

Plot on 30L14

e. m. peto associates ltd.

YOUR REFERENCE:-

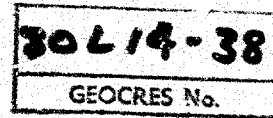
OUR REFERENCE:- 55225

BA ~~1813-B~~
1813-B

1287 caledonia road,
TORONTO 19, ONTARIO
Telephone: 789-1126

SITE 30-54

November 26th, 1965



The County of Welland,
102 Main Street East,
Welland, Ontario.

Attention: Kr. W.L. Smith, P.Eng.,
County Engineer

Gentlemen:

Re: Robbins Bridge
Welland River - Ontario

We submit six copies of our report on the stability problems arising on the north approach of this bridge in August 1965, and authorized by purchase order No. 603.

The report deals with the initial failure, the remedial measures recommended following the initial slip, and with the subsequent failure of the berms recommended in the remedial work.

It is concluded that the initial failure was due to raising the original approach embankment some six feet, this additional surcharge being sufficient to create unstable conditions within the underlying soft organic clayey silt.

The subsequent failure arising within the berm on the east side of the approach where it adjoins the abutment is attributed to insufficient horizontal space between the bridge abutment and the river bank.

The County of Welland

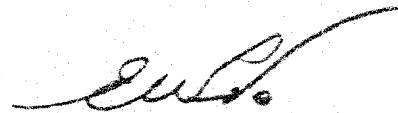
-2-

This lack of space does not allow such geometry of embankment and berm next to the bridge abutment which would ensure stability without overstressing the soil. For this reason the eventual remedy resolved itself into displacing the soft organic soil to achieve stability of the bank on the east side where it adjoined the river. Elsewhere the original proposals were satisfactory.

We believe this report to be complete within your terms of reference, however, if you have any questions in connection with this work please contact us.

Yours very truly,

E.M. PETO ASSOCIATES LTD.,



E.M. Peto, P.Eng.

CEP/hf

Soil Investigation Report
Of
ROBBINS BRIDGE
WELLAND RIVER - ONTARIO
For
The County of Welland

DISTRIBUTION:

6 c.c. The County of Welland
1 c.c. File

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A. INTRODUCTION

The work described in this report was authorized by purchase order No. 603 dated 4th August 1965.

Following the recent replacement of an old bridge on Welland River, known as Robbins Bridge, by a new structure, a failure of the reconstructed embankment on the north side of the new bridge took place. This Company was retained to analyze the causes of the failure and to suggest remedies.

B. GENERAL INFORMATION

1. The subsoil conditions at the site and the geology and topography of the area were first described in our report No. 63115, dated August 8, 1963. The report, apart from bridge foundation design data, included recommendations regarding the reconstruction of the bridge approach embankments.
2. The additional field work was carried out at the beginning of August 1965, by our field crew No. 3, using a standard skid-mounted diamond drilling rig, adapted for soil sampling. Standard drilling and sampling procedures were followed, but undisturbed samples were taken at more frequent intervals, to determine as thoroughly as possible the shear strength of the soft soil.
3. Five testholes were put down at the locations shown on the enclosed drawing. The depth of the holes was only sufficient to prove the foundation strata possibly involved in the embankment failure, and varied from 31.5 to 41.5 feet.
4. Ground elevations at the borehole locations were measured by our field engineer, and were referred to the level of the deck of the new bridge, assumed to be at elevation 100.0. In order to convert the assumed elevations to Geodetic, 480.1 must be added to the reported levels.

B. GENERAL INFORMATION - cont'd.

-2-

5. Details of soil conditions encountered in the testholes are described on the appended borehole logs, which also include the results of in-situ standard penetration tests, water contents, Atterberg tests and shear strength data.
6. Records of laboratory tests, such as particle size distribution, shear strength, density and consolidation, are included in Appendix A.

C. SOIL CONDITIONS

The subsoil conditions and the geology of the area were described in our Report No. 63115, of August 8, 1963. The present testholes confirmed the stratification within the depth affecting the embankment stability. The subsoil profiles, in the form of sections through the testholes, are presented on the enclosed drawing.

The cause of the embankment distress is in all likelihood the layer of soft organic silty clay. This material commences a foot or two below the floor of the valley and has an indicated thickness of about 16 ft. close to the bridge. Two holes put down 75 ft. from the bridge showed a thickness of only about 10 ft., so that the material apparently tapers out in the northerly direction, away from the river. At this latter location, the organic clay was also considerably stronger than nearer the bridge, which probably explains the limited length of the embankment affected by instability.

The organic clay is underlain by a stratum of reddish-brown, layered clay. This material forms the main overburden over bedrock in the general area, and at this site is about 62 ft. thick, as determined in the earlier investigation. The red-brown clay is soft at depth, but possesses a stiffer crust immediately below the lower boundary of the overlying organic clay. Because within this crust the clay is appreciably stronger than the overlying organic clay, it was concluded that the lower limit of a potential slip zone is the top of the red-brown clay stratum.

C. SOIL CONDITIONS - cont'd.

-3-

The new embankment fill, consisting of silty clay mixed with some sand, was placed over the old embankment, and directly on top of the existing grade outside the limits of the old embankment. Following the failure, the uppermost six feet of the new embankment material were spread outside the embankment toe, and the testholes were put down through a varying depth of this fill as indicated on the subsoil profiles.

D. STABILITY OF EMBANKMENT

1. Analysis of Initial Failure: The Report No. 63115 drew attention to the presence of the soft organic clay layer. To ensure stability of the embankment, it was recommended to place a granular drainage blanket below the new fill, and to build up the grade slowly, preferably before completion of the new bridge. At the time, our information was that the roadway elevation of the old bridge would be maintained, but we recommended not to raise the height of the embankment.

A drainage blanket was not included and the new embankment was placed over a relatively short period, and to a height some six feet greater than that of the old embankment. Moreover, it was reported that the new fill consisted of clay which was poorly compacted and may have been too wet at the time of placement.

A bulging out of the embankment slopes was observed, which would indicate that the fill material was itself unstable, possibly due to water content wet of the optimum and poor compaction.

Following extensive cracking and some subsidence of the embankment, the uppermost six feet were removed. However, later reports indicated that slippage was continuing, in spite of the unloading. This probably shows that a shear in the foundation soil has been initiated under the full weight of the embankment. Further, some cracking and movement of fill below

the bridge deck has been reported. According to profiles supplied by the County of Welland, this fill was placed on a 4:1 slope, and should have been stable at this angle even with poor compaction. Hence, its instability is probably caused by the weakness of the subsoil.

The above observations may be interpreted to indicate a two-fold cause of instability, namely poor quality of compaction, resulting in slope failure and shearing of foundation soil, caused by excessive embankment height.

Observations of the cracking and subsidence of the embankment were recorded in a report by W.A. Johnston, of the Office of the County Engineer, Welland, dated July 28, 1965. The report describes the extent of tension cracks at various depths, as the uppermost six feet of the embankment were scraped off. This work was carried out carefully, in one foot layers. Sketches and dimensions of the cracks observed at the various levels were included in the above report. Extent of the cracks could be followed thanks to the presence of gravel, which was used to fill out the cracks in the embankment as they first appeared and before the bridge was closed to traffic. Although the gravel could not arrest the embankment failure, it served the very useful if unintended purpose of facilitating a study of the instability.

