

#58-F-229-C

MERIVALE RD.

É OTTAWA,

QUEENSWAY

BRIDGE No. # 36

MCROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA 1

CANADA

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BA 831
58-F-229C

FOUNDATION REPORT - MERIVALE ROAD

1. TERMS OF REFERENCE

We were requested by the Ottawa office of DeLeuw, Cather & Co. of Canada Ltd. to investigate the foundation conditions at the site of a proposed bridge known as No.36 and to prepare a report. Proposed foundation elevations, road grades, and fill heights were discussed during the progress of the work.

2. RECOMMENDATIONS

2.1 Bearing Capacities

At about elevation 232 which is at the proposed bottom of foundation, bearing capacities are low. A value of 1,300 pounds per square foot would have to be used if the weakest areas are considered. Construction protection would be necessary as discussed later.

2.2 Soil Compressibility

The medium soft clay layers between elevation 232 and elevation 214 have a net preconsolidation of the same order as the possible bearing capacities. Consolidation settlements due to footings would therefore not be significant but the consolidation of these same clay layers due to the weight of the approach fills up to a height of elevation 257 would be significant. In areas immediately adjacent to the abutment where the footing loads would combine with the fill loads, the maximum amount of consolidation settlement would occur.

Settlement calculations were made using the results of consolidation tests performed on typical samples, and the estimated settlement of the center of the approach fill is 0.6 feet. The combined settlement of approach fill together with the footings would be estimated at about 1.3 feet.

The timing of settlements is difficult to estimate due to unknowns in the degree of continuity of pervious layers observed in the clays, but the settlements could be expected to occur within the first year after loading.

The desirability of scheduling construction of the approach fills with a preload period can be considered. A preload period of several months would considerably reduce the expected differential final position between a pile supported structure and its approach fill. However, preliminary data for the preloaded fill at Highway 17 and Montreal Road East of Ottawa indicate that settlement estimates based on laboratory consolidation tests have been greater than the settlements actually observed. A further study of the results still being obtained at the Montreal Road site would be warranted before incurring special costs of rescheduling work to provide a preload period.

2.3 Foundation Type

The remarks in our previous report covering the Carling Avenue West bound site apply equally well at this location and are repeated here along with other applicable sections.

If spread footings are considered for support of the structure, the effect of the consolidation settlements of footings and embankment must be considered. In view of these settlements and the differential amounts associated with them, is unlikely that any structure sensitive to movement would be feasible.

If pile supported foundations are considered, allowance should be made for the probability that piles will penetrate a few feet into the weathered and interbedded rock and payment clauses should be made sufficiently flexible to cover variations which may be encountered.

2.4 Pile Loads

In addition to the design loads on piles, it should be recognized that the inescapable consolidation of the clay layers due to the weight of the adjacent approach embankment will produce a load on the piles by negative friction. The load will, of course, be distributed across the depth of the clay layer and will develop over the same period as that estimated for consolidation settlements. The estimated maximum value of the load due to negative friction is approximately 25 tons for a 14 inch diameter pile. Other maximum values can be estimated by multiplying the pile surface area in compressible clay by the shear strength of the clay itself, but considerable judgement must be exercised in evaluating the modifying factors of pile spacing, strength regain of the clay, and disturbance due to driving.

2.5 Construction Precautions

If the clay soils at elevation 232 are used for support of all or part of the structure, these soils are sensitive and must be protected from disturbance during construction. A protective layer of lean concrete or crushed stone placed immediately after excavation would serve the purpose.

3. SITE INVESTIGATION

3.1 Field Work

Six boreholes were made at the site using the staff and equipment of J.B. Dufresne Ltd., drilling contractors. A member of our technical staff was continuously with the drill rig to keep adequate records and to control the sampling operations.

Two inch thin wall tube samples were recovered in the cohesive soil layers and two inch split barrel samples in the non-cohesive layers in conjunction with the standard penetration test. Samples were visually classified in the field and later given laboratory confirmation. The underlying

rock was diamond drilled and 1 1/8" diameter cores recovered for inspection. A careful watch was kept for drops or discontinuities in the drilling and groundwater levels were also recorded during the field programme.

3.2 Laboratory Testing

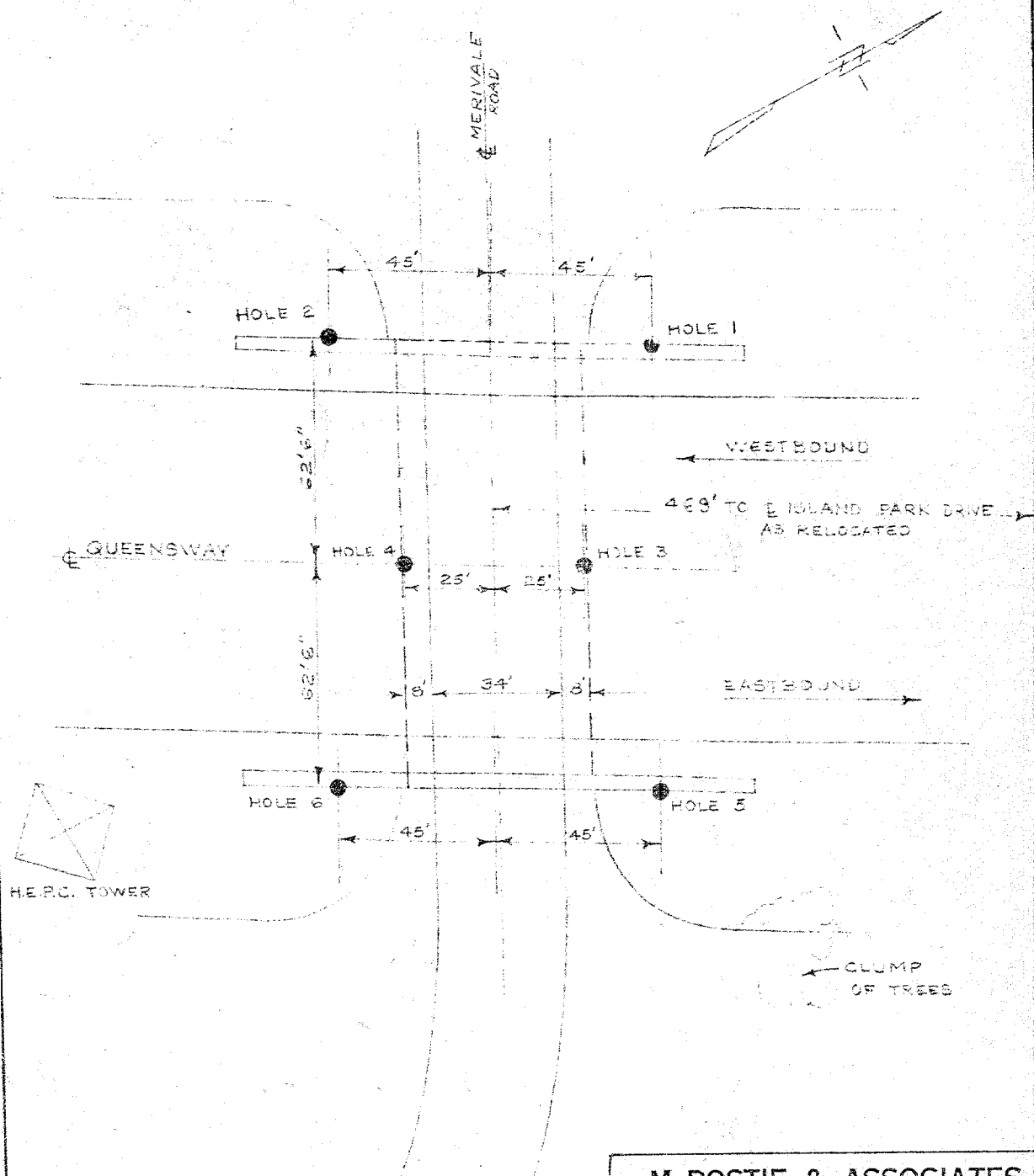
The tube samples were extruded and tested in our laboratory. Moisture content, unconfined compression test, and small penetrometer tests were made on all samples to investigate strengths of the soil and variations within the deposit. Representative samples were given mechanical analysis to confirm soils types and to aid estimating the construction behaviour of the soils. Two samples from the typical compressible layer were given consolidation tests to provide data for the computation of expected settlements.

