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LOCATION HERON RD/C.P.R.
GRADE SEPARATION

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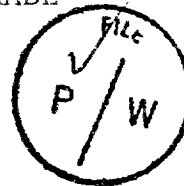
31G5-117
GEOCRES No.

SUB-SURFACE INVESTIGATION
HERON ROAD/C.P.R. GRADE
SEPARATION

for

CITY OF OTTAWA

c/o M.M. Dillon & Co.Ltd.



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JOB NO. 67 F98

NOVEMBER, 1967

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1287 caledonia road,

TORONTO 19, ONTARIO

Telephone: 789-1128

November 3, 1967.

City of Ottawa,
c/o M. M. Dillon & Co. Ltd.,
Consulting Engineers,
280 Metcalfe Street,
Ottawa 4, Ontario.

Attention: Mr. J.H. Kearney, P.Eng.

Dear Sirs:

Re: Sub-surface Investigation
Heron Road/C.P.R. Grade
Separation
Ottawa, Ontario

We are pleased to submit herewith six copies of our report covering this investigation.

We regret the delays involved in the production of the complete report, these being due in part to the time consumed in computer analysis of slopes.

We have made recommendations concerning the founding of the C.P.R. bridge on piles to rock at approximately 130 ft., the safe depths of cut for various side slopes and for drainage schemes to control ground water and assist stability.

In addition, we have discussed such matters as pavement design, installation of storm water sewer and general construction difficulties.

While we believe this report to be complete within our terms of reference, we appreciate that due to the complexity of this project, some further queries are likely. We look forward

to the privilege of being of further assistance with these matters.

In particular we are examining the question of the use of excavated material for bulk fill to form embankments for the adjacent southern access freeway project and shall let you have our opinion by letter, when we have been able to reach some conclusions.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

A handwritten signature in cursive script, appearing to read "C. F. Freeman".

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

ANSB/jw

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GRADING CURVES

BOREHOLE LOGS

SITE PLAN AND PROFILES

1. INTRODUCTION

We were requested by M. M. Dillon & Co. Ltd., acting on behalf of the City of Ottawa, to carry out a sub-surface investigation at the junction of Heron Road and the C.P.R. tracks in Ottawa. The purpose of the investigation was to provide sufficient data, concerning the sub-surface conditions, to enable a grade separation to be designed. Although Heron Road may be taken either over or under the railway, no change in elevation of the rail tracks is acceptable.

While this investigation is solely concerned with the grade separation of Heron Road and the C.P.R. tracks, the proximity of this crossing to the proposed interchange of the Southern Entrance Freeway and Heron Road is such that the solution applied to this grade separation affects the Freeway interchange. Aesthetic considerations make it desirable for the Freeway to pass over Heron Road, which in turn requires that Heron Road be taken under the C.P. Railway rather than over it. Hence, unless soil conditions are such as to strongly favour an overpass, other considerations make an underpass desirable.

Preliminary soil studies have already been carried out in this vicinity by E.M. Peto Associates, both for the Rideau Heights Development (Report 60239) and for the Southern Entrance Freeway (Report 6249/50). The findings of these reports and those of the Functional report on the Southern Entrance Freeway, 1962 by M. M. Dillon & Co. Ltd., are of relevance to the present investigation.

2. FIELD WORK

2.1 Location and Elevation of Boreholes: The boreholes were located with reference to the centre of the intersection of Heron Road and the C.P.R. lines and to the centre line of either the railway or Heron Road. Borehole 5 was the exception, being referred to Bronson Avenue and the C.B.C. access road. Elevations were taken by one of our technicians and were referred to geodetic bench mark No. 2794 (Elevation 227.74) located at the south-east corner of the Dunbar bridge.

2. FIELD WORK - cont'd

Thirteen boreholes were put down, numbers 1 to 4 being taken to bedrock, at the proposed pier and abutment locations. The bedrock was cored at each of these four locations. Borehole 5 was put down to the east of the C.B.C. building to allow consideration of the possible effects on the stability of that building resulting from construction of access ramps north of Heron Road and west of Bronson Avenue. Of the remaining boreholes, numbers 6 to 12 serve to show conditions over the length of Heron Road, whose elevation will be changed to facilitate passing beneath the C.P.R. without excessively steep gradients. This distance was taken from the tentative Heron Road profile indicated in the Dillon Functional report.

Borehole 13 was located just west of Saw Mill Creek on the south side of Heron Road and was taken to sufficient depth to give an indication of conditions which would be encountered if the storm water sewer, serving the Bronson Avenue-Freeway-Heron Road interchange area, were to feed into Saw Mill Creek.

2.2 Boring: Both a standard diamond drilling rig converted for soil sampling and a track mounted continuous flight auger rig were used for the boring. The work was carried out by two of our field crews, together with two field technicians.

A 3½ inch diameter auger was used for boreholes 2, 5 to 13 inclusive. "H" casing was used throughout, reducing to BX or AX for diamond drilling of the bedrock.

2. FIELD WORK - cont'

The standard rig was used for boreholes 1, 3 and 4, 4 inch diameter pipe being used to case the hole, reducing to BX or AX for coring.

In each borehole, approximately the upper 15 ft. was sampled by taking Shelby tube samples and carrying out standard penetration tests. Below this level less than 1 blow was required to drive the split spoon 18 inches, thus rendering this test useless. Below 15 ft., therefore, in boreholes other than 3, 4 or 5, an in situ vane shear test was carried out and a Shelby tube sample obtained at approximately 8 ft. intervals until stiffening of the soil occurred at about 85 ft. below surface.

In boreholes 3 and 4, no sampling was carried out between depths of 15 ft. and 100 ft. In boreholes 1 to 4 standard penetration tests were resumed at approximately 100 ft. and carried out at regular intervals down to bedrock. Occasional boulders were encountered below about 85 ft., requiring diamond drilling to advance the hole.

In borehole 5, little sampling was carried out below 15 ft., a few in situ vane tests being performed to check consistency. Bedrock was assumed on refusal to the augers, but was not proved.

In many cases the Shelby tube samples were obtained using a piston sampler in an endeavour to minimize sampling disturbance.

3. SUB-SURFACE CONDITIONS

Geologically, the site is located on the Ottawa valley clay plain. The silty clay deposits were deposited in the marine environment of the late glacial Champlain Sea. The underlying till deposits with boulders were laid down during the Wisconsin glaciation period. The drift rests upon Billings black shale bedrock which dates from the Ordovician age.

- 3.1 Fill: In boreholes 6, 7 and 8 to depths of 2 ft. 6 inches; 4 ft. 6 inches and 3 ft. 0 inches mixed fill consisting of silty clay with sand, stone and cinder content was found. This fill probably results from the time of construction of the eastbound slip-road in front of the Department of Insurance building. Fill was also found in borehole 13 to a depth of 4 ft. 6 inches. This fill is part of the embankment built up to carry Heron Road across the valley of Saw Mill Creek. Two feet of till was also found in borehole 5, drilled in the C.B.C. lawn.
- 3.2 Topsoil: A few inches of topsoil were found in borehole 8 above the fill and in boreholes 9, 10, 11, 12 above the naturally occurring clay.
- 3.3 Grey-Brown Silty Clay: At every location drilled, the deposit of marine clay was penetrated. Approximately the upper 10 ft. of this clay was found to be weathered to a grey-brown colour and to offer resistance to the standard penetration test of about 15 blows/ft. near ground surface, decreasing to 3 or 4 blows towards a depth of 10 ft. This stratum was not present in boreholes 2 and 4, which were drilled in the C.P.R. cut, adjacent to the railway lines.

3. SUB-SURFACE CONDITIONS - cont'd

3.4 Grey Silty Clay: Beneath the weathered zone extending down to between elevations 202 ft. and 213 ft., (between 38 ft. and 44 ft. in the borehole), is a grey silty clay, which is essentially similar to the grey-brown material, but is unweathered. The transition from weathered to unweathered material is readily discernible (a) as a colour change and (b) as the standard penetration resistance falls to less than 1 blow/ft. In some cases these two criteria are not exactly compatible and the profiles show colour changes; thus, the uppermost penetration value in the grey clay may be shown as 2 blows, which is nevertheless very soft.

The standard penetration test was discontinued in this stratum as being no longer of value. Strength determinations rely on in situ vane tests and laboratory tests on Shelby tube samples.

3.5 Blue-Grey Silty Clay: This stratum was found in the five deep boreholes and also in borehole 13, which was commenced at a relatively low elevation (230.9) adjacent to Saw Mill Creek. It is a characteristic blue-grey colour with black organic specks. This black mottling is characteristic of fresh samples obtained from below the water table. Previous work, by the Division of Applied Biology of the National Research Council, has shown the mottling to be due to the presence of colonies of anaerobic bacteria.

This stratum seems to terminate more or less uniformly at elevation 160 ft. or at a depth of approximately 94 ft. in boreholes 1 to 5 inclusive. It was not fully penetrated by borehole 13, which terminated in this stratum at 44 ft. 9 inches.

3. SUB-SURFACE CONDITIONS -- cont'd

3.6 Grey Clayey Silt: Below elevation 100 ft. approximately a change occurs. The colour of the material reverts from blue-grey to grey and the black specks cease to occur. Although this stratum has been named clayey silt, this may be something of a misnomer as the particle size distribution of a single sample from this stratum does not show significant variation from the other Leda clay components above. In fact, the clay size particles predominated in the sample tested. It should be mentioned that the grain size distribution of Leda clay is quite variable and that, in some samples the clay size predominates whereas in others the silt size does.

This stratum is something of a transition stratum. It properly belongs to the sequence known as Leda clay, but is beginning to have more shearing resistance, presumably due to lower moisture content, which in turn results from drainage above the bedrock.

3.7 Sandy Silt Till: The transitional clayey silt stratum is only 6 to 12 ft. in thickness and is underlain by a sandy silt till with gravel seams and pockets. This stratum is initially loose to compact but becomes compact to dense with depth and occasionally very dense. Standard penetration tests were resumed in this stratum. In this stratum large random boulders were encountered, which had to be cored to allow the hole to be advanced.

3.8 Black Clayey Silt Till: Overlying the bedrock and beneath the sandy silt till is a layer of silt till which has become black, presumably from contact with the upper weathering zone of the rock. This stratum is approximately 7 ft. thick at borehole 2

3. SUB-SURFACE CONDITIONS - cont'd

reducing to 1 ft. thick at borehole 1. In boreholes 2 and 3 this stratum was found to contain water under artesian pressure. Presumably the same should be true of boreholes 1 and 4, where it is possible that the use of wash water resulted in lack of observation of this phenomenon.

