general usefulness.

A longitudinal expansion joint, wide enough to permit the transverse differential settlement to take place, will be provided at the centreline of the structure.

An additional 6 inches in vertical clearance will be provided to allow for settlement of the structure with respect to Main Street.

Estimated Cost

Estimated Construction Cost = $145,000

GSS:nm

INSTRUMENTATION AT OVERPASSES ALONG QUEENSWAY -

at (a) O'Connor St.

(b) Metcalfe

(c) Elgin

(d) Rideau Canal

and (e) Main St.

Date: 20 June 1962.

Place: (a) De Leuw Cather & Co., Ottawa - preliminary talk 8.30 - 9.00 a.m.

(b) National Research Council - 9.30 - 12.00

Present:	J. Saunders L. Marshall	De Leuw Cather & Co.
	Carl Crawford Bill Eden Ken Burn	National Research Council.
	K. Y. Lo	Department of Highways, Ontario.

It was agreed that N.R.C. instrument any sections which they want to for their own interest, and D.H.O. supplement any sections required for construction purposes. It was made clear that results obtained by N.R.C. should be made available to D.H.O. immediately, especially when construction reaches a critical stage. As far as construction is concerned, instrumentation requires settlement plates only for the fill and piezometers only for pile driving at the banks of Rideau Canal. At this section, the piles are to be driven to till first and the fill placed later on according to the rate of dissipation of pore pressure set up by pile driving.

It was agreed that N.R.C. write up the details of instrumentation for review and suggestions.

KYLo

20 June 1962

MEETING WITH L. WALKER, DISTRICT ENGINEER AT OTTAWA

2.30 - 3.00 p.m. - June 20, 1962.

(a) Informed him of the results of discussion with N.R.C. and De Leuw Cather on instrumentation along Queensway overpasses.

(b) Discussion Cumberland slide with him.

Kyfto.

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R. W. MIDDLEMISS, B. ENG., P. ENG., J.E.I.C.

393 BELL STREET
TELEPHONE CE. 2-5334

February 12, 1962.

A. G. Sternac,
Principal Foundation Engineer,
Materials and Research Division,
Ontario Department of Highways,
Queenspark, Toronto.

Dear Tony:

Re: Subsurface investigation reports for O'Connor Street and Metcalfe Street at Queensway, Ottawa, Ont.

Thank you for the reminder last week that we have not yet provided some of the information as we promised during our meeting of December 12th, 1961. The Queensway work is currently proceeding at an accelerated pace as you know, and we have been busy with the current investigations.

I felt however that the basic agreement was reached at the December 12th meeting that the structure at Queensway and O'Connor Street, our report SF-570, should be supported on the clay soils and that tolerable differential settlements would be encountered in either a closed or an open ended structure.

Similarly regarding the structure at Queensway and Metcalfe, our report SF-487, although the report covers the preliminary stage of the investigation and will be followed by a report on the design stage investigation, it appeared to me that we agreed it was reasonable to continue to plan the structure with soil supported foundations. Any assumptions regarding soil strengths or compressibilities would be subject to confirmation by the design stage investigation.

..............2

One point which we felt might be more fully explained was the settlement calculations comparing open and closed end structures. As suggested in your earlier memo, we are enclosing copies of our calculations for your review. I think that we both agree however, that the quantitative amounts of settlement calculated by normal methods are much too great. There are a few examples of actual observations however, which show us that settlements due to loads less than the preconsolidation loads do actually occur. You will recall the results which we showed for the Montreal Road at Highway 17 crossing east of Ottawa where total settlements of the order of 1/10 of a foot have occurred at loads below the preconsolidation pressures. We also discuss Kars bridge fill where observations are showing settlements of 0.5 to 0.7 feet at loads approximating the preconsolidation load. All of the foregoing emphasize of course the need for further instrumentation of actual fills in this area so that a sound basis for judgement may be achieved.

As agreed during the discussions we are including a section of a report from another project in the Ottawa marine leda clay. In this appendix A to the Outfall Sewer Report, the basis for considering a possible 25% reduction in undrained shear strength with time is discussed.

Also as discussed at our meeting, the reference to a pertinent article on the arching of granular embankments is given below:

"Stress Systems Within Simple Slopes of Granular Materials" by Trollope and Morgan, Institution of Engineers of Australia, N 1314 Civil Engineering Transactions, March, 1959.

We trust you will find the information supplied sufficient for present purposes.

Yours very truly,
McRostie & Associates Ltd.,

Per: Gordon McRostie

G. C. McRostie.

GCM/aa
encl.

APPENDIX A

Time-Strength of Clay Testing Program.

Report on

Subsurface investigations (1958 and 1959)

Stage One (Keefer St. to Sewage Treatment Plant Site)

Outfall Sewer Tunnell

Sewage Disposal Project

Corporation of the City of Ottawa

to

De Leuw Cather & Company of Canada Limited.

PART TWO

Design and Construction Recommendations

McROSTIE & ASSOCIATES Ltd.

APPENDIX A: TIME-STRENGTH OF CLAY TESTING PROGRAM

Objective

The testing program was initiated so that more specific information would be available to assist in appraising the effect of time on the strength of the clay around the tunnel opening. This factor was considered critical owing to - the low safety factors which the preliminary analyses had indicated, the high sensitivity of the clay and the grave consequences of failure in the soil.

Previous Work

The previous work done on this subject of time effects has not been great. Its importance, however, has received increasing recognition over the past few years. Research work on clays, as well as other materials such as concrete and rock, have shown that the reduction in strength with time can be substantial.

Casagrande and Wilson (1951) reported that for the clays they had tested the decrease in undrained shear strength could be from 15% to 60% after one day and from 20% to 90% after one month. In addition, their work indicated that materials which were susceptible to loss of strength with time also showed a relatively low ratio of secant modulus of deformation (at 30% of normal compressive strength) to instantaneous modulus. In slow stress rate tests it was found that for all the clays tested there was an increase in strain at failure with a decrease in rate of loading.

Skempton and Bishop (1954) suggested that there is a stress, for any given clay, below which failure will not occur, no matter how long the load is maintained, and that this limiting stress is about 75% of the strength as measured in the standard undrained test. However, this view was only based on work by Taylor (1943) and Casagrande and Wilson (1951). Even with this limited data an exception to the rule existed in the report of Casagrande and Wilson (1951).

Goldstein and Ter-Stepanian (1957) stated that undrained tests of different clays showed that their strength decreased during one to three months by 30% to 70%. However, samples which were allowed to drain did not fail under the same loads even when the duration of the tests lasted much longer. In addition, they reported that samples subjected to different loads and failing at different times did so at the same deformation. Goldstein and Misumsky (1958) commented that in some tests the reaction of soft clay relaxed to zero.

Bjerrum (1958) reported that the undrained shear strength of the clay they tested decreased by about 50% in one day and by about 60% after one month compared to the strength obtained in a test of ten minutes duration. The pore pressure parameter, Af, was found to increase as the time to failure increased. This increase in pore pressure at failure with time was not sufficient, however, to account for the whole decrease in undrained shear strength with time, and therefore a reduction in the true cohesion and or the true angle of internal friction was considered to have occured. Consolidated drained triaxial tests showed that the drained strengths were independent of the different rates of loading. This suggested that the decrease in the fundamental strength parameters with time were being offset by an increase due to the decrease in moisture content with time. For both drained and undrained tests it was found that the actual deformation at failure decreased with an increase in time to failure. The modulus of elasticity for the undrained tests, calculated at 50% of failure strain, appeared to be independent of the rate of loading. Bjerrum (1959) mentioned that in some of their work they had dug a test shaft in clay to a depth which would produce a safety factor of one against bottom heave. The bottom of the shaft did not fail with time indicating that any decrease in the strength of the clay with time was more than compensated for by the increase in the strength of the clay with drainage.

Crawford (1959) tested a block of Leda clay and found that the undrained strength decreased with increasing time to failure. In addition the pore pressure parameter, Af, increased significantly with time to failure. The amount of increase seemed to depend somewhat on the pressures at which the samples were consolidated. This work also indicated that drain tests did not satisfactorily evaluate this sensivive clay. It was felt that the effective stress Mohr failure envelope is best determined for critical field conditions by a one hour test. The strain at failure seemed to be essentially constant for this clay. The angle of shearing resistance in terms of effective stresses increased from 17.5° to 23° as the time to failure increased from 0.1 hour to 10 hours. A corresponding decrease in the cohesion occured. Of even more importance, however, was the lowering of the effective stress Mohr envelope for specimens under a confining pressure below the preconsolidation pressure as the testing time increased.

Housel (1939) pointed out the similarity in the reaction of clays to stress to certain rheological models. He had devised a test which showed the "yield stress" in clay. This test was essentially a double ring shear test with the load applied in constant increments each increment being maintained for a constant period of time. The increments of strain occurring with time were found to be proportional to the level of shear stress until a point was reached when they increased disproportionately. This level of stress was called the "yield stress". This test had shown that the yield stress" was about 20% of the ultimate strength from unconfined compression tests for some Lake Michigan clays.

Henkel and Skempton (1955) analysed a landslide and suggested that the effect of overconsolidation on the shear strength of clays may largely disappear on a geological time scale. In later work Henkel (1956) indicated that the higher strength of overconsolidated clays seems to be due to the resultant negative pore pressures which occur on shearing.

Program

The best test to use for the purpose of this study was considered to be the consolidated, undrained, triaxial compression test with the measurement of pore pressures. Unconfined compression (Qu), quick triaxial (Q) and field vane shear tests were run on samples from adjacent boreholes. The purpose of these tests was to correlate the results with the results from previous routine testing and to have information which would be useful in judging how representative each series of tests might be of field conditions. In addition, for the same purposes the sampling methods were varied using a normal 2" thin-walled sampler for the unconfined and quick triaxial test samples and 2" piston-sampler for the consolidated triaxial test samples and a second series of unconfined compression tests.

amples were taken of the clay profile at one station along the tunnel. It was recognized that the clay was layered with variations in moisture content up to 18% within 4". This would make the correlation of tests on different samples difficult. However, it was considered most important to examine the characteristics of the actual clay involved in the engineering problem.

The consolidation pressure for the triaxial tests was selected to minimize the additional consolidation that would be produced on the samples. For this reason a value on the low side of the overburden pressure was selected to account for a lateral earth pressure coefficient at rest being less than one. At the same time, it was considered more importand to use one constant value for all tests, so that this would not be a variable between tests, rather than using some constant ratio of the overburden pressure. For these reasons, a consolidation pressure of 1 tsf was selected. After the consolidation pressure had been applied to the samples and equilibrium conditions in the pore pressures achieved a back pressure of 1 tsf was applied to the sample with a corresponding increase in confining pressure of 1 tsf. This procedure was to eliminate any air in the system and to obtain a quick response in the pore pressure readings when the shear stress was applied. Three series of consolidated triaxial tests were run.

The first series of tests was run at a controlled strain rate (Qt). These tests were done by the Division of Building Research, N.R.C. The rates of strain were varied from about 20% to 0.2% strain per minute giving times to failure for most of the samples between six minutes and six hours. These tests were considered essential as the most pertinent work of others with respect to our engineering problem had been done using this test method. Consequently, comparison was considered to be best through the medium of the same test. Furthermore, the test is of particular value as a failure stress and time can be obtained for each sample, which is not always the case in creep tests, and the clay reaction or strength after failure can be measured.

The second series of consolidated triaxial tests was performed at constant stress (Qp). These tests were also done by the Division of Building Research, N.R.C. These were essentially creep tests. It was thought that they would be a better analogue of the field conditions. Furthermore, this type of test is more easily analysed in terms of rheological parameters. However, owing to the variations in the clay from sample to sample a complete series of these tests to give a wide variation in failure times would require much duplication. Such a complete series was not considered necessary in view of the supplementary information that would be obtained from the other two series of tests.

The third series of consolidated traixial tests was run using a constant increment of stress applied for a constant increment of time (Qy). The purpose of this series was to determine the "yield stress" after Housel's concept. Whereas the first two series of tests should be made on samples of equal strength so that the variations in material does not provide an important variable between individual tests, this third series provided tests which were complete within themselves. In other words, each test indicated the level of "yield stress" with respect to the ultimate strength. Initially deviator stress increments were set at about 5% of ultimate deviator stress with time increments of 5 minutes (samples at 35', 40', 45' and 50' depths). When it was seen that the "yield stress" could be expected to be quite high these increments were changed to 10% and 10 minutes (samples at 20', 25', 55', 60', 65', 70' and 75'). Analysis of the viscous action indicated that these increments should be adequate to obtain strain increments greater than that corresponding to the relaxation time. As a separate control on ultimate strength a sample from each tube was also tested in the normal manner for a consolidated quick test with a controlled strain rate of 20% per minute.

Results and Discussion

The results of this special investigation are summarized on Plates 1 to 7.

By examining Plate No. 1 we can obtain some information on the probable depth of dried crust. It can be seen that the moisture content is less than the liquid limit down to 15'. The quick triaxial test results (Q) are greater than the unconfined compression tests (Qu) down to 20'. (Unless otherwise stated deviator stresses are being compared.) Similarly the consolidated quick triaxial test results (Qc) are greater than the unconfined compression tests (Qu) down to 20'. The vane tests can be read to mean that the shear strength from 20' to 30' inclusive is about 2400 psf. from 35' to 50' inclusive about 2100 psf and from 55' to 65' inclusive about 1800 psf. These observations indicate that the clay crust is about 20' to 30' thick. This corroborates other work on this area which indicated the effect of drying on preconsolidation load to start somewhere between 20' and 30'.

Also on Plate No. 1 we see that the consolidated quick triaxial results are greater than the quick triaxial (Q) results from 35' to 75'. Also over this range the consolidated quick triaxial tests (Qc) are approximately equal to the vane results. The first comparison might mean that it takes time for the pore pressure to reach equilibrium in the sample. However, as consolidation of the samples produced little measurable change in the moisture content this is not a strong argument. On the other hand, during the consolidation period it is possible that weaker material adjacent to fissures or joints might be reconsolidated to produce a stronger sample. The agreement between the consolidated quick and vane results would support this argument if it is assumed that the vane test is a valid measure of the in situ shear strength. The fact that there is little moisture change during the consolidation period of the consolidated quick tests also indicates that the difference between these tests and the quick tests is not due to the preconsolidation load being lower than the consolidating pressure. Consolidation tests have shown that it is reasonable to expect the preconsolidation load to be well above one ton psf - the confining pressure in the triaxial tests.

By examining the broad trends of shear strength with depth on Plate No. 1 it can be seen that there is some tendency for the strength to decrease with depth below 30'. This is contrary to what is normally assumed by other workers in this field. In this case, of course, the material is also changing with depth - the plastic index is decreasing, the liquidity index increases in an erratic manner and the activity coefficient decreases with depth.

Although the evidence presented here is not exhaustive it does confirm our views that the unconfined compressive test is as good a test as the quick triaxial test for determining the undrained shear strength (in total stresses) of our clays below the dessicated crust. In the crustal soil the triaxial test is preferable. At the same time it is not known how representative these tests are of the actual shear strength even of the clay below the crust. It is possible that the vane or consolidated triaxial tests are more representative. Furthermore, although we have been reluctant to use the high values obtained from vane testing in the local clays the comparison with the consolidated triaxial results and the suggested explanation mentioned above does provide some basis for using those higher values.

On the merits of normal thin-walled sampling versus piston sampling it would seem by comparing the unconfined compression (Qu) results from Hole 126 using normal sampler with those from Hole 145 using a piston sampler that no conspicuous advantage was obtained in using the piston sampler. In other words, our usual sampling method seems quite satisfactory for strength testing.

On Plate No. 2 there is plotted some of the results of the tests run at the constant strain rate (Qt). The curves show that the decrease of strength as the time to failure increases from 0.1 hour to 1.0 hour is between 6% and 13%. As the time to failure increases from 0.1 hour to 10 hours the strength decrease varied from 12% to 27%. In view of the results of the yield stress tests (Qy) an attempt was made to decrease the strain rate still further. However, attempts at testing with a lower strain rate than 0.2% per hour resulted in difficulties in the apparatus which invalidated the results.

The curves on Plate No. 2 also contain the results of the constant stress tests (Qp). As these tests seem to fall close to the curves connecting the points from the constant strain rate tests (Qt) it was possible to adhere to the original plan of only conducting one series of these tests. At the same time some substantiation was provided for assuming that the constant strain rate tests were representative of the constant stress condition (the more normal field condition).

On Plate No. 3 a typical strain time curve for the constant stress (Qp) tests is shown. The time assumed for failure in reporting the N.R.C. results, was that corresponding to the vertical portion of the end of the curve. It is possible that a more representative time would be where the point of inflection occurred in the curve. It is reasonable to assume that at the point of inflection something occurs in the material which leads to the increasing rate of strain with time. Up until this point the strain rate is decreasing. It is possible that the mechanism involved is one of a critical strain beyond which the natural structure on which the strength is dependent breaks down. This would mean, in effect, that less time than indicated would be required to cause a breakdown in the clay. The fact that the central parts of these strain-time curves were often straight for a considerable range indicates that this concept should be examined further.

On Plate No. 4 typical results of the yield tests (Qy) are presented. The curves of time strain increments (Et) plotted against deviator stress show a pattern similar to Housel's double ring shear test. The curves of deviator stress versus total strain (E) show the same type of curve obtained from the unconfined, quick, and consolidated quick tests. These curves show the material to be quite brittle. The curves of deviator stress versus instantaneous strain (Ei), i.e. this excludes all increments of strain due to time, plot as straight lines for a surprising portion of the curve. This suggests that some type of rheological model, such as a Maxwell or a Kelvin body, might represent the clays behaviour with time under stress. The straight line portion of these curves represent a deformation modulus varying from 3,400 psi to 9,300 psi.

