

66-F-248M

S. NATION RIVER

FINCH

BA. 2392  
Site 31-276

# McROSTIE SETO GENEST

& ASSOCIATES LTD. - CONSULTING ENGINEERS - 393 BELL ST., OTTAWA, ONTARIO  
& ASSOCIÉS LTÉE - INGÉNIEURS CONSEILS - 393, RUE BELL - TEL. 232-5334

66-F-248M

## 1. TERMS OF REFERENCE

We were requested by C.C. Parker and Associates Limited, Consulting Engineers, to carry out a subsurface investigation at the site of a proposed bridge crossing the South Nation River in Lot 3, Concession V and VI, Township of Finch, Counties of Stormont, Dundas and Glengarry.

The proposed structure would consist of a continuous girder with two piers and two abutments. The approach embankment on the west side would be about 20 feet high at the abutment. On the east side the approach embankment would be about 5 feet high at the abutment. A new culvert of reinforced concrete or steel was also proposed at about 1,000 feet east of the east abutment of the new bridge.

## 2. RECOMMENDATIONS

### 2.1 Structure Foundation

The most likely economical and feasible type of foundation for the proposed bridge at this site is a steel H-pile foundation. High capacity precast concrete piles such as Herkules piles could prove to be economical. Of course any alternate type of end bearing piles should also be considered if they prove to be more economical.

The driving of piles at this site might be difficult due to the presence of boulders as encountered in boreholes 1, 2, 3 and 4. It should therefore be expected that some of the piles will need to be relocated within the abutment and pier supports.

## 2.2 Extra Pile Load Due to Downdrag

Where piles are driven through a compressible layer which is continuing to consolidate over a period of time due to a new load, some of the weight of the soil surrounding the piles is gradually transferred onto the piles by "negative friction". This additional load is commonly known as pile downdrag and considerable judgment is required in estimating its occurrence and its eventual magnitude. After considering all the factors of the subsurface profile, the average pile size and surface area and the type of material supporting the piles, we can recommend that the design loads on each pile of the west abutment be increased by 25% to provide an allowance for downdrag.

## 2.3 Soil Strength

Since soils are not recommended for support of a footing type foundation for the proposed structure, detailed bearing values are not given here. It can be stated, however, that the underlying boulder till and bedrock below this site will provide adequate support for an H-pile foundation or any type of end bearing pile which is likely to be used at this site.

## 2.4 Embankment Foundations

### 2.4.1 Consolidation Settlements

#### 2.4.1.1 West Embankment

A settlement analysis considering the clay stratum between elevations 210 and 200, as determined in the boreholes at the west abutment and using results of consolidation tests made on clay soils of similar properties, showed that a total settlement of 0.2' to 0.3' could be expected below the centre of a 20

foot high embankment at this location. It was assumed in the analysis that the highly compressible layer of organic material was removed and replaced by compacted granular material. The timing of settlement is difficult to determine accurately since coefficients of consolidation determined for various layers in the compressible stratum must be representative. However, we estimate that settlements are likely to occur over a period of one to two decades.

#### 2.4.1.2 East Embankment

Consolidation settlements of the clay foundation under the east embankment are not expected to be significant since the clay soils have been preconsolidated to pressures greater than the proposed embankment load.

#### 2.4.1.3 Embankment at New Culvert (Station 6 + 20)

A settlement analysis considering the clay stratum between elevation 222 and 205 as determined in borehole No. 5 and using results of consolidation tests done on clay soils of similar properties, showed that a total settlement of 0.2' to 0.3' could again be expected below the centre of the 20 foot high embankment at this location.

As mentioned previously, the timing of settlement is difficult to determine. We estimate, however, that the settlements are likely to occur over a period of one to two decades.

In view of the settlement expected, a flexible pipe culvert bearing on a granular cushion would appear to be more desirable. Any alluvium or organic deposits would of course need to be removed prior to the placing of a granular cushion.

A rigid concrete box culvert could be used; however, the estimated settlement would need to be recognized in the design of such a structure.

#### 2.4.2 Embankment Stability

##### 2.4.2.1 West Approach Fill (Laterally)

A stability analysis using a total stress approach was made considering a failure slip lateral to the proposed 20 foot high embankment.

The first analysis was made assuming that the embankment fill would be placed above the existing natural soils after removal of the 2.5 to 5 foot thick topsoil layer.

The results of this analysis showed that in order to obtain a suitable factor of safety against instability of the embankment foundation, the existing layer of organic material would need to be removed. This analysis considered, however, a single stage construction. If construction proceeds in stages, a further study of the stability under stage construction could be made.

An analysis of the embankment stability after removal of the organic material and replacement by compacted granular material proved to be stable. A further analysis of the embankment considering a slip circle through the underlying clay soils also proved to be stable. In the analysis a reduction of 20% in the shearing resistance of the embankment was made for compatibility of stress - strain characteristics of the granular embankment and clay foundation. (See Plate No. 13)

#### 2.4.2.2 West Abutment (Closed-end Structure)

A stability analysis using the total stress method was made considering a slip circle longitudinal to the proposed 20 foot high embankment at the west abutment. Boreholes 1 and 2 revealed the presence of a clay stratum at that location. The results of the analysis showed that the embankment should be built with characteristics of density and angle of internal friction of 130 p.c.f. and  $\phi = 37^{\circ}$  respectively. These properties would ensure a factor of safety of 1.6 for the stability of the foundation. Similarly to 2.4.2.1 a reduction of 20% was made in shearing resistance through embankment for compatibility purposes. Of course scouring and erosion protection would be required in the river bed east of the west abutment. (See Plate No. 14).

#### 2.4.2.3 Embankment Fill at Culvert (Station 6 + 20)

A stability analysis using the total stress method was also made considering a failure circle lateral to the 20 foot high embankment. It was assumed in the analysis that any alluvium or organic soil deposits would be removed, if encountered, prior to the placing of the embankment fill. Also a reduction of 20% in shearing resistance strength through embankment was made for compatibility purposes.

The results of this analysis showed that the embankment foundation would be stable under a 20 foot high embankment.

In the analysis the characteristics of density and angle of internal friction  $\phi$  of the embankment were assumed to be 120 p.c.f. and  $\phi = 32^\circ$  respectively. These properties would ensure a factor of safety of 3.1 for the stability of the foundation. (See Plate No. 15).

#### 2.5 Construction Precautions

Construction inspection of pile driving operation should be considered. Similarly, supervision of the removal of organic material and compaction control of embankment fills should also be considered. An allowance provided to cover the cost of a pile load test on any pile which inspectors might feel was substandard is a useful addition to construction control of any piling contract.

Alluvium or organic soil deposits encountered in the creek bed should be removed before placing of culvert and embankment fill. We would suggest that the

compaction around the pipe be done by first placing vertical struts inside the pipe and then compacting the granular fill on both sides of the culvert until the struts become loose.

Protection of the foundation and embankment materials against erosion by flowing water and wave action should be considered. Hence, the culvert entrance and exit could be protected by a boulder pavement.

Variations between boreholes could be expected at this site because of the irregular subsoil encountered. If significant variations are discovered at the time of construction, they should be reported to the supervising authority for appropriate action.