3.3 Observations

A few feet of fill was found in some of the boreholes as part of the former railway and recent shopping centre construction. The natural soils consist firstly of 25 to 30 feet of clay, the upper ten feet^{off} which is stiff and fissured, the remainder is softer with silt included and sand or silt interbedding. Beneath the clay a thin layer of till was found in some holes and beneath this was shaly limestone rock of the Ottawa formation. Several drops were noticed during the drilling in the rock and this coupled with some low core recoveries indicates that the upper rock surface is weathered and that seams exist in the section of the rocks that were investigated.

4 COORDINATION

We would be pleased to provide any additional information that is available concerning the site or to discuss any points which may arise from this report.



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BOREHOLE LOCATIONS
QUEENSWAY & MERIVALE RD.

SCALE 1" = 40'

PLATE 1

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SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QWY, MERIVALE RD.

 ELEVATION OF GROUND SURFACE (ZERO DEPTH) 240.2
 REMARKS See: plate #2

HOLE NO.

2

DATE 30/10/54

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	PENETRATION TEST	
							LB. HAMMER INCH DROP	NO CASING INCH DIA. ROD BLOWS PER FOOT
				GROUND SURFACE				
				TOP SOIL - 10'	00	240.2	OVERNIGHT WATER LEVEL	
				VERY STIFF				0.0
4.5	6.6-7.2 73-R-48		2-1	BROWNISH GRAY CLAY				1
				STIFF BROWNISH				
3.1	5.2-4.0 72-R-18		2-2	GRAY CLAY				1
				MEDIUM SOFT				
1.1	2.8-1.6 1.7-R-03		2-3	BROWNISH GRAY CLAY				1
				MEDIUM SOFT	100	230.2		1
4.2	1.4-1.1 1.6-R-0		2-4	FISSURED GRAY				1
				SILTY CLAY				1
1.5	1.3-1.6 0.5-1.0 1.4-R-0		2-5	MEDIUM SOFT				1
1.6	1.0-0.6 1.1-1.2 1.0-R-0		2-6	GRAY CLAY				1
1.6	1.0-0.9 0.6-1.3 1.4-R-0		2-7	WITH SANDY	20.0	220.2		1
1.4	0.8-1 1.1-1.3 2.0		2-8	CLAY LAYERS				1
				AND SILTY				
				CLAY LAYERS	25.0			
				LOOSE TILL	26.5			
			2-9	MEDIUM DENSE TILL	27.5			
				SHALEY LIMESTONE	30.0	210.2		
				(CORE RECOVERY 52%)				
				BOTTOM OF HOLE	32.1	-207.1		

REMOVED - R

5

4 ft 6"

 % WATER CONTENT
 WATER CONTENT

PLATE

3

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SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QWY / MERIVALE RD.

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 244.2'

REMARKS See plate #2

HOLE NO.

3

DATE Nov 5/59

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	PENETRATION TEST	
							LB. HAMMER INCH DROP	NO CASING INCH DIA. ROD
							BLOWS PER FOOT	
				GROUND SURFACE	0.0	244.2		
			5-1	FILL (BROWNISH GRAY CLAY WITH SAND)	50'			
			27 5-2	HR 10 BROWNISH GRAY CLAY	15'			
1.7	67-67 55-116		3-3	VERY STIFF fissured BROWNISH GRAY CLAY WITH SOME SILT	20.0	234.2		
1.8	52-42 38-17 R-0.4		3-4	VERY STIFF BROWNISH GRAY CLAY WITH SOME SILT	12.5			
1.1	18-16 15-17 R-0.0		3-5	MEDIUM SOFT fissured BROWNISH GRAY SILTY CLAY				
1.3	18-13 13-2.0 10-20.0		3-6	SAND & SILT - 15.8 MEDIUM SOFT fissured GRAY SILTY CLAY - 12.5				
0.9	16-13 16-15 15-20.0		3-7	MEDIUM SOFT GRAY	20.0	224.2		
1.3	0.3-1.3 0.5-1.4 1.3-1.6 R-0.0		3-8	SILTY CLAY				
0.5	0.9-1.4 1.2-1.4 R-0.0		3-9	MEDIUM SOFT fissured SAND & SILT - 12.5 TILL - 12.5 SHALEY LIMESTONE - 28.0	37.2	207.2		
				SHALEY LIMESTONE (CORE RECOVERY 88%)	53.0	204.2		
				BOTTOM OF HOLE	94.9	209.3		

0 20 40 60 80 100
 % WATER CONTENT
 @ MOISTURE CONTENT

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SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

QWY / MERIVALE RD.