3.9 Black Shale Bedrock: Bedrock was encountered and proved in boreholes 1 to 4 at depths of 119 ft. 6 inches; 130 ft. 6 inches, 126 ft. 5 inches and 133 ft. 0 inches respectively. Five feet of core was recovered in boreholes 1, 2 and 3 and 4 ft. in borehole 4.

3.10 Water Conditions: Water levels were observed after completion of the boreholes and as the drilling programme occupied considerable time, many of the boreholes were open long enough for the long term condition to have been observed.

In boreholes 2 and 3 an artesian condition was found above the rock. In borehole 3 when the casing was being pulled from 120 ft., water rose 3 to 5 ft. above ground level.

Observed water levels are given in Table I.

4. LABORATORY TESTING

Some Atterberg limit tests and particle size distribution analyses were carried out to assist classification. The results of these tests are given in Tables II and III. A number of quick

4. LABORATORY TESTING - cont'd

undrained triaxial tests were carried out and the strengths so obtained, compared with the in situ vane shear strengths. These are indicated on the borehole logs and are given in Table IV.

Consolidated-undrained triaxial tests with measurement of pore pressure were carried out in order to assess values of effective stress parameters for use in long term stability analyses. The results of these tests are summarized in Figure 1.

Consolidation tests were also carried out on a number of samples, in order to examine the stress history of the soils and to provide settlement parameters. The void ratio--pressure relationships are given at the end of this report.

5. RAILWAY BRIDGE

Although it is possible to consider carrying the bridge on conventional isolated footings, the allowable bearing capacity of approximately 1 ton/sq.ft. is thought to be too low to allow the anticipated bridge loads to be carried on sensibly sized footings. Further objections to the use of footings are (i) some settlement would ensue, probably of a magnitude incompatible with railway working and (ii) the excavations for such footings would seriously complicate the question of slope stability for the road cut and would in themselves be difficult to install without unacceptable disturbance within the potential failure arc of the slope.

It is recommended, therefore, that the bridge structure be carried on end-bearing piles taken to bedrock. It is thought that a minimum displacement driven pile would be suitable, for example, a steel "H" pile.

5. RAILWAY BRIDGE - cont'd

However, there is evidence that steel embedded in Leda clay is subject to corrosion, as discussed below in paragraph 5.1. An alternative proposal would be to use a precast concrete pile such as the Johnson Herkules pile which has been used successfully in similar soils in Scandinavia and Eastern Canada including the Gatineau area. Should such a pile be used, the question of corrosion of the steel joints between the pile sections ought to be considered, but in applications of these piles to date, cathodic protection has not been considered necessary.

- 5.1 Corrosion: There is some reason to believe that steel immersed in Leda clay may be subject to corrosion. Data supplied to us by the National Research Council based on tests adjacent to Riverside Drive bridge at Heron Road and interpreted in the light of Norwegian experience indicates that these soils are of medium to intermediate corrosivity. This means that corrosion of the order of .02 mm per year may be expected. This level may be considered harmful, with failure for normal piles under normal loads occurring within 150 years.

Although the arguments in favour of cathodic protection are not complete, it is suggested that consideration might be given to its use as a precautionary measure. It is believed that such measures have been taken on at least one structure in Ottawa, but details are not forthcoming.

- 5.2 Installation Difficulties: No difficulty will be encountered in penetrating the clay to approximately 90 ft. However, boulders were encountered in boreholes 1 and 2 at depths of 113 ft., 97 ft. and 101 ft. These were of sufficient magnitude to cause apparent refusal to pile driving. Piles should be taken to rock. The

5. RAILWAY BRIDGE - cont'd

four boreholes taken to bedrock may be used to assess the likely depth to rock. Where apparent refusal is reached at an earlier depth, the pile should be relocated and redriven, unless shown by a pile test to be acceptable.

The actual elevation of the rock surface has been verified at each of the borehole locations 1 to 4 and on the appended profile the rock surface has been shown varying linearly between these measured elevations. It must be stated that this is conjectural and that as this area is known to be faulted, it is possible for the actual rock surface to deviate from that portrayed in the section.

5.3 Design Requirements: The piles should be designed to withstand the whole load solely in end bearing, no account being taken of any frictional component of resistance. During driving of the piles the clay immediately surrounding the piles will be remoulded and consequently, as the clay is of high sensitivity, will lose most of its shearing resistance. Although some strength will be regained with time, this should be neglected for design purposes and will serve merely to increase the design safety factor.

Although in the case of an underpass it is unlikely that any negative skin friction will act on the piles, it should be pointed out that in the case of a bridge founded on piles and having approach embankments, the settlement of the embankments--which would be large were the preconsolidation pressure of the Leda clay exceeded--would transmit negative skin friction to the piles, which would need to be designed to withstand this. This situation is likely to arise at the Bronson Avenue/Heron Road overpass

5. RAILWAY BRIDGE - cont'd

Base dimension is highly desirable?

Where lateral thrust is likely to come onto the piles, as for example at the abutments, the use of raker piles is recommended.

6. CUT

In order to lower the road to pass beneath the railway, it will be necessary to excavate a substantial cut. The actual depth of cut required is not known exactly, but it is thought that 25 to 30 ft. will be of the right order. Such a cut in Leda clay presents stability problems. As it was required to examine a number of cuts having different depths, slopes, etc., the only way in which a proper analysis could be undertaken was to use an electronic digital computer. Accordingly, data was prepared for analysis at the I.B.M. Scientific Data Centre in Toronto using a programme based on that originally written by Little and Price (Geotechnique 1958). The analysis uses the method of slices and applies to the long term condition based on effective stresses. The factor of safety F is given by: -

$$F = \frac{1}{\Sigma W \sin \alpha} \cdot \Sigma \left[\left\{ c' b + (W - ub) \tan \phi' \right\} \cdot \frac{\sec \alpha}{1 + \frac{\tan \phi' \cdot \tan \alpha}{F}} \right]$$

The effective stress parameters used were based on the laboratory test results interpreted in the light of work reported by Crawford (National Research Council). They are $\phi' = 20^\circ$ and $c' = 300$ lbs./sq.ft. The water table was taken as being 3 ft. below existing ground surface and it was assumed that the excavation of the cut was an instantaneous process compared with the time required for significant pore pressure dissipation in the Leda clay. Hence, for analysis the water table was taken as being coincident with the slope face below 3 ft. and also with the bottom of the cut.

6. CUT - cont'd

Pore pressures were then calculated from the piezometric surface. It is believed that, provided that the drainage recommendations are successfully followed, this represents the most unstable case which will occur. It should be noted, however, that if for some reason pore pressures are allowed to exceed these values, a more unstable case will result.

6.1 Water Table: In carrying out the analyses the water table in every case has been taken as being parallel to ground surface at a depth of 3 ft. prior to excavation and that after excavation remains thus and follows the surface of the sloping face of the cut and the bottom of the cut. This will be the state of affairs immediately on completion of the excavation, due to the low permeability of the clay. With time a new equilibrium water surface will result. This will generate lower pore pressures on the failure surface than the configuration assumed and in the long term should add slightly to the factor of safety.

6.2 Slopes Analyzed: The results of the stability analyses are summarized in the graphs given in Figures 3 to 7 inclusive. It is considered realistic, for slopes carrying no surcharge, to assume that tension cracks will develop. The depth of such tension cracks may be estimated from the relationship: -

$$Z_c = \frac{2c \times \sqrt{N\phi}}{\gamma}$$

Using $c' = 300$ lbs/sq.ft., $\phi' = 20^\circ$ and $\gamma = 105$ lbs/ft.³ this gives a theoretical depth $Z_c = 8.2'$. A round figure of 8 ft. was adopted for computation.

For slopes carrying a surcharge, the depth of tension cracks is given by

6. CUT - cont'd

$$Zc\phi = \frac{(2c \times \sqrt{N\phi}) - q}{\gamma}$$

This means that no tension cracks will develop if $q = 860$ lbs/sq.ft. or a height of surcharge of 6.4 ft. at a density of 135 lbs/ft.³. As any useful height of embankment required will probably exceed 6.4 ft. in height, tension cracks were assumed not to develop when surcharged cuts were analyzed.

6.2.1 Cut Without Surcharge: Slope stability analysis is largely a matter of trial and error and therefore the more potential failure surfaces which are analyzed, the more certain one may be of having located the most critical surface. However, consideration of cost makes it desirable to attempt to predict the radius and centre of the critical circle for each configuration of slope analyzed. Initially a cut of 2.5:1 to depth 25 ft. was considered in detail and it was found that a failure circle tangent to approximately 1.4 times the depth of cut, below ground surface, was critical. Also a variation of ± 5 ft. in depth of failure circle produced a variation in factor of safety of only 1.5% and it was therefore assumed for all cuts, not carrying surcharge, that failure surfaces tangent only to this level need be considered, thus eliminating a number of other trial surfaces, without dangerous loss of accuracy. However, as only one level of tangency has been investigated for each slope, it is possible that more critical circles might exist. It is, therefore, important that the slopes finally selected be checked for stability at various levels of tangency. 2

6. CUT - cont'd

Slopes of 2.5, 3.0, and 3.5 horizontal to 1 vertical were analyzed for depths of cut of 25, 30 and 35 ft., all having tension cracks to 8 ft. below existing ground surface. These results are summarized in Figure 3. It is recommended that a factor of safety of not less than 1.25 be adopted, therefore, the maximum depths for each slope are 2.5:1, 25 ft.; 3.0:1, 30 ft.; 3.5:1, 37½ ft.

Figure 4 indicates the reduction in factor of safety arising from the incidence of tension cracks to a depth of 8 ft. for a slope of 2.5:1 to various depths of cut.

The effect of depth of tension cracks on reduction in factor of safety was investigated and is shown in Figure 5. The rate of decrease in factor of safety, for a given slope, increases as depth of tension cracks increases. As has been shown, the relevant depth in this case is 8 ft.

6.2.2 Cuts With Surcharge: As it is possible that, in the required configuration of roads in the area of Sawmill Creek Parkway - Heron Road interchange, it will be necessary to have ramps rising on embankment placed near the edge of the Heron Road cut, this matter has been examined.