Finally, variations in depth to the dense till stratum can be expected and contract payment procedures should make clear which party is to bear the cost of these variations.



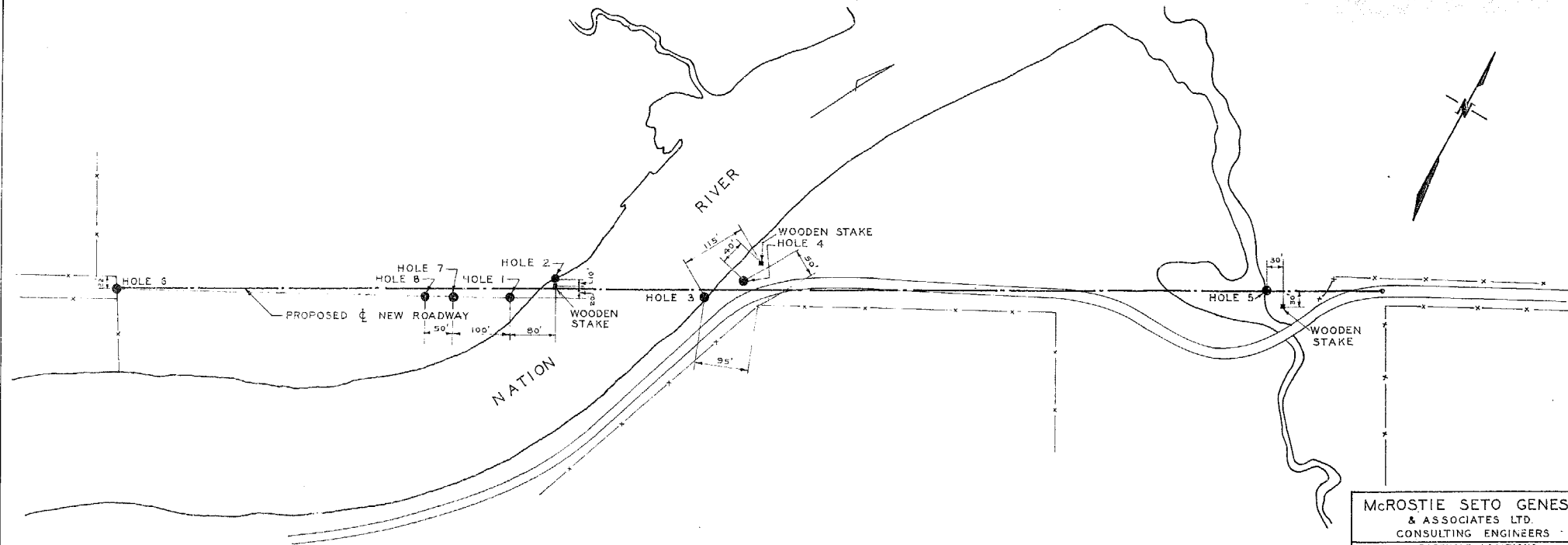
### 3. SITE INVESTIGATION

Eight boreholes were made at the site with our test drilling equipment in the locations shown on Plate No. 1. Sampling in the cohesive soils was by 2 inch thin wall Shelby tube sampler with small scale penetrometer tests made at the end of the tube sample as they were taken out. In situ vane tests were made between the tube sampling. Sampling in non-cohesive soils was by 2 inch split barrel sampler with standard penetration resistance tests carried out simultaneously. Where casing refusal was encountered, the material was diamond core drilled to confirm the presence of bedrock. During the diamond drilling operations a careful watch was kept for any loss of drill water or drop of drill rods since these help to indicate the soundness of the rock. All soil and rock samples were field identified, logged and then brought to our laboratory for further study.

Moisture content and visual classification tests were made on all samples and a series of small scale penetrometer tests was made on all tube samples. Three Atterberg Limits and Mechanical analysis were made on representative samples of the subsoil strata.

The soils and rock encountered at borehole locations are shown in detail on Plates No. 2 to 9. On the west bank of the river between chainage 19 + 00 and 22 + 00 the soils can be generalized as consisting of a layer of topsoil 2.5 to 5.0 thick, underlain by about 7 feet of clayey silt overlying a layer of organic material about 3.5 feet thick. Below the organic materials is a layer of clay about 8 feet thick and a layer of till about 15 feet thick which overlies bedrock. On the east shore the subsoil consists of a two foot

layer of topsoil underlain by hard to medium soft clay, which overlies a layer of dense till above rock. Bedrock is predominantly a limestone with some limy dolomite layers. Artesian pressures were encountered in borcholes No. 3 and 5. However, these were sealed off before leaving the site.



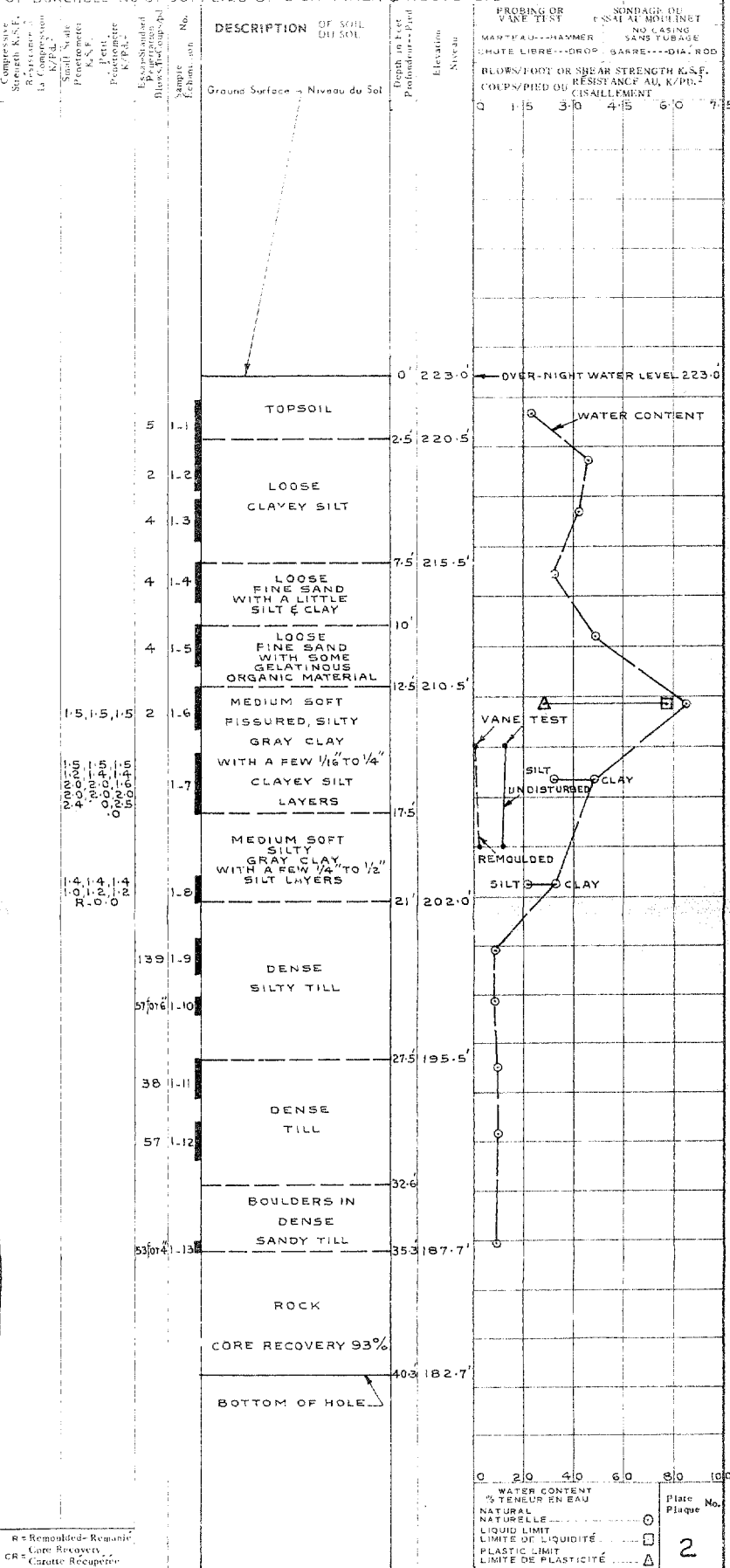
McROSTIE SETO GENEST  
& ASSOCIATES LTD.  
CONSULTING ENGINEERS  
BOREHOLE LOCATIONS  
CHESTERVILLE AREA - BRIDGE No.3  
SCALE:- 1"=100' | PLATE No. 1