On Plate No. 5 the results of the yield tests (Qy) as they vary with depth are presented. By comparing the "yield stress" (2Ty) with the ultimate strengths obtained in both the yield tests (Qy) and consolidated quick tests (Qc) it can be concluded that the clay "yields" between 76% and 88% of the ultimate strength. The tests on samples at depths of 35', 45' and 50' were not completed using the standard incremental procedure right up to the "yield stress". The points that have been plotted at these depths are estimates, based on the test results, of the probable "yield point".

Plate No. 5 also shows failure strains from the yield tests (Qy), consolidated triaxial tests (Qc) and creep tests (Qp). The variations of failure strain corresponding to the ultimate strength in the yield tests (Qy) and the consolidated quick tests (Qc) vary widely. It is considered that these have no particular significance as their method of selection is not such as to produce consistant results (being the abscissa corresponding to the maxima on a rather flat curve). It is condidered that the strain at which "yielding" occurred is a more significant number. The plate shows that the failure strain here varied between 0.65% and 2.4%. Considering the strength of the sample on which the 2.4% figure was obtained it seems probable that disturbance affected the results. By examining the results on the samples at 25' and 40' it seems possible that these results may have been affected by fissuring in the samples. Consequently, if the results from these three tests are eliminated the failure strain at "yielding" then varies between 1.2% and 1.4%. It is felt that this suggests that there is a critical strain in this soil beyond which the natural structure starts to break down and the natural strength is reduced. By assuming that the failure strain in the constant stress tests (Qp) is represented by the point of inflection in the strain-time curves it is seen on Plate No. 6 that the variation here is between 1.0% and 1.8%. This is fair corroboration of the results of the yield tests (Qy). Using the same concept a preliminary examination of the constant strain rate tests showed critical strains varying between 0.7% and 1.0%. In addition, below 50' the continuing pressure in the triaxial tests was less than the overburden pressure. This is known to affect test results, the more erratic strain plot below 50' might be accounted for to some extent by this factor.

On Plate No. 6 the results of cycling the stress increment in Qy tests on two samples are shown. The curves show that the instantaneous strain is of the same order of magnitude as the time strain. They also show that the recovery of instantaneous strain varies from about 30% to 80% of the original instantaneous strain. The recovery of time strain is substantially zero. The instantaneous strain with the second application of the stress increment seems to be about the same as the recovery of instantaneous strain. The increase time strain on the second application of the increment is of the order of 20% of the original time strain. These data indicate that for reversible effects no simple rheological model would be representative. They also indicate that besides elastic and viscous reactions to stress there is also the plastic reaction as well.

On Plates No. 7 and 8 are shown incremental stress tests run with time increments sufficient to obtain a constant strain at any given stress level. These curves indicate there is some substantiation for assuming that at stress levels below the "yield stress" failure will not occur, and a finite strain will be obtained given some minimum amount of time but that this strain will not increase with periods of time greater than this minimum. The normal deviator stress-strain curves are added for reference purposes. In addition, they show stress reactions after failure that are surprisingly high. Other tests for stress re-action after failure on a neighbouring project showed much lower values. Furthermore, it is known that the completely remoulded strength is almost zero in these soils. It is possible that these results were influenced by the previous stress program - migration of water away from the potential failure plane is possible.

Conclusions

It is possible that the average undrained quick shear strength of the clay is closer to 2100 psf, considering the vane and the consolidated triaxial tests, than the 1400 psf based on the unconfined compression tests. It is also possible that the minimum shear stress at the tunnel elevation may be closer to 1800 psf, based on vane and consolidated triaxial tests, rather than 1000 psf based on the unconfined compression tests. On the other hand, if the differencesin results of the quick and consolidated quick triaxial tests are due to joints or fissures in the clay, adjacent to an opening the unconfined compression test results would be the more representative strength. Whatever the correct conclusions is on this point, it is clear that, considering the state of the art, current policy of using normal thin-walled 2" samplers and unconfined compression tests, if carefully done, provides good quality results for normal purposes.

Although not conclusive the test results do suggest that the decrease in undrained shear strength with time is not likely to be more than about 25%. It might be preferable to be discussing shear strength in terms of effective stresses. However, the fact that only one confining pressure was used in this series of tests and the complexity of the drainage flow pattern to be expected around a tunnel heading made it impractical to deal in other than total stresses for this particular problem.

On the basis of the analyses that have been done for this report it seems clear that the maximum strain which could occur in this clay without failure would be very close to 1%. This should be valid regardless of the amount of time during which the shear stresses exist. On the other hand, variations in the intermediate and minor principal stresses might affect this figure.

REFERENCES

CASAGRANDE, A. & WILSON, S. "Effect of Rate of Loading on the Strength of Clays and Shales at Constant Water Content", Geotechnique, Vol. 2, June 1951.

SKEMPTON, A. & BISHOP, A. Chapter X, "Soils" in "Building Materials, Their Elasticity and Inelasticity", Interscience 1954.

GOLDSTEIN, M. & TER-STEPANIAN, G. "The Long-term Strength of Clays and Deep Creep of Slopes" Proc. 4th ICSMFE, Vol. 2, 1957.

GOLDSTEIN, M. & MISUMSKY, V. "Discussion" Proc. Brussels Conference 58 on Earth Pressure Problems, Vol. 3, 1958.

BJERRUM, L., SIMONS, N. & TORBLAA, I. "The Effect of Time on the Shear Strength of a Soft Marine Clay" Proc. Brussels Conference 58 on Earth Pressure Problems, Vol. 1, 1958.

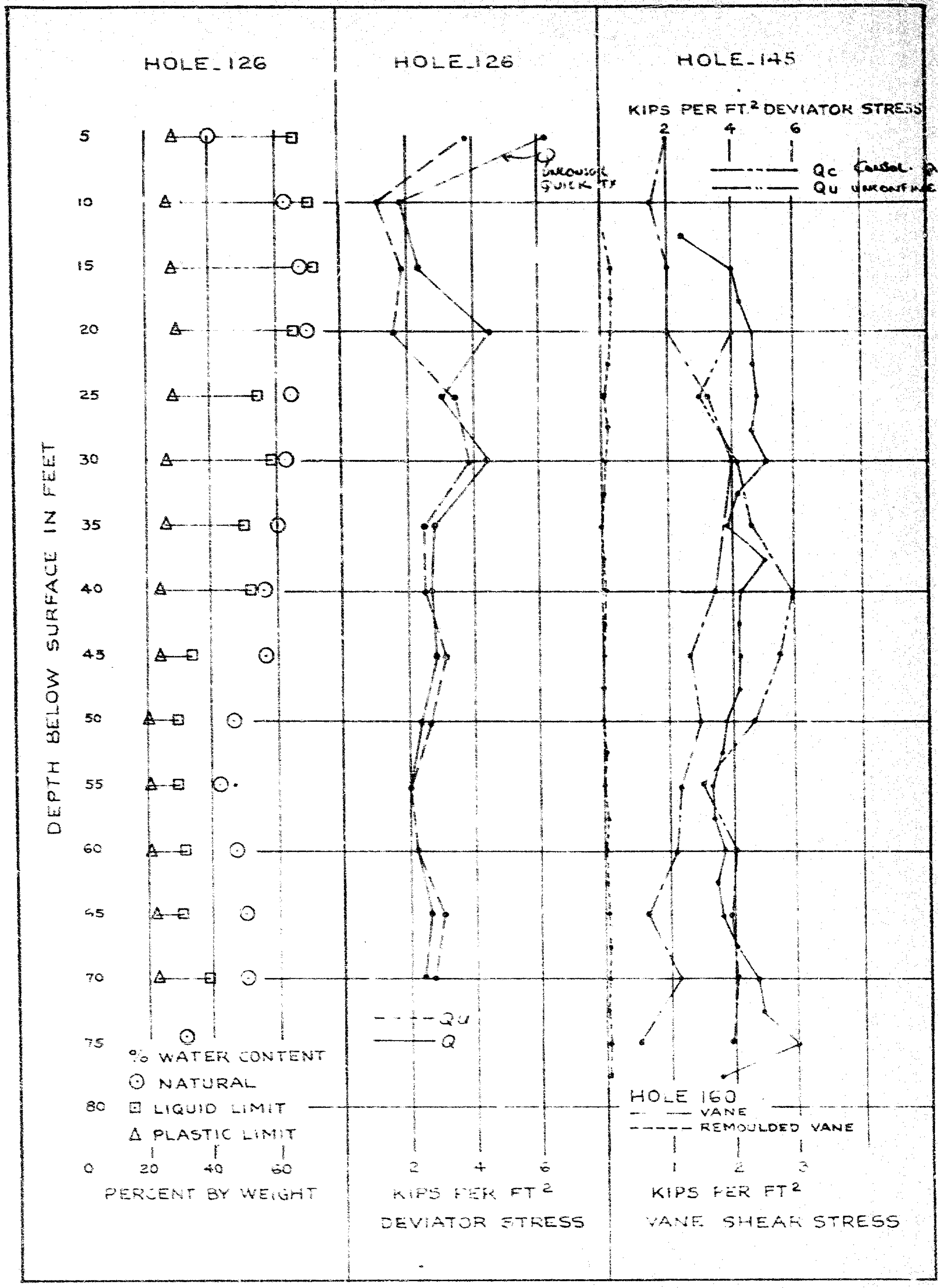
BJERRUM, L. "Construction of a Subway through Soft Clay in Oslo" a talk to Ottawa Soil Mechanics Group, 31 August, 1959.

CRAWFORD, C. "The Influence of Rate of Strain on Effective Stresses in Sensitive Clay" preprint of paper for 62nd Annual Meeting of ASTM, June 1959.

HENKEL, D. & SKEMPTON, A. "A Landslide at Jackfield, Shropshire in a Heavily Over-consolidated Clay" Geotechnique, Vol. 5, June, 1955.

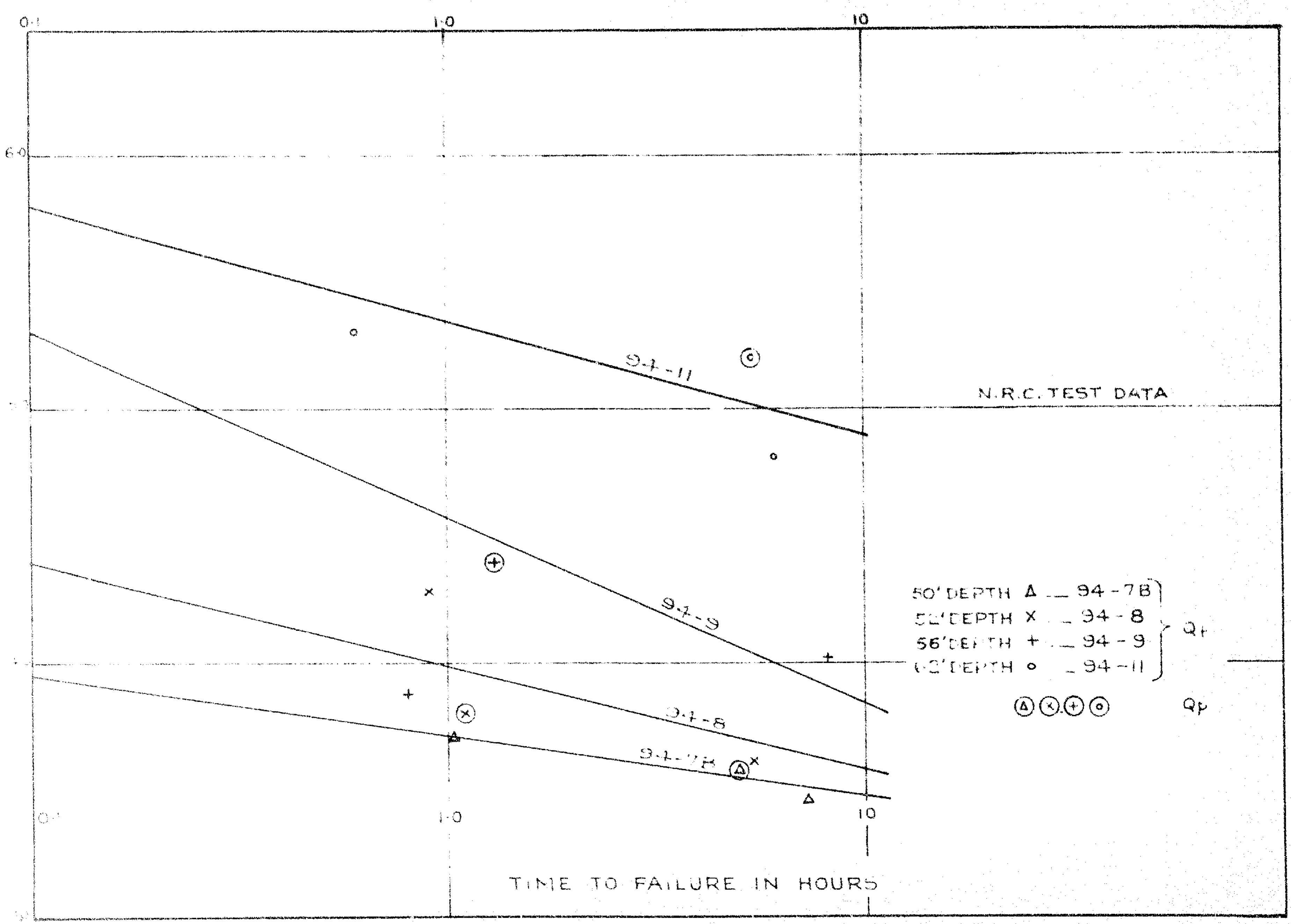
HENKEL, D. "The Effect of Over-consolidation on the Behaviour of Clays during Shear" Geotechnique, Vol. 6, December, 1956.

HOUSEL, W. "The Shearing Resistance of Soil - Its Measurement and Practical Significance" Proc. ASTM, Vol. 39, 1939.