First, a surcharge commencing immediately at the top edge of the cut, was considered. The surcharge was considered to have a density of 135 lbs/ft.³ and to contribute nothing to the

6. CUT - cont'd

shearing resistance on the potential failure surface. The surcharge was taken as rising from the crest of the cut at a slope of 2 horizontal to 1 vertical. Slopes of 2.5 to 1 and 3.0 to 1, each to a depth of 25 ft. were analyzed for various heights of surcharge and the results summarized in Figure 6. These show that factor of safety decreases sharply as height of surcharge increases and that to maintain an adequate factor of safety (1.25) the maximum permissible heights of surcharge would be 2½ ft. and 5 ft. on slopes of 2.5:1 and 3.0:1 respectively. These values are not strictly valid, as they have been obtained assuming zero depth of tension cracks, a condition not satisfied until the height of surcharge is 6.4 ft. However, the conclusion may be drawn that no useful height of surcharge may be placed immediately on the crest of a cut to 25 ft. or greater depth.

Having reached this conclusion, the possibility of increasing factor of safety by moving the toe of the embankment back from the crest of the cut, was examined for a slope of 2.5:1 cut to 25 ft. Heights of surcharge of 10, 20 and 30 ft. were considered for distances between crest of cut and toe of embankment of 5, 12½ and 20 ft. The results of these analyses are summarized in Figure 7. These show that for separations of up to 20 ft. between crest of cut and toe of embankment, very little increase in factor of safety is achieved (although the failure surface is changed). The only positive conclusion to be drawn from these results is that a surcharge not exceeding 10 ft. in height may be placed not closer than 20 ft. from the top edge of a 2.5:1 cut to 25 ft. without reducing the factor of safety below 1.25. In addition various negative conclusions may be drawn.

7. DRAINAGE

Recent slides in the Ottawa area (see for example Crawford and Eden on "Stability of Natural Slopes in Sensitive Clay" Proceedings ASCE July, 1967) have underlined the importance of pore pressures in the region of a potential slip surface. Although most slopes which are normally required are quite stable when analyzed in terms of total stresses, as pore pressures change, so do effective stresses and in time failure may be induced. The foregoing comments indicate (i) that stability of slopes should be checked in terms of effective stresses and (ii) adequate drainage should be provided to ensure that pore pressures assumed for stability analysis are not exceeded.

The critical area for pore pressure development is around the toe of the slope. It is proposed that adequate drainage measures be undertaken to control development of pore pressures and, hence, to assist greatly in the establishment and maintenance of stable slopes.

7.1 Drainage to Assist Slope Stability: When a cutting is made such that its bottom is below the water table prior to excavation, equilibrium of the ground water is disturbed and flow occurs to establish a new equilibrium compatible with the changed conditions. Once the cutting has been excavated, water tends to flow under gravity towards the cut, which tends to act as a sink. This rate of flow is a function of permeability, which in the case of Leda clay is very low. There may be considerable local variation in permeability caused by the variable presence of sand particles.

If seepage is allowed to occur without check, it is probable that sufficient seepage forces would develop on the potential failure surface to reduce the shearing resistance to the point at which failure occurs. The logical step to prevent this is to

7. DRAINAGE - cont'd

control seepage by intercepting it with drains before it can reach the potential failure zone. The actual choice of drainage system will depend on cost and feasibility of installation. The first of these is beyond the scope of this report and, although some comment will be made on the second, ultimately this will depend on the availability of suitable equipment and of suitably experienced contracting personnel.

Three theoretical schemes have been worked out which should ensure full control of ground water and seepage. However, as it appears likely that all of these would prove unacceptable on the grounds of either cost or technical difficulty, it is not proposed to present details here. Accordingly, only the basis of each scheme is presented, and a very much simpler scheme is detailed. It is thought that this simple scheme should provide enough control to ensure stability. Should further details of the sophisticated schemes be required, these would be furnished on request.

7.1.1 Summary of Sophisticated Schemes: Combination of interceptor drain, road subdrain and horizontally drilled longitudinal drains. Generally as per Figure 8. Longitudinal drains installed by horizontal boring and jacking.

Full depth interceptor trench together with road subdrain, generally as per Figure 9. Excavation of trench to necessary depth could be difficult.

7. DRAINAGE - cont'd

Deep Counterfort drains, generally as per Figure 10. These also act as buttresses to the slope, as the friction of the potential slip material against the sides of the Counterforts supports the bank. These in combination with surface herring-bone drains would probably be the ultimate in effectiveness. Installation would be best carried out prior to excavation of the cut and could be technically difficult. This scheme would be extremely costly.

- 7.1.2 Simple Scheme: This scheme is shown in Figure 11 for a slope of 2.5:1 to 25 ft. For other slopes the configuration will be the same, but the dimension may need to be varied. This scheme will not reduce seepage forces to zero everywhere on the failure surface, but should do so in the critical toe area. The components of the scheme are (i) a longitudinal interceptor drain in the shoulder of the slope, further back from the crest of the slope than the intersection of the potential failure surface and the ground surface. This should be generally as detailed in Figure 11, i.e. 10 ft. deep, 2 ft. wide at the bottom with a side slope of 1 horizontal to 10 vertical outwards towards the top, having a 6 inch diameter perforated pipe laid on at least 3 inches of crushed stone filter material, (ii) a toe drainage ditch, of the shape indicated in Figure 11, filled with graded granular material and (iii) shallow Counterforts 2 ft. wide at 15 ft. centres as indicated by area ABC on Figure 11, also filled with graded granular material. These three components will act in conjunction with the road subgrade drain and a surface run-off collector in the slope shoulder.

7. DRAINAGE - cont'd

The use of a combined drain for both surface run-off and subdrainage is not recommended as experience has shown that this allows surface water to enter the subgrade in the case of a road or the ground itself in this case. The surface run-off collector should be an open ditch rendered impervious by a skimming of concrete or sand asphalt lining. The ground surface should be sloped towards this ditch at about 2% slope and grassed to reduce water ingress.

7.2 Road Drainage: Two types of road drainage should be provided, one to deal with surface run-off and the other for depressing the ground water table beneath the road to ensure a dry road bed and minimize the danger of frost action. These systems should be separate for the reasons outlined in 7.1 above.

7.2.1 Subgrade Drains: Subgrade drains should be provided under each hard shoulder and also under the median. These should be at least 6 ft. deep and generally as specified for the interceptor drains in 7.1.2 above (see Figure 11), dimensions being reduced in the ratio 6:10 to take into account the reduction in depth. They should be sealed to prevent ingress of surface water.

As the road itself will have to be constructed on a relatively thick granular sub-base, this will disperse laterally any water rising to this height and this should then reach the subgrade drains. However, if the subgrade drains are functioning properly, the ground water should not normally reach the sub-base.

7. DRAINAGE - cont'd

- 7.2.2 Surface Run-off: Presently available data indicate that a 2% fall will be provided away from the crown of each carriageway in both lateral directions. Surface run-off will therefore be picked up through gratings in the kerbs and will collect in catch basins, which in turn will discharge into the main storm water sewer placed at depth beneath the road.

The slopes of the cut should be grassed to increase their run-off factor and this water will then collect in the toe drainage trench which should also be connected at intervals to the main storm water sewer. Some consideration might be given to providing this ditch with a relatively impervious lining such as sand asphalt, which would prevent any tendency for the water collected to enter the natural soil, particularly in time of severe storm.

It has already been mentioned in paragraph 7.1.2 that a surface water collector ditch should be placed in the shoulder of the slope, say 10 ft. back from the crest of the cut slope and that this should be lined to render it impervious. The water collected in this ditch will have to be disposed of and it is thought that a convenient method would be to run pipes at intervals from the invert of this ditch down the shallow Counterforts proposed in paragraph 7.1.2. This would obviate the necessity of digging further trenches specifically for placement of such

7. DRAINAGE - cont'd

pipes. Disposal of water from the shoulder interceptor drain could be similarly treated. It should be noted that discharge of water, from these two sources, into the toe drainage ditch should be avoided. Instead the pipes could be fed to the catch basins located under the hard shoulder of the road for collection of surface run-off from the road itself. The catch basins which would have to deal with this extra inflow should be designed to have the necessary capacity.

These proposals need not be considered rigid but rather as an indication of practices to avoid and above all to indicate that all drainage systems should be connected to a suitable permanent outlet, which will probably in all cases be the main storm water sewer.

8. PAVEMENT DESIGN

Selection of an acceptable pavement design will be influenced by a number of factors including cost (installation and maintenance), technical feasibility and availability of suitably experienced and competent contractors. Actual designs will themselves be dependent on strength and type of subgrade, potential frost action and design traffic intensity.

The concern of this report is to provide the soil parameters, which in conjunction with other data, will be needed for pavement design. Provision of these parameters is not a straight forward matter since the usual criterion, the California Bearing Ratio test is not applicable to sensitive materials, although some information might be gleaned from in situ C.B.R. tests.

8. PAVEMENT DESIGN - cont'd

8.1 Rigid Construction - Concrete Pavement: Modern thinking on the subject of subgrades for concrete roads places the emphasis on the requirement of a high degree of uniformity of subgrade support, rather than on that of high subgrade strength. This follows from the fact that loads on concrete pavement are distributed over a large area of subgrade, thus resulting in low contact stresses. *Call* Core should be taken to provide gradual transitions where abrupt changes in soil type are encountered. Provided that they are adequately reinforced, concrete pavements should be better able to withstand frost heave than would asphalt pavements. Nevertheless, efforts should be made to avoid frost action, wherever possible.

Current thinking in Ontario seems to be away from the use of reinforcement and towards the use of unreinforced sections having frequent random dowelled joints at intervals of between 12 ft. and 19 ft. (average 15 ft. 6 inches). For this class of road a minimum concrete thickness of 9 inches would be required and the concrete should have a crushing strength of 3,500 p.s.i. and a flexural strength of 450 p.s.i. It is thought that a minimum of 24 inches of compacted granular sub-base will be needed in this application. If it is properly installed the granular working mat will provide the basis for this sub-base.

The trend away from reinforcement seems to make it even more imperative to protect the pavement from frost action. In this instance we believe this will be achieved by (a) use of adequate thickness of sub-base and (b) a good drainage system.

8. PAVEMENT DESIGN - cont'd

8.2 Asphalt Pavement Designs: In order to use the thickness design method of the Asphalt Institute, it is necessary to have an estimate of the traffic intensity for which the road is to be designed. This should take into account the growth of traffic over the design life of the road. Once this has been established, relationships supplied by the Asphalt Institute enable a Design Traffic Number to be assigned. This, together with a C.B.R. value allows a design to be obtained from the design curves of the Institute.

As has been said previously, it is difficult to assign a C.B.R. value to a sensitive material. However, as a thick sub-base is to be provided, it would be possible to measure the C.B.R. of this and to use this value as the basis for design. This would undoubtedly result in the minimum permissible thicknesses being used.