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 223.0'  
NIVEAU DU SOL (PROFONDEUR ZÉRO)

DATE JUNE 6, 1966

HOLE FORAGE No. 1

NOTES: B.M. (E.L. 232.4) GEODETIC - PAYMENT SURFACE OVER CULVERT BOREAST  
OF BOREHOLE No. 5, SUPPLIED BY C.C. PARKER & ASSOC. LTD.



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ELEVATION OF GROUND SURFACE (ZERO DEPTH) 222.0  
NIVEAU DU SOL (PROFONDEUR ZÉRO)

NOTES SEE PLATE No 2

## SOIL PROFILE & TEST SUMMARIES

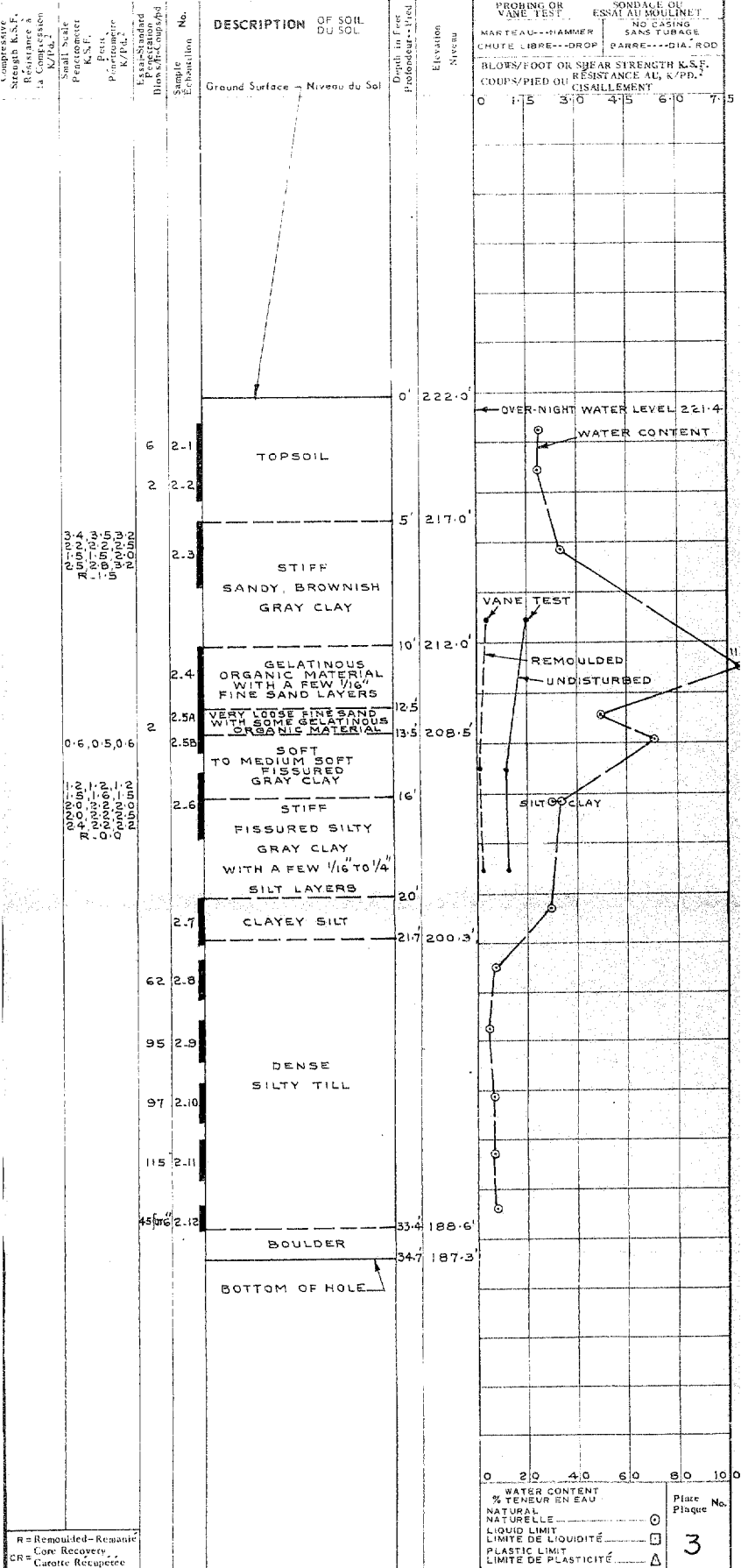
PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

### CHESTERTVILLE AREA BRIDGE No. 3

DATE JUNE 3, 1966

HOLE FORAGE No.