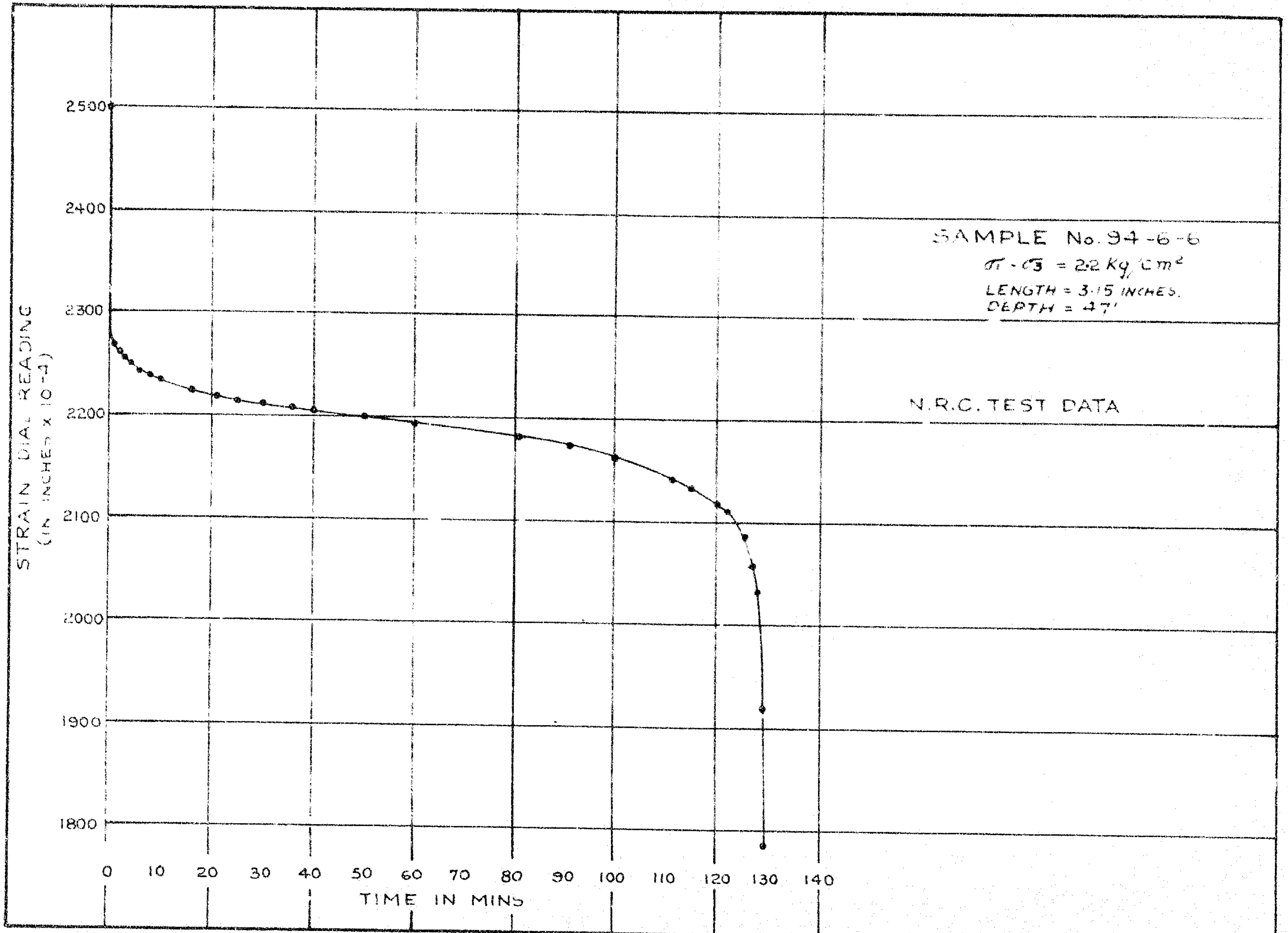


STRENGTH & CONSISTENCY PROFILE

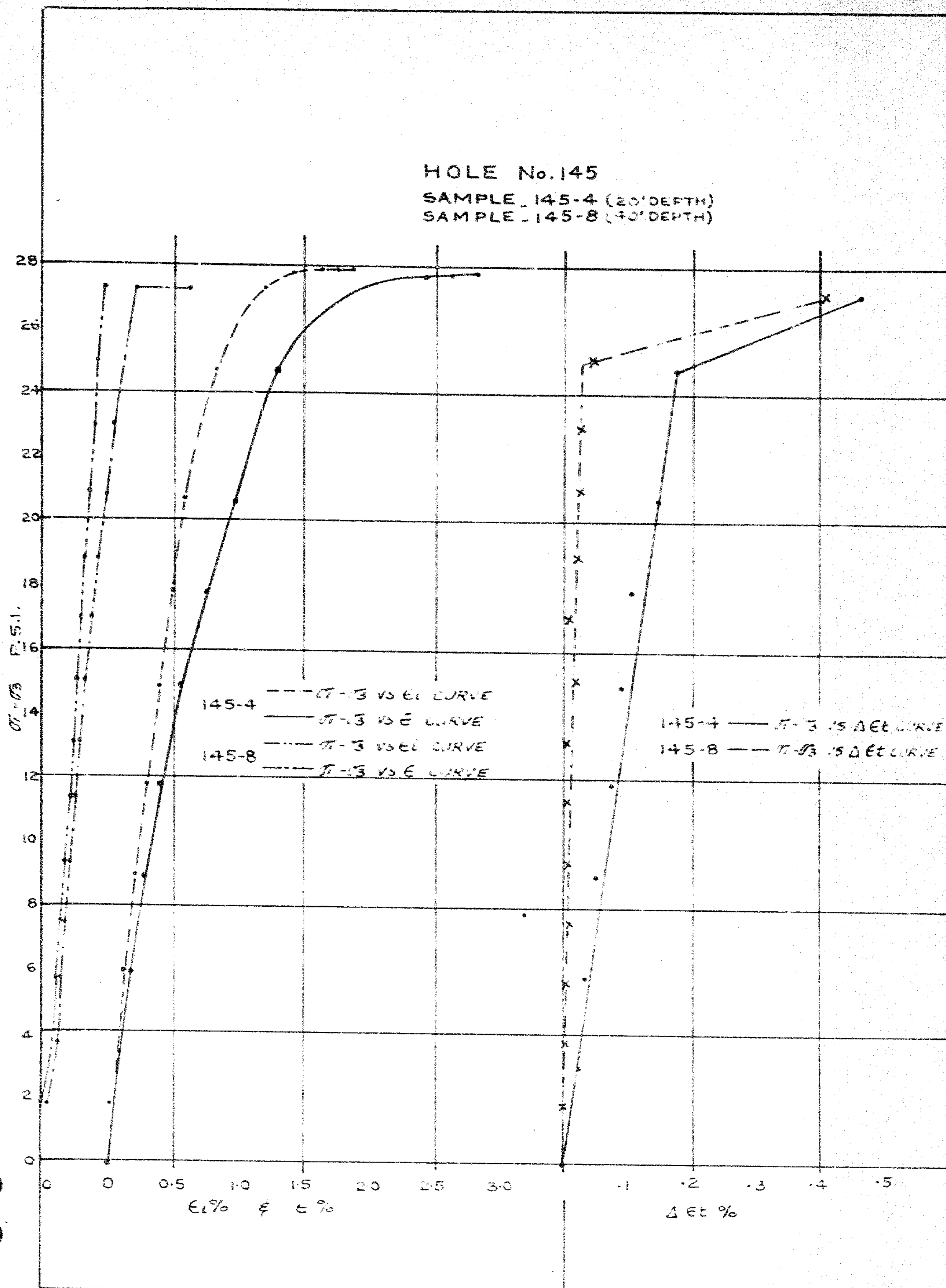
PLATE



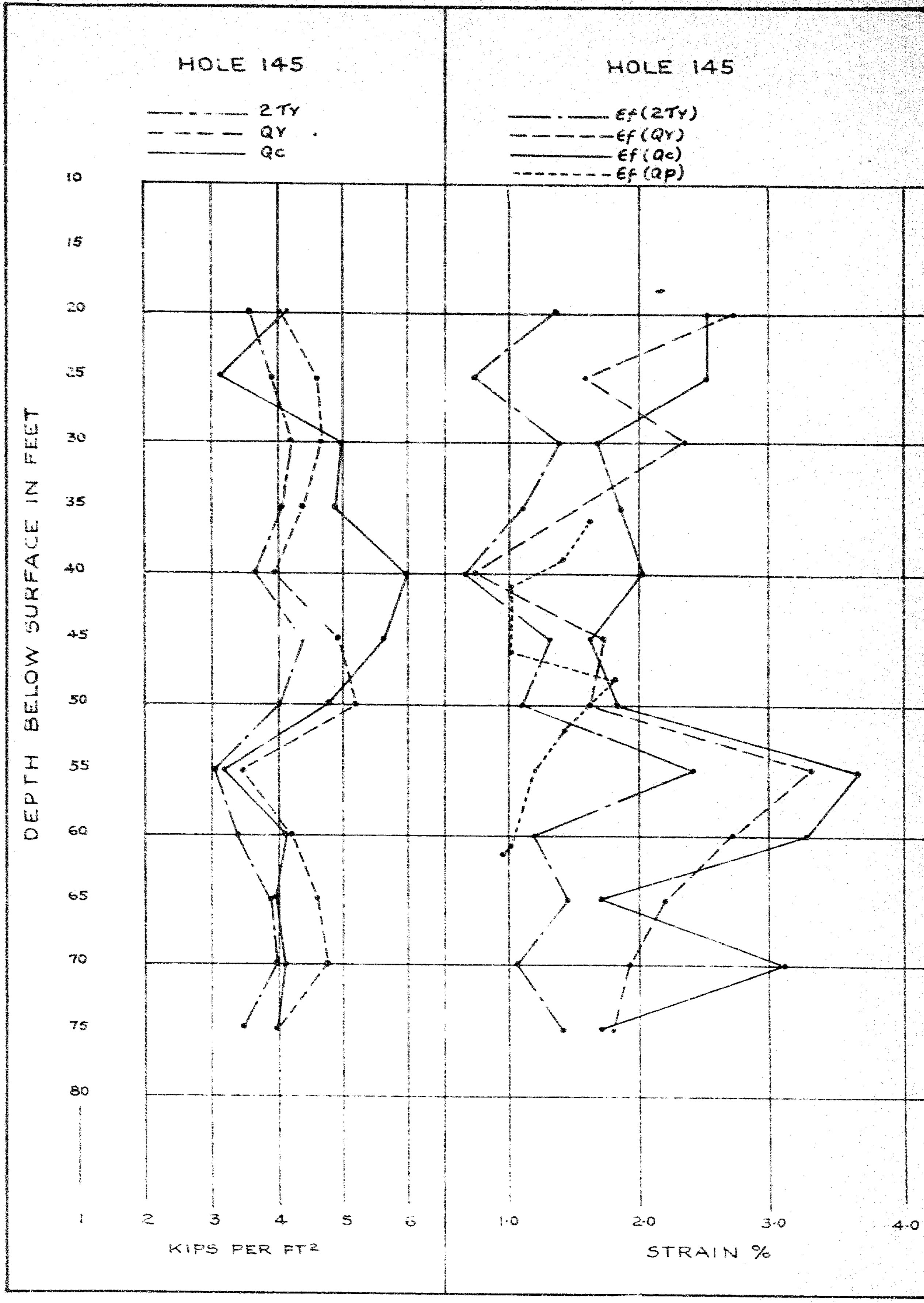
0·1
1·0
10
6·0
N.R.C. TEST DATA
94-11
94-9
94-8
94-7B
50' DEPTH Δ _ 94-7B
56' DEPTH + _ 94-9
Qp
TIME TO FAILURE IN HOURS