Using the reported observations, we have traced the extent of the tension cracks on a profile of the embankment and subsoil and determined probable locations of slip surfaces in the subsoil. Slip circle analysis was then carried out on alternative slip surfaces consistent with the observed locations of the tension cracks. Assuming that the factor of safety must have been near or just below unity with the embankment to its full height, the probable value of the shear strength of the organic clay was calculated. This value was found to be around 250 lbs./sq.ft., which is consistent with the shear strength on clay samples measured in the laboratory. The strength below and outside the slope of the embankment, where it has not gained strength as a result

D. STABILITY OF EMBANKMENT - cont'd.

-5-

of consolidation under the weight of either old or new fill, was probably closer to 150 lbs/sq.ft., which is again consistent with the laboratory data.

Observations indicated that the depth of the tension cracks was about 8 ft. which would be consistent with an average shear strength of the cohesive fill of 700 lbs/sq.ft. This value was included in the stability analysis along the probable slip plane through the fill, below the tension crack. Zero shearing resistance in the tension cracks was assumed.

The stability of the old bridge embankment was also analyzed, employing the shear strength data determined as above. The factor of safety was found to be above unity, verifying the shear strength assumptions.

The shear strength values, obtained from laboratory tests and from analysis of the embankment failure, were used for the design of berms for the reconstruction of the embankment.

2. Remedial Measures Recommended Following The Initial Failure: Calculations were carried out, which showed that berms must be provided to ensure stability of the embankment in the section which has exhibited cracking.

As the soil consolidates and gains strength under the weight of the berms, the factor of safety of the embankment would increase. This is a slow process because of the very low permeability of the clayey soil, and the most critical period would be during and for a time after the reconstruction. A factor of safety of 1.3 was considered appropriate during the critical period. This led to the recommendation of the following geometry of the berms.

D. STABILITY OF EMBANKMENT - cont'd.

-5-

Height: 8 ft. below crown of embankment, at toe of embankment.

Width: 60 ft. at bridge abutment, progressively decreasing at a rate of 1 ft. of width for every 4 ft. of distance along the embankment to the north of the bridge abutment. This width is between the toe of the embankment on top of the berm and the toe of the berm.

Slopes: 2.5:1, horizontal to vertical slopes are suggested both for the embankment and for the berms. A slope of at least 0.25 in. per ft. should be provided for drainage and should be maintained during settlement of the berm.

Extent: The berms should extend at least 80 ft. from the abutment, or further if cracks beyond this distance have been observed.

It was believed that berms with the above dimensions would somewhat extend past the Right-Of-Way. Should this have been the case and the County would have preferred to avoid buying out more land, the following alternative was presented for consideration.

A single-lane roadway, some 20 ft. wide, could be provided as a temporary measure. An alternative berm width for a factor of safety of 1.3 would then be possible within the present Right-Of-Way if the centreline of the 20 ft. roadway was located 3 ft. west of the centreline of the bridge. These dimensions should be checked. The embankment could then be extended to the full width of 40 ft. after a few months, when the soil has consolidated under the weight of the berms. It would not be necessary to extend the berms at that time.

The following general remarks should be observed during the reconstruction of the embankment.

D. STABILITY OF EMBANKMENT - cont'd.

-7-

The fill should be placed starting with the berms. The central portion of the embankment should be extended above the berm height only after the berms have been completed.

In the section where failure has occurred, the existing embankment material should preferably be spread out and the reconstruction should proceed over the entire width of both berms. The fill should then be placed in not more than one foot lifts, evenly from end to end of a section. A compaction to at least 80 percent of Modified Proctor maximum density should be achieved, and there should be not less than 10 percent of air voids. A gradient should be provided if stoppage during rain is necessary, so that rain water should drain freely and not stand in pools on the surface of fill.

If only a 20 ft. wide embankment will be provided temporarily then during the subsequent enlargement the new fill should be properly keyed to the existing embankment, the outside foot or two of which should be scarified. To avoid differential settlements, it would be even preferable to tear down the 20 ft. embankment and to build the 40 ft. section evenly across.

3. Subsequent Failure of Berms: Before the recommendations contained in the previous section could be presented in writing, the County proceeded to reconstruct the embankment commencing with the berms.

On September 17th, it was reported that the berm which was just placed on the east side of the embankment had failed, by slipping towards the river. The berm on the west side of the embankment developed a tension crack, almost symmetrical with the failure on the east side.

The undersigned inspected the site on the same day. It was observed that the failure had resulted in the displacement of the soft organic clay, which emerged above the river surface to a distance of some 60 ft. from the shoreline. From the apparent configuration of the soil masses involved in the slide, it was concluded that the failure had the form of slippage within the organic clay stratum, at or above the interface with the stiffer crust of the underlying reddish clay deposit.

Calculations described in Chapter E have previously shown that fill extending to elevation 571 and facing the river would have a marginal stability, if it had a 4:1 slope and providing the river bank profile shown on County of Welland's "Plan of the North Approach to Robbins Bridge", dated August 16th, 1965, was accurate.

Inspection of the fill which was being placed on the west side of the embankment and which had not failed but showed a tension crack, disclosed that in places the level of top of this fill approached the level of the bridge beam set at elevation 574, thus exceeding the computed safe elevation 571. Also, the fill was heaped up on a steep slope facing the river, in preparation for spreading out. It was obvious that this fill was not being properly compacted.

It was reported that the failed fill on the east side of the embankment had similar geometry prior to failure. It is considered that the immediate cause of the failure was the excessive height and slope of the fill. Also, the fill was probably placed too rapidly.

However, the basic reason for the difficulties experienced is the lack of sufficient horizontal space between the bridge abutment and the river bank. This shortage of space does not allow such geometry of embankment and berm next to the bridge abutment which would ensure stability without overstressing the soil.

D. STABILITY OF EMBANKMENT - cont'd.

-9-

As was shown by computations described in Section E, stability of the fill next to the river bank could be considerably improved only by the placement of an underwater berm, consisting of gravel or rock fill, against the bottom of the river bank. Because of the anticipated large cost of such a measure, we hesitated recommending it.

Following inspection of the failure, it was concluded that probably the most practicable course of action was to continue replacing the fill in the failure zone, so that the soft organic clay would be displaced. It was recommended to place the fill on as flat a slope as possible, and also to flatten out as quickly as possible, the heaped-up fill on the west side of the abutment, which was on the verge of failure. It was also recommended to rebuild at first only a narrow part of the embankment, allowing one lane access to the bridge.

The rest of the embankment should be added after consolidation of the soil under the present fill. It is impossible to predict when further fill can be added without any risk. Computations showed that 80 percent consolidation of the organic clay stratum would take approximately two years. In the failed section, where much of the clay was displaced, the process will be considerably faster.

It is recommended to enlarge the embankment only in the spring of 1966.

There is a danger that erosion of the material displaced into the river, particularly by spring floods, may decrease the passive resistance and lead to further instability. On the other hand, the fill within the river banks may have consolidated sufficiently by that time to allow stability even with erosion of the displaced material.

D. STABILITY OF EMBANKMENT - cont'd.

-10-

Should there exist further danger of instability because of erosion, sufficient sliding resistance could probably only be provided by excavating a trench through the fill at the river bank to elevation 540 and to backfill with compacted sand and gravel. This would provide a key extending to the underlying firmer soil.