2



R = Remoulded - Remanié  
CR = Core Recovery - Carotte Recupérée

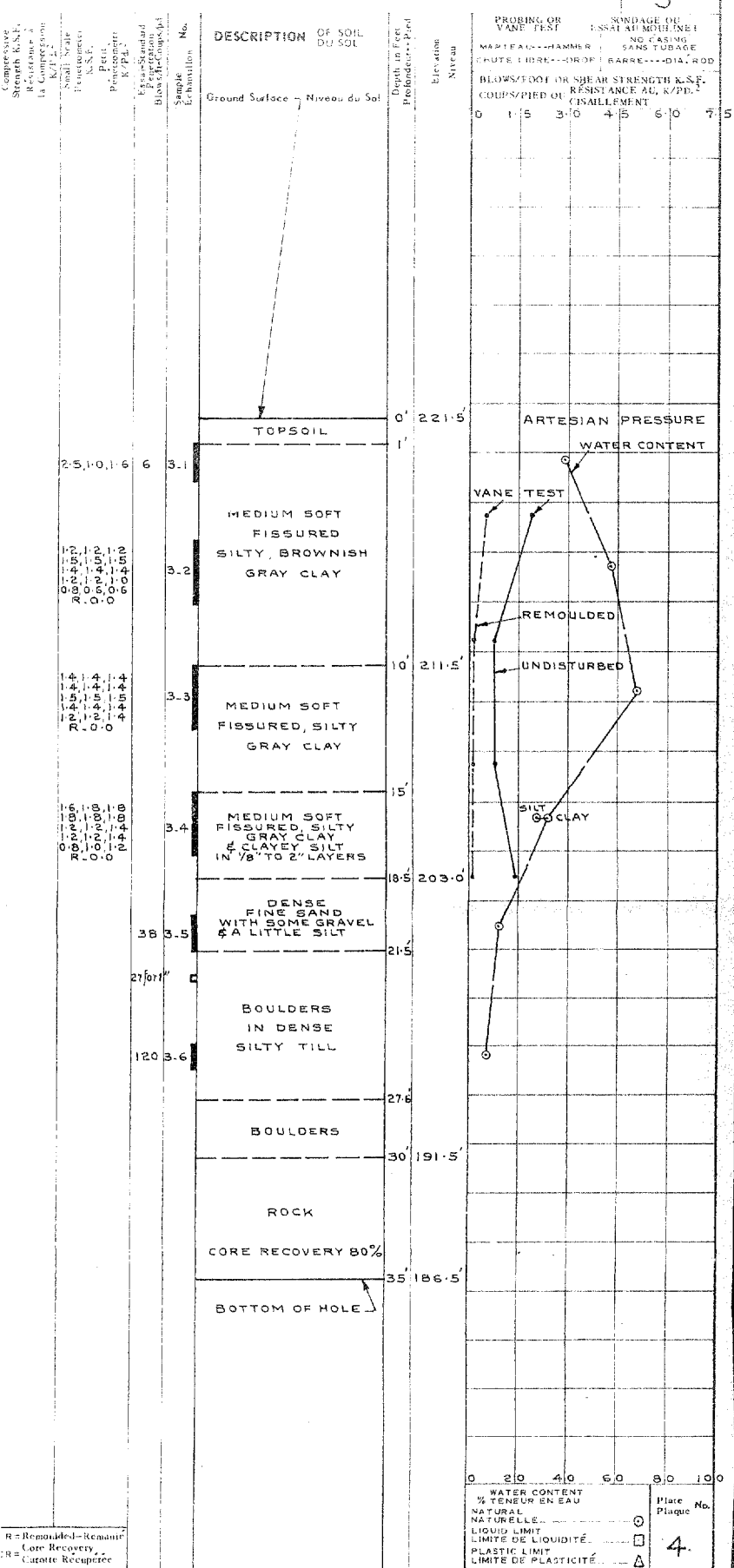
MONOSTIE SOTO GENEST

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CONSULTING ENGINEERS - INGENIEURS CONSEILS  
OTTAWA CANADA

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 221.5'  
NIVEAU DU SOL (PROFONDEUR ZÉRO)  
NOTES SOIL PLATE No. 2

SOIL PROFILE & TEST SUMMARIES  
PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS  
CHESTERTVILLE AREA  
BRIDGE No. 3

DATE JUNE 1, 1966 HOLE FORAGE No. 3



# MONROE SPTO GENEST

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CONSULTING ENGINEERS - INGÉNIEURS CONSEILS

OTTAWA CANADA

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 234.1

NIVEAU D. SOL (PROFONDEUR ZÉRO)

NOTES SEE PLATE No. 2

## SOIL PROFILE & TEST SUMMARIES

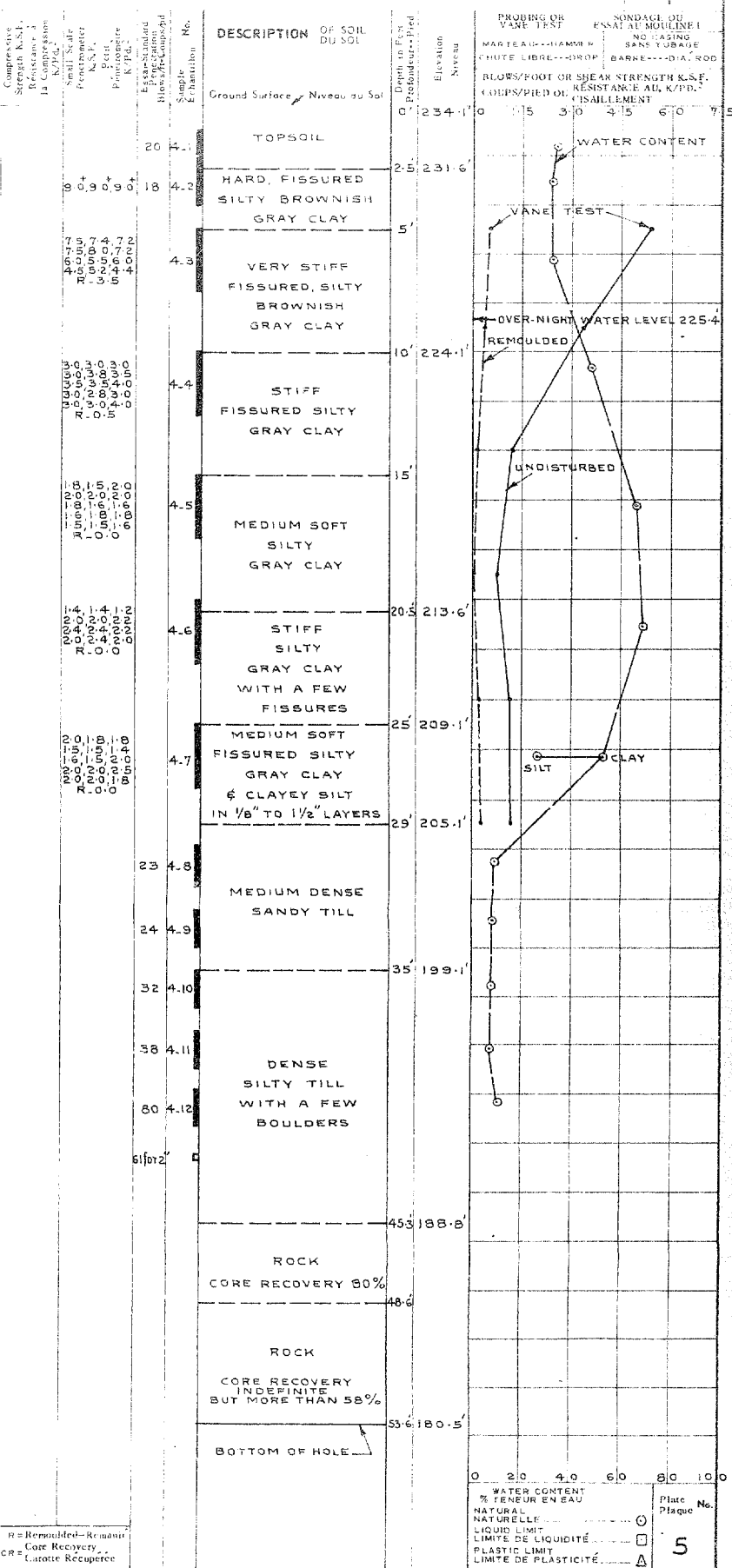
PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

CHESTERTVILLE AREA  
BRIDGE No. 3

DATE MAY 31, 1966

HOLE FORAGE No.

4



R = Remoulded - Remanié  
CR = Core Recovery  
Carotte Récupérée





Core Recovery  
Catotte Récupérée

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**CONSULTING ENGINEERS - INGÉNIEURS CONSEILS**  
**OTTAWA CANADA**

## PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

CHESTERVILLE AREA  
BRIDGE No.3

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 223.0'  
NIVEAU DU SOL (PROFONDEUR ZERO)

DATE JUNE 7, 1966

HOLE No.  
FORAGE 7

NOTES SEE PLATE No. 2

[illegible]

# MONROD SEED GENEST

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OTTAWA CANADA

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 221.0'  
NIVEAU DU SOL (PROFONDEUR ZÉRO)

NOTES: SEE PLATE NO. 2

## SOIL PROFILE & TEST SUMMARIES

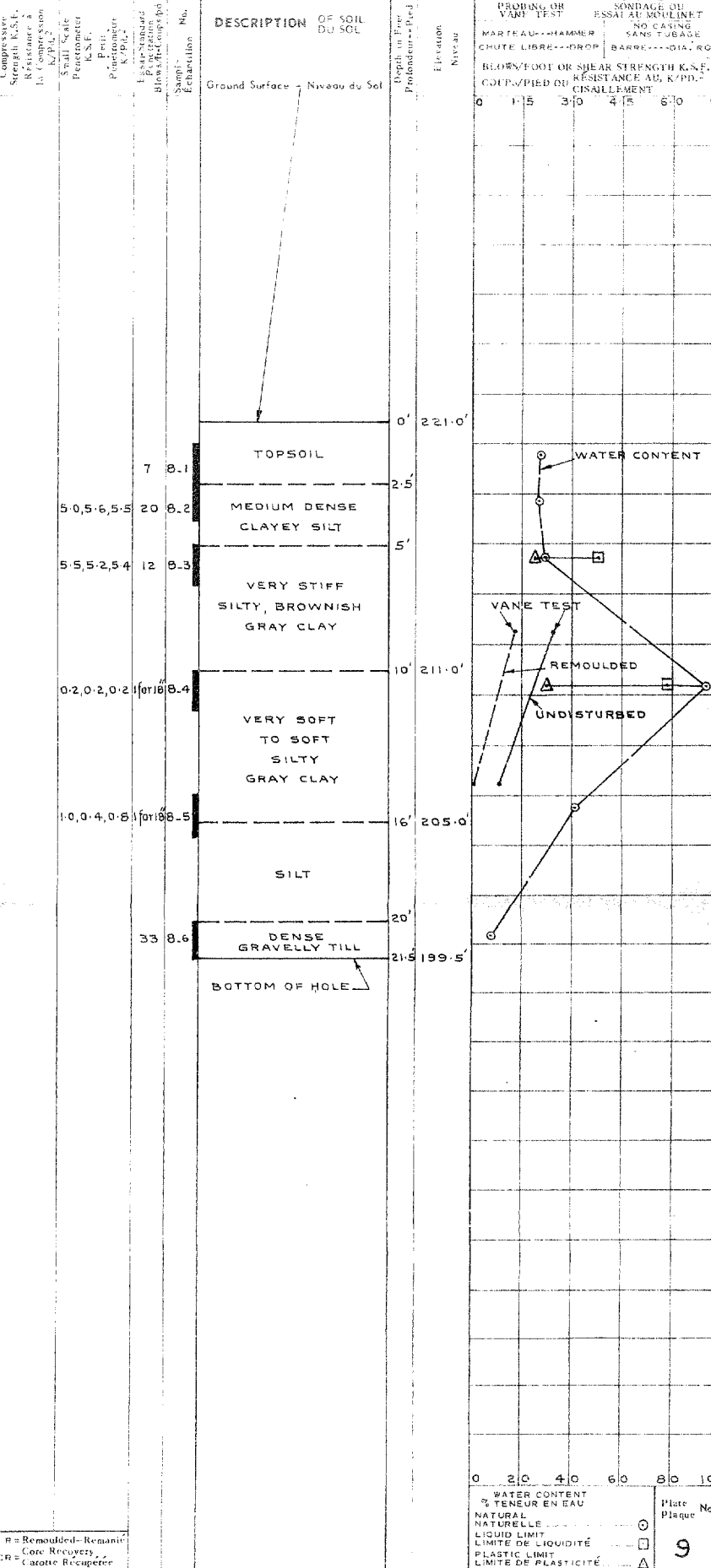
PROFIL DE TERRAIN ET RÉSUMÉ DES ESSAIS

CHESTERTVILLE AREA  
BRIDGE No. 3

DATE JUNE 7, 1966

HOLE FORAGE No.

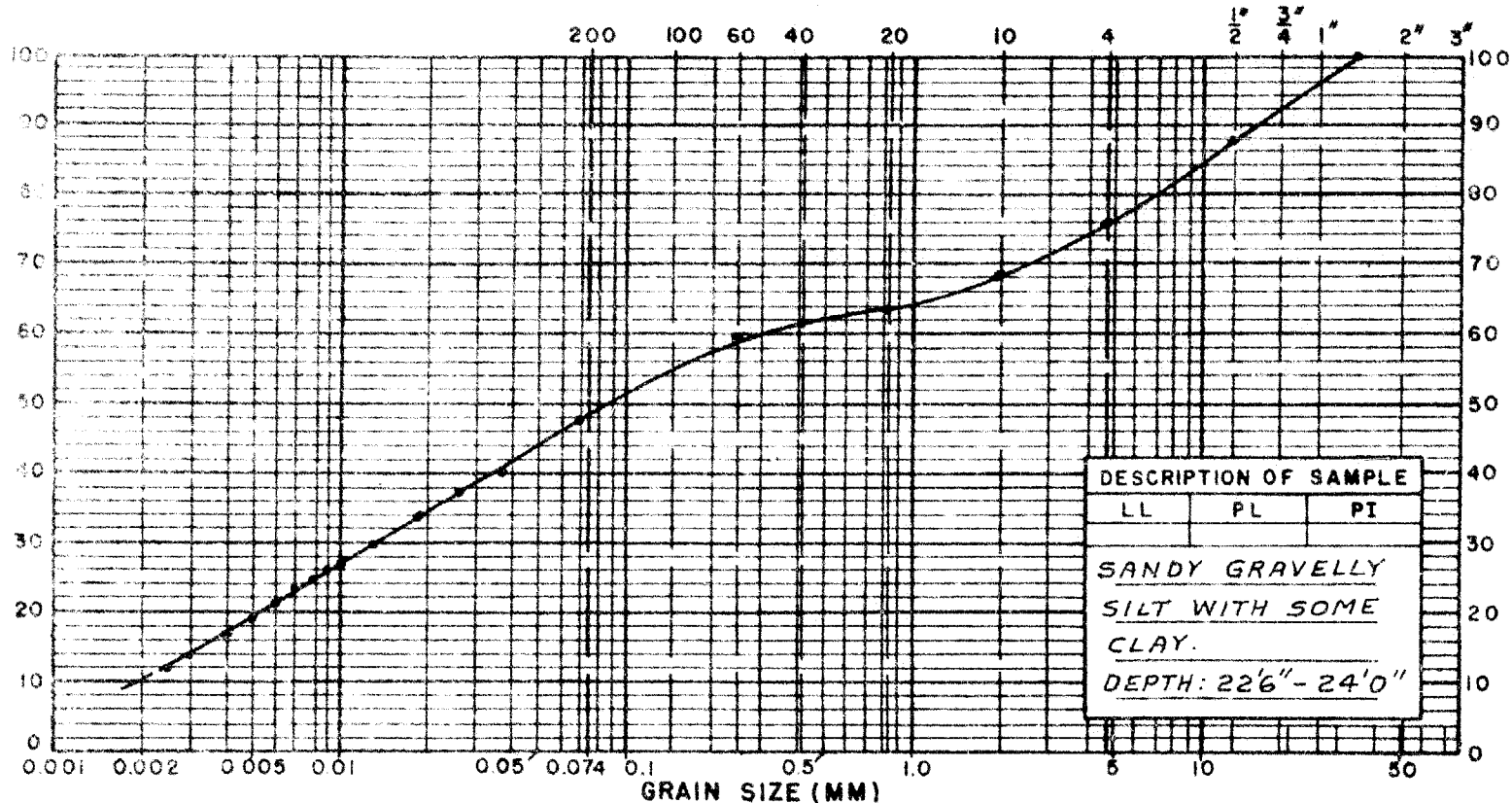
8



R = Remoulded - Remanié  
Core Recovery  
Carotte Récupérée

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

PERCENT FINER BY MASS



DESCRIPTION OF SAMPLE

LL	PL	PI
SANDY GRAVELLY SILT WITH SOME CLAY.		
DEPTH: 22'6" - 24'0"		

CLAY OR SILT		SAND			GRAVEL	
		FINE	MEDIUM	COARSE	FINE	COARSE
48%		27%			25%	

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

PROJECT E-1608 CHESTERVILLE

SAMPLE No 2-B

PLOTTED A.G. DATE 22-6-66

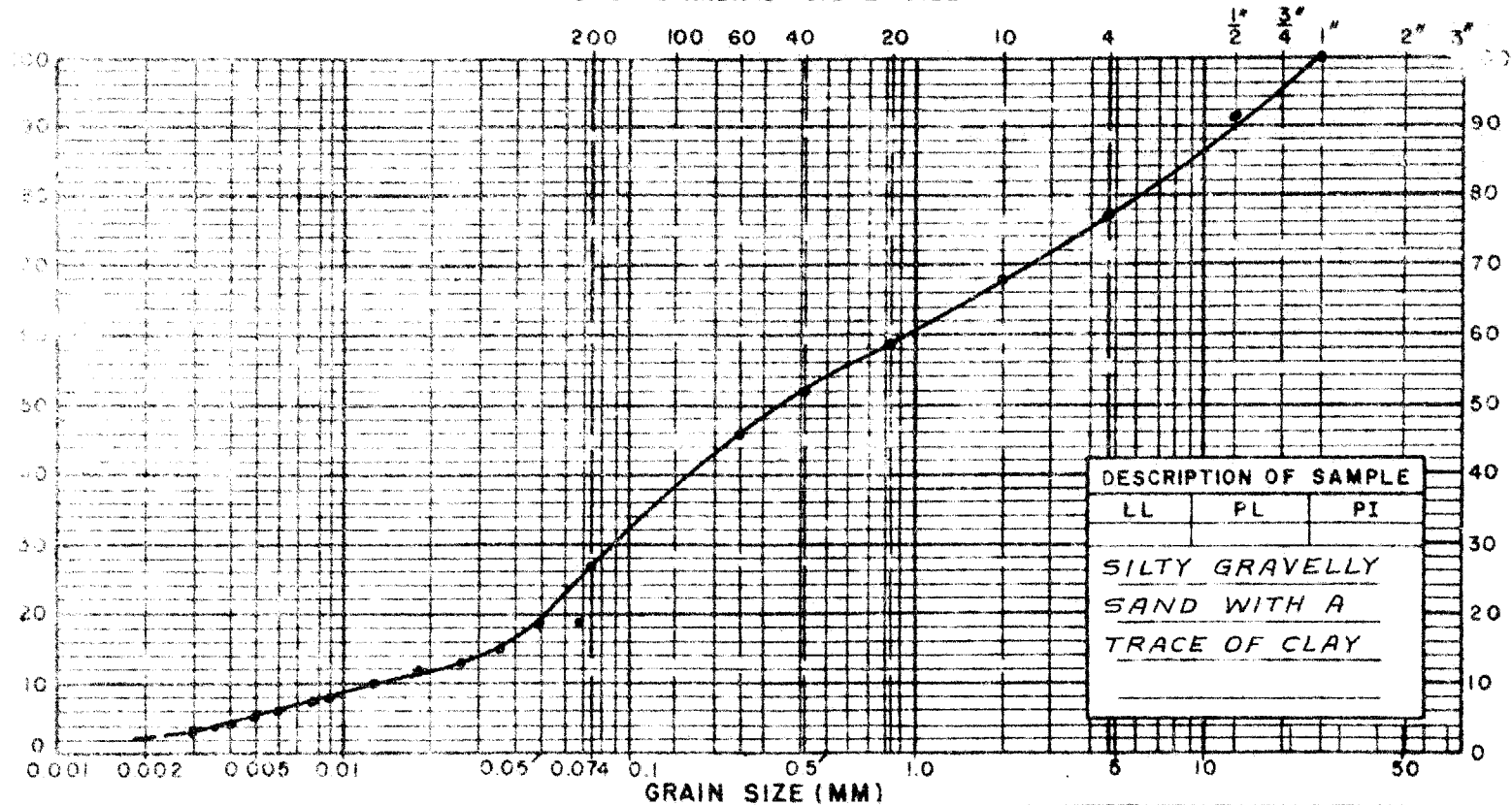
REMARKS .002 TO .105 = 42%

CHECKED L.B. DATE 22-6-66

McROSTIE & ASSOCIATES LTD.  
CONSULTING ENGINEERS  
OTTAWA, CANADA

SAMPLE No 2-B

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



DESCRIPTION OF SAMPLE		
LL	PL	PI
SILTY GRAVELLY SAND WITH A TRACE OF CLAY		

CLAY OR SILT	SAND			GRAVEL	
	FIN	MEDIUM	COARSE	FINE	COARSE
27%		50%			23%

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

PROJECT E-1608 CHESTERVILLE

SAMPLE No 3-5

PLOTTED A.G. DATE 22-6-66

REMARKS .002 TO .105 = 30%

CHECKED L.B. DATE 23-6-66

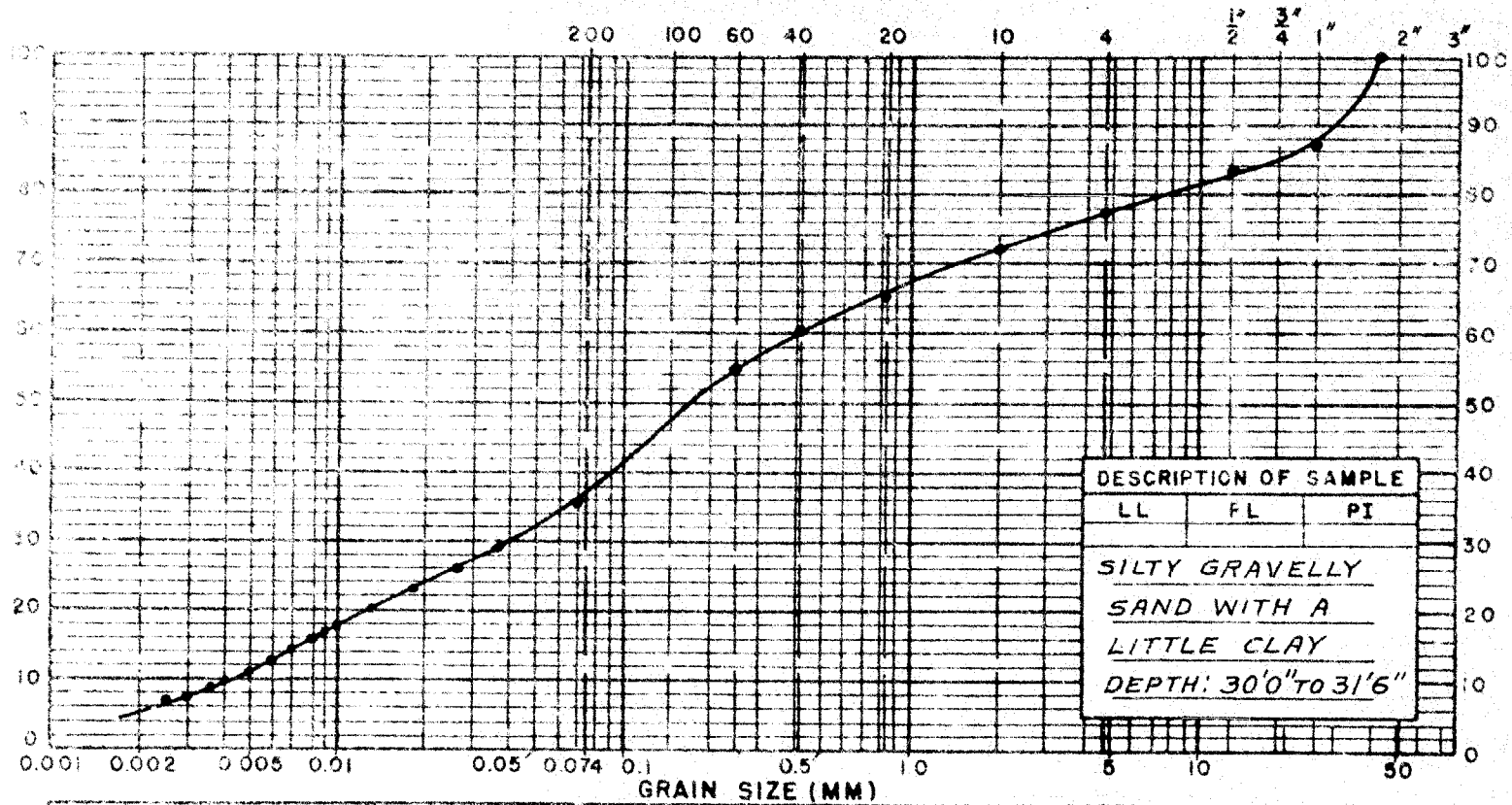
DEPTH: 20'0" TO 21'6"

McROSTIE & ASSOCIATES LTD.  
CONSULTING ENGINEERS  
OTTAWA, CANADA

SAMPLE No 3-5

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

PERCENT FINER (UNITED STATES METHOD)



DESCRIPTION OF SAMPLE

LL	FL	PI
SILTY GRAVELLY SAND WITH A LITTLE CLAY		
DEPTH: 30'0" TO 31'6"		