TYPICAL STRAIN VS TIME CURVE FOR Qp TESTS

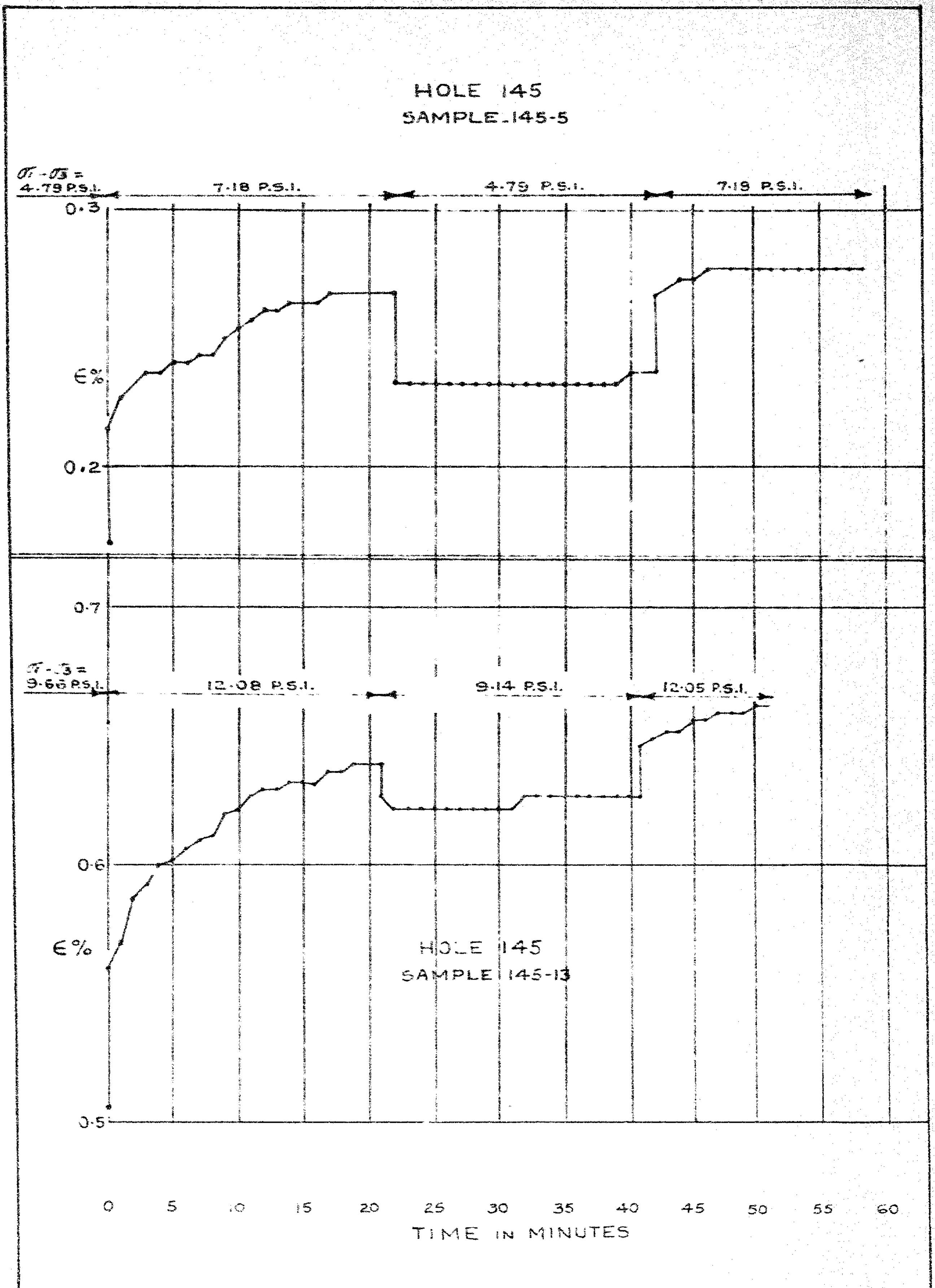


INCREMENTAL STRESS - TIME TESTS (QY) PLATE No. 4



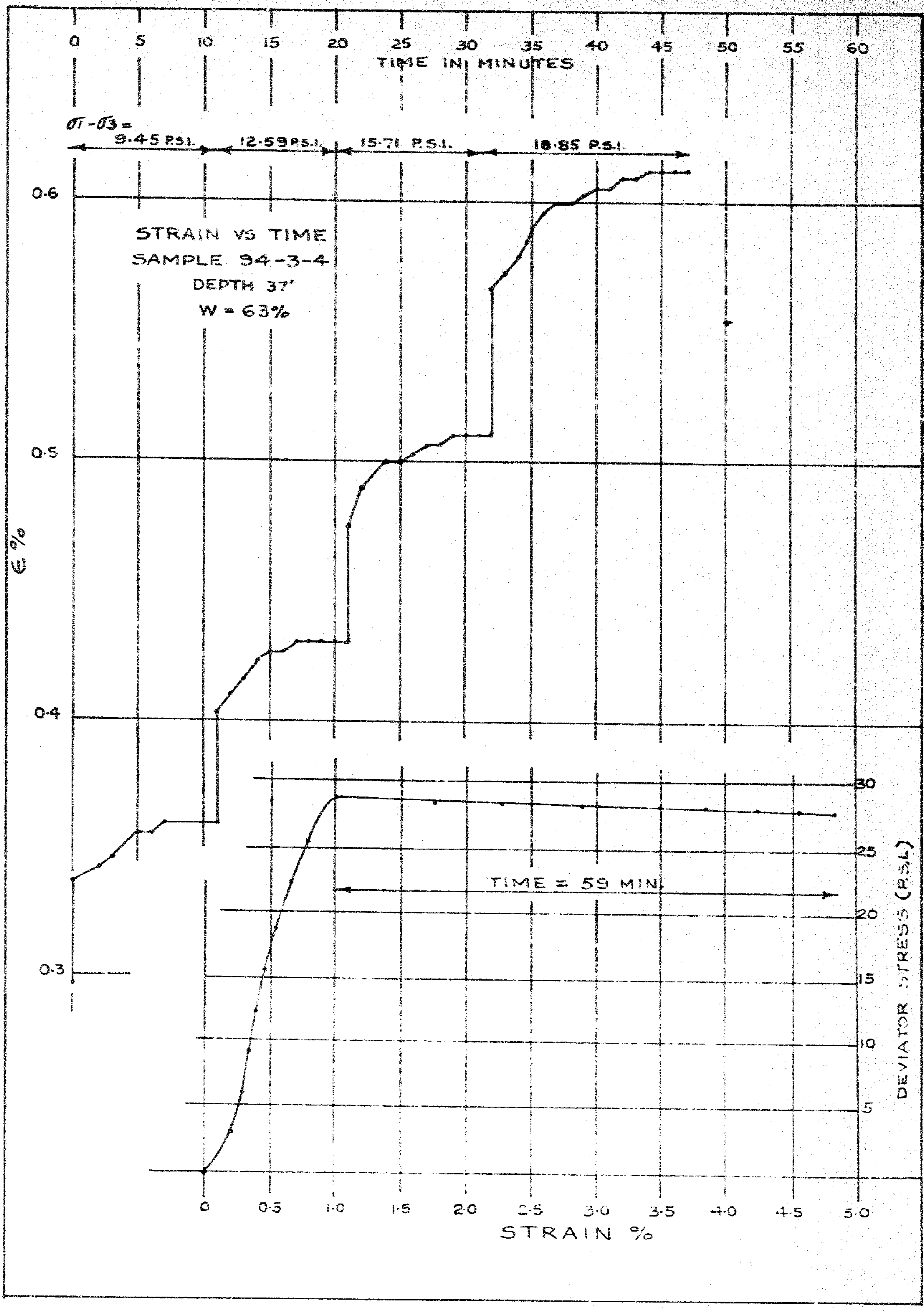
"YIELD" STRESS-STRAIN PROFILE

PLATE No. 5



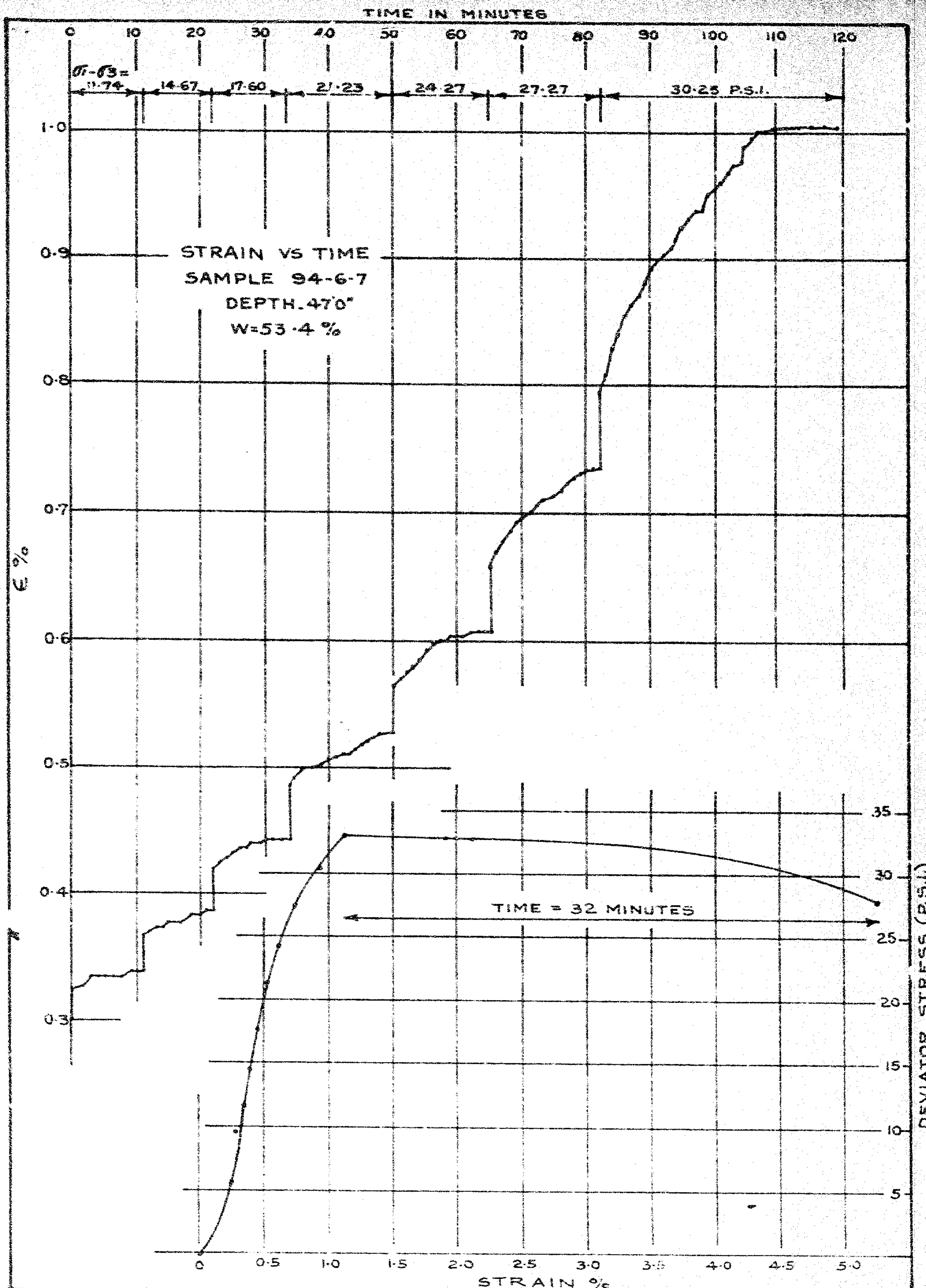
CYCLING OF STRESS INCREMENTS (QY)

PLATE No. 6



TIME EFFECTS ABOVE & BELOW "YIELD STRESS"

PLATE No. 7



TIME EFFECTS ABOVE & BELOW "YIELD STRESS"

PLATE No. 8

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SUBSURFACE INVESTIGATION FOR A PROPOSED STRUCTURE AT METCALFE STREET AND THE QUEENSWAY, OTTAWA, ONTARIO

1. INTRODUCTION

We were requested by the Ottawa Office of De Leuw, Cather & Company of Canada Limited, to carry out a subsurface exploration at the site of a proposed structure to carry the Queensway over Metcalfe Street. The results of a pilot hole study at this site were submitted in a preliminary stage report in October, 1960. This report contains final recommendations pertaining to the structure only, based on results from two boreholes. Recommendations regarding the embankment at this location were submitted in September, 1961. However, the report includes for completeness all the data and results of analyses contained in the preliminary report for this site.

2. RECOMMENDATIONS

2.1. Foundation Type

A footing type of foundation is recommended for the structure at this site and could bear at about elevation 216 on the in-situ brownish gray clay stratum determined in the two boreholes. The structure should be designed to allow differential movements not exceeding 0.3 feet perpendicular to the Queensway centreline. Settlements of the structure could be lessened by a pile type of foundation but larger differential settlements between the embankments and the structure would result. Also, settlement calculations indicate that smaller differential settlements of the structure would result with a closed end type of abutment rather than an opened end structure. Hence, closed abutments would appear preferable.

2.2. Soil Strength

Results from a shear strength study at this site indicate that a net allowable bearing capacity of 4,000 POUNDS PER SQUARE FOOT may be assumed for design purposes for footings bearing on the brownish gray clay near elevation 216. Since no attempt was made to determine the appropriate absolute value of shear strength of the cohesive stratum, a factor of safety of 3 is incorporated in the recommended bearing capacity.

A valuation of the shear strength of the bearing soil in the critical zone was obtained by penetrometer readings, unconsolidated triaxial testing or piston type tube samples and by field vane tests. A plot of stress distribution below the abutments showed that the induced stress pattern remained within the scatter of shear strength results.

2.3. Soil Compressibilities

The results of a settlement study made in the preliminary stage report are shown on the accompanying Plate No. 4. The calculations were limited to the compressible strata between elevation 213.7 and 183.7. The values of settlement calculated following present day standard calculation methods are known to be larger than the values to be expected. For this reason we recommend that the lower values of settlement, calculated using the void ratio at the present overburden, be adopted as a guide in design.

Field observations of settlements on structures built on similar subsoil (Highway 17 at Green Creek and Kars Bridge) substantitate the fact that predicted settlements are greater than those actually observed. There are probably many reasons for this difference; one is the difficulty in establishing the maximum load imposed on the cohesive strata during their geological history. The laboratory void ratio vs log of pressure curve for an "undisturbed" soil sample is known to have lower coordinates than the actual field curve. The laboratory curve yields a preconsolidation load which corresponds to an appreciable amount of settlement whereas if the field curve could be determined, the preconsolidation load would be somewhat higher and the change in void ratio would be correspondingly smaller. Thus lower settlement values would be predicted.

The basic one dimensional consolidation theory utilizes the so called "initial" void ratio. This void ratio should represent the condition of the soil in-situ. Calculating initial void ratios by means of basic soil properties such as water content and specific gravity, for samples below the groundwater table, a realistic in-situ void ratio may be obtained. However, some authors use as initial void ratio the void ratio obtained from an e vs log p curve at present overburden pressure. The accuracy of the result is questionable since the method is based on a plot which very often yields erratic results. Nevertheless, settlement computations in which the actual initial void ratio is substituted by the void ratio at the present overburden pressure give considerably smaller settlement values to be expected; this, by virtue of a lower void ratio value at present overburden pressure than the calculated initial void ratio based on actual soil properties. At this time, however, we recommend that the lower settlement values predicted, as shown on Plate No. 4, be used as a guide in design.

It can be seen from the above discussion that a settlement observation device installed on a structure abutment, such as this one, is urgently needed to help improve the state of knowledge in the profession and to increase our ability to forecast settlements. If such a device could be installed on this structure, observations of settlement could be made and compared to computed settlements obtained by other methods, such as applying the elastic theory assuming an adequate Poisson's ratio and determining moduli of elasticity from triaxial test results obtained to date on samples from this site. Recompression settlements could be calculated also based on an extended tangent to the initial part of recompression curves already determined from consolidation tests at this location.

A study of the intereffects of abutments and embankments was made in the preliminary stage investigation for a closed end structure considering a depth of 40 feet below existing ground surface. The results showed that no appreciable stress concentration occurred within this depth. Therefore, it may be concluded that this intersection would not increase the settlement values reported on Plate No. 4.

2.4. Stability of Foundation Soils Adjacent to Abutments

The abutments and backfill approaches in a closed end structure should be stable at this site. Borehole No. 2 confirmed the nature of the subsoil as determined in the pilot hole and therefore no slip is anticipated in the foundation soils due to the abutment and backfill loads. This conclusion was originally obtained from a stability analysis made in the preliminary stage study. (see Plate No. 5).

The effects of creep movements in the foundation soils were considered but were not felt to have a significant effect on a footing type foundation at this site.

2.5. Embankment

The results of a preliminary analysis on the foundation conditions for the embankment at the structure on this site were submitted in report No. SF-487. The results of a complete analysis on the foundation conditions for the Queensway embankments between Bell Street and Elgin Street were also submitted in a report No. SF-562.

3. SITE INVESTIGATION

3.1. Preliminary Stage Investigation

3.1.1. Field Work

A pilot boring (Hole No. 1) was made at the site with our test drilling rig in the location shown on Plate No. 1 Eight Shelby tube soil samples were recovered from cohesive soil strata at 5 foot intervals down to a depth of 30 feet below ground surface and at 10 foot intervals, below 30 feet, down to more granular soil layers. Five two-inch split barrel samples were recovered in granular soil strata in conjunction with standard penetration resistance tests at 10 foot intervals. All the samples were brought to our laboratory for examination and classification. An indication of the relative densities of the granular layers encountered was supplied by the results of the standard penetration tests performed. During the boring operations groundwater level was observed and recorded.

Bedrock encountered at 104.3 feet below ground surface was diamond drilled. The cores were recovered for inspection and logging. Core recovery percentages were determined in an effort to evaluate the structural properties of the rock. The presence of seams in the bedrock formation was detected by careful watch for drops of drill rods and loss of drill water during the rock drilling operations.

3.1.2. Laboratory Testing

Classification tests were made on most samples, two consolidation tests were made at depths representing the mid height of increments of the compressible layer, and preliminary strength determinations were made, a group of unconfined compression tests were made to provide correlation with those tests in other areas, but it is recognized that in the fissured zones the results are lower shear strength values than may be confirmed with other techniques.

Small scale penetrometer tests (Soiltest Pocket Penetrometer) were made at each six inch interval in the soil before it was extruded from the tube. This simple procedure provides useful information on the variations in strength in a vertical direction. In fissured clays we feel that the penetrometer may provide a more representative answer than the unconfined test since the soil is confined during the penetrometer reading by the lateral pressure of the tube. Penetrometer tests on remoulded samples were attempted at each six inch interval to check that the remoulded strength had not changed. A zero reading on the penetrometer means that the remoulded strength was less than 0.1 kips per square foot.

3.2. Second Stage Investigation

3.2.1. Field Work

Boreholes Nos. 2 and 2A were made with our test drilling rig in the locations shown on Plate No. 1. Three split barrel samples were taken at 2 1/2 ft. intervals from the very stiff cohesive crust for classification purposes. Five thin wall tube samples, 2 inches in diameter were retrieved by a piston type sampler, at 5 ft. intervals, in the clay soils down to 37 ft. below ground surface. A thin wall open tube sample was taken at 10 ft. depth in Hole 2A when the sample at this depth was not recovered in Hole No. 2. Six borehole vane tests were made at 5 ft. intervals in between tube samples to estimate the in-situ shear strength of the cohesive soil layers encountered. All samples were brought to our laboratory for classification, examination and testing. An overnight groundwater level was observed and recorded during the boring.

3.2.2. Laboratory Testing

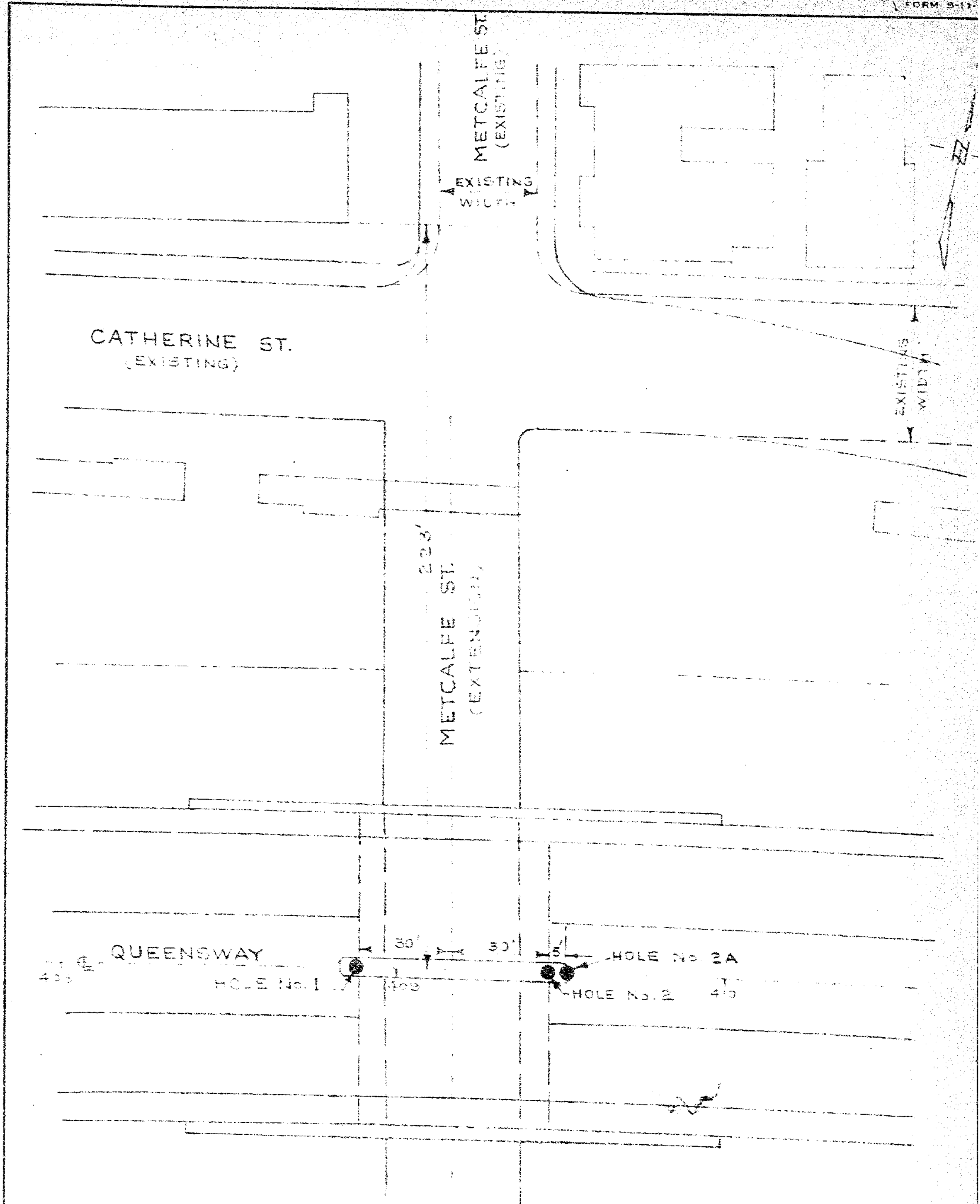
Eight unconsolidated undrained triaxial tests were performed on samples from within the critical zone to determine the shear strength of the soils encountered. A triaxial test could not be made on the sample at 10 ft. depth because of the small amount of soil retrieved. Small scale penetrometer tests were made at six-inch intervals in each tube to verify the variation of shear strength in a vertical direction. Small scale penetrometer readings were also taken from the end of split barrel samples in the crust only to ascertain the consistency of the clay at that depth. Classification tests were also made on most samples.

3.3. Observations

The geotechnical profile of the subsoil as revealed by the two boreholes is shown on the accompanying Plates No. 2 and 3. It can be generalized as consisting of an average of 4 ft. of fill, mostly fine sand, underlain by 63 ft. of gray silt and clay mixture and clay, decreasing in consistency from very stiff to medium soft with depth. About 31 ft. of sand and silt mixtures underlie these strata and overlie approximately 5 ft. of loose to medium dense glacial till underlain by bedrock. The rock encountered at elevation 119.4 consists of black shale with the upper layers possibly fractured. High percentages of core recovery from the lower strata would indicate bedrock to be structurally sound.

An overnight groundwater level reading in Hole No. 1 in December 1959, showed the groundwater to be at about 21 ft. below ground surface.

However, in April, 1962, an overnight groundwater level reading in Borehole No. 2 indicated that the groundwater table was at about 1 ft. below ground surface. This level can be considered high groundwater and would certainly be lower in drier seasons.