However, the need for and the size of such a key, or of an alternative gravel berm placed against the toe of the bank below the river, could only be determined quantitatively following a detailed survey of the configuration of the river bank profile and disposition of the displaced soil masses. Also, additional testholes, on the river bank and below the river, would be needed to determine the soil profile and the shear strength.

The alternative measure would be to construct wing walls to contain the fill next to the abutments. However, the walls would have to be supported on to the piles extending into the dense till, and such a solution would undoubtedly be very costly.

E. STABILITY OF FILL BELOW THE BRIDGE DECK

Although no failure of this slope was reported, some cracking and subsidence had been observed. Since the fill material should be stable at the reported 4:1 slope, the instability probably originated in the soft organic clay stratum. Interpolation of the soil data from Report No. 63115 indicates that this material extends below the river banks.

Stability analysis was performed, using configuration of the river bank as shown on Welland County Plan of the North Approach to the Robbins Bridge, dated August 16, 1965. Calculations have shown that the most critical potential surface of sliding is deep-seated, extending well below the submerged portion of the slope, tangentially to the lower boundary of the soft organic clay at a depth of 32 ft. below the bridge deck. The factor of safety is near unity if the shear strength of the organic clay is taken as 200 lbs/sq.ft. Should the observed signs of instability be connected with this potential surface of sliding, then placement of fill or riprap

E. STABILITY OF FILL BELOW THE BRIDGE DECK - cont'd.

-11-

on the river bank would not improve the stability, and may even worsen it. Only placement of gravel against the river bank below water would improve the stability.

Before such, presumably expensive, measure is taken, it is recommended to maintain careful observation of this slope. Should further movement occur, then the top of the slope should be cut off, even at the risk of exposing the bridge abutment wall. It is doubtful whether this fill contributes much to passive resistance against this wall; in fact, the cracking may feasibly have been caused by the observed movement of the wall. Also, if it is possible to control water level in the river, then it is recommended to maintain it at a high level while the soil consolidates under the fill. Any lowering of level should be very gradual, not faster than at the rate of one foot per week.

F. SETTLEMENT OF EMBANKMENT

Based on consolidation test results, included with this report, the final settlement of the embankment would theoretically reach 13 to 24 inches. This is in addition to shear deformation settlement, which mostly takes place during the construction stage. The large settlements are caused not only by the highly compressible layer of organic clay, but also by the considerable compressibility of the 62 ft. thick stratum of red-brown clay.

Approximately one half of the above settlement will occur within the first two years after construction, while the remainder will develop over the next 30 to 50 years.

G. EARTH PRESSURE ON PILES

Because of the settlement of the deep deposits of soft materials under the weight of the fill, horizontal pressure will be exerted on piles supporting the bridge abutments. The magnitude of this pressure cannot be reliably predicted, as proven theories are lacking and little experimental data has been published. Tschebotarioff in Leonard's "Foundation Engineering" suggests that, as a rough approximation, a triangular distribution of the lateral pressure be assumed in the soft clay, reaching a maximum value at the centre of the soft strata, and equal to:

$$P_{max} = K_0 \cdot \gamma \cdot H$$

where K_0 = earth pressure coefficient
at rest
= 0.6, assume

γ = density of fill
= 120 lbs/sq.ft.

H = height of embankment above
level of pile cap
= 15 ft., assume.

Hence, P_{max} was computed as 1.25 kips/sq.ft., and the maximum bending moment, at the middle of the 70 ft. length of pile in soft clay, assumed simply supported at both ends, was computed as 6.2×10^6 in.-lb., giving a maximum fibre stress of about 54,000 psi. to be added to the axial stress. This appears to be very high, but

- i. the method is probably over-conservative
- ii. deflection at the pile-cap reduces the bending moment
- iii. as the pile deflects, passive pressure is mobilized in the clay, reducing the bending moment.
- iv. The fixity at the pile cap reduces the bending moments.

G. EARTH PRESSURE ON PILES - cont'd.

-13-

Nevertheless, the computation may explain the observed movement of the abutment wall away from the bridge deck, which was reported. It is recommended to maintain careful measurements of any deflections. Should the movements continue, it may be necessary to slow down the placement of fill to allow the soil to consolidate gradually. In the worst case, should progressive movement give cause for alarm, it may be necessary to cut back the embankment some distance from the bridge and to provide an additional span as access to the bridge.

Yours very truly,

E.M. PETO ASSOCIATES LTD.,

C. F. Freeman

C.F. Freeman, P.Eng.,
Chief Engineer.

RK/hf

LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		
W.T.P.L.	WETTER THAN PLASTIC LIMIT		D.T.P.L.	DRIER THAN PLASTIC LIMIT
	A.P.L. ABOUT PLASTIC LIMIT			

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H.	SAMPLE ADVANCED HYDRAULICALLY	
	P.M.	SAMPLE ADVANCED MANUALLY	

SOIL TESTS

Q _u	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Q _{cu}	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q _d	DRAINED TRIAXIAL		

LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

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VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

W.T.P.L. WETTER THAN PLASTIC LIMIT

O.T.P.L. DRIER THAN PLASTIC LIMIT

A.P.L. ABOUT PLASTIC LIMIT

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

SOIL TESTS

QU	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
QCU	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL		

JOB NO. 65225

NOVEMBER 26TH, 1965

APPENDIX "A"UNCONFINED COMPRESSION TEST DATA

Hole No.	Sample No.	Depth	M.C. %	Density Wet	p.c.f. Dry	Degree of Saturation %	Void Ratio e	% strain at Failure	U/C shear Strength p.s.f.
1	2	2'4"-2'8"	20.5	116.4	96.7	74.0	.745	2	580
1	7	8'8"-9'	21.0	125	103.2	90.0	.632	20	1040
1	10	12'-12'4"	94.5	82	42.1	85.0	3.00	11	145
1	13	15'4"-15'8"		Not able to take test					
1	16	20'-20'4"	101.0	83.5	41.5	89.0	3.06	20	260
1	21	25'4"-25'8"	48.6	93.5	62.9	78.0	1.68	20	250
1	24	30'-30'4"	28.6	126	98.0	100.0	.721	20	280
1	28	35'-35'6"	28.3	124.2	96.8	100.0	.743	20	730

APPENDIX "A" - cont'd.