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 240.9 DATE NOV. 5 1988REMARKS See: plate #2 FOR MECHANICAL ANALYSIS SAMPLES S-4, S-7

HOLE NO.

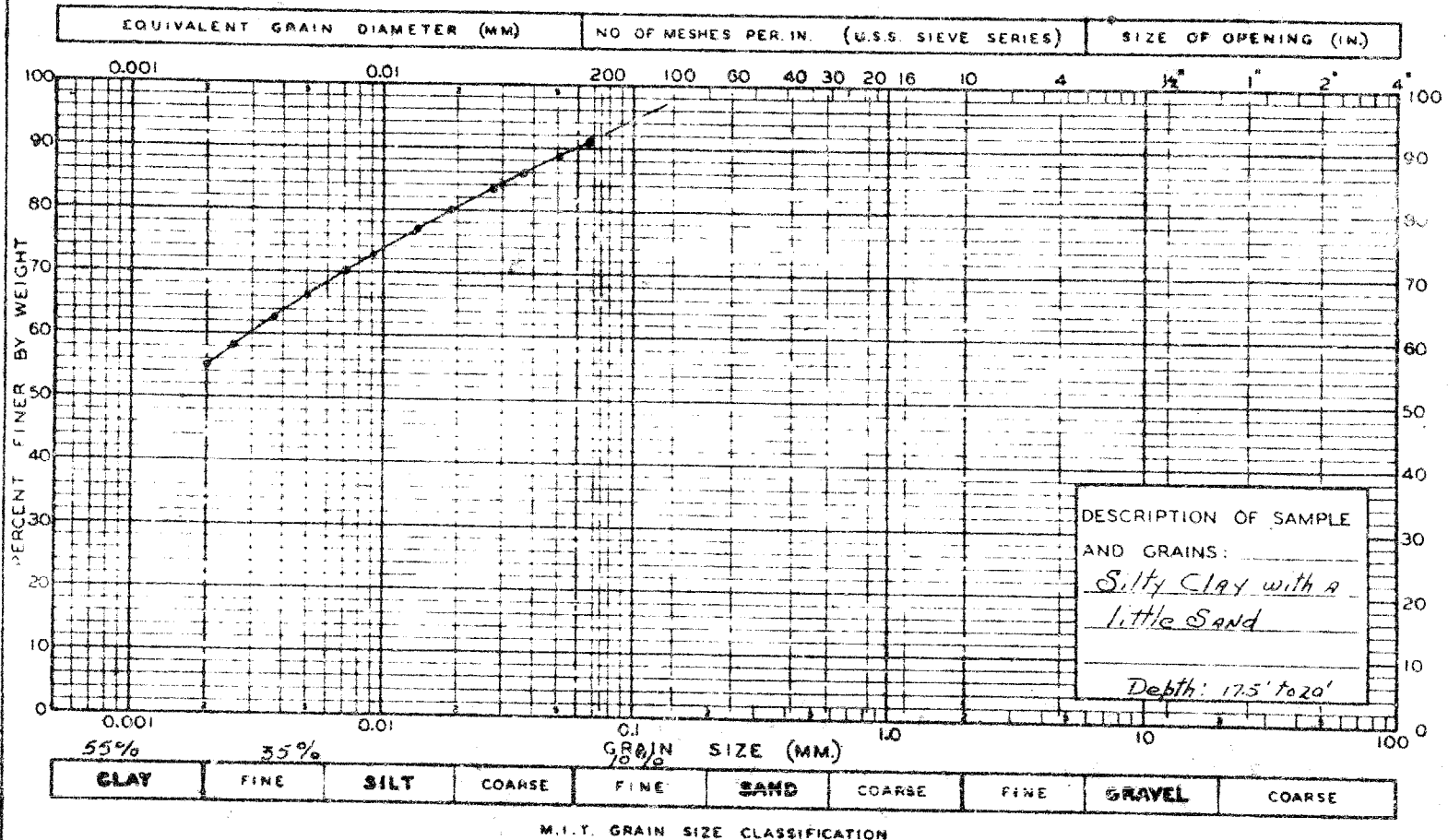
5

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 INCH DROP	NO CASING INCH DIA. ROD SHEAR STRENGTH IN KIPS PER FT. ²
				GROUND SURFACE				
				FILL	0	240.9		
								OVERNIGHT WATER LEVEL 20'
2.7	7.5-7.1 5.2-5.22		26 5-1	HARD BROWNISH GRAY CLAY				
			5-2	VERY STIFF BROWNISH GRAY CLAY WITH SOME SILT				
1.4	2.8-4.0 2.5-5.01		5-3	STIFF BROWNISH GRAY CLAY WITH SOME SILT				
1.2	1.5-2.1 1.3-5.00		5-4	MEDIUM SOFT BROWNISH GRAY SILTY CLAY WITH SOME SAND & SAND DOCKETS & SAND LAYERS	10'	230.9		
1.4	0.7-1.1 1.1-1.8 1.1-2.00		5-5	MEDIUM SOFT				
1.4	1.1-0.9 1.1-1.5 R-0.0		5-6					
1.2	1.3-1.0 1.3-1.4 R-0.0		5-7	GRAY SILTY CLAY				
0.5	1.4-0.8 0.7-0.6 R-0.0		5-8	SOFT FISSURED GRAY SILTY CLAY AND SILT IN LAYERS	20'	220.9		
				SHALEY LIMESTONE DROP (CORE RECOVERY 93%)				
				BOTTOM OF HOLE	27.5	213.4		