McROSTIE & ASSOCIATES
CONSULTING ENGINEERS

BOREHOLE LOCATIONS
METCALFE ST. & QUEENSWAY

SCALE 1" = 40'
PLATE 1

FORM S-10-4

McROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

METCALFE & QUEEN [illegible]

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 223·7' DATE [illegible]

REMARKS B.M. (EL. 219·5') CITY B.M. AT CORNER OF [illegible] & [illegible]

HOLE No. 1

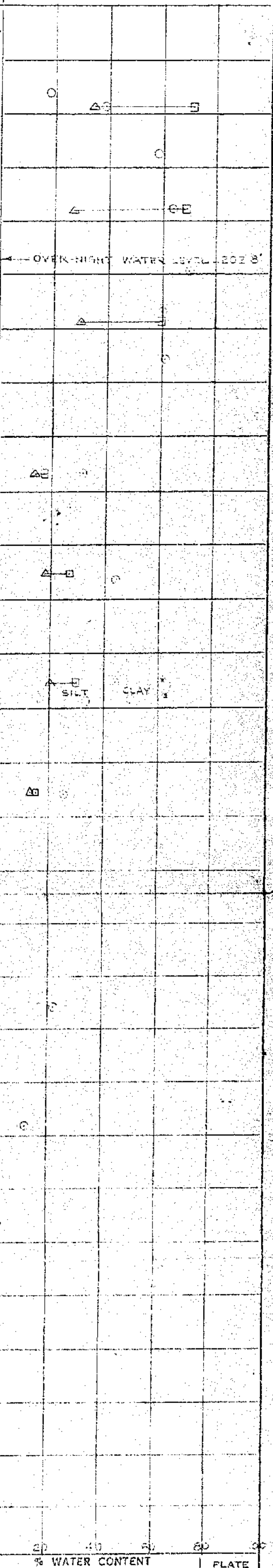
UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	223·7'
				FILL	5'	
		1 for 6"	1-1A	~~FINE SAND VERY LOOSE~~	5·5'	218·2'
	7·2, 7·6, 7·8	7	1-1B	CLAYEY SILT BROWNISH GRAY FISSURED, VERY STIFF HIGH PLASTICITY (MH)	10'	213·7'
2·4	4·0, 4·0, 4·0 3·4, 3·2, 3·6 2·4, 3·0, 2·4 R-0·2		1-2	CLAY, BROWNISH GRAY, FISSURED STIFF, HIGH PLASTICITY (CH)	15'	
2·3	1·8, 1·6, 2·0 1·6, 1·6, 1·4 2·8, 3·0, 3·0 2·2, 2·2, 2·2 2·2, 2·0, 2·4 R-0·0		1-3	CLAY, GRAY STIFF HIGH PLASTICITY (CH)		
2·0	2·4, 2·4, 2·2 3·5, [illegible], 3·6 2·0, [illegible], 2·0 3·4, 3·0, 3·2 2·8, 3·2, 2·8 R-0·0		1-4			
2·8	2·2, 2·5, 2·6 2·2, 2·4, 2·6 2·6, 2·4, 2·6 2·6, 2·6, 2·2 1·4, 1·6, 1·4 R-0·0		1-5			
2·5	2·6, 2·8, 2·6 2·6, 2·9, 3·0 3·2, 2·9, 2·8 3·4, 3·0, 3·0 3·2, 3·0, 3·5 R-0·0		1-6		40'	183·7'
1·2	1·6, 1·6, 1·8 1·0, 1·0, 1·2 1·2, 1·2, 1·2 1·6, 1·6, 1·6 1·5, 1·5, 1·6 R-0·0		1-7	SILT WITH SOME CLAY & A LITTLE FINE SAND, GRAY, MEDIUM SOFT LOW PLASTICITY (ML)	50'	
2·1	2·0, 1·9, 1·6 2·2, 2·0, 1·8 1·8, 1·8, 1·8 2·0, 2·0, 1·8 R-0·0		1-8	CLAY, GRAY, MEDIUM SOFT LOW PLASTICITY (CL)		
2·1 0·6	2·0, 3·0, 1·8 1·8, 1·8, 2·0 1·8, 1·8, 2·0 1·2, 1·4, 1·6 R-0·0		1-9		61·3'	162·4'
				SILT & CLAY IN 1/2" LAYERS GRAY, MEDIUM SOFT (ML & CL IN LAYERS)	68'	
		0	1-10	SANDY SILT WITH SOME CLAY LOW PLASTICITY VERY LOOSE (ML)	80'	143·7'
		21	1-11	SILT WITH SOME FINE SAND & A TRACE OF COARSE SAND MEDIUM DENSE		
		18	1-12		99'	124·7'
		9 7 for 6"	1-13	TILL, LOOSE	101'	
				TILL, MEDIUM DENSE	104·3'	119·4'
				SHALE CORE RECOVERY INDEFINITE BUT GREATER THAN 59%	110'	
				SHALE CORE RECOVERY 90%	11?'	
				SHALE CORE RECOVERY 95%	118·3'	105·4'
				BOTTOM OF HOLE		

PROBING OR VANE TEST

.......LB. HAMMER ....INCH DROP

NO CASING ....INCH DIA. ROD

BLOWS PER FOOT OR ~~SHEAR STRENGTH IN KIPS PER FT.²~~



% WATER CONTENT
NATURAL ○
LIQUID LIMIT □
PLASTIC LIMIT △

PLATE 2

L.M. CO.—A33078

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

METCALFE & QUEENSWAY

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 223·6' DATE APRIL 25, 1962

REMARKS SEE PLATE No. 2

HOLE No. 2 & 2A

TRIAXIAL ~~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
				GROUND SURFACE	0'	223·6'
	7·0, 6·5, 7·0	13 for 6" 15	2-1	FILL SAND & CINDERS	3'	
	7·5, 8·0, 7·0	12	2-2	CLAY BROWNISH GRAY SILTY, VERY STIFF FISSURED	7·5'	216·1'
	8·5, 8·5, 8·5	20	2-3	CLAY BROWNISH GRAY HARD TO STIFF FISSURED		
	5·3, 6·0, 6·0 3·5, 4·0, 3·6 R-0·8		2A-4		13·5'	
2·74	2·4, 2·4, 2·4 1·8, 1·8, 1·6 2·8, 3·0, 2·8 3·2, 3·2, 3·2 1·5, 2·0, 1·8 R-0·0		2-4	CLAY GRAY STIFF FISSURED HIGH PLASTICITY (CH)	20'	
2·30	2·2, 2·0, 2·0 2·2, 2·2, 2·0 2·9, 2·6, 3·0 R-0·0		2-5	CLAY GRAY STIFF SLIGHTLY FISSURED HIGH PLASTICITY (CH)		
2·74	3·5, 3·5, 3·4 3·0, 3·4, 3·5 3·3, 3·0, 3·3 3·0, 3·2, 3·2 2·8, 3·0, 3·3 R-0·0		2-6			
	1·8, 2·0, 2·4 2·2, 2·5, 2·0 2·5, 2·5, 2·4 2·0, 2·2, 2·0 1·2, 1·3, 1·3 R-0·0		2-7			
	1·9, 1·8, 1·6 2·4, 2·4, 2·4 2·2, 2·2, 2·0 1·4, 1·4, 1·5 1·5, 1·5, 1·6 R-0·0		2-8		37'	186·6'
				CLAYEY SILT MEDIUM SOFT	39'	184·6'
				BOTTOM OF HOLE		

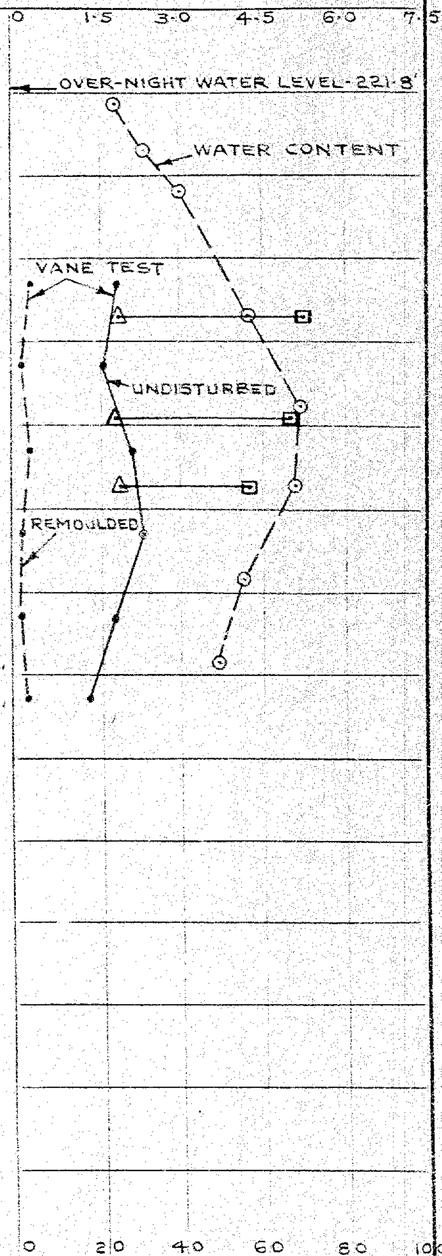
PISTON TUBES FROM 2-4 TO 2-8 INC.

~~PROBING OR~~ VANE TEST

.......LB. HAMMER ....INCH DROP

NO CASING ....INCH DIA. ROD

~~BLOWS PER FOOT OR~~ SHEAR STRENGTH IN KIPS PER FT.²



% WATER CONTENT
NATURAL ⊙
LIQUID LIMIT ⊡
PLASTIC LIMIT △

PLATE 3

R - REMOULDED

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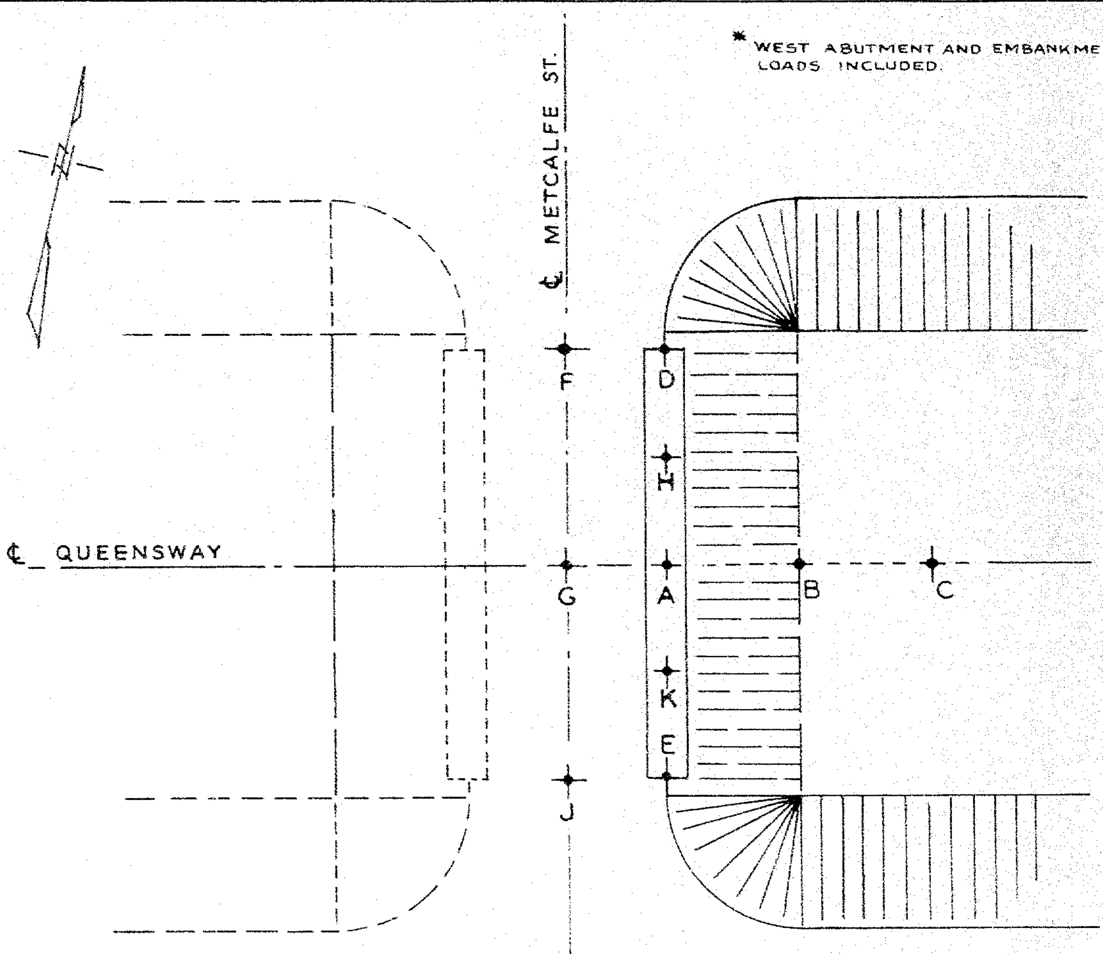
CONSULTING ENGINEERS
OTTAWA, CANADA

TYPE OF STRUCTURE	FOOTING SIZE	VOID RATIO	SETTLEMENT AT POINT						
			A	B	C	D & E	* F & J	* G	H & K
OPENED END	9.5'x130'	INITIAL	1.08'	1.29'	1.62'	0.93'			
		PRESENT OVERBURDEN	0.42'	0.66'	0.99'	0.27'			
CLOSED END	12'x130'	INITIAL	1.41'	1.51'	1.53'	1.08'	0.75'	0.75'	1.41'
		PRESENT OVERBURDEN	0.75'	0.84'	0.87'	0.42'	0.08'	0.11'	0.75'

* WEST ABUTMENT AND EMBANKMENT LOADS INCLUDED.



SETTLEMENTS AT VARIOUS POINTS FOR OPENED AND CLOSED END TYPE OF STRUCTURE.

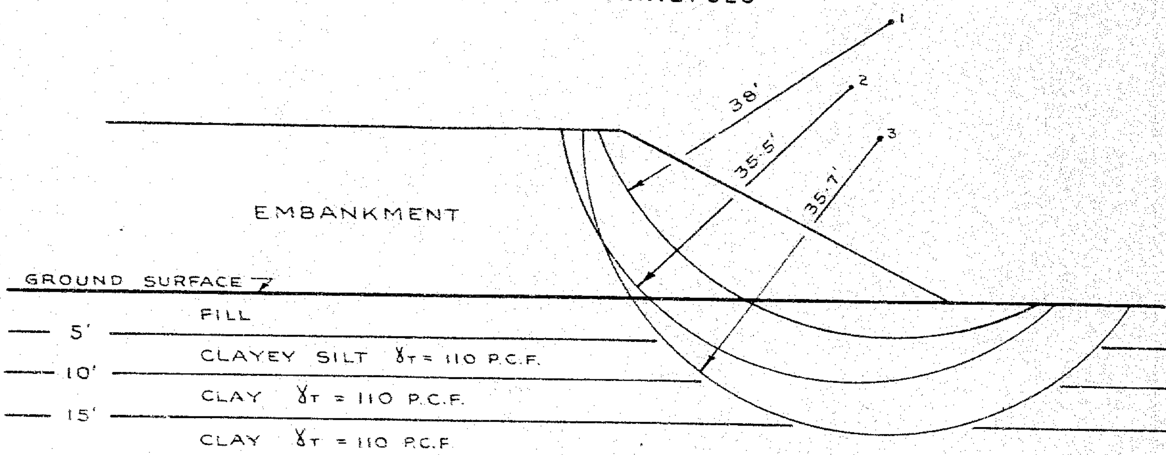
SCALE: 1" = 50'

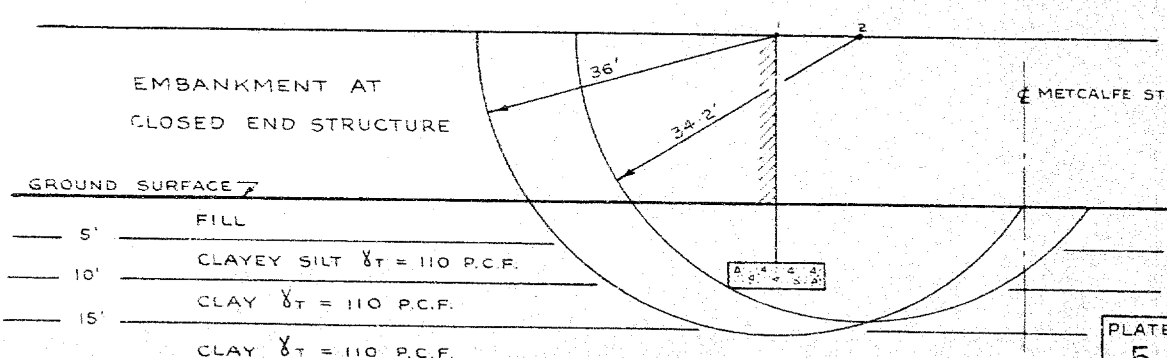
PLATE 4

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA, CANADA

STABILITY ANALYSIS OF	SLIP No.	EMBANKMENT		EXISTING FILL		COHESION ALONG ARC K.S.F.	F.S.	F.S. 25% LOSS OF STRENGTH
		φ°	γ_T P.C.F.	φ°	γ_T P.C.F.			
EMBANKMENT	1	28	120	28	100		1·29	
		30	120	28	100		1·32	
		32	120	28	100		1·36	
		30	120	REMOVED AND COMPACTED AS EMBANKMENT			1·44	
		32	120				1·56	
	2	28	120	28	100	3·5	5·3	4·0
		28	120	28	100	1·5	2·5	2·0
	3	28	120	28	100	3·5, 1·5, 1·25	3·8	2·8
STRUCTURE CLOSED END	1	28	120	28	100	3·5, 1·5	3·2	2·5
	2	28	120	28	100	3·5, 1·5	2·8	2·1