The design thicknesses evaluated by the method of the Asphalt Institute assume (a) subgrade compaction criteria of 18 to 24 inches compacted to 95% of AASHO standard density for cohesive subgrades and 100% for cohesionless subgrades and (b) the use of an insulation course of minimum thickness 2 inches, to be placed between the pavement and the subgrade to prevent either mud-pumping or ingress of the asphalt binder into the subgrade.

As Leda clay is not susceptible to compaction, the subgrade requirements must be met by the sub-base.

8. PAVEMENT DESIGN - cont'd

A possible compromise between use of the C.B.R. values for the clay on the one hand and the granular sub-base on the other, is the adoption of an arbitrary C.B.R. value to be used in conjunction with an adequately prepared sub-base.

Various design solutions are then possible. These would cover pavements composed of entirely asphalt concrete and all combinations of asphalt concrete, granular base and/or sub-base which satisfy the design requirements.

As an example of these possibilities, four designs are given below for an arbitrary C.B.R. value of 6 together with a D.T.N. of 1000. These are used as an example and the actual figures selected will be at the discretion of the Consulting Engineers.

8.2.1 Full Depth Asphalt Concrete Pavement: For a C.B.R. of 6 and a D.T.N. of 1000 the total required thickness of Asphalt Concrete is 10½ inches of which a minimum of 2 inches is needed for the surface course. Hence, the design is: -

Asphalt Concrete Surface	2.0 inches
Asphalt Concrete Base	8.5 inches
Total Construction	<u>10.5 inches</u>

Placed, of course, in this case on 24 to 36 inches of compacted granular sub-base:

8. PAVEMENT DESIGN - cont'd

- 8.2.2 Asphalt Concrete Surface & Base with Granular Base: The minimum permissible asphalt concrete thickness for a D.T.N. of 1000 is 6 inches. The difference between the total required thickness ($10\frac{1}{2}$ inches) and minimum permissible asphalt concrete thickness is $10.5 - 6.0 = 4.5$ inches. This is the amount of asphalt concrete for which granular base may be substituted. The substitution ratio for granular base is 2:1, therefore, if the whole 4.5 inches of asphalt concrete which may be replaced by granular base is so replaced a thickness of $4.5 \times 2 = 9$ inches of granular base will be required. The minimum required thickness of surface course is 2 inches. Hence this alternative design is: -

Asphalt Concrete Surface	2.0 inches
Asphalt Concrete Base	4.0 inches
Granular Base	9.0 inches
Total Construction	<u>15.0 inches</u>

Further variation between designs 8.2.1 and 8.2.2 is permissible by varying the amount of asphalt concrete replaced within the limits 0 and 4.5 inches, the only condition being that each inch of asphalt concrete omitted must be replaced by 2 inches of granular base.

- 8.2.3 Asphalt Concrete Surface & Base with Granular Base & Granular Sub-Base: A further alternative design may be made using both granular base and sub-base. As in the examples 8.2.1 and 8.2.2 above, the total required asphalt concrete thickness is $10\frac{1}{2}$ inches and the minimum permissible asphalt concrete thickness is 6.0 inches. The value used to determine the

8. PAVEMENT DESIGN - cont'd

thicknesses of granular base and sub-base permissible in the pavement structure is 10 inches.

The amount of asphalt concrete which may be converted to granular base is $10.0 - 6.0 = 4.0$ inches. The thickness of granular base which must be substituted for this asphalt concrete is $4.0 \times 2 = 8.0$ inches.

The amount of asphalt concrete which may be converted to granular sub-base is given by $10.5 - 10.0 = 0.5$ inches. The required thickness of substituted granular sub-base is $0.5 \times 2.7 = 1.35$ inches or $1\frac{1}{2}$ inches in round figures. The minimum required thickness of asphalt concrete surface course is 2 inches, therefore this design is: -

Asphalt Concrete Surface	2.0 inches
Asphalt Concrete Base	4.0 inches
Granular Base	8.0 inches
Granular Sub-base	1.5 inches
Total Construction	15.5 inches

- 8.2.4 Asphalt Concrete Surface & Base with Granular Sub-Base: A further alternative would be to omit the granular base in which case the design would be: -

Asphalt Concrete Surface	2.0 inches
Asphalt Concrete Base	8.0 inches
Granular Sub-base	1.5 inches
Total Construction	11.5 inches

8. PAVEMENT DESIGN - cont'd

Further variations on alternatives 8.2.3 and 8.2.4 are permissible by altering the proportion of asphalt concrete converted to granular base within the permissible limits 0 to 4.0 inches. Each inch of asphalt concrete replaced by 2 inches of granular base substituted for it.

As the maximum constituent size of material in any layer should not exceed half the thickness of the layer, the use of a sub-base only $1\frac{1}{2}$ inches thick would not be advisable or worthwhile.

As the present application involves a deep sub-base, it seems likely that the only design solution worth considering would be 8.2.1 placed directly on the granular sub-base, compacted to a suitable standard, i.e. 100% of AASHTO standard density. The thickness of sub-base being determined partly by this requirement.

9. STORM WATER SEWER

A major storm water sewer will be necessary to carry away surface run-off from Heron Road and ultimately also from the Saw Mill Creek Parkway - Heron Road interchange.

There are two possible outlets for such a sewer, (i) Saw Mill Creek at Heron Road, approximate water elevation 220 ft., and (ii) the Rideau River at the George Dunbar Bridge on Bronson Avenue, approximate water elevation 183 ft.

9. STORM WATER SEWER - cont'd

The distances from the interchange to these two points are approximately 1,750 ft. and 2,000 ft. respectively. The ground level at Heron Road and the C.P.R. is 253 ft. and the road surface when the underpass is constructed is likely to be between 223 ft. and 228 ft. Hence, although the distance to Saw Mill Creek is shorter, it appears from information currently available that the creek water level is too high to permit discharge into it. Should this be verified by a careful study of exact dimensions, then the sewer will have to discharge into the Rideau River.

The sewer will be at a depth of 40 ft. to 45 ft. below existing ground level at Heron Road and the C.P.R., i.e. elevation 213 ft. to 218 ft., as it must be at least 10 ft. below the underpass.

Although a short section under the nadir of the underpass will be only of the order of 10 to 15 ft. below bottom excavation level and could be constructed in trench, for the most part the depth of the sewer below ground surface will be more like 40 ft. and for this operation in Leda clay we recommend tunnelling.

Instrumentation and study of a recently constructed tunnel sewer of outside diameter 10 ft., inside diameter 8 ft. have shown that such a tunnel can be satisfactorily made using a rotary tunnelling machine followed by immediate lining with corrugated segmental steel liner rings. The liner should be in close contact with the clay and any spaces between liner and clay should be grouted. In practice it was found that the tunnelling machine gave a well trimmed excavation and that little grouting was necessary.

9. STORM WATER SEWER - cont'd

The flexible temporary lining should be left in place and the permanent relatively rigid structure could be of reinforced concrete cast inside the temporary lining.

Although the use of compressed air does not seem to be strictly necessary, it has been used previously in Ottawa for deep tunnels in Leda clay. Its use could be made necessary by the presence of thin water bearing sand seams, in which case a pressure of 4 to 5 p.s.i. would probably suffice. In general the permeability of the clay is so low that ground water should not present a problem during the time required to install the temporary liner.

Experience indicates that a tunnel of this type, if properly installed, should cause very little surface settlement. Such ground settlement should be due only to elastic and recompression settlements brought about by the change in effective stress.

The tunnel may be expected to act as a drainage sink and thus have an effect on the prevailing piezometric regime. Due to the low permeability of the clay, the quantities of water involved will be very small.

In spite of its high sensitivity, the Leda clay tends to behave as a brittle elastic material and its response time to pressure changes appears to be of the order of months rather than years.

9.1 Earth Pressures: The flexible temporary liner suffers a decrease in vertical diameter and a corresponding increase in horizontal diameter as equilibrium is approached under the earth pressures

9. STORM WATER SEWER - cont'd

developing after installation. The liner will have to resist an all-round of approximately $2/3$ of full overburden within a short time of installation and this will tend to increase with time. For a depth of 40 ft. $2/3$ of full overburden is about 2,800 lbs/sq.ft. It is suggested that allowance be made for the temporary lining having to resist full overburden with time (4,200 lbs/sq.ft. at 40 ft.).

The permanent, relatively rigid reinforced concrete lining does not yield in the same manner as the flexible lining and consequently is subject to a different pressure distribution.

The average vertical pressure on the permanent lining may be expected to be approximately $3/4$ of full overburden and the corresponding lateral pressure about .7 of this value. It is suggested, however, that for design purposes the rigid lining should be able to resist full overburden in the vertical direction acting in combination with (a) $2/3$ full overburden and (b) full overburden in the horizontal direction. If the ultimate horizontal pressure is less than $2/3$ full overburden, a very small radial yield will increase it to $2/3$ full overburden and the design conditions will be fulfilled.

10 DISCUSSION

10.1 Leda Clay: It has been known for a considerable period of time that the Leda clay, found in the Ottawa and St. Lawrence river valleys, exhibits

10. DISCUSSION - cont'd

certain characteristics which give rise to difficulties both in interpretation of standard soil mechanics tests and of construction in this medium. It is beyond the scope of this report to go into details of this phenomenon, suffice it to say that the site under consideration is situated in Leda clay and that, consequently, even more care than usual should be exercised in dealing with the soil.

Although the causes of sensitivity have not been completely explained, the most significant contributory factor appears to be decreasing salt concentration. However, this report is not so much concerned with the causes of sensitivity, as with their effects and the steps which need to be taken in dealing with them. From this viewpoint, it is necessary to carry out sufficient field and laboratory testing to determine certain characteristics of the particular soil at this site and then to correlate these to more extensive work already carried out on similar material. The principal source of published information on Leda clay in the Ottawa area is the National Research Council.

84.
Sensitivity is described as the ratio of undisturbed to remoulded shear strength. One method of ascertaining this ratio is from the results of in situ vane tests. Unfortunately, the remoulded strength of Leda clay is too low for it to be reliably determined with the vane equipment used and so the method is not available to us. However, on the basis of tests on Norwegian Marine clays, Bjerrum has given a relationship between sensitivity and liquidity index, see Figure 12. It can be seen from

10. DISCUSSION - cont'd

Table II that the strata can be divided into at least two types on the basis of plasticity and also that as the plasticity index decreases, so the liquidity index increases. These observations fit in well with Bjerrum's theories on sensitivity arising, at least in part, from leaching.