0 20 40 60 80 100
 % WATER CONTENT
 NATURAL ☐
 LIQUID LIMIT ☐
 PLASTIC LIMIT ☐

PLATE
6

MECHANICAL ANALYSIS OF SOILS



PROJECT: *Qwy + Merivale Rd*

SAMPLE NO. *9-7*

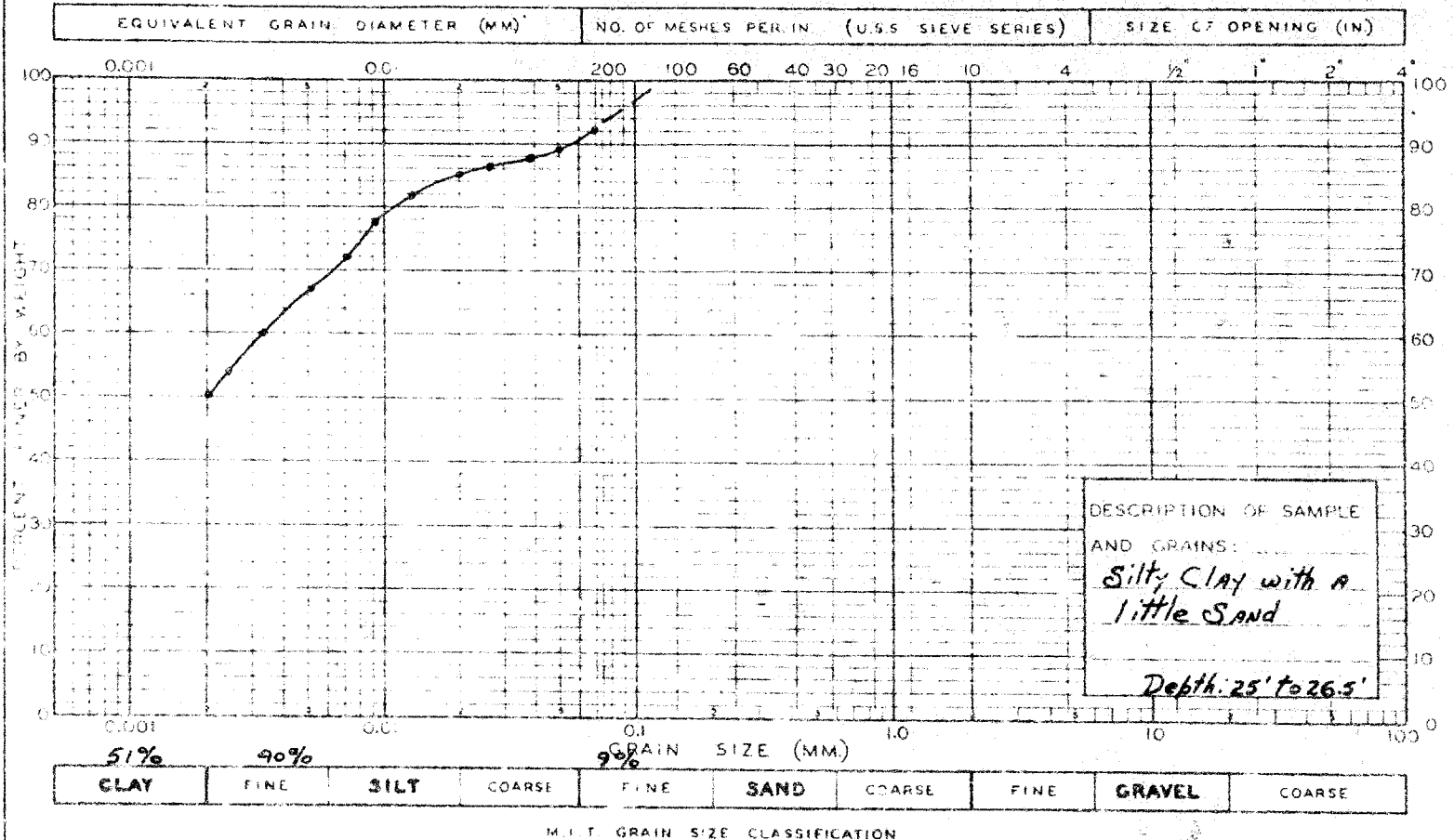
PLOTTED: *D.M.* DATE: *3-11-58*