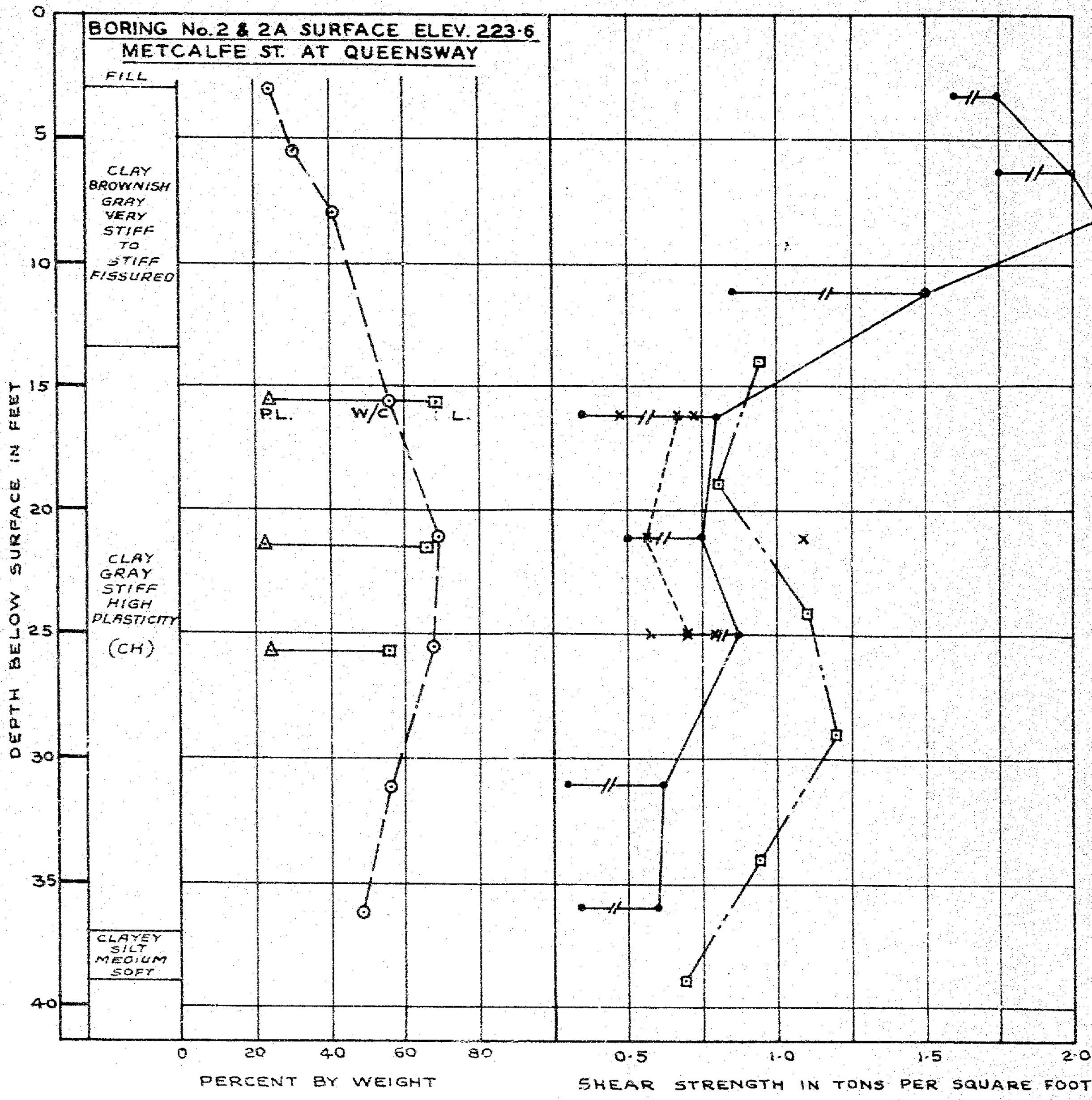
STABILITY ANALYSES





SCALES: 1" = 20'

PLATE 5



LEGEND

- HIGHEST PENETROMETER READINGS.
- RANGE IN PENETROMETER READINGS.
- BOREHOLE VANE TESTS.
- UNCONSOLIDATED UNDRAINED TRIAXIAL TESTS CHOSEN.
- × UNCONSOLICATED UNDRAINED TRIAXIAL TESTS.

McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA

TRIAXIAL COMPRESSION TEST

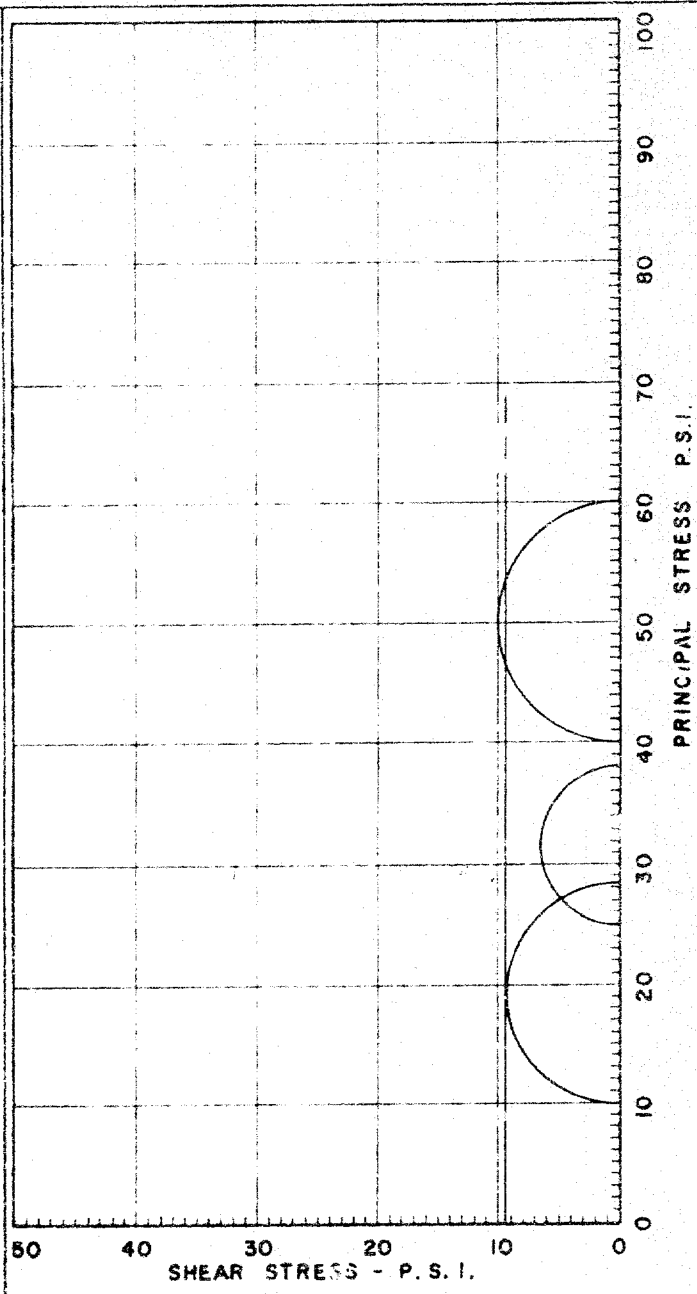
TEST SERIES SUMMARY SHEET

PROJECT: E_417 SITE: METCALFE & QUEENSWAY DATE: MAY 4/62

HOLE NO. 2 LOCATION: DEPTH:

TEST NO.	DEPTH	LATERAL PRESSURE (P.S.I.)	COMPRESSIVE STRESS (P.S.I.) - qc	WATER CONTENT - W %	WET DENSITY - γm
2_4A	16'0" TO 16'6"	10	18.5	72.7	
2_4B	15'6" TO 16'0"	25	13.1	70.7	
2_4C	15'0" TO 15'6"	40	20.2	74.6	
AVERAGE					

$C = 9.5$ P.S.I. $\phi = 0°$



MOHR DIAGRAM

CLAY HIGHLY PLASTIC (CH)

TESTED: DATE:
COMPUTED: DATE:
PLOTTED: G.B. DATE: 4/5/62
CHECKED: D.M. DATE: 4/5/62

McROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

FORM S-24

TR[illegible]XIAL COMPRESSION TEST

[illegible]EST SERIES SUMMARY SHEET

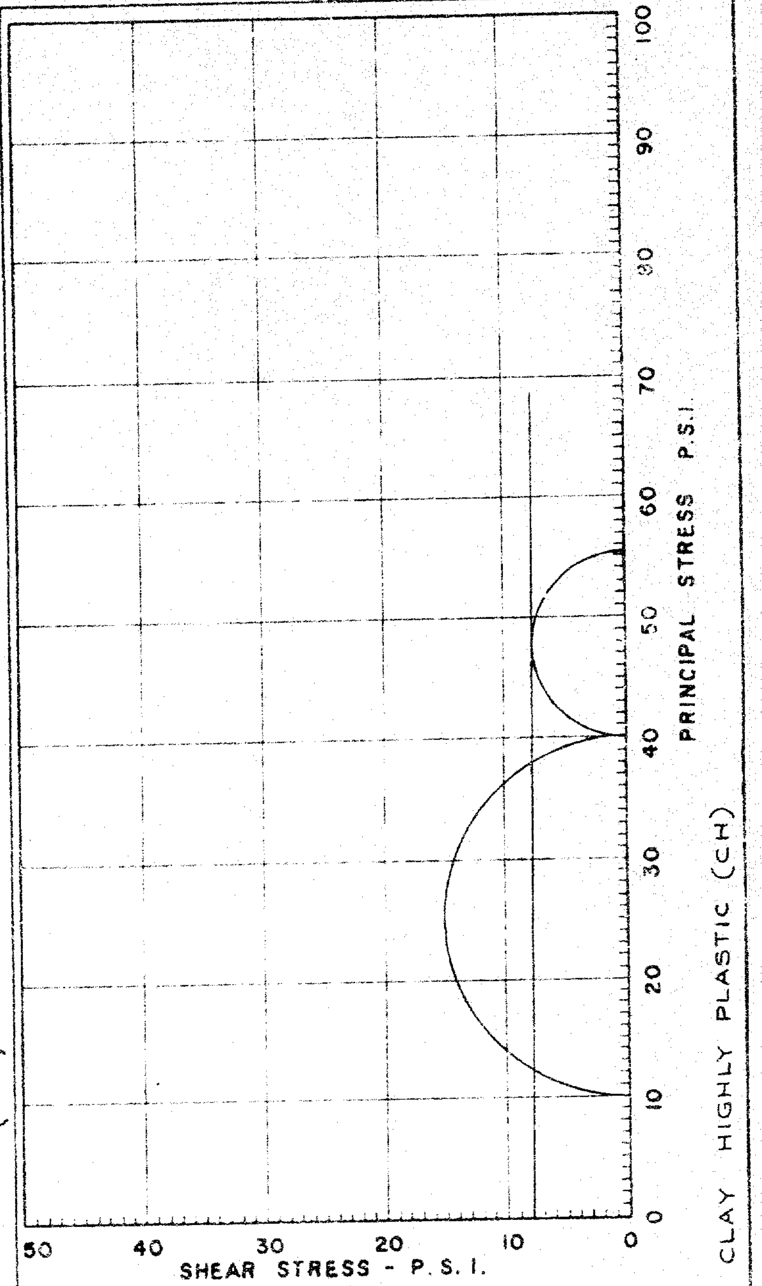
PROJECT: E-417 SITE: METCALFE & QUEENSWAY DATE: MAY 4/62

HOLE NO. 2 LOCATION: DEPTH:

TEST NO.	DEPTH	LATERAL PRESSURE (PSI)	COMPRESSIVE STRESS (P.S.I.) - qc	WATER CONTENT - W %	WET DENSITY - γm
2-5A	21' 0" TO 21' 6"	10	30.0	69.3	
2-5B	20' 3" TO 20' 7"	40	15.7	71.4	
AVERAGE					

$\Phi = 0°$

C = 8 P.S.I. (MIN.)



MOHR DIAGRAM

CLAY HIGHLY PLASTIC (CH)

TESTED:		DATE:	
COMPUTED:		DATE:	
PLOTTED:	G.B.	DATE:	4/5/62
CHECKED:	D.M.	DATE:	4/5/62

McROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

FORM S-14

TRIAXIAL COMPRESSION TEST

TEST SERIES SUMMARY SHEET

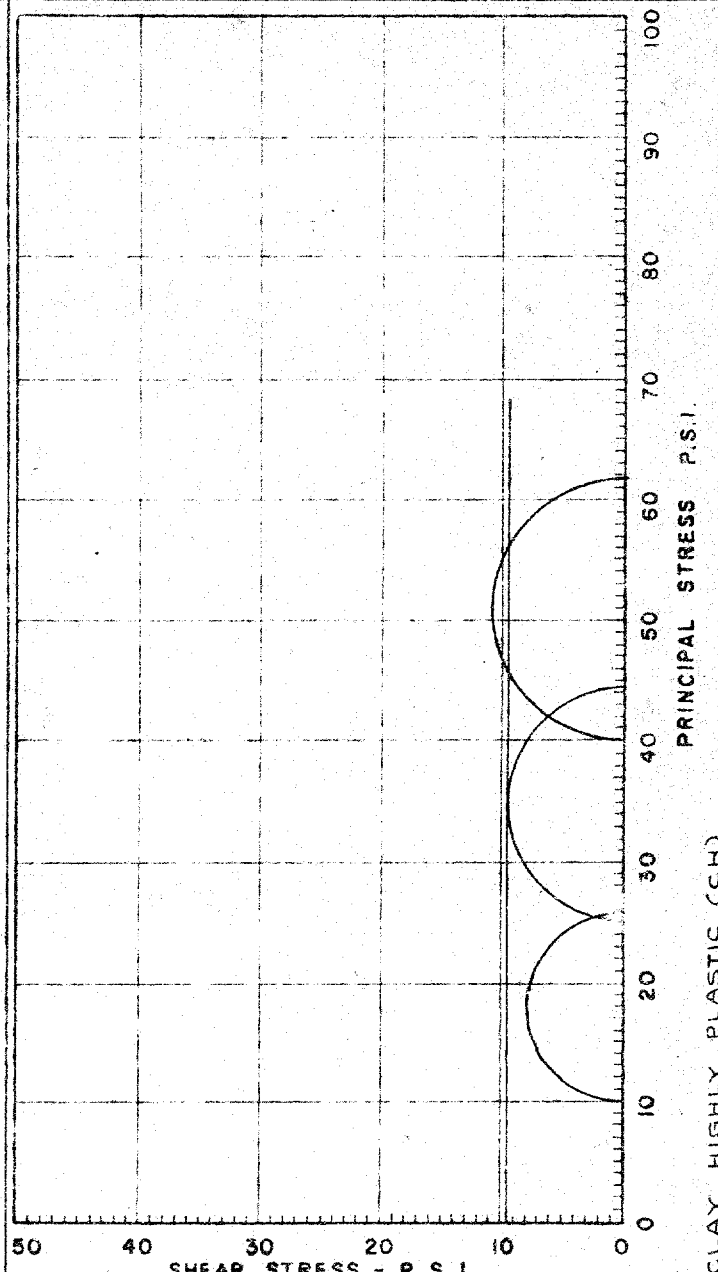
PROJECT: E-417 | SITE: METCALFE & QUEENSWAY | DATE: MAY 4/62

HOLE NO. 2 | LOCATION: | DEPTH:

TEST NO.	2-6A	2-6B	2-6C						AVERAGE
DEPTH	26'0" TO 26'6"	25'6" TO 26'0"	25'0" TO 25'6"						
LATERAL PRESSURE (P.S.I.)	25	40	10						
COMPRESSIVE STRESS (P.S.I.) - qc	19.8	21.8	35.8						
WATER CONTENT - W %	64.7	68.7	68.4						
WET DENSITY - γm									

C = 9.5 P.S.I.