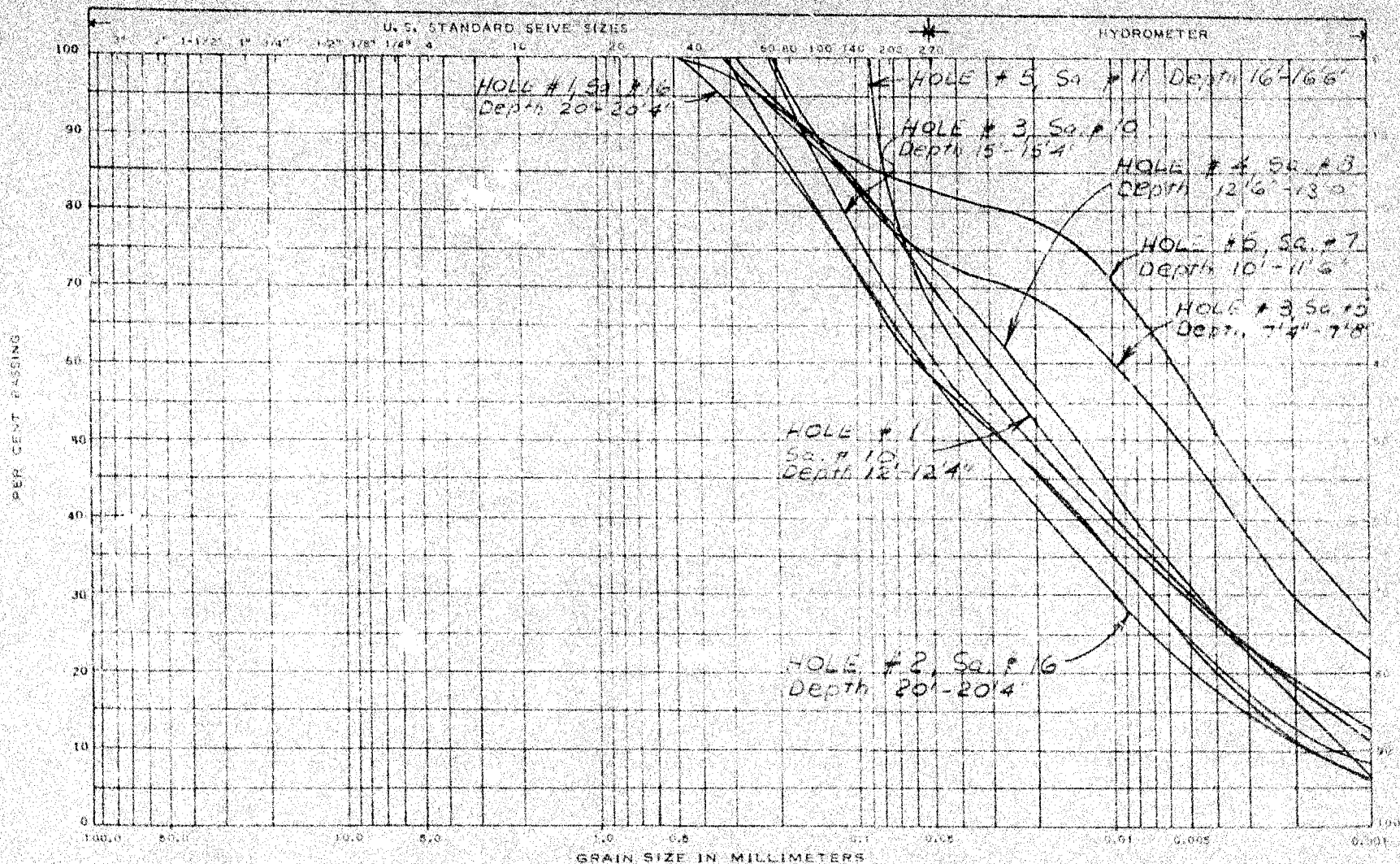
Hole No.	Sample No.	Depth	M.C. %	Density Wet	p.c.f. Dry	Degree of Saturation %	Void Ratio e	% Strain at Failure	U/C shear Strength p.s.f.
2	2	2'-2'4"	10.7	117.2	106.0	49.0	.592		
2	8	10'-10'4"	56.5	106.3	67.9	100.0	1.48	20	410
2	12	15'-15'4"	65.5	81.5	49.3	73.0	2.41	10	120
2	16	20'-20'4"	91.5	81.2	42.4	83.0	2.97	20	115
2	20	25'-25'6"	28.3	122.2	95.3	99.0	.770	20	520
2	24	30'-30'6"	27.3	125.2	98.4	100.0	.715	20	630
3	5	7'4"-7'8"	99.5	102.9	51.6	100.0	2.26	20	420
3	8	10'-12'4"	62.7	80	49.1	70.0	2.43	11	160
3	10	15'-15'4"	118.0	86.2	39.5	98.0	3.27	10	32
3	14	20'-20'4"	78.0	90	50.5	89.0	2.37	10	32
3	18	25'-25'6"	33.5	122.2	91.6	100.0	.897	20	490
3	22	30' - 6"	29.2	124.3	96.2	100.0	.809	20	355

Appendix "A" - cont'd.

Hole No.	Sample No.	Depth	M.C. %	Wet Density p.c.f.	Dry Density p.c.f.	Degree of Saturation %	Void Ratio e	% strain at Failure	U/C shear Strength p.s.f.
4	4	4'-5'6"	16.9	130.0	111.1	89.0	.515	7	475
4	6	8'-9'9"	27.0	112.2	88.5	80.0	.910	20	970
4	8	12'6"-13'	63.0	100.5	61.6	98.0	1.73	10	780
4	12	18'-18'6"	20.0	131.0	109.0	99.0	.545	20	1540
4	14	20'6"-21'	24.0	127.5	103.0	100.0	.635	20	480
4	17	25'-25'6"	30.0	124.5	95.7	100.0	.880	17	1100
4	21	30'30'6"	26.8	126.3	99.7	100.0	.804	20	1140
5	5	6'-7'6"	22.7	117.0	95.5	80.0	.770	20	1700
5	7	10'-11'6"	48.5	108.0	72.5	99.0	1.32	13	990
5	11	16'-16'6"	124.0	92.2	41.2	100.0	3.08	10	276
5	14	20'6"-21'	23.2	130.0	105.0	100.0	.645	10	890
5	17	25'-25'6"	27.3	122.4	96.2	98.0	.753	20	320

e. m. peto associates ltd.