REMARKS: *0.02 to 0.105 size = 41%*

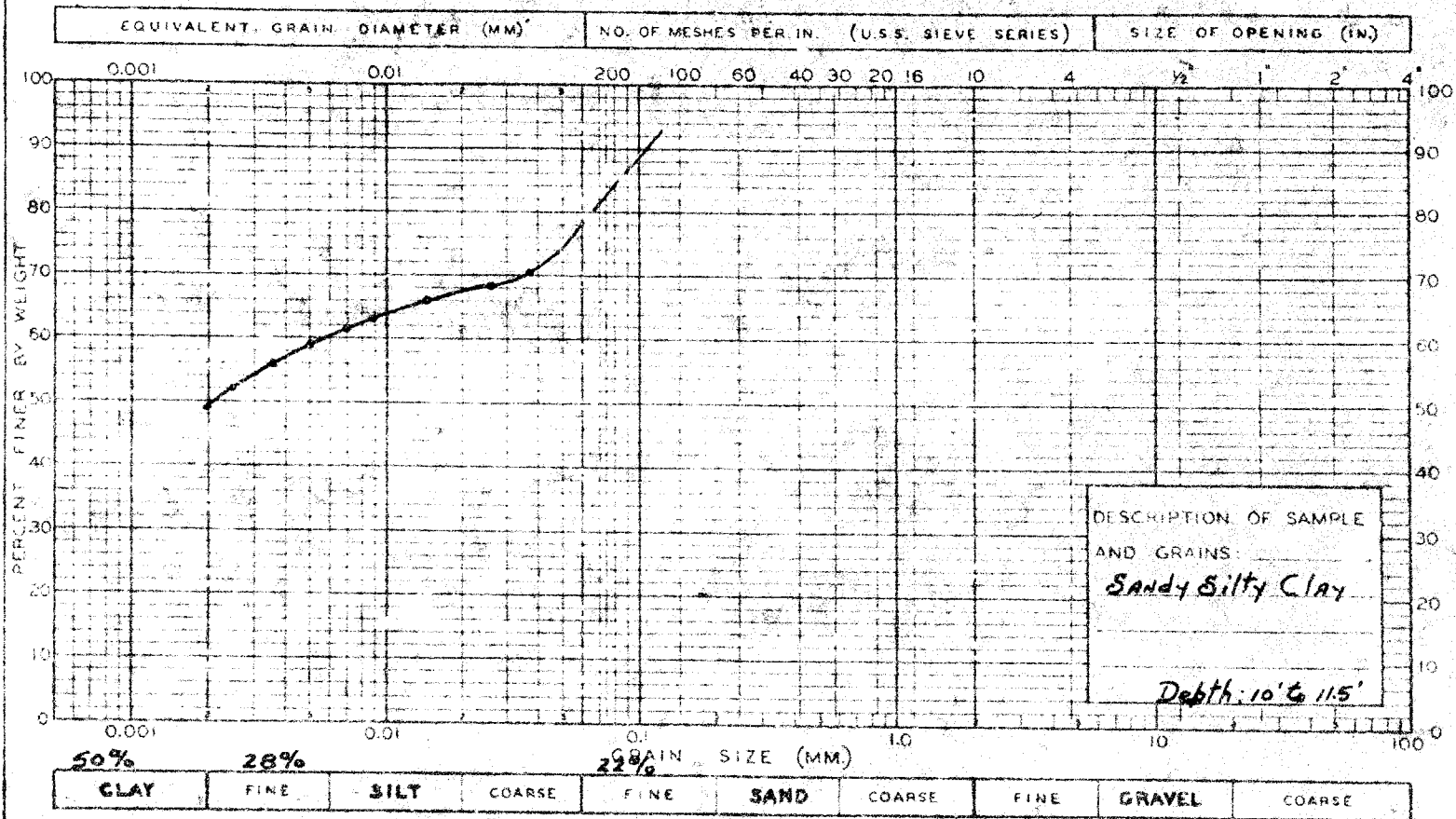
CHECKED: *C.J.* DATE: *5-11-58*

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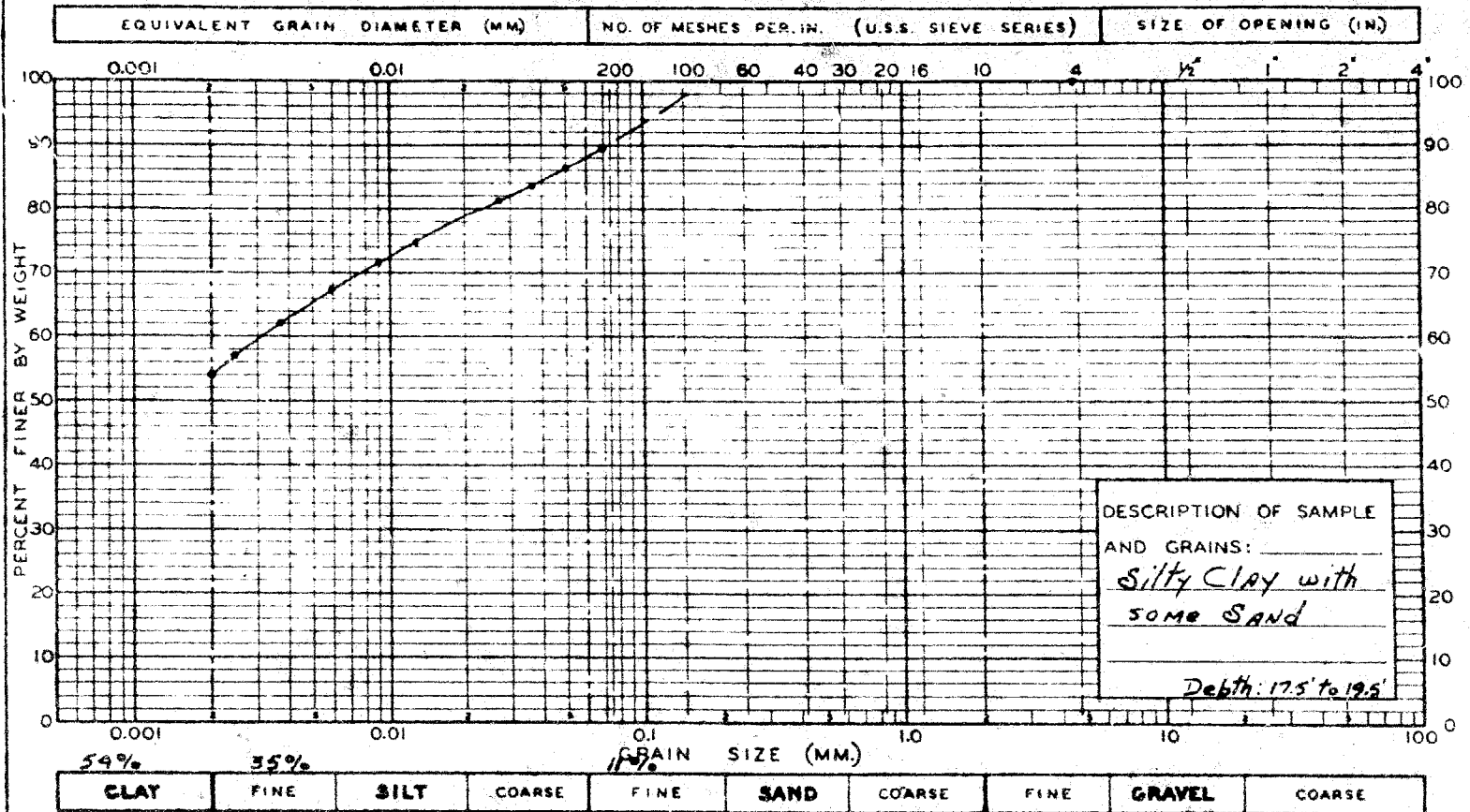
MECHANICAL ANALYSIS OF SOILS