$\phi = 0°$



MOHR DIAGRAM

TESTED: ______ DATE: ______
COMPUTED: ______ DATE: ______
PLOTTED: G.B. DATE: 4/5/62
CHECKED: D.M. DATE: 4/5/62

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MAR. 14/55 BR 822

PLATE No. 9

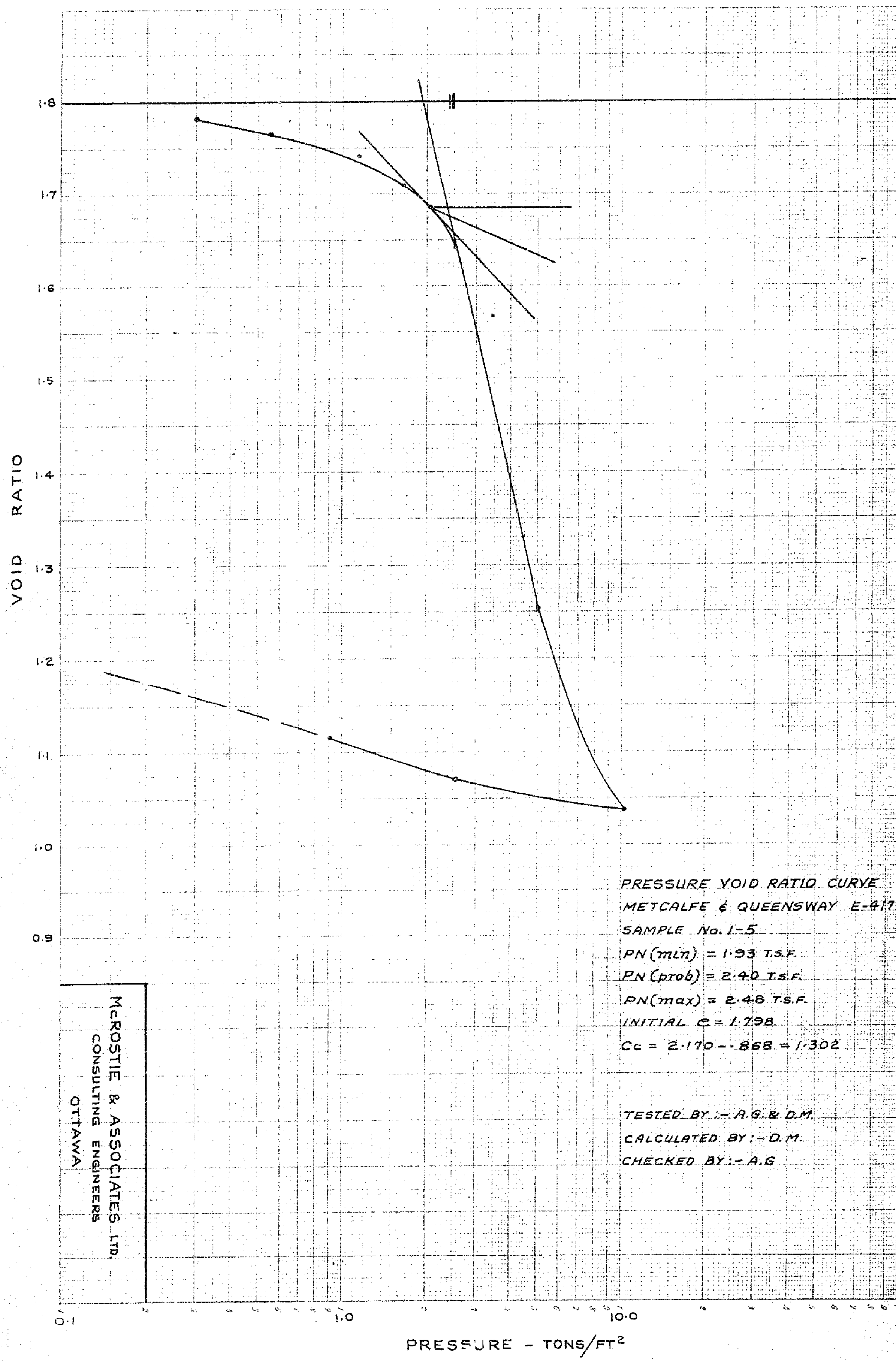
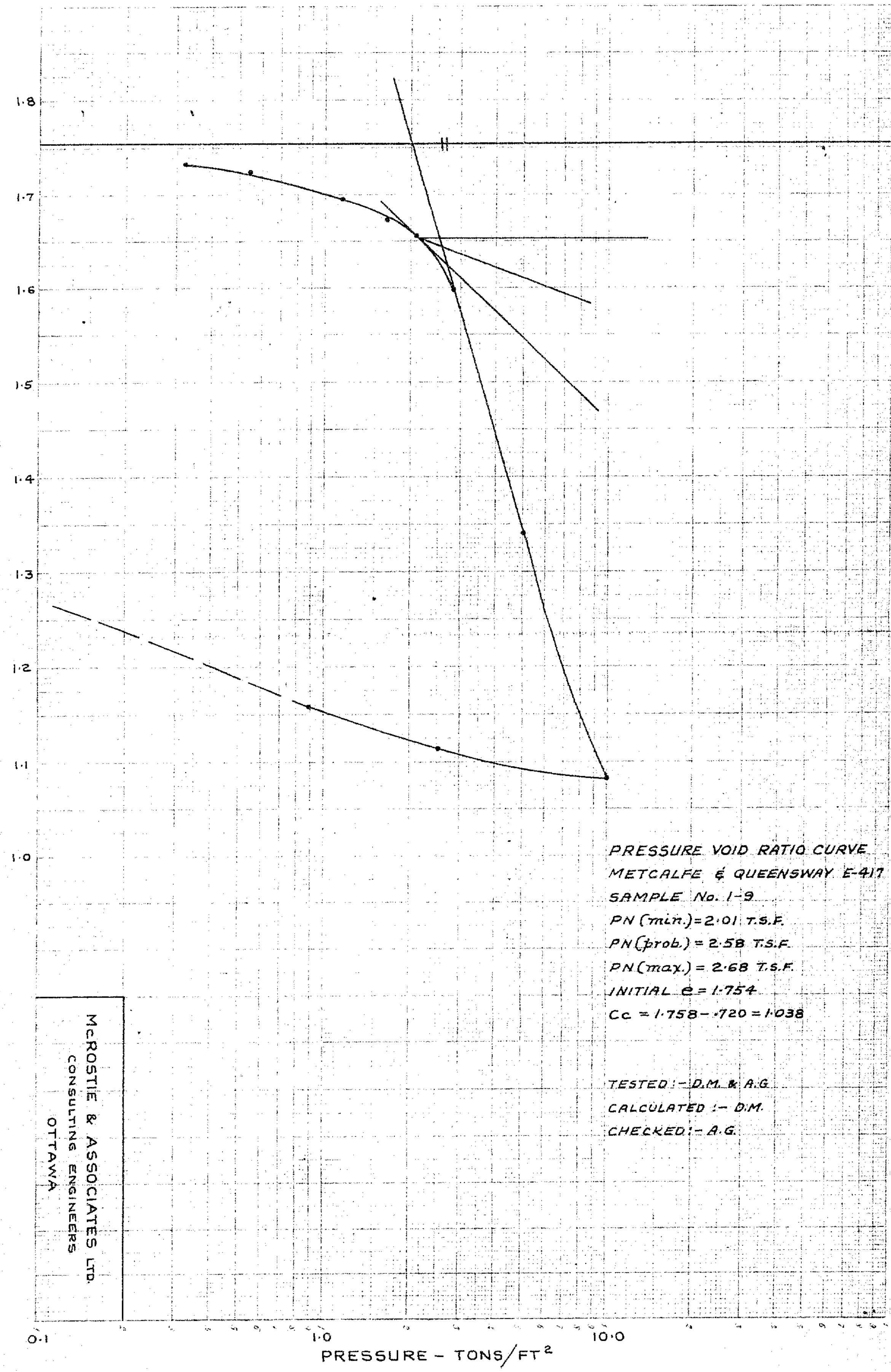

VOID RATIO
PRESSURE - TONS/FT²
1·8
1·7
1·6
1·5
1·4
1·3
1·2
1·1
1·0
0.9
0·1
1.0
10·0
PRESSURE VOID RATIO CURVE
METCALFE & QUEENSWAY E-417
SAMPLE No. 1-5
PN (min) = 1·93 T.S.F.
PN (prob) = 2·40 T.S.F.
PN (max) = 2·48 T.S.F.
INITIAL e = 1·798
Cc = 2·170 - ·868 = 1·302
TESTED BY :- A.G. & D.M.
CALCULATED BY :- D.M.
CHECKED BY :- A.G
McROSTIE & ASSOCIATES LTD
CONSULTING ENGINEERS
OTTAWA

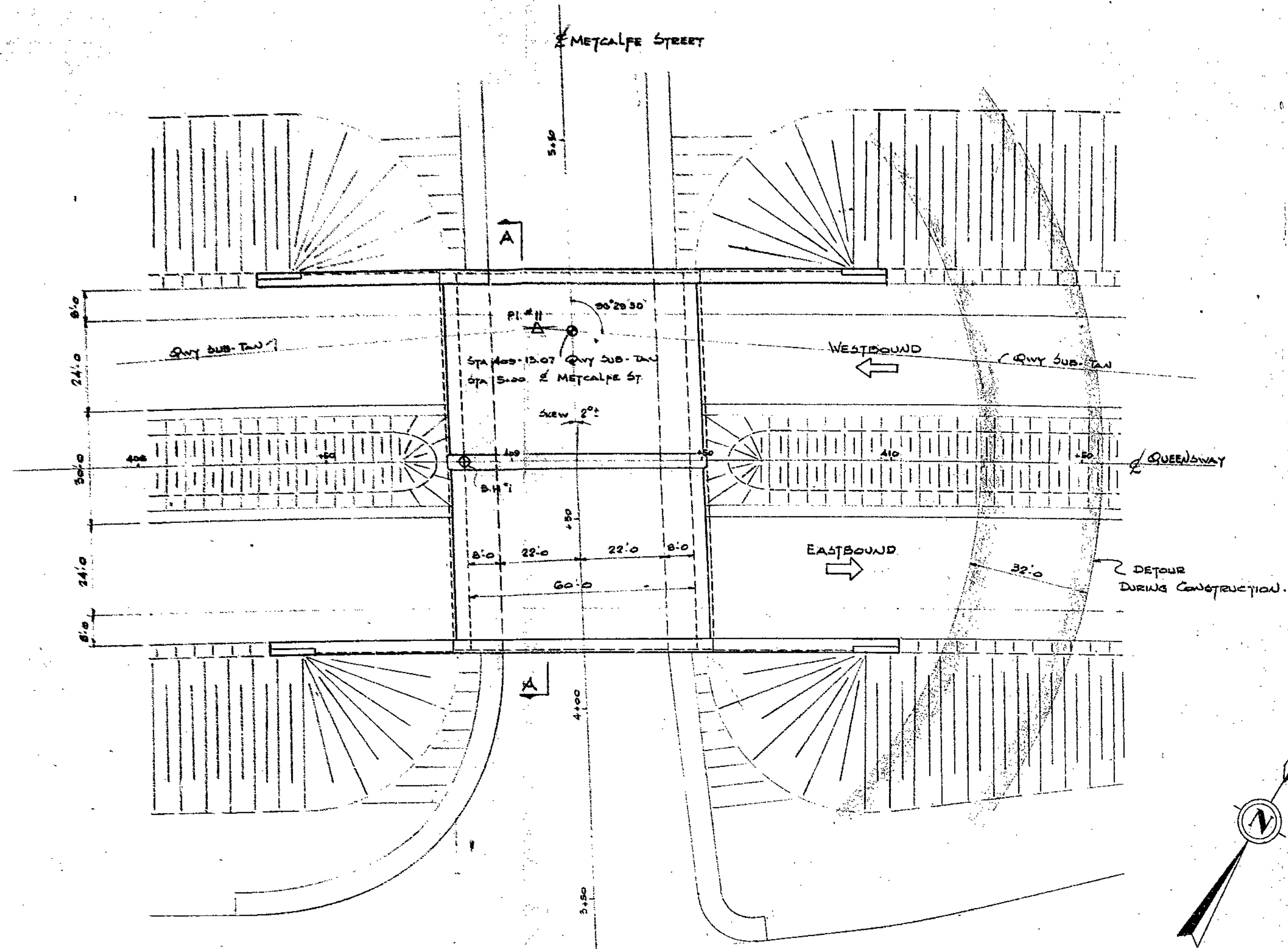
PLATE No. 10



VOID RATIO
1·8
1·7
1·6
1·5
1·4
1·3
1·2
1·1
1·0
0·1
1·0
10·0
PRESSURE - TONS/FT²
PRESSURE VOID RATIO CURVE
METCALFE & QUEENSWAY E-417
SAMPLE No. 1-9
PN (min.) = 2·01 T.S.F.
PN (prob.) = 2·58 T.S.F.
PN (max.) = 2·68 T.S.F.
INITIAL e = 1·754
Cc = 1·758 - ·720 = 1·038
TESTED :- D.M. & A.G.
CALCULATED :- D.M.
CHECKED :- A.G.
McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA

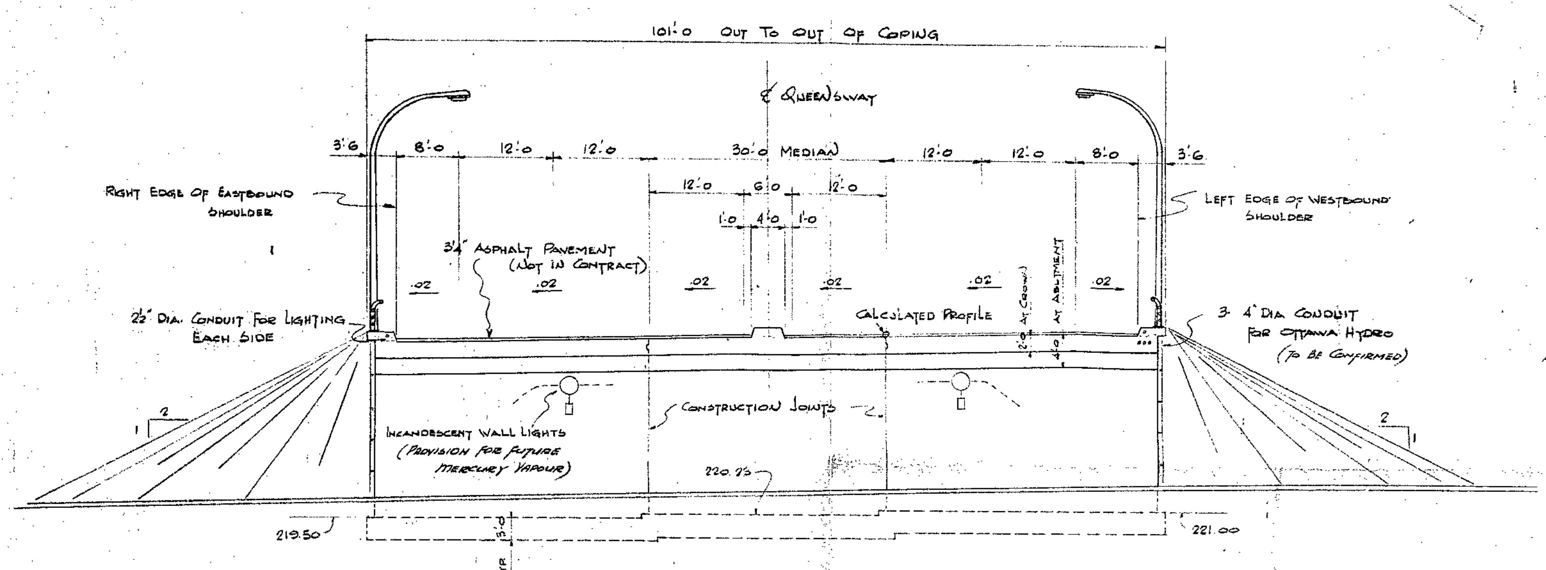
PLATE No. 11

#60-F-253C
W.P. #950-59-1
METCALFE ST. &
THE QUEENSWAY
(OTTAWA)



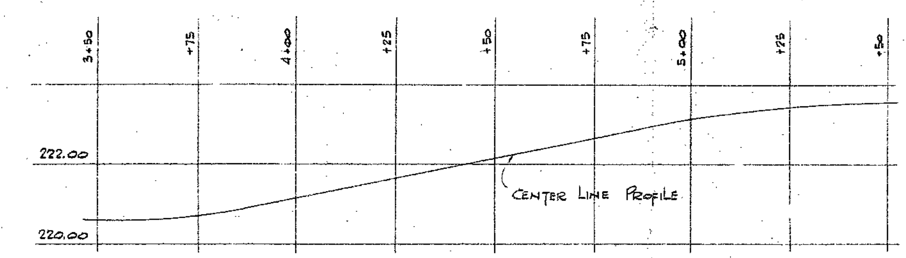
Bridge Plan

Scale: 1" = 20'-0

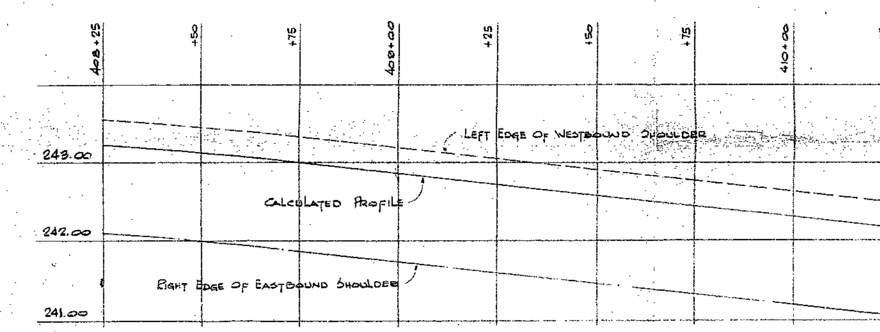


Section A-A'

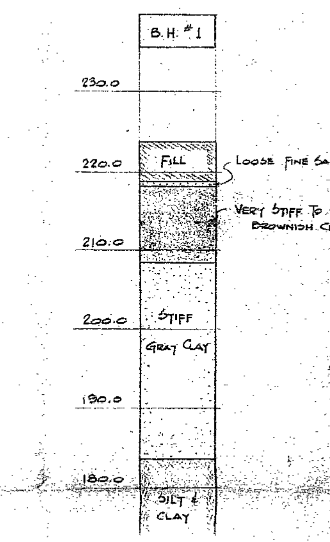
Scale: 1" = 10'-0



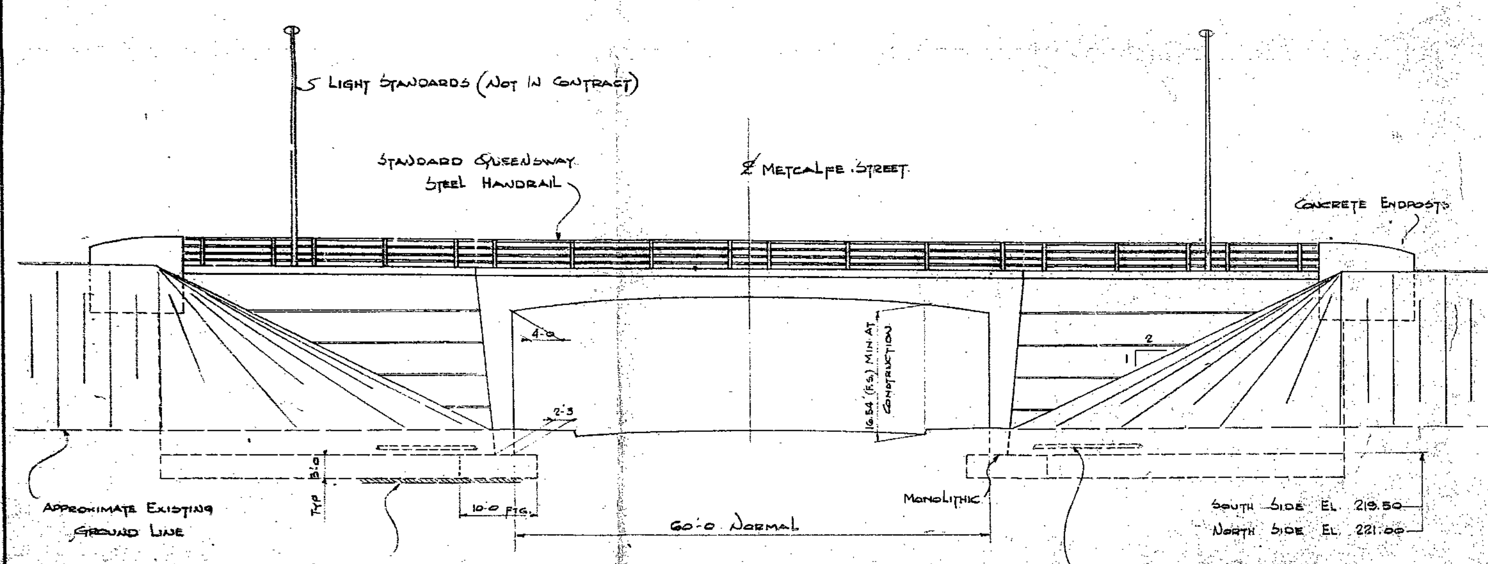
Metcalfe Street Profile — Scale { Vert'l: 1" = 2'-0 ; Horiz'l: 1" = 20'-0 }



Queensway Profile — Scale { Vert'l: 1" = 1'-0 ; Horiz'l: 1" = 20'-0 }



Borehole Log



Elevation - South Side

(North Side Similar)

Scale: 1" = 10'-0

Notes:

Design Specifications: A.A.S.H.O. Specifications For Highway Bridges - 1957.