From Bjerrum's graph shown in Figure 12 it will be seen that sensitivities, corresponding to the range of liquidity indices 1.4 to 3.4, have values between 16 and in excess of 250. These values may only be taken as indicative rather than absolute, as they are results from single tests. Where the plasticity index is low, the equation for liquidity index is ill-conditioned and the values of 3.0 and 3.4 may be too high. However, on a qualitative basis these findings agree well with published work on similar material.

From the results of the consolidated-undrained triaxial tests with pore pressure measurements (summarized in figures 1 and 2) values of effective stress parameters of $c' = 300$ p.s.f. and $\phi' = 20^\circ$ were selected as appropriate for effective stress analyses. These values are also in accordance with current thinking at N.R.C.

From a practical point of view the significant aspects of sensitivity are (i) although in its undisturbed state the clay has appreciable shear strength, on remoulding most of this strength is destroyed and (ii) although the modulus of recompression is low, once the preconsolidation pressure has been exceeded, further settlement is excessive.

*data is different
if the plasticity is
14 to 140*

10. DISCUSSION - cont'd

The passage of contractors' vehicles across Leda clay is not possible because of the effect of remoulding by the wheels or tracks, thus destroying strength. Hence, excavation has to be carried out remotely from a firm surface, for example by drag-line. Once the excavation is complete, 2 to 3 ft. of clean granular material should be placed in compacted layers of 6 to 9 inches finished thickness. This will then facilitate movement of construction plan.

Although it is desirable to compact this working surface as mentioned, it is likely that initially 9 to 12 inches will have to be just tipped and spread. The best technique for development of this granular working surface should be determined by field experimentation. A sufficient thickness should be tipped.

10.2 Construction Problems: These arise from the necessity of avoiding any movement of plant on exposed Leda clay surfaces. The problem has to be resolved by (a) selection of appropriate excavation techniques and equipment and (b) adherence to a carefully planned and executed sequence of operations.

A particular problem arises in connection with the installation of the drainage ditch at the toe of the slope, as outlined in the simple drainage proposal, paragraph 7.1.2. In the interests of safety, general excavation of the cut should not be allowed

10. DISCUSSION - cont'd

to proceed to a greater depth than 20 ft. prior to the installation of the toe ditch. If the general excavation were completed first and this ditch then excavated, the toe of the slope would be unloaded and instability could well occur.

Excavating equipment must not operate on the slope face unless experience has shown this to have no detrimental effects -- for example, when the ground is severely frozen.

While a drag-line is probably very suitable for bulk excavation of this type, it is unlikely that it could be used for finishing operations which will require use of some machine capable of more precise control and of operation from the floor of the cut on which the granular mat has been developed.

In connection with the development of this working surface, it is suggested that consideration be given to the planning of operations so that the main excavation proceeds inwards from the shallow ends of the cut. In this way the granular material is dumped from the existing Heron Road pavement at the point where excavation has commenced and subsequently from vehicles reversing onto the already placed granular material. In this way the incoming loaded trucks compact the granular material and also always avoid trespassing on the newly exposed clay surface. Hence, the construction mat is developed towards the deepest point of the excavation. Suffice it to say, without further detail, that careful planning will be necessary to establish the working surface.

How can a toe trench be
excavated before the
depth of the cut is
determined?

10. DISCUSSION - cont'd

2. | 10.3 Alternative Proposals: It has been assumed that if possible the underpass will be constructed in cut with normal side slopes. No mention has been made of benching, as it is not thought that sufficient land is available for such a measure, which could otherwise be used to achieve stable steeper slopes. However, if the problems created by slopes to these depths in Leda clay appear to be proving excessively expensive and difficult to solve, there is a further possibility. The sides of the cut could be supported by vertical concrete retaining walls braced at the existing ground level, at intervals and at the base by a structural concrete road. This type of construction may be seen in the Decarie Expressway in Montreal, where its use was dictated by lack of space rather than soils problems.

11. SUMMARY OF PRINCIPAL RECOMMENDATIONS

The bridge should be founded on end-bearing piles taken to bedrock. Steel "H" piles or Johnson Herkules piles might be considered and care should be taken in ascertaining terminal depth, owing to the presence of boulders above the rock.

In order to maintain a minimum factor of safety of 1.25 against slope failure, cuts should not exceed the following depths depending on the slope: 25:1, 25 ft.; 3.0:1, 30 ft.; 3.5:1, 37½ ft. The maintenance of an adequate factor of safety is dependent on provision of good control of ground water and seepage.

11. SUMMARY OF PRINCIPAL RECOMMENDATIONS

A simple drainage scheme is proposed and is shown in Figure 11. This consists of a granular toe drain together with an interceptor drain in the slope shoulder in conjunction with shallow Counterforts and the road subgrade drainage.

Ranges of design pressure for both temporary and permanent sewer tunnel linings are proposed, based on recent Ottawa experience and the theories of Terzaghi.

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JOB NO. 67 F98

BH	1	2	3	4	5	6	7	8	9	10	11	12	13
Water													
Level	6'5"	7'0"*	9'6"*	0'4"	2'2"	3'5"	2'11"	3'4"	5'8"	3'5"	7'0"+	2'0"	7'0"

* Artesian pressure above rock

+ Approximate

TABLE I - Water Levels

BH	Sa.	Depth	L.L.	P.L.	P.I.	L.I.	M nat.
2	5	8'6"-10'0"	60	22	38	1.44	77
2	8	13'-15'	32	18	14	2.07	47
2	9	23'-25'	50	23	27	1.59	66
2	10	31'-33'	33	21	12	2.92	56
1	11	39'-41'	23	10	13	--	--
2	11	39'-41'	25	17	8	2.5	37
2	12	47'-49'	22	16	6	2.17	29
2	13	55'-57'	23	16	7	3.4	40
1	14	65'-66'	23	16	7	3.0	37
2	14	67'-69'	25	17	8	--	--
2	17	87'-89'	50	20	30	.93	48

TABLE II - Atterberg Limits

JOB NO. 67 F98

BH	Sa.	Depth	Sand	Silt	Clay	Description
2	5	8'6"-10'0"	1	24	75	Silty clay
2	8	13'-15'	1	56	43	Very silty clay
2	9	23'-25'	0	57	43	Very silty clay
2	10	31'-33'	2	36	62	Silty clay
1	11	39'-41'	3	31	66	Silty clay
2	11	39'-41'	0	45	55	Very silty clay
2	12	47'-49'	10	52	38	Very silty with a trace of sand
2	13	55'-57'	8	50	42	Very silty with a trace of sand
2	14	67'-69'	10	53	37	Very silty with a trace of sand
2	17	87'-89'	4	32	64	Silty clay

TABLE III - Distribution of Principal
Constituent Sizes

JOB NO. 67 F98

BH/Sample	Depth	Shear Strength lbs/sq.ft.
2/4	7'-8'6"	1130
2/5	8'6"-10'0"	725
2/8	13'6"-15'0"	925
2/10	31'-33'	1020
1/11	39'-41'	625
2/12	47'-49'	1640
2/13	55'-57'	1100
2/14	67'-69'	860
2/15	72'-74'	1530
2/16	79'-81'	1350
2/17	87'-89'	1220

TABLE IV - Shear Strengths from Quick
Undrained Triaxial Tests

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

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

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
I_p	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_C	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma'}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma'}$
T_v	TIME FACTOR = $\frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_f	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e \sigma$ OR $\ln \sigma$	NATURAL LOGARITHM OF σ
$\log_{10} \sigma$ OR $\log \sigma$	LOGARITHM OF σ TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