MECHANICAL ANALYSIS OF SOILS



MECHANICAL ANALYSIS OF SOILS



PROJECT: *Qwy f Merivale Rd*

SAMPLE NO. *5-7*

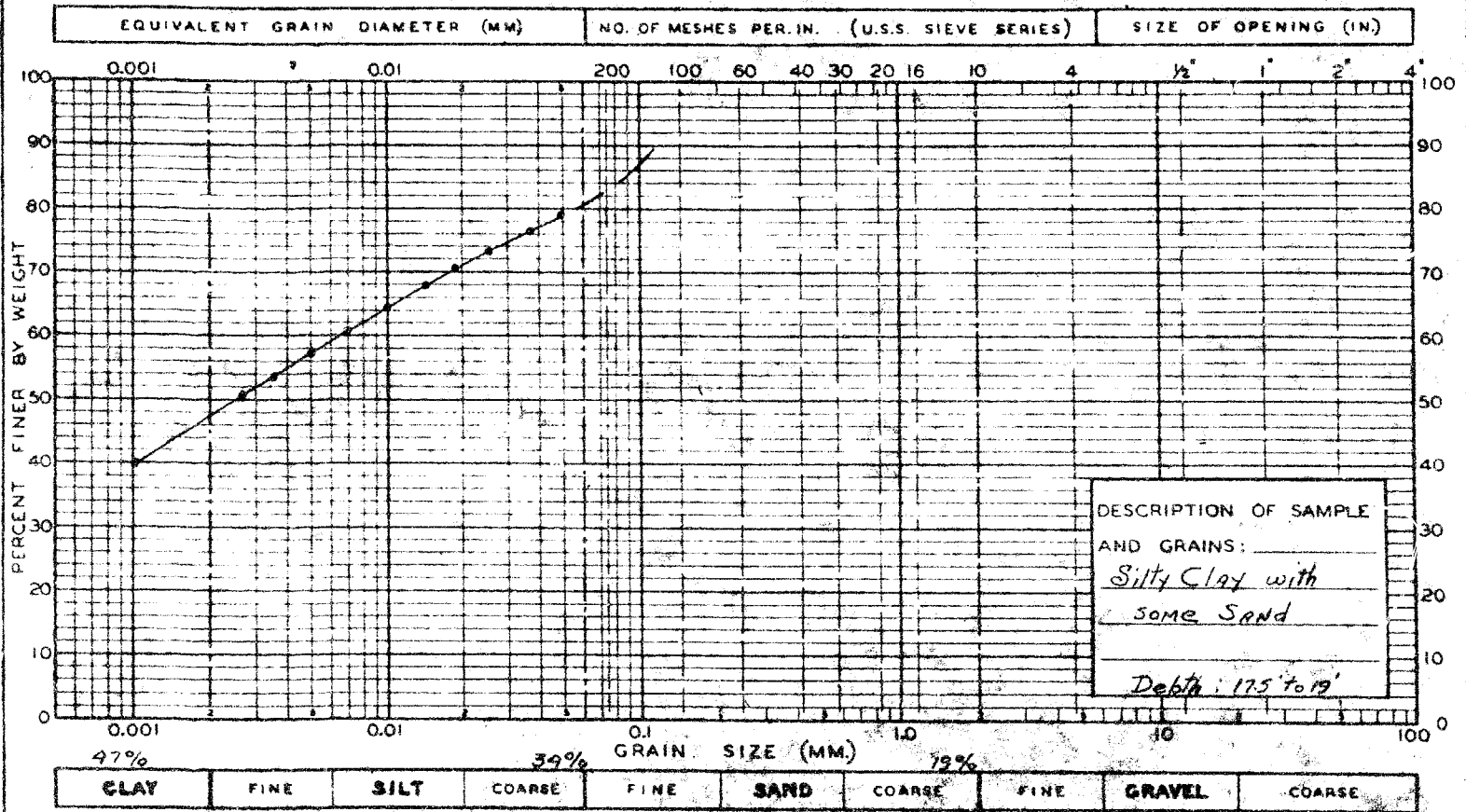
PLOTTED: *D.M.* DATE: *3-11-58*

REMARKS: *.002 to .105 size = 92%*

CHECKED: *C.J.* DATE: *5-11-58*

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MECHANICAL ANALYSIS OF SOILS



DESCRIPTION OF SAMPLE AND GRAINS:

Silty Clay with some Sand

Depth: 175 to 19'

M.I.T. GRAIN SIZE CLASSIFICATION

PROJECT: Qwy f Merivale Rd

SAMPLE NO. 5-2

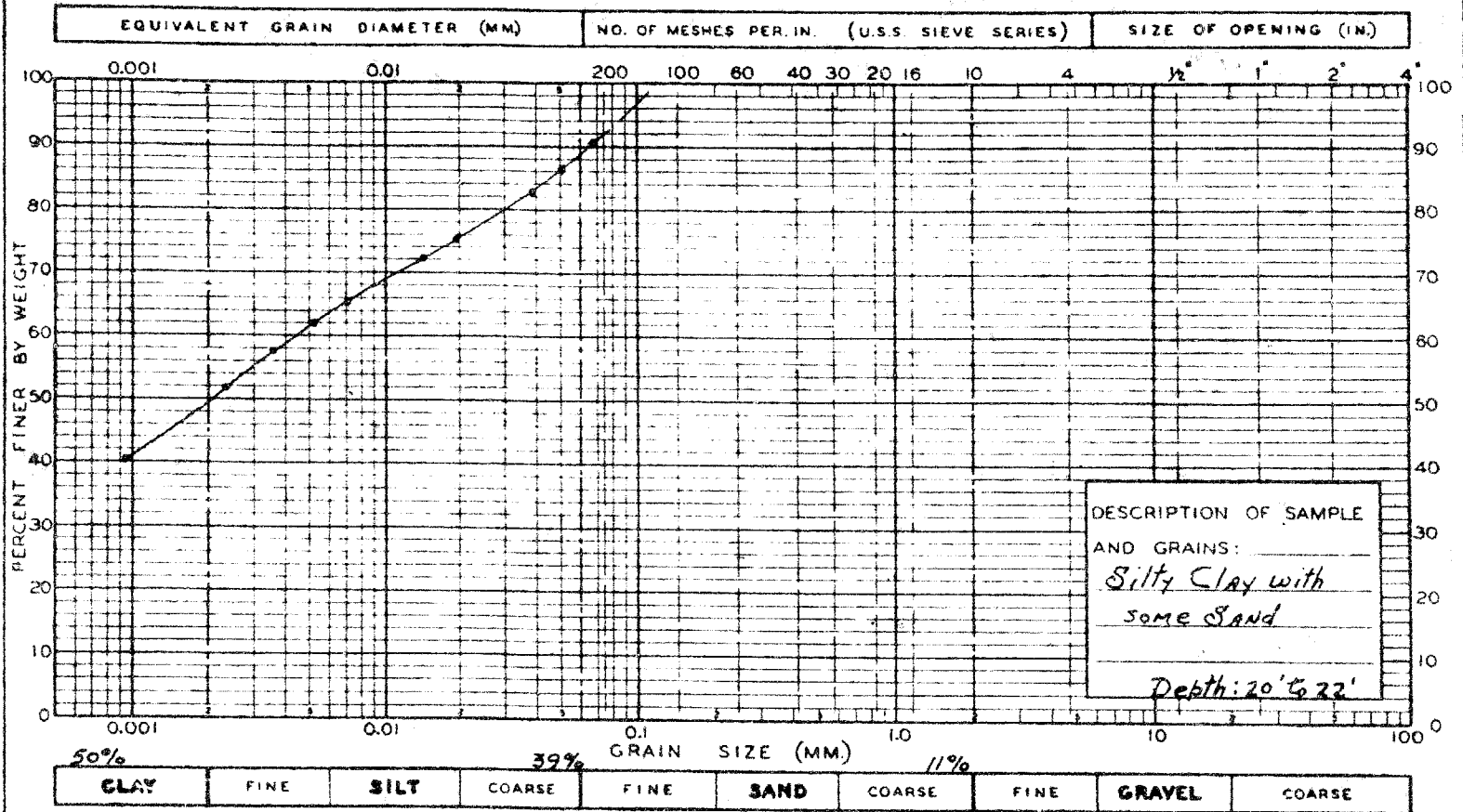
PLOTTED: D.M. DATE: 25-10-58

REMARKS: 002 to 105 Size = 97%

CHECKED: A.G. DATE: 26-10-58

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MECHANICAL ANALYSIS OF SOILS