Live Load: H20-S16-44

Concrete Strength: 3000 P.S.I. Throughout

Refer To BA 1154 For Preliminary Soils Report.

Foundations: Spread Footings On Clay Crust. (Net Allow' Brg Capacity 4000 P.S.F.)

Superstructure: R.C. Rigid Frame

Construction Of This Bridge Is Included In The O'Connor St To Rideau Canal Grading Contract.

Settlement Gauge To Be Installed On West Abutment Before Backfilling Commences.

DISTRICT Nº 9
W.P. Nº 950-59

No	Revisions	By	Date

DEPARTMENT OF HIGHWAYS OF ONTARIO

OTTAWA QUEENSWAY
LIMITED-ACCESS HIGHWAY
OTTAWA CANADA

BRIDGE Nº 37 AT METCALFE ST.
PRELIMINARY PLAN

DE LEUW CATHER & CO. OF CANADA LIMITED
Consulting Engineers

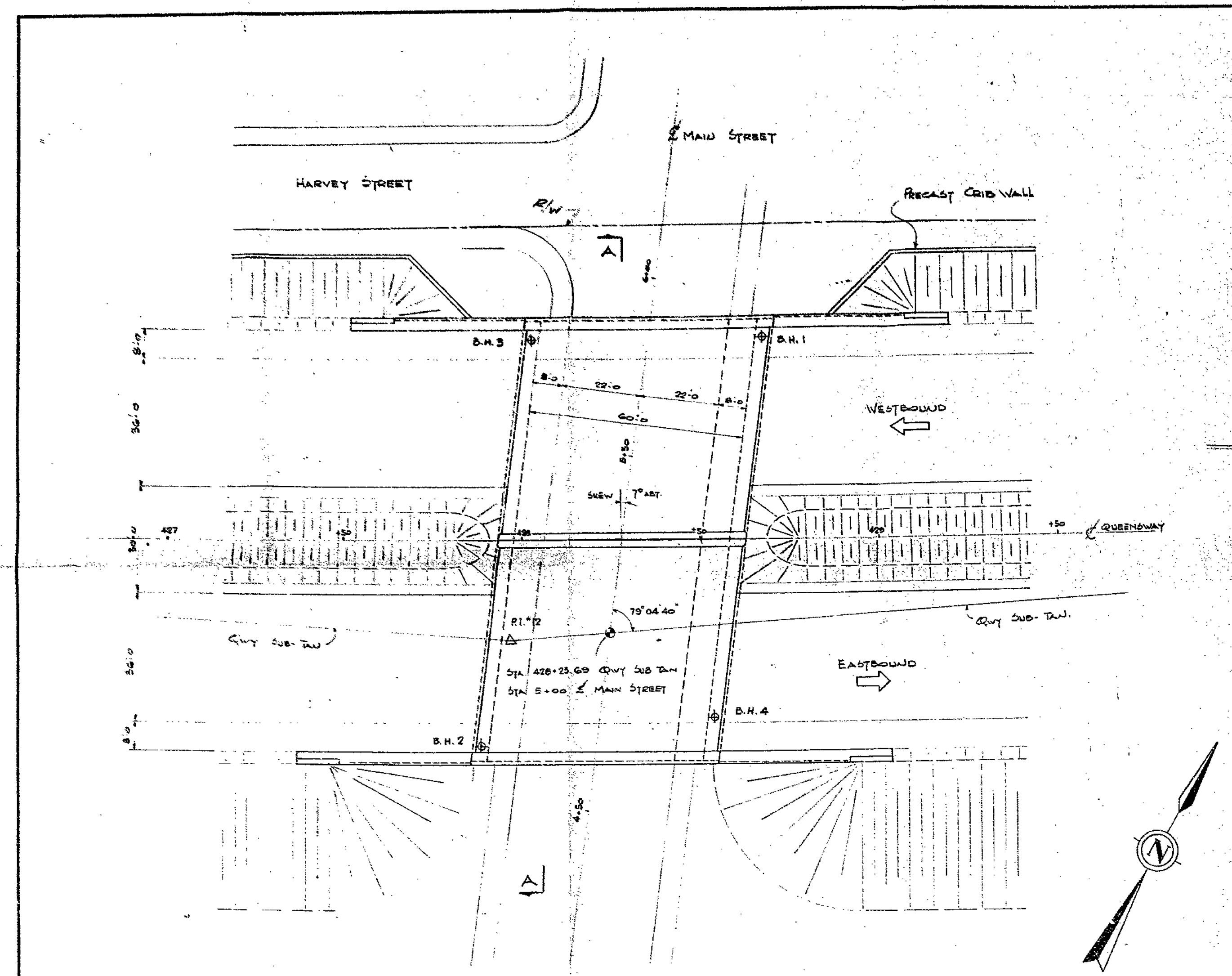
DEPT. OF HIGHWAYS OF ONTARIO

Leon J. Marshall
Director of Planning & Design

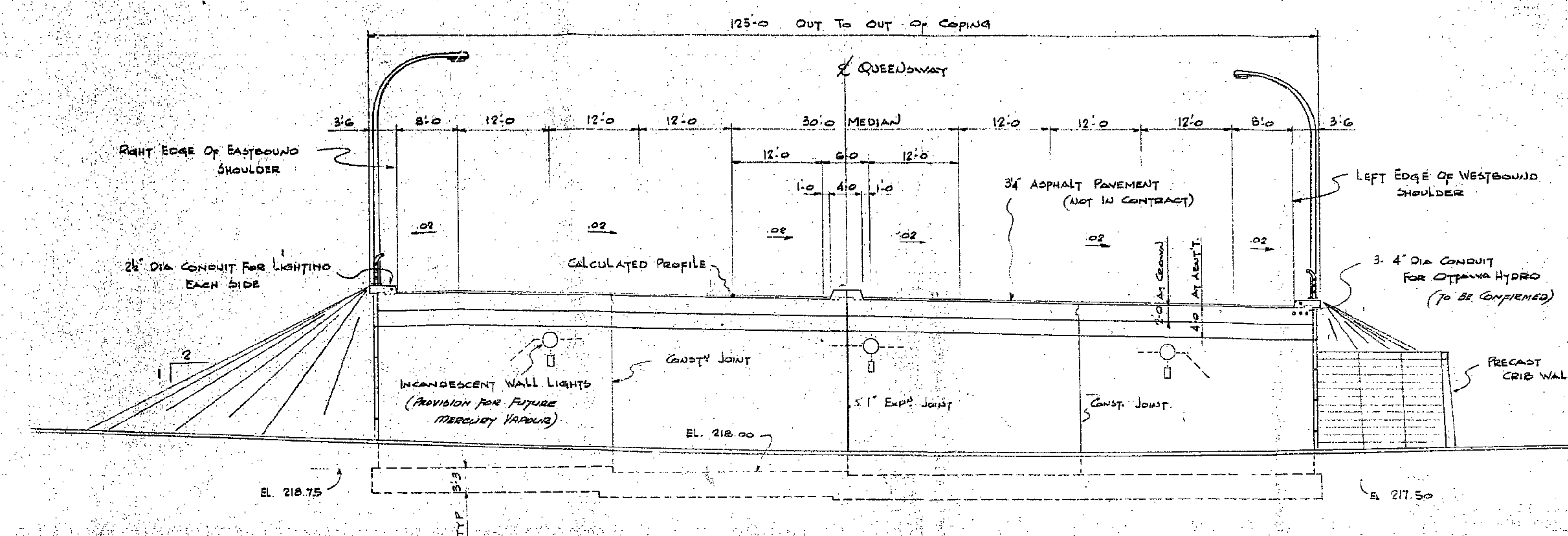
Designed by: G.S.S.	Date: APR. 27, 62	DWG No. D5067-P1
Drawn by: P.T.		
Checked by: G.G.S.	Scale: AS SHOWN	Sheet: 1 of 1

C45F-P1

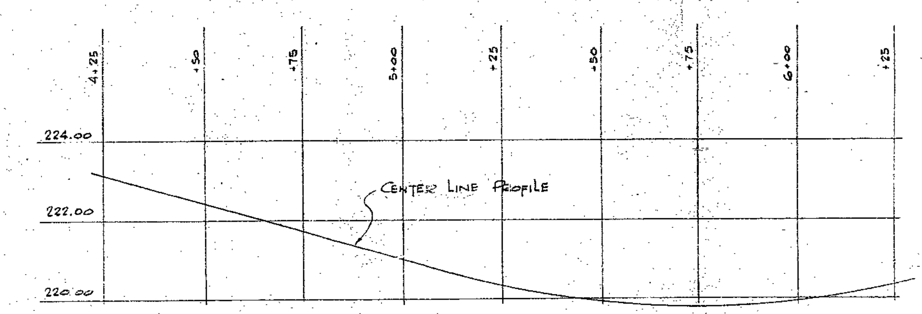
PRINT RECORD		
NO.	FOR	DATE



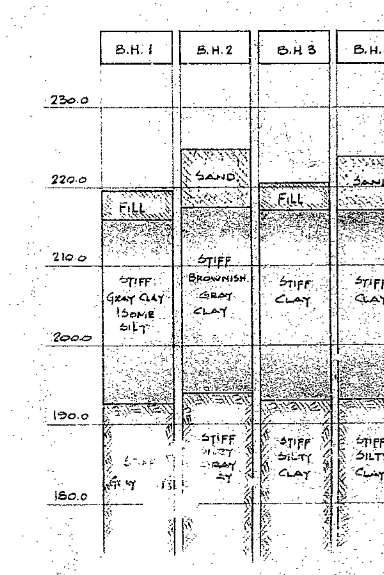
BRIDGE PLAN
SCALE: 1" = 20'-0



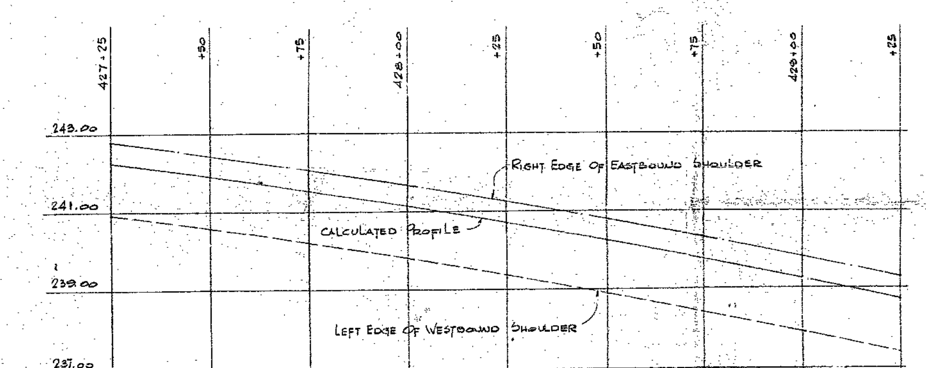
SECTION A-A
SCALE: 1" = 10'-0



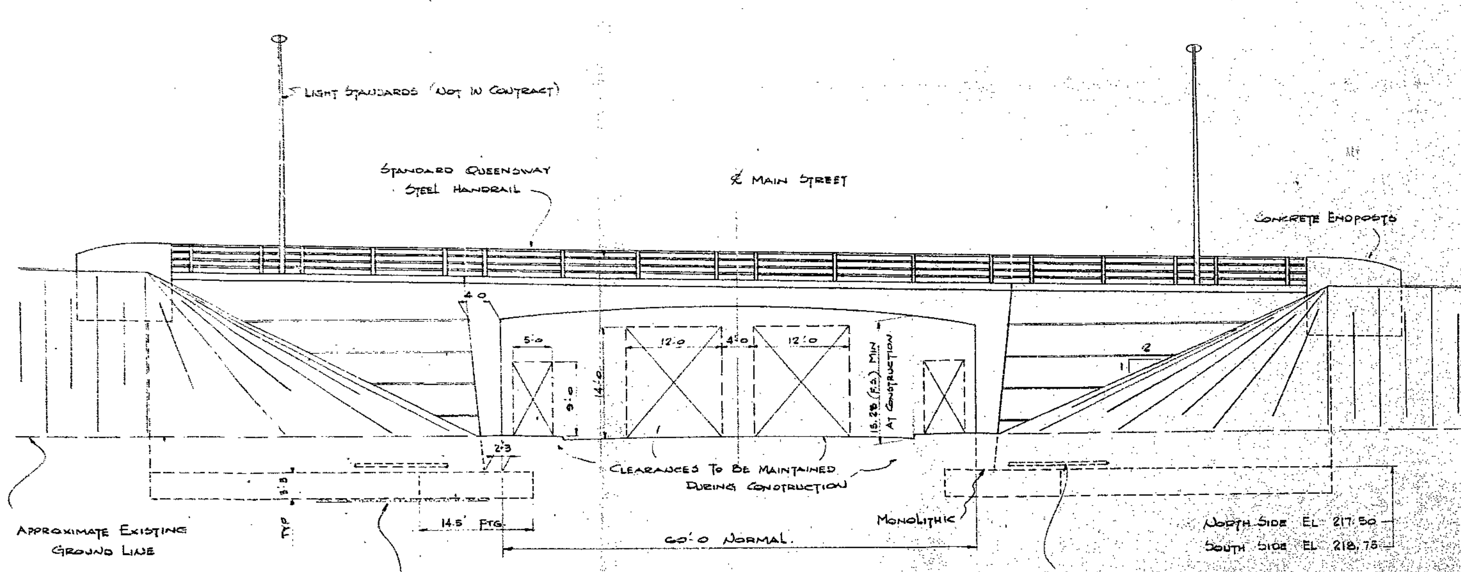
MAIN STREET PROFILE SCALE { VERT'L: 1" = 2'-0, HORIZ'L: 1" = 20'-0 }



BOREHOLE LOG



QUEENSWAY PROFILE SCALE { VERT'L: 1" = 2'-0, HORIZ'L: 1" = 20'-0 }



ELEVATION - SOUTH SIDE
(NORTH SIDE SIMILAR EXCEPT FOR CRIB WALLS)
SCALE: 1" = 10'-0

NOTES:

DESIGN SPECIFICATIONS: A.A.S.H.O. SPECIFICATIONS FOR HIGHWAY BRIDGES. 1957

LIVE LOAD: H20-S16-44

CONCRETE STRENGTH: 3000 P.S.I. THROUGHOUT

FOUNDATIONS: SPREAD FOOTINGS ON CLAY. (NET ALLOW BRG. CAPACITY 3000 P.S.F. - TO BE CONFIRMED IN SOILS REPORT BEING PREPARED BY H.Q. GOLDER & ASSOC.

SUPERSTRUCTURE: R.C. RIGID FRAME.

PIEZOMETERS AND SETTLEMENT GUAGE TO BE INSTALLED NEAR WEST ABUTMENT BEFORE BACKFILLING COMMENCES.

CONSTRUCTION OF THIS BRIDGE IS INCLUDED IN THE RIDEAU CANAL TO CONCORD AVE. GRADING CONTRACT.

DISTRICT Nº 9
W.P. Nº 953-59

MAY - 4 1962

No	Revisions	By	Date

DEPARTMENT OF HIGHWAYS OF ONTARIO

OTTAWA QUEENSWAY
LIMITED-ACCESS HIGHWAY
OTTAWA CANADA

BRIDGE Nº 25 AT MAIN ST.
PRELIMINARY PLAN

DE LEUW CATHER & CO. OF CANADA LIMITED
Consulting Engineers
Leon Marshall

DEPT. OF HIGHWAYS OF ONTARIO
Director of Planning & Design

Designed by: G.S.S.	Date: APR. 27, 62	DWG. No. P5070-P1
Drawn by: P.T.	Scale: AS SHOWN	Sheet 1 of 1
Checked by: G.S.S.		

C45E-P1