CLAY OR SILT		SAND			GRAVEL	
		FINE	MEDIUM	COARSE	FINE	COARSE
37%		40%			23%	

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

PROJECT E-1608 CHESTERVILLE

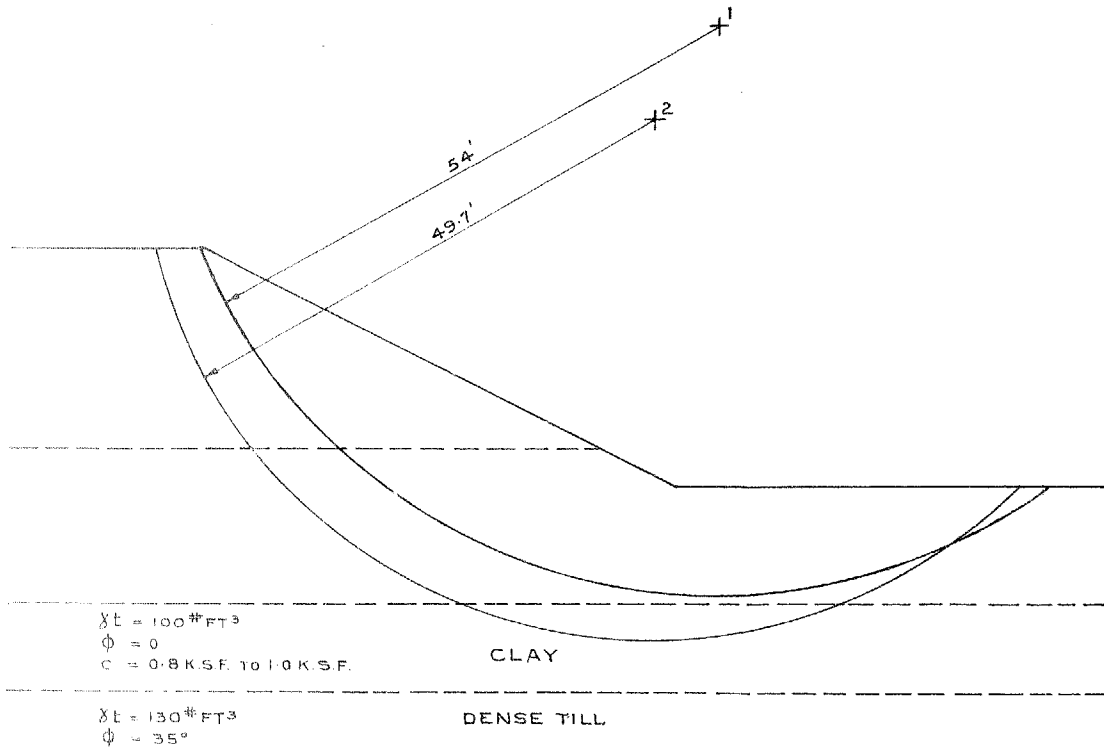
SAMPLE No 4-8

PLOTTED A.G. DATE 22-6-66

REMARKS .002 TO .105 = 37%

CHECKED L.B. DATE 23-6-66

McROSTIE & ASSOCIATES LTD.  
CONSULTING ENGINEERS  
OTTAWA, CANADA



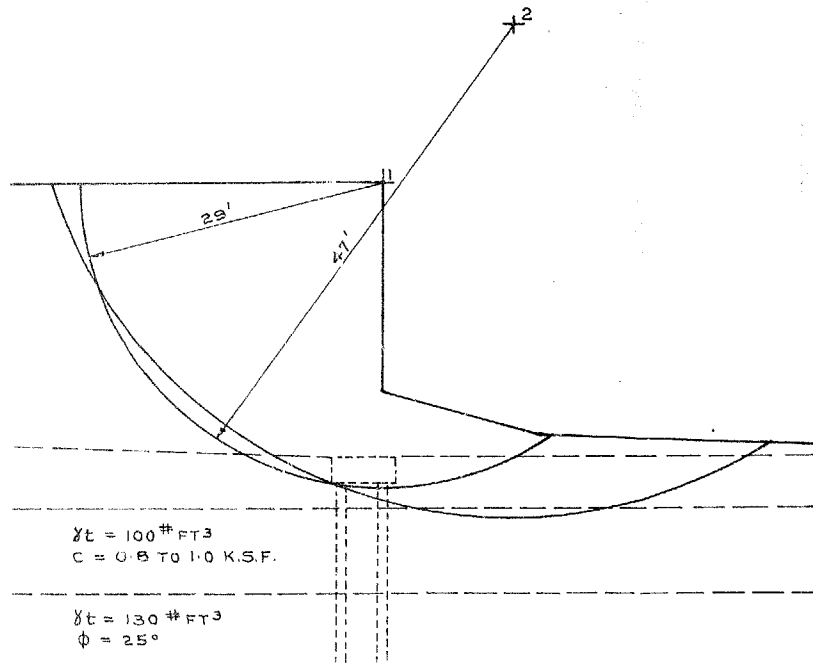
STABILITY ANALYSIS OF	SLIP No.	EMBANKMENT		SUBSOIL		COHESION ALONG ARC K.S.F.	F.S.
		$\phi^\circ$	$\gamma_t$ P.C.F.	$\phi^\circ$	$\gamma_t$ P.C.F.		
	1	32°*	120	28°*	100	0	0.8
				0°	60	0	
		32°*	120	28°*	100	0	1.3
EMBANKMENT	2			0°	60	0	
		32°	120	REMOVED & REPLACED SUBSOIL TO ELEV. 210 AS EMBANKMENT		0	2.0
		32°*	120	0	100	0.8	1.9

\*80% OF TAN  $\phi$  USED TO CALCULATE EFFECTIVE SHEARING RESISTANCE THROUGH GRANULAR EMBANKMENT AT INCIPIENT FAILURE.

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STABILITY ANALYSIS  
LATERAL TO WEST EMBANKMENT  
CONNAUGHT BRIDGE

SCALE 1"=10' PLATE No. 13



STABILITY ANALYSIS OF	SLIP No.	EMBANKMENT		SUBSOIL		COHESION ALONG ARC K.S.F.	F.S.
		$\phi^\circ$	$\gamma_t$ P.C.F.	$\phi^\circ$	$\gamma_t$ P.C.F.		
STRUCTURE CLOSED END	1	32°	120	32°	120	0	1.5
	2	32°*	120	0	100	0.8	1.3
		38°*	130	0	100	0.8	1.5
		38°*	130	0	100	1.0	1.6

\* 80% OF  $\tan \phi$  USED TO CALCULATE EFFECTIVE SHEARING RESISTANCE THROUGH GRANULAR EMBANKMENT AT INCIPIENT FAILURE.

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& ASSOCIATES LTD.  
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STABILITY ANALYSIS  
AT WEST ABUTMENT

CONNAUGHT BRIDGE

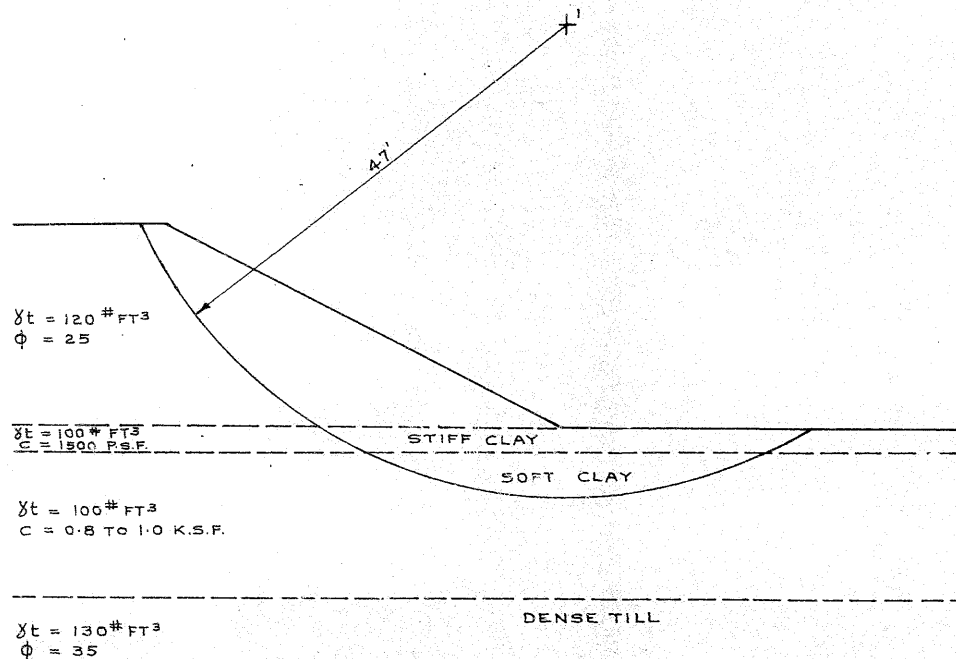
SCALE: 1"=10'

PLATE No. 14



STABILITY ANALYSIS OF	SLIP No.	EMBANKMENT		SUBSOIL		COHESION ALONG ARC K.S.F.	F.S.
		$\phi$	$\gamma_t$ P.C.F.	$\phi$	$\gamma_t$ P.C.F.		
EMBANKMENT	1	32°*	120	0 0	100 100	1.5 0.8	3.1

\* 80% OF TAN  $\phi$  USED TO CALCULATE EFFECTIVE SHEARING RESISTANCE THROUGH GRANULAR EMBANKMENT AT INCIPIENT FAILURE.



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STABILITY ANALYSIS  
OF EMBANKMENT  
AT NEW CULVERT (STA. 6+20)  
CONNAUGHT BRIDGE