M.I.T. GRAIN SIZE CLASSIFICATION

PROJECT: *Qwy f Merivale Rd*

SAMPLE NO. *6-8*

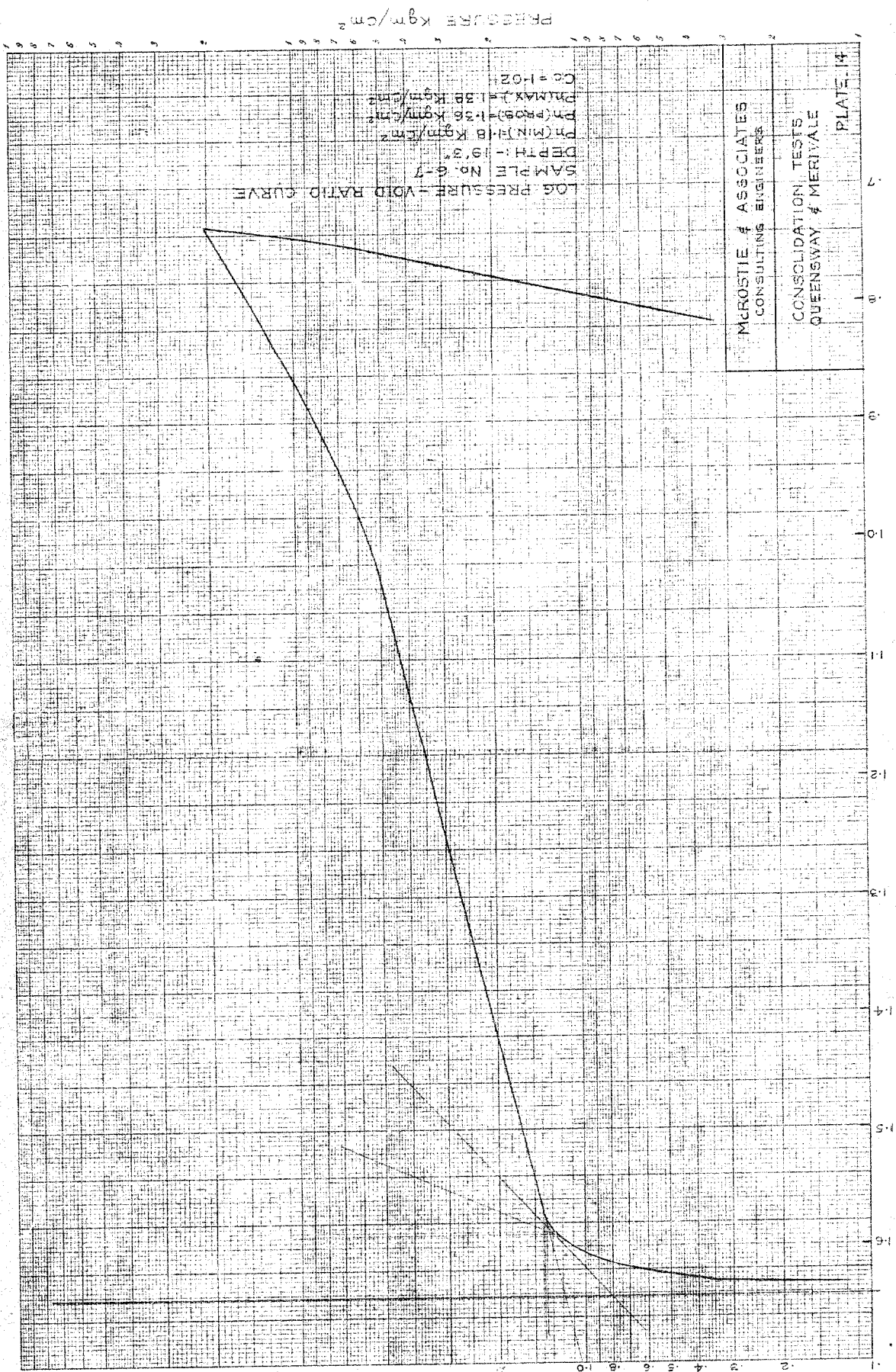
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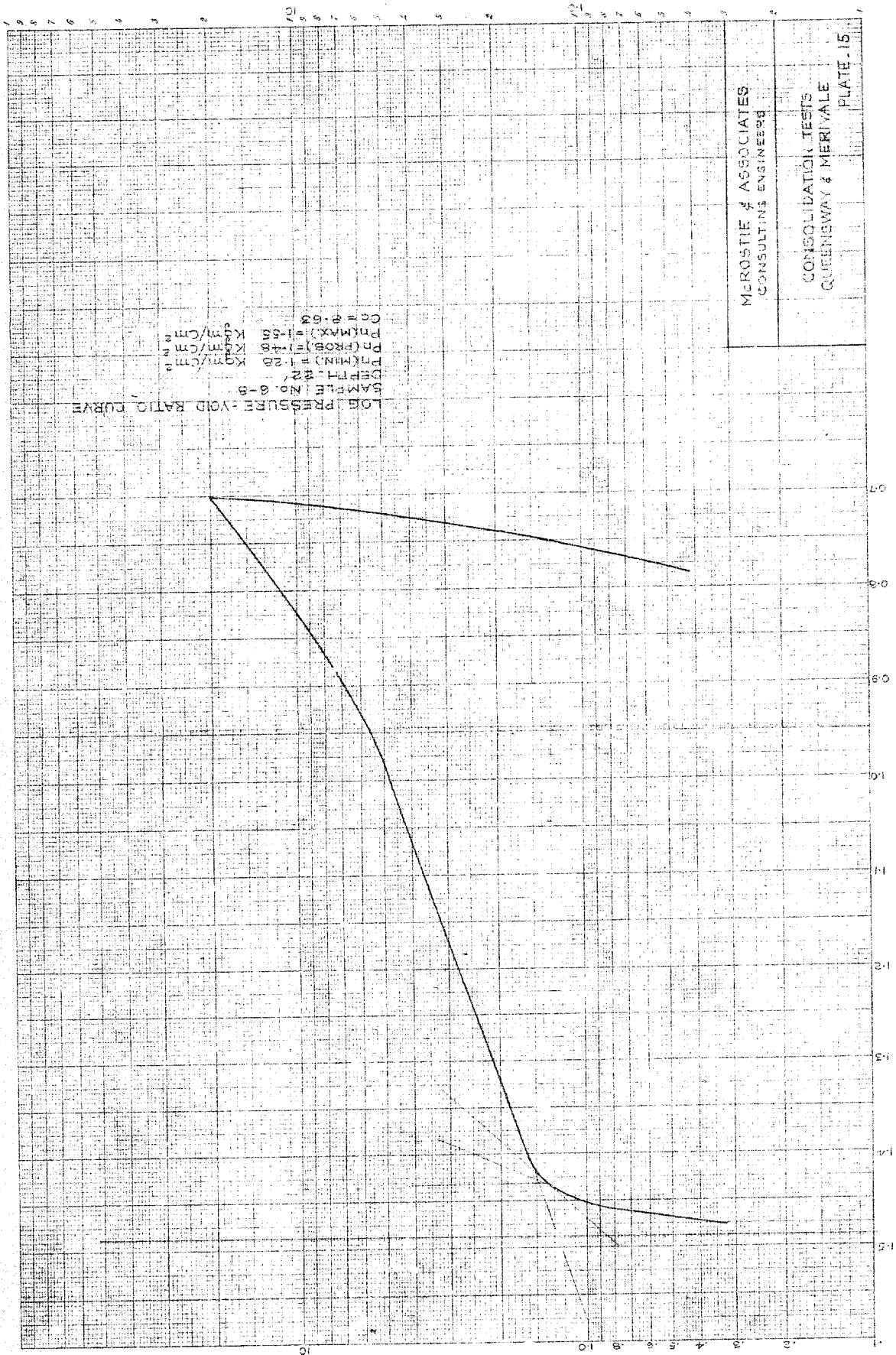
REMARKS: *002 to 105 size = 50%*