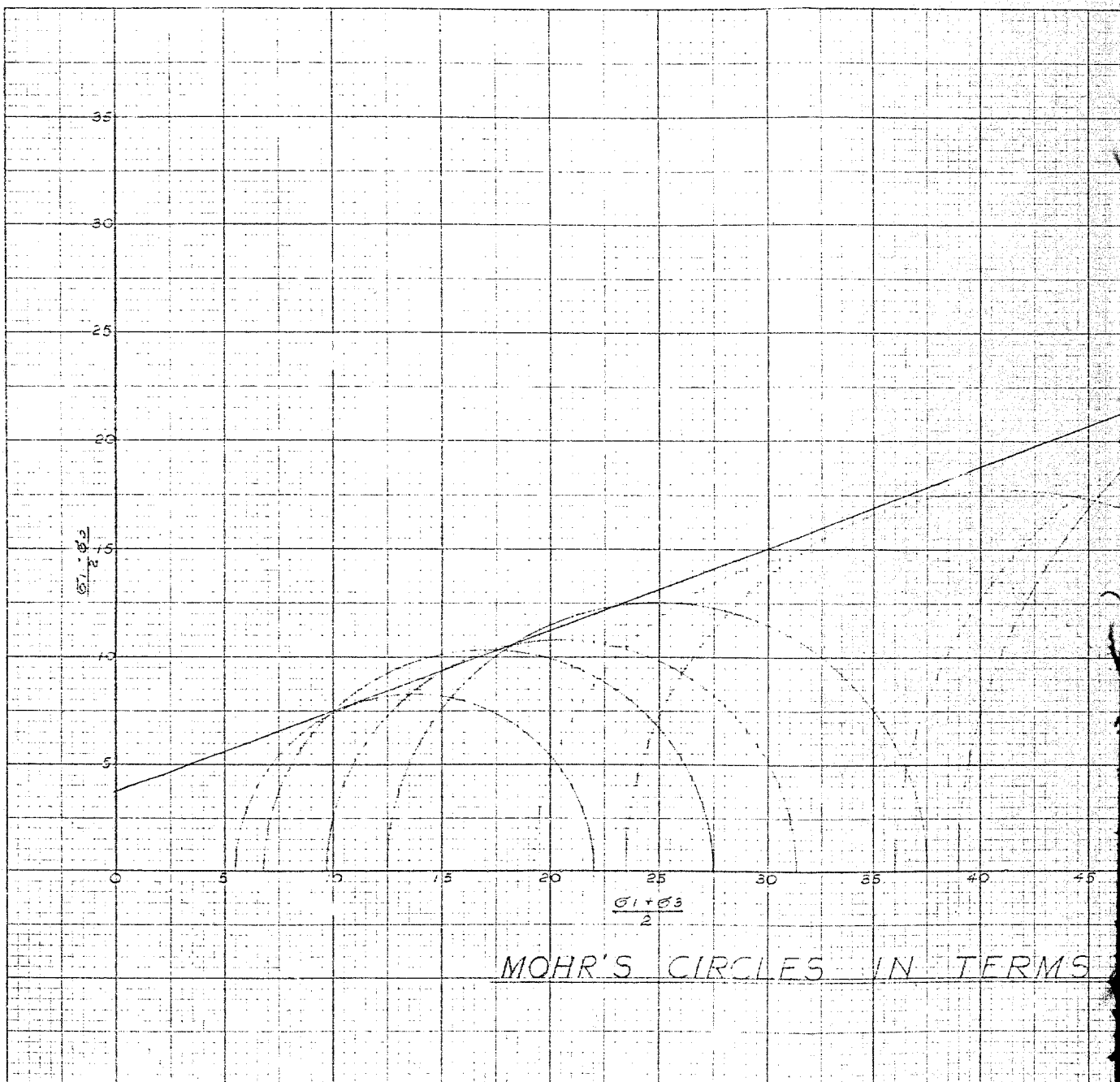
SLOPES

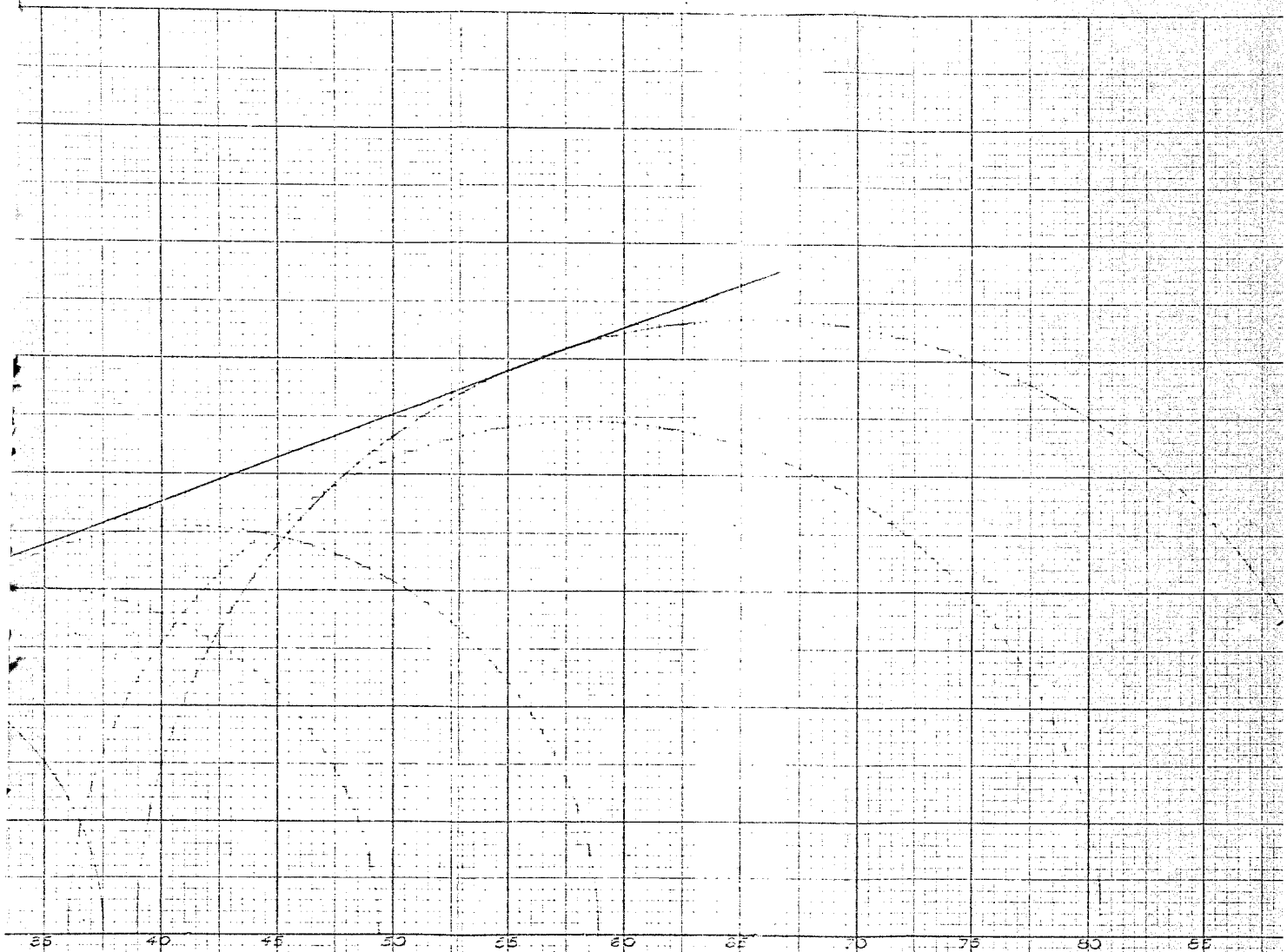
H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

$\frac{\sigma_1 - \sigma_3}{2}$

$\frac{\sigma_1 + \sigma_3}{2}$

MOHR'S CIRCLES IN TERMS

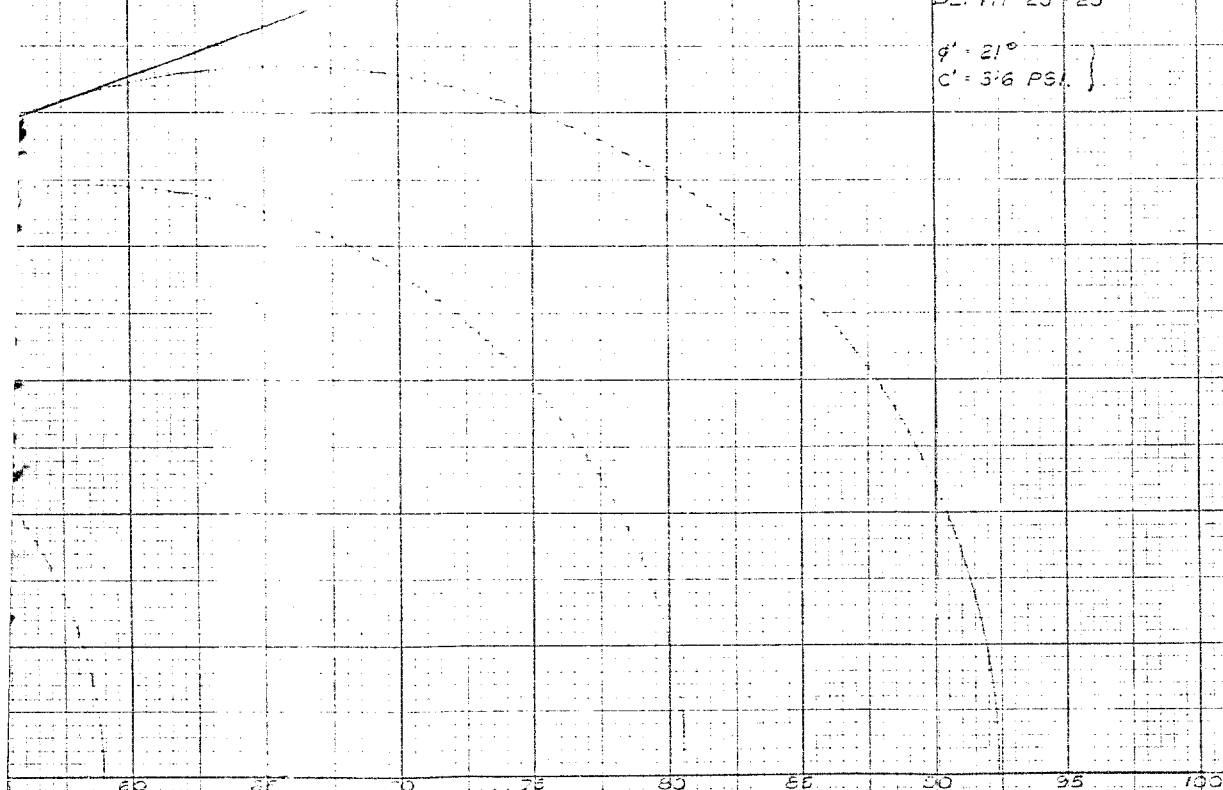




IN TERMS OF EFFECTIVE STRESSES

BORE HOLE NO. 2
SAMPLE NO. 9
DEPTH 23'-25'

$\phi' = 21^\circ$
 $C' = 3.6 \text{ PSI}$

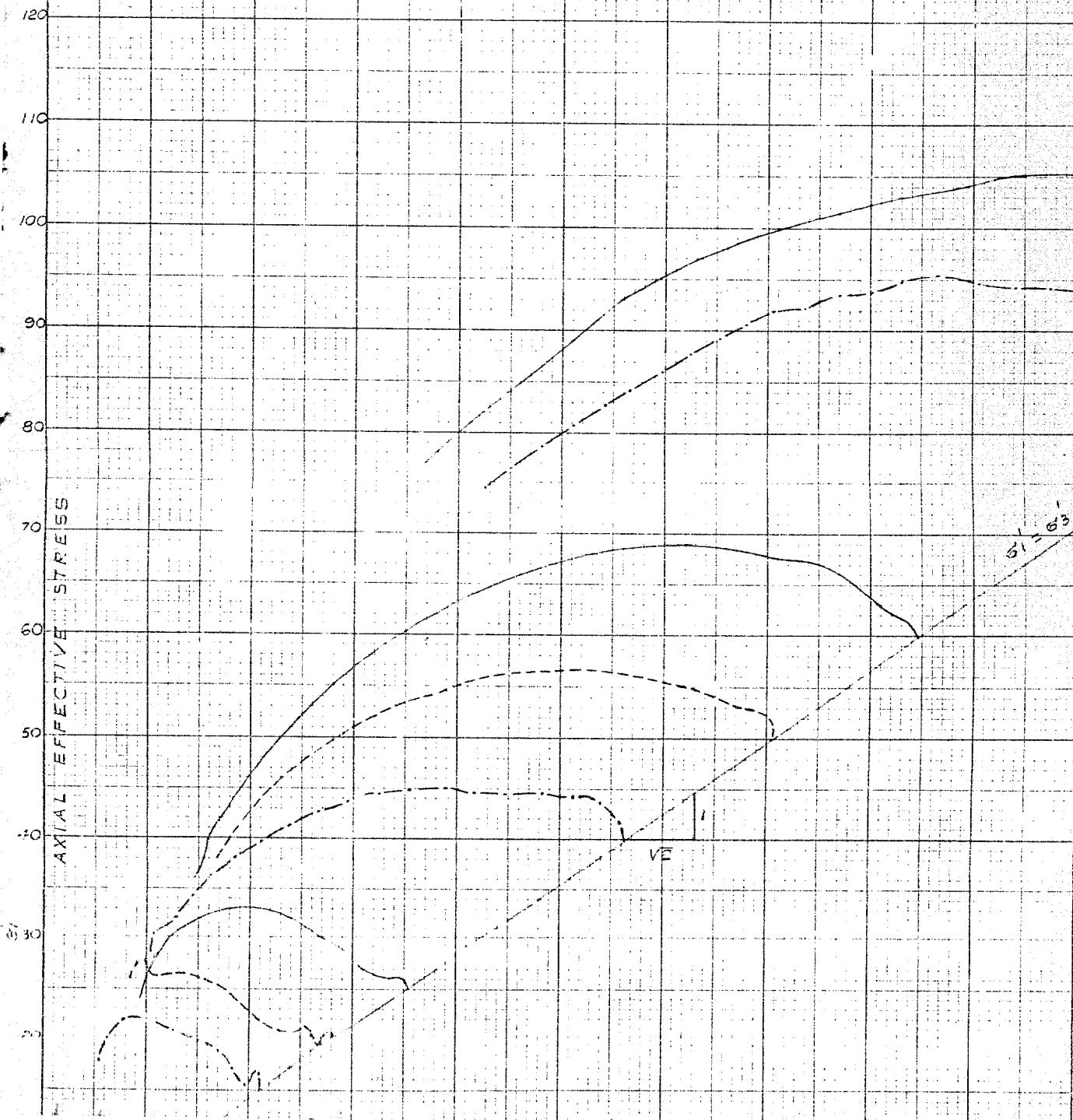


EFFECTIVE STRESSES

FIG. 1

JOB NO. 67-F99
EMP. GEO. ASSOC. INC.
AUG. 1967

RENDULIC PLOT



RENDULIC PLOT

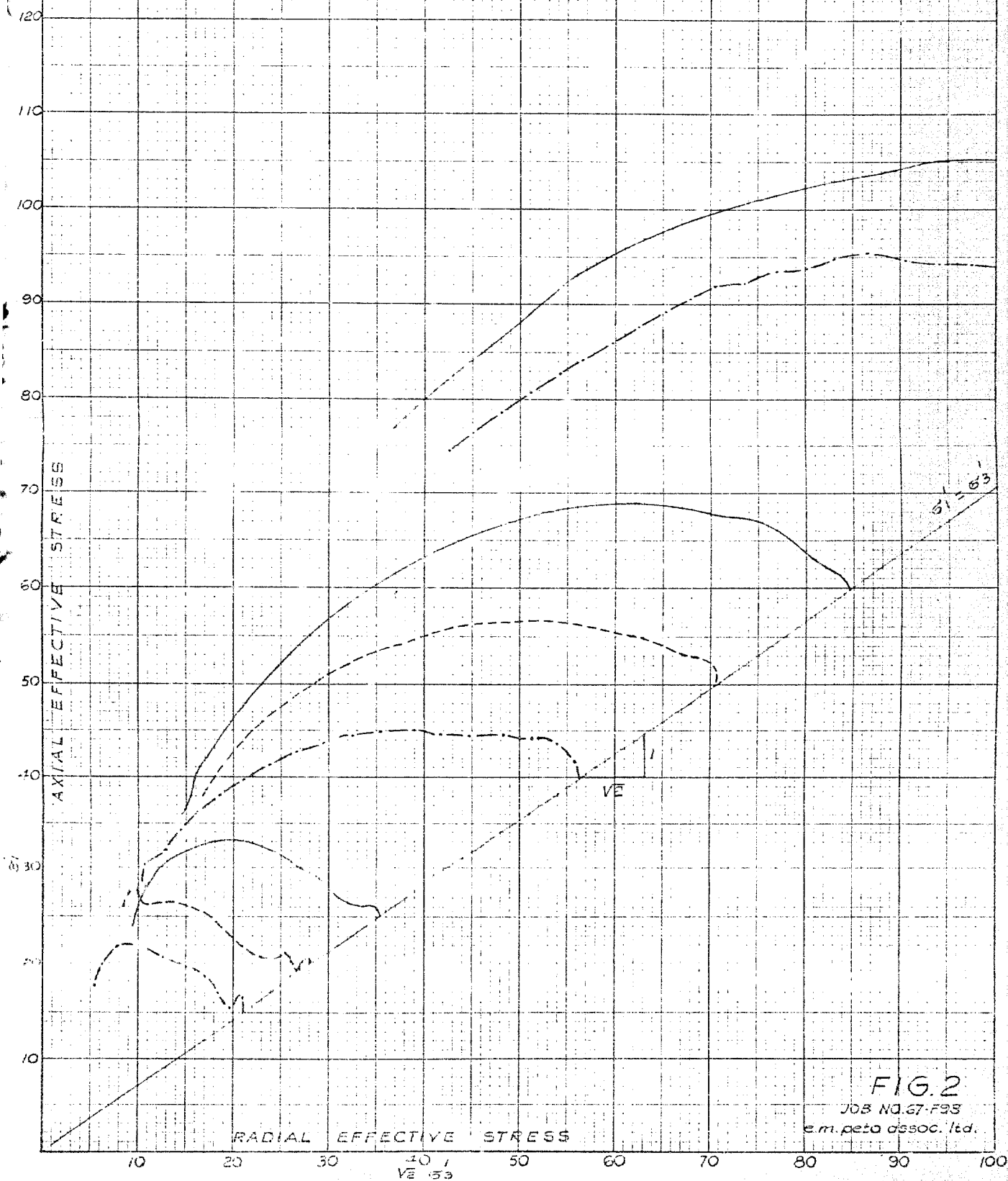


FIG.2

JOB NO.67-F98

c.m. petra assoc. ltd.

FIG. 3

FACTOR OF SAFETY VERSUS DEPTH OF CUT FOR
3 SLOPES WITH TENSION CRACKS TO 8'

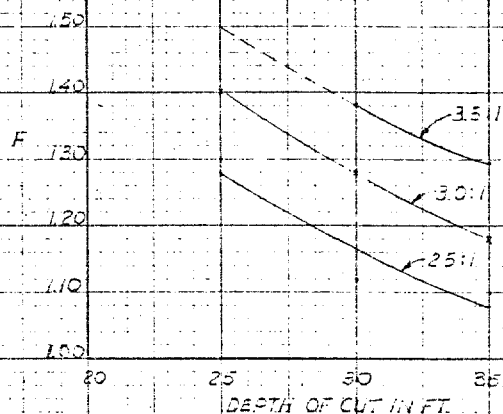


FIG. 4

FACTOR OF SAFETY VERSUS
DEPTH OF CUT FOR SLOPE 25:1
WITH AND WITHOUT TENSION CRACKS

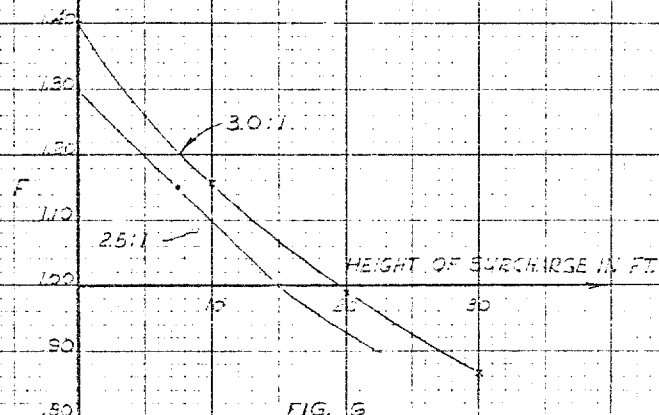
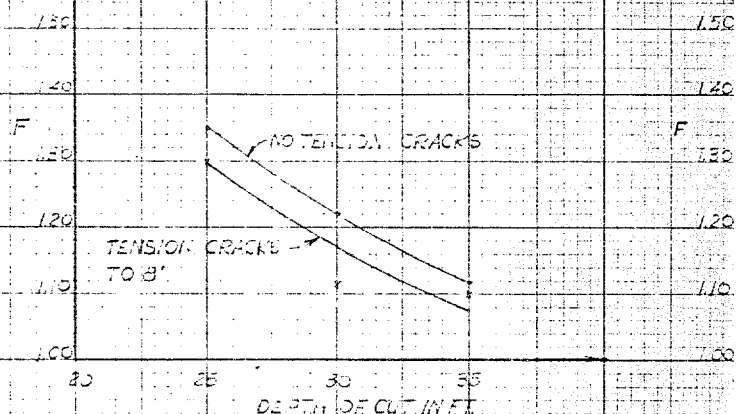


FIG. 5

FACTOR OF SAFETY VERSUS HEIGHT
OF SURCHARGE (135 100 / H²) RISING
FROM SLOPE CREST AT 2:1
CUT DEPTH 25'

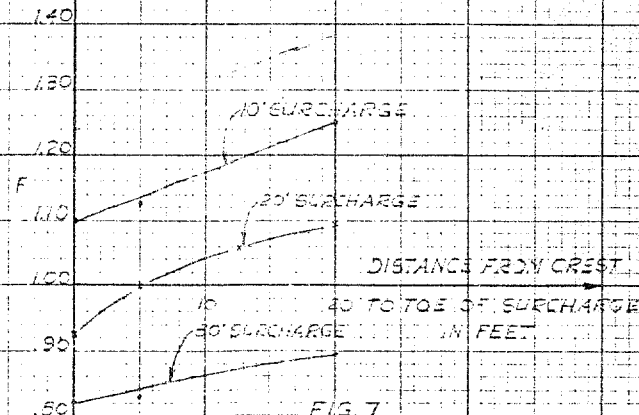
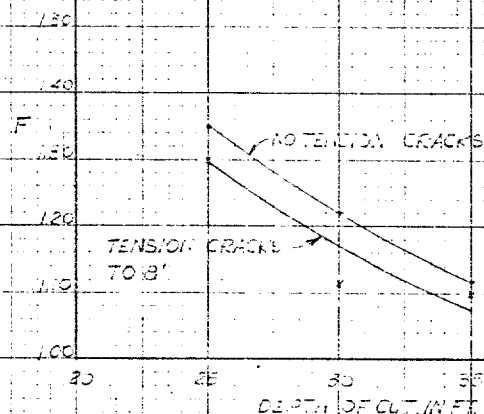


FIG. 7

FACTOR OF SAFETY VERSUS DISTANCE
OF TOE OF SURCHARGE FROM CREST
OF SLOPE 25:1 TO 25' DEPTH

FIG. 4

FACTOR OF SAFETY VERSUS
DEPTH OF CUT FOR SLOPE 2.5:1
WITH AND WITHOUT TENSION CRACKS



F.3.5

FACTOR OF SAFETY VERSUS DEPTH
OF TENSION CRACKS FOR SLOPE
2.5:1 TO 25' DEPTH

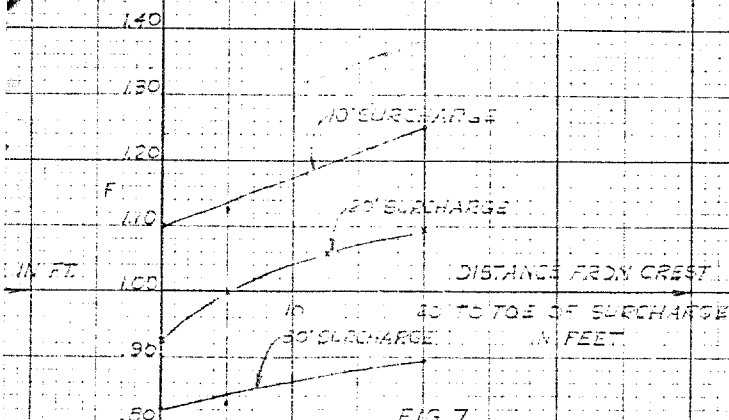
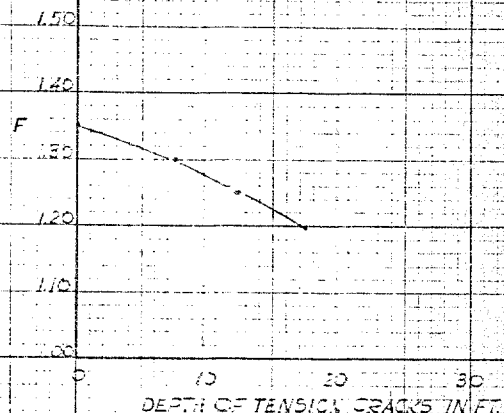


FIG. 7

FACTOR OF SAFETY VERSUS DISTANCE
OF TOE OF SURCHARGE FROM CREST
OF SLOPE 2.5:1 TO 25' DEPTH

FIG. 3, 4, 5, 6 & 7

JCS NO. 67-198
E.M. PETS ASSOCIATES, INC.

COMBINED INTERCEPTOR DRAIN, ROAD SUBGRADE DRAIN AND HORIZONTAL LONGITUDINAL DRAINS.

SLOPE 2.5 : 1 , 25' CUT

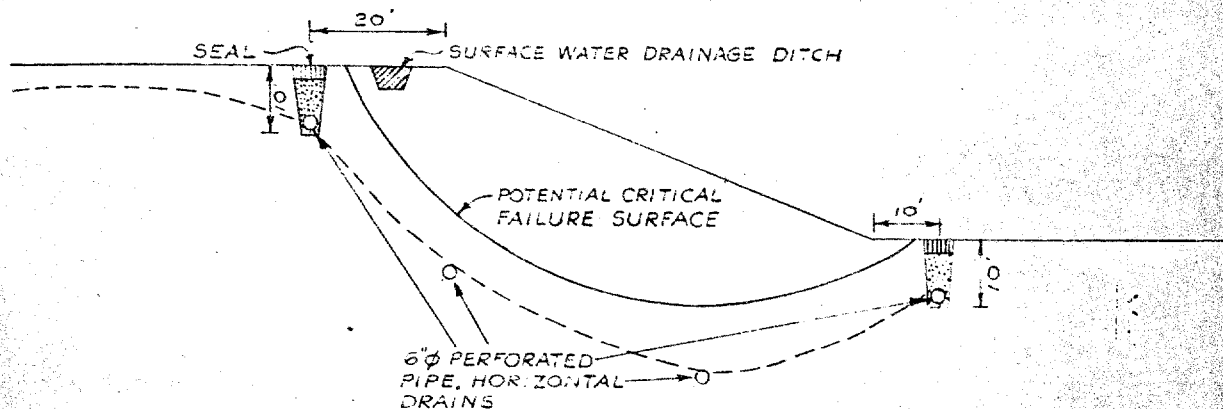


FIG. NO. 8

FULL DEPTH INTERCEPTOR TRENCH TOGETHER WITH ROAD SUBGRADE DRAIN

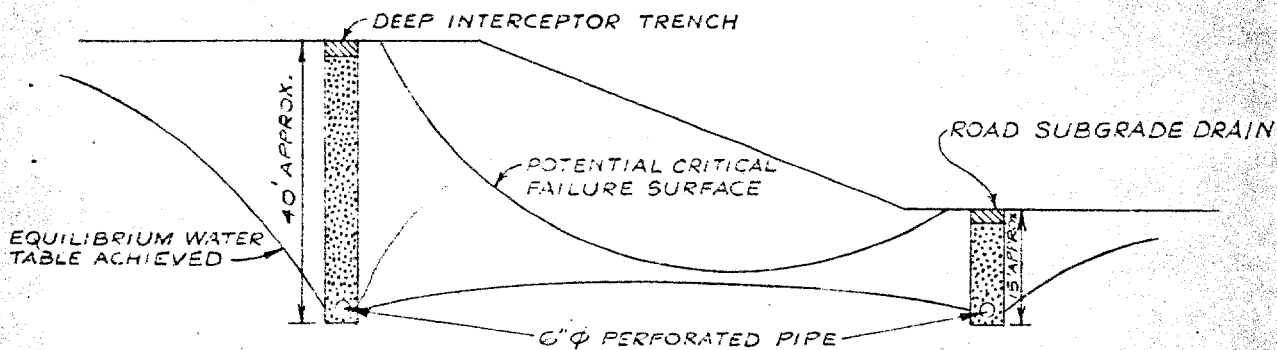
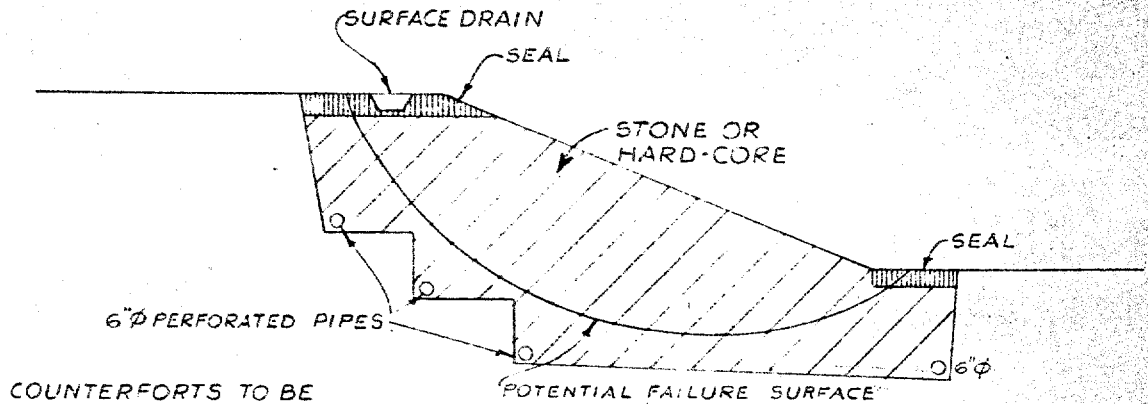


FIG. NO. 9

COUNTERFORT DRAINS

SLOPE 2.5 : 1 TO 25'



COUNTERFORTS TO BE
2' WIDE AT 15' CENTRES

FIG. NO. 10

SIMPLE DRAINAGE PROPOSAL

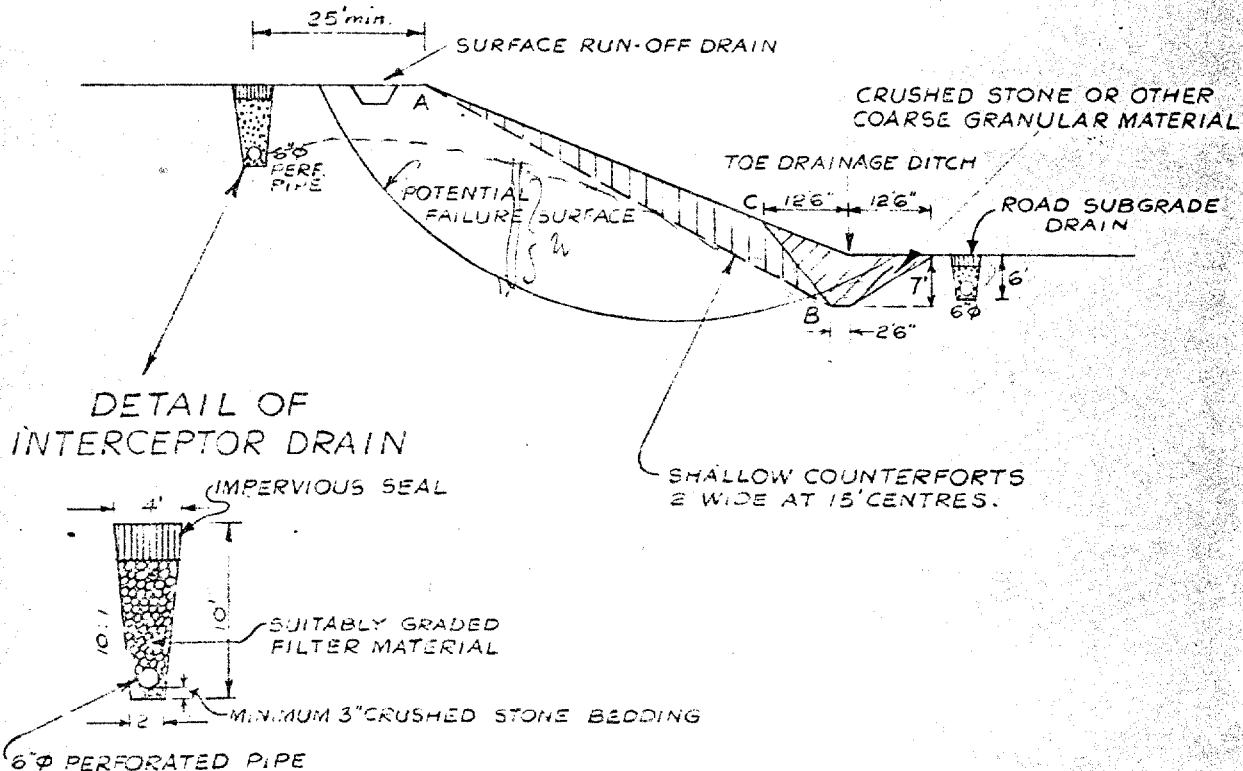