Toronto 19, Ontario

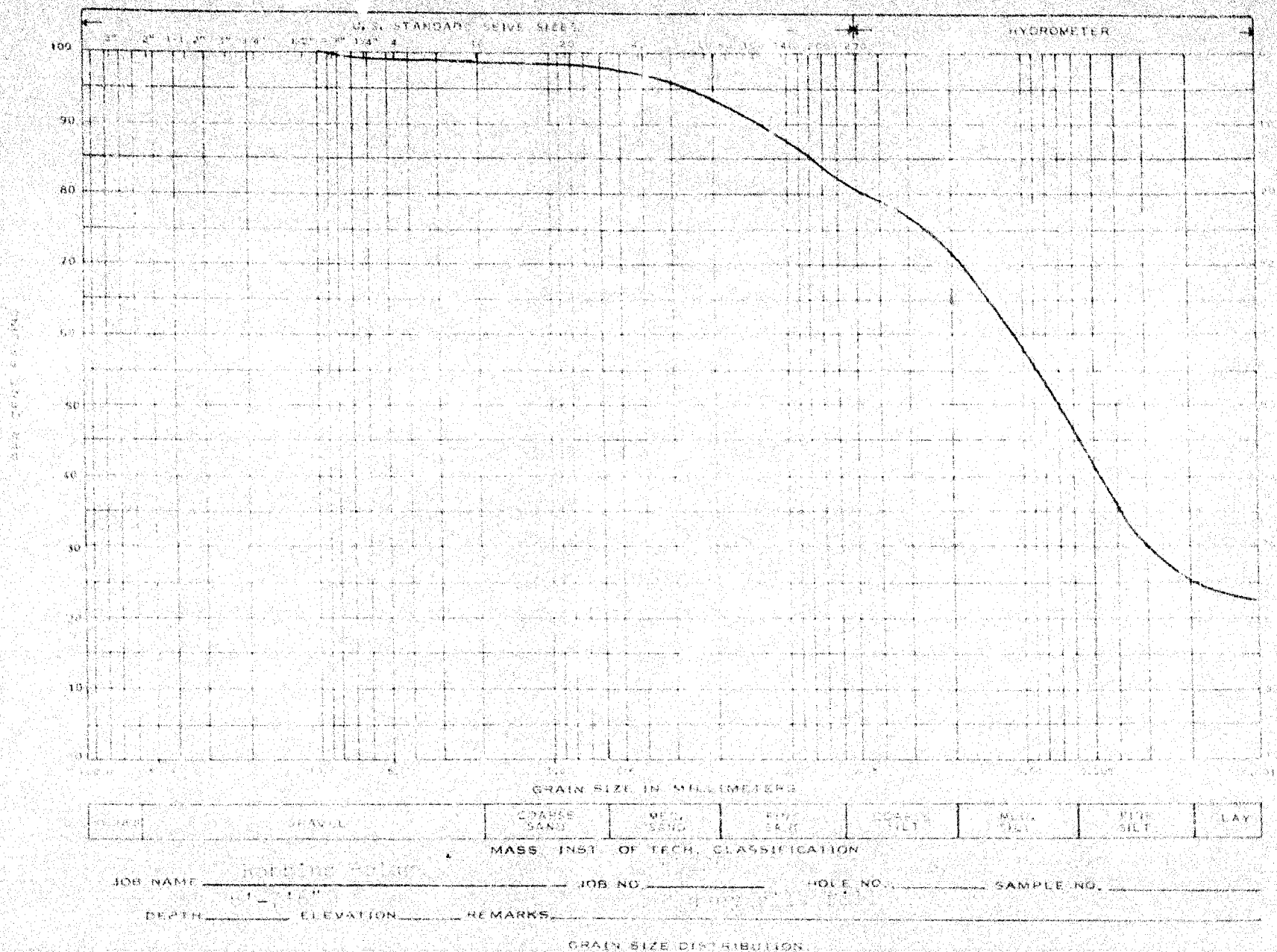


JOB NAME Roblin Bridge JOB NO. 1225 HOLE NO. _____ SAMPLE NO. _____

DEPTH _____ ELEVATION _____ REMARKS organic clayey silt to silty clay stratum

GRAIN SIZE DISTRIBUTION

Toronto 19, Ontario



CONSOLIDATION TEST RESULTS

BOREHOLE #1 SAMPLE #1A
DEPTH 13.8' - 16'
K = 8.7 PCF
W = 91.5%

VOID RATIO - e

2.260
2.100
1.940
1.780
1.620
1.460
1.300
1.140

APPLIED PRESSURE KIPS/SQ. FT. - p

UOD # 65215

E. M. DeTo Associates, Ltd.

0.1

1.0

10.0

100.0

CONSOLIDATION TEST RESULTS

BOREHOLE #1 SAMPLE #21
DEPTH 25' 8" - 26'
X = 91.2 P.C.F.
W = 75.7%

VOID RATIO $e \rightarrow$

1.720
1.560
1.400
1.240
1.080
0.920
0.760
0.600

Job #65025

Empeto Associates Ltd.

APPLIED PRESSURE KIPS/SQ. FT. \rightarrow

0.1

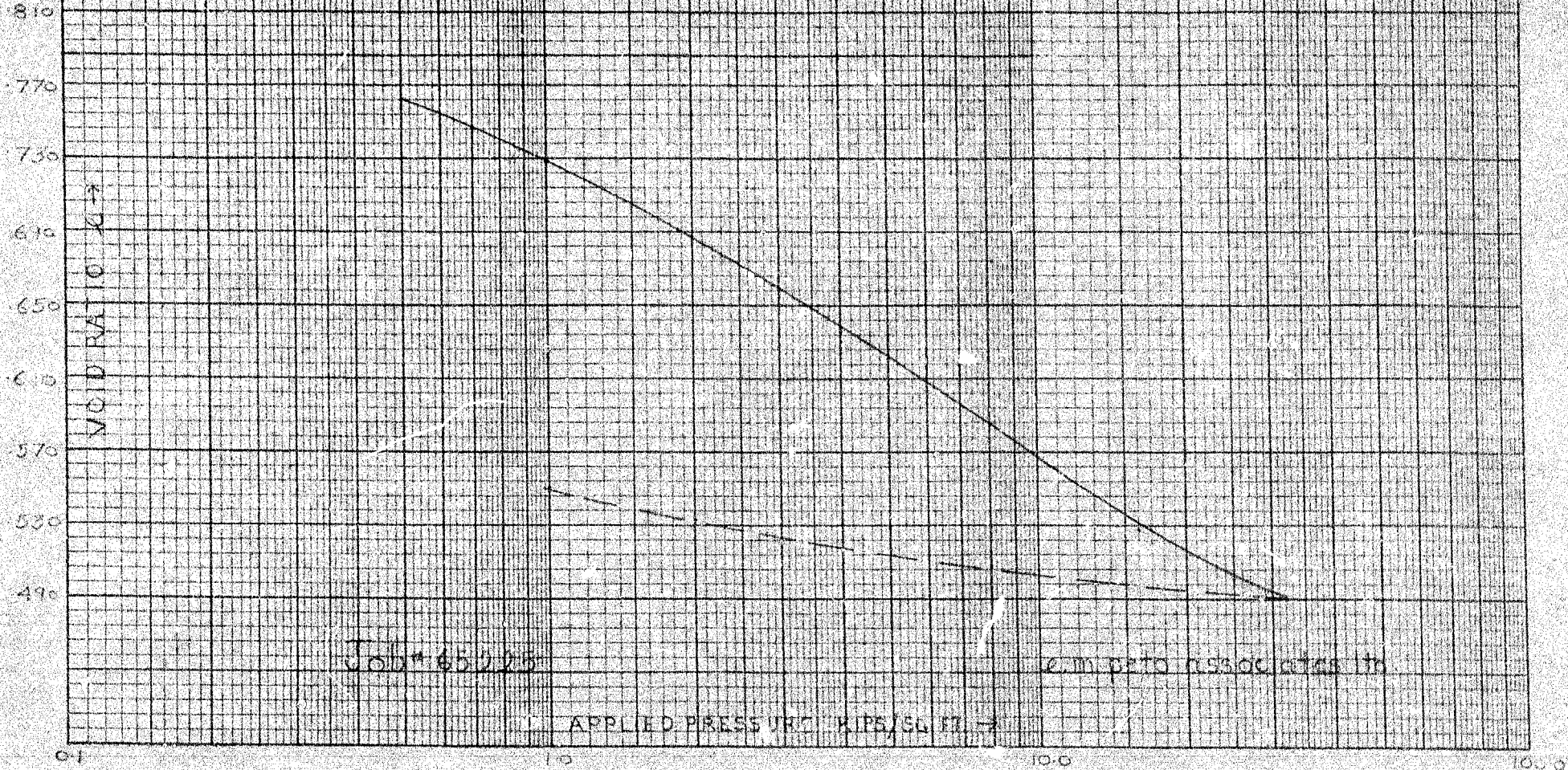
1.0

10.0

100.0

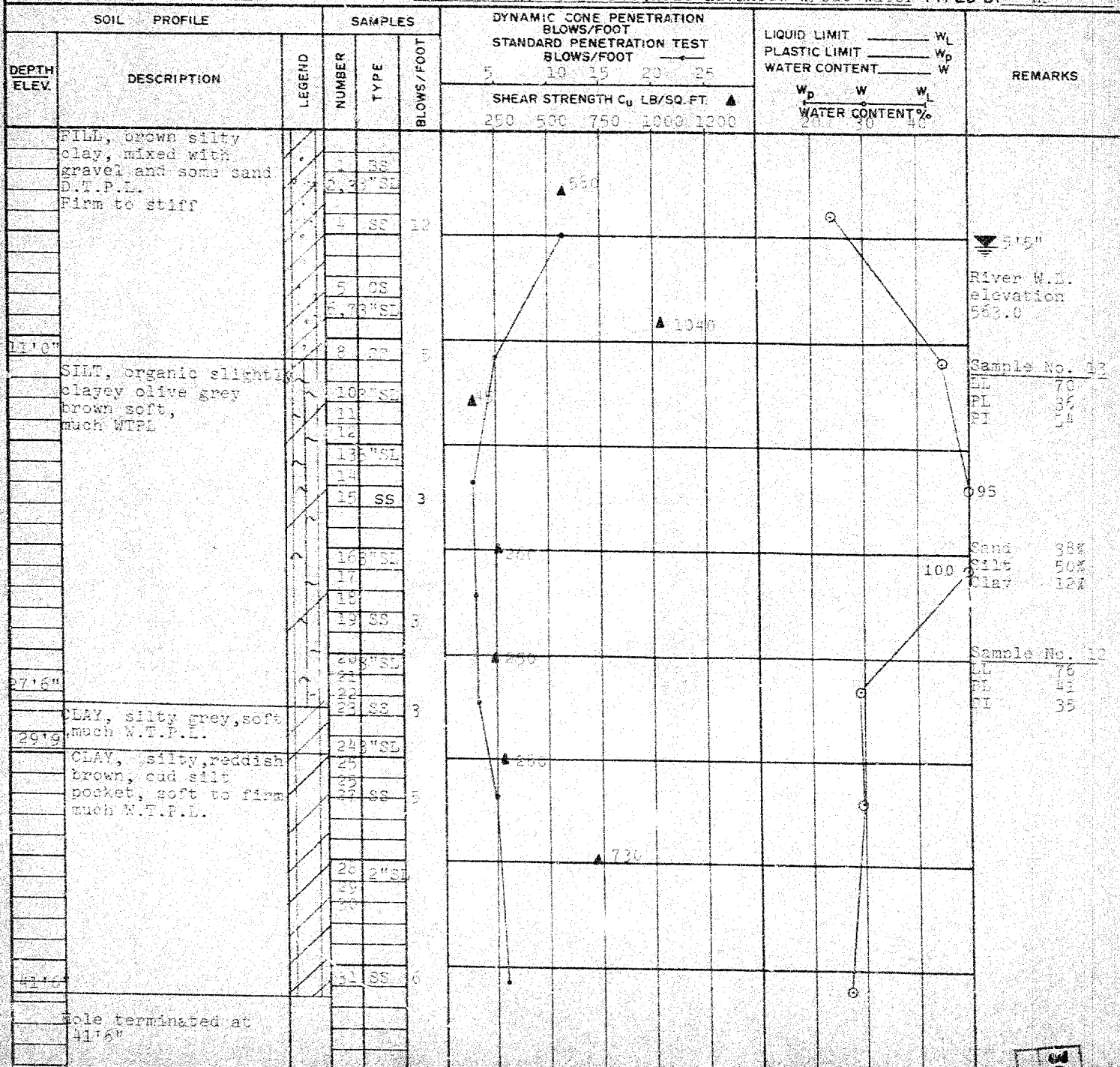
CONSOLIDATION TEST RESULTS

BOREHOLE #1 SAMPLE #26
 DEPTH 30' 8" - 31'
 8 - 120 3/8" C.U.T.
 LI 27.5%
 REDDISH GREY SILTY CLAY



RECORD OF BOREHOLE NO. 1

JOB NO. 65225 JOB NAME Robbins Bridge - Welland River TECHNICIAN JL
 BORING DATE 4,5 Aug. '65 CLIENT Welland County ENGINEER GFF
 GROUND ELEV. 574.5 (geodetic) BOREHOLE TYPE 4" casing, to 30 ft. hole advanced w/out water TYPED BY HP



e.m.peto associates ltd.

RECORD OF BOREHOLE NO. 2

Consulting soil engineers

JOB NO. 65225

JOB NAME Robbins Bridge - Welland River

TECHNICIAN JL

BORING DATE 5 Aug. '65

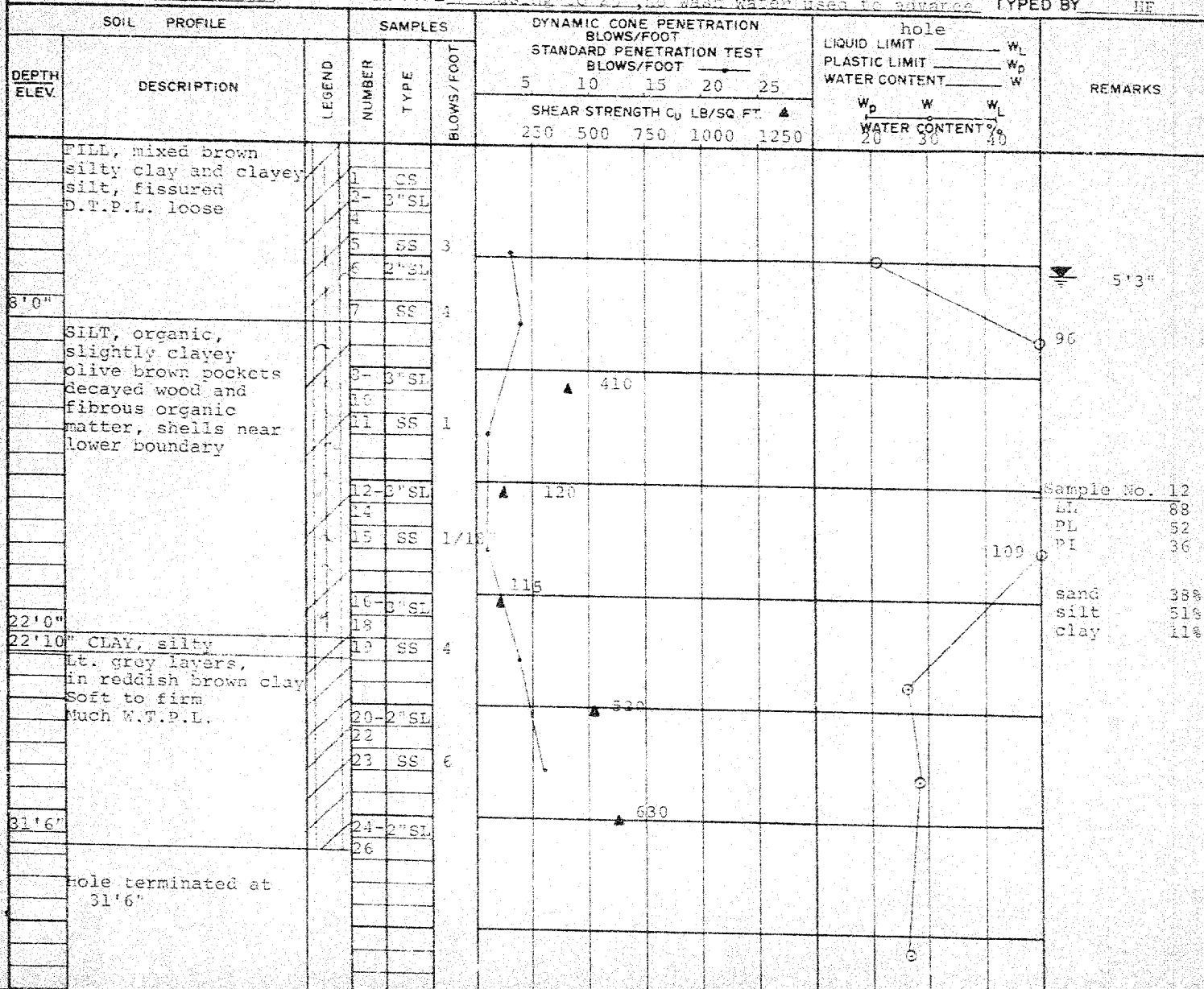
CLIENT Welland County

ENGINEER CEN

GROUND ELEV. 559.2

BOREHOLE TYPE 4" casing to 25', no wash water used to advance

TYPED BY DE



e. m. peto associates ltd.

RECORD OF BOREHOLE NO. 3

Consulting soil engineers

JOB NO. 65225

JOB NAME Robbins Bridge - Welland River

TECHNICIAN JL

BORING DATE 9 Aug. 1965

CLIENT Welland County

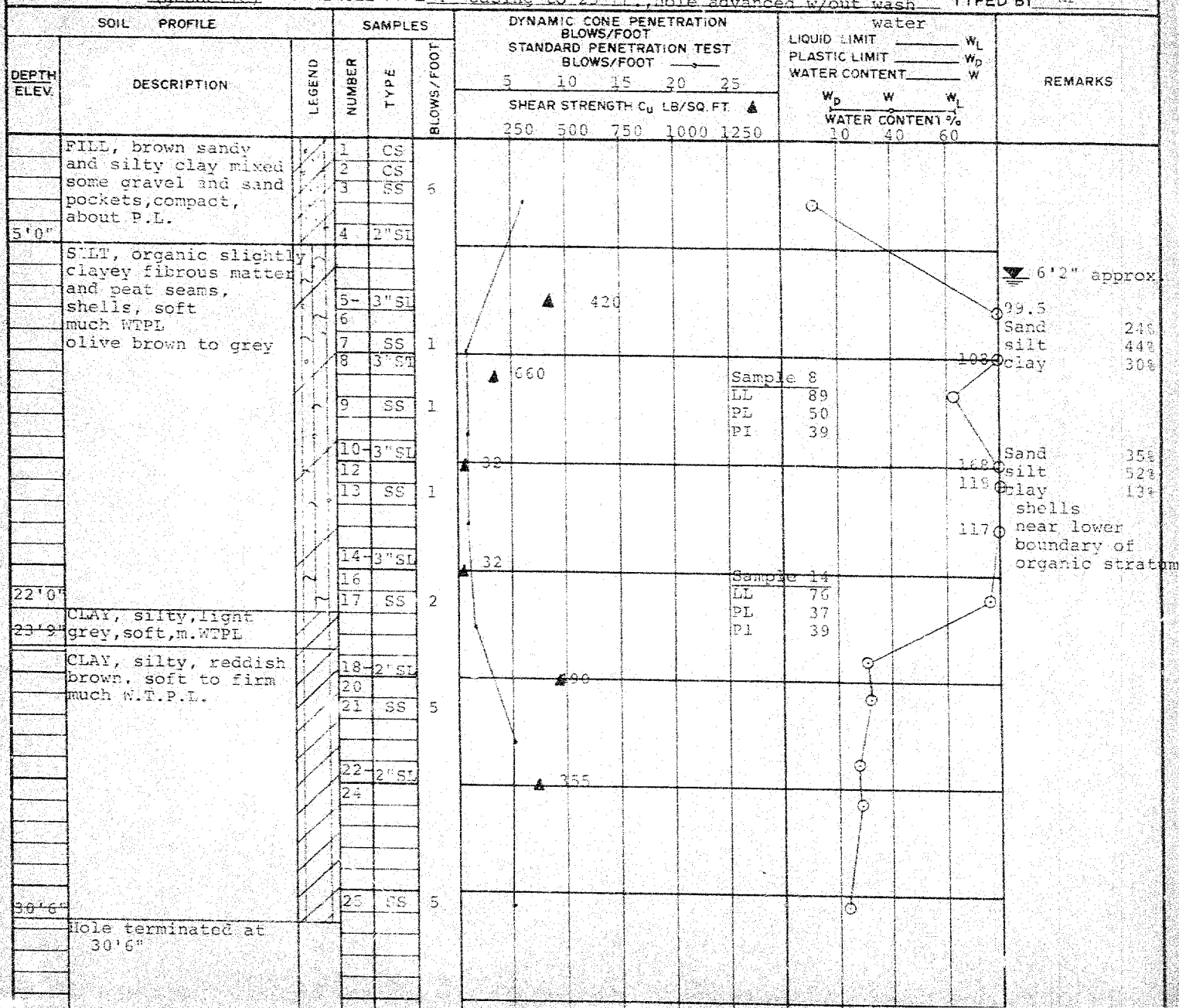
ENGINEER CFF

GROUND ELEV. 568.3

(geodetic)

BOREHOLE TYPE 4" casing to 25 ft., hole advanced w/out wash

TYPED BY NF



e. m. peto associates ltd.

RECORD OF BOREHOLE NO. 4

Consulting soil engineers

JOB NO. 65225

JOB NAME Robbins Bridge, Welland River

TECHNICIAN JL

BORING DATE 6 Aug./65

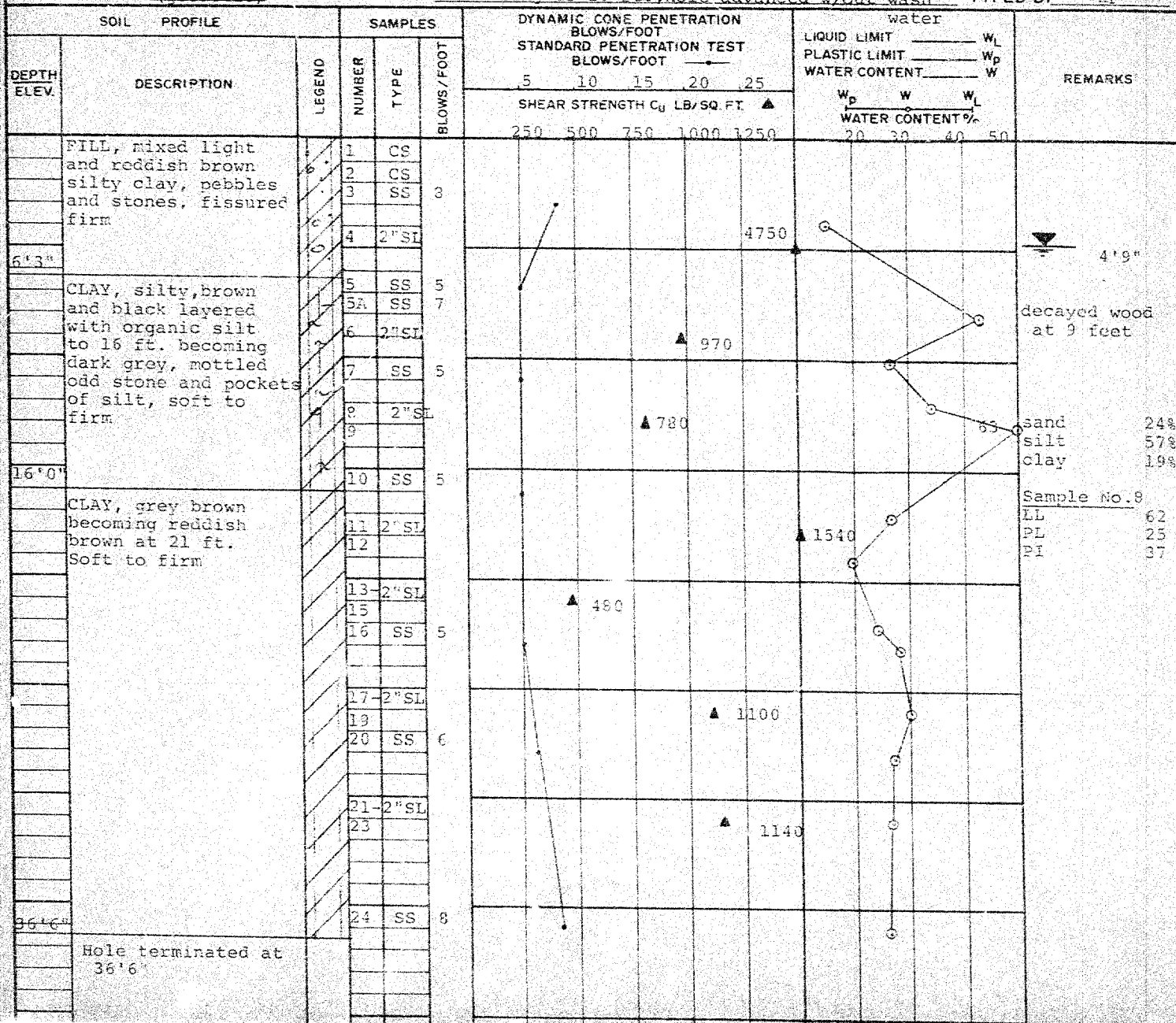
CLIENT Welland County

ENGINEER CFF

GROUND ELEV. 569.3
(geodetic)

BOREHOLE TYPE 4" casing to 20 ft., hole advanced w/out wash

TYPED BY HF



e. m. peto associates ltd.

RECORD OF BOREHOLE NO. 5

Consulting soil engineers

JOB NO. 65225

JOB NAME Robbins Bridge - Welland River

TECHNICIAN JT

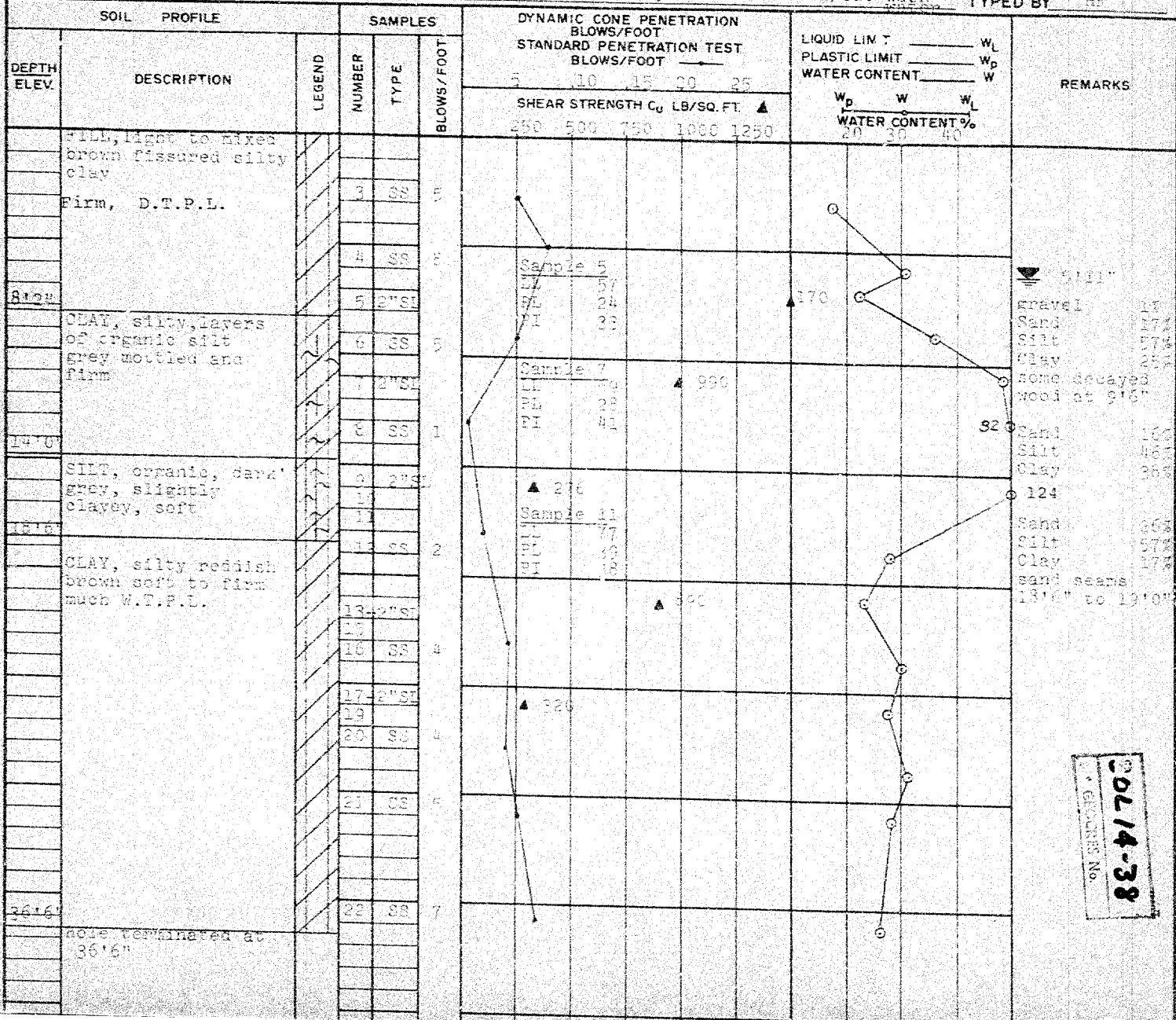
BORING DATE Aug. 7/65

CLIENT Welland County

ENGINEER CFF

GROUND ELEV. 569.7 (Geo. BOREHOLE TYPE 4" casing, to 25', hole advanced w/out wash

TYPED BY HF



DOCUMENT MICROFILMING IDENTIFICATION

GEOCREs No. 30614-38

DIST. 4 REGION CENTRAL

W.P. No. _____

CONT. No. _____

W. O. No. _____

STR. SITE No. 34-54

HWY. No. _____

LOCATION LOT 26 CON. 7 WHARFLES

TWP.

OVERSIDE DRAWINGS TO BE INCLUDED WITH THIS REPORT 1

REMARKS: Documents to be unrolled after

microfilmed