CHECKED: *C.J.* DATE: *5-11-58*

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FORM 58





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FOUNDATION REPORT NO. 2 - MERIVALE ROAD

1. TERMS OF REFERENCE

We were requested by the Ottawa Office of De Leuw Cather & Company to provide any information which had become available on the timing and magnitude of consolidation settlements of the Highway 17 at Montreal Road East embankment. This fill had been instrumented and under observation for approximately one year and the use of results from its study was mentioned in our first report for Bridge No. 36.

The consolidation settlements previously predicted were to be reviewed and the feasibility of a preload schedule considered. Three other items were introduced into the study at this time; firstly a consideration of the settlements which might occur in a large drainage conduit for Cave Creek Drain crossing beneath the approach embankments, secondly the possibility of creep or flow of the clay beneath abutment foundations due to stress from the approach fill, and finally the possibility of loss of strength of embankment foundations in the sensitive clays due to the driving of abutment foundation piles.

2. RECOMMENDATIONS

2.1 Review of Predicted Settlements.

In light of the fact that the actual settlements at the Montreal Road fill have been less than half of those predicted on the basis of conventional settlement analysis, and since similar factors can be expected to apply at the Merivale Road site,

we are now prepared to recommend that the estimated consolidation settlements given in our Report No. SF-385 can be divided by two. The majority of the settlement can now be expected to occur during the first six months rather than one year as originally estimated.

As further knowledge is gained from other projects where actual observations of the settlements of fills on local marine clays are being made, more refined methods may be developed and the conventional settlement analyses may then be modified.

2.2 Preload Schedule for Approach Fill

The estimated settlements are still of sufficient magnitude that a preload schedule for the approach fill should be considered. As previously stated, it is now expected that a preload period of six months would be sufficient to produce most of the consolidation settlements but confirmation of the decrease in settlement rate is an important part of a preload technique. Suitable settlement gauges should be installed so that the movements of the original ground surface and the movements of the upper surface of the preload fill can be observed both at the site of the future abutment and at a location at least fifty feet from the abutment which would not be affected by the abutment construction.

An important item in the planning of the preload fill is that the fill should be constructed so that it covers the future abutment area with an embankment of full height. This procedure naturally requires the removal, at the time of construction of the abutment, of a portion of the fill which had covered the abutment area. The cost of double handling this material must be considered as a cost of the preload technique but if the method outlined above is not followed, much of the benefit from the preload procedure is lost.

2.3 Settlement of Draining Conduit

The previous settlement studies indicate that the centre of the fill would settle approximately 0.6 feet. The actual settlements might be less than this figure, but since some unknowns exist regarding construction procedures, we would consider a camber of 1.0 feet to be a reasonable figure to use for the conduit. We would recommend that a flexible type conduit be used since the actual settlements of the foundation are not then critical to the structural safety of the conduit.

You will no doubt have experience in writing the specifications for the installation of such culverts. The bedding and the compaction of the adjacent fill are the two most important factors. The specification for the bedding of the pipe usually presents no problem. The specification for the compaction of the adjacent fill is more difficult. We would suggest that this be done by requiring the pipe to be strutted and the compaction of the adjacent fill to be sufficient to cause the struts to become loose.

2.4 Creep of Abutment Foundation Soils

The slow shear failure or creep of abutment soils was studied and the interaction of the abutment foundation piles with these soils. The horizontal resistance of the abutment foundation batter piles contributes to the long term stability of the embankment foundation. To achieve this purpose, we recommend that the batter of the piles shown on the preliminary drawings be increased from 2 on 12 to 4 on 12 provided that sufficient vertical capacity remains.

2.5 Effect of Pile Driving

The effect of driving abutment foundation piles through the sensitive clay soils which prevent the lateral yield of the embankment foundation soils was considered. The marine clay soils of the Ottawa area have not as yet been observed to lose strength at considerable distances from the piles being driven even though this effect has been observed in other Ontario clays. A study is underway at the Kars Bridge which rests on local marine clays and the effect of pile driving on soil strengths will be better known at the conclusion of this project in approximately six months time. We would suggest that the Kars Bridge results be examined when they are available for confirmation of our present assumptions. Fortunately this confirmation can be available prior to the actual construction of Bridge No. 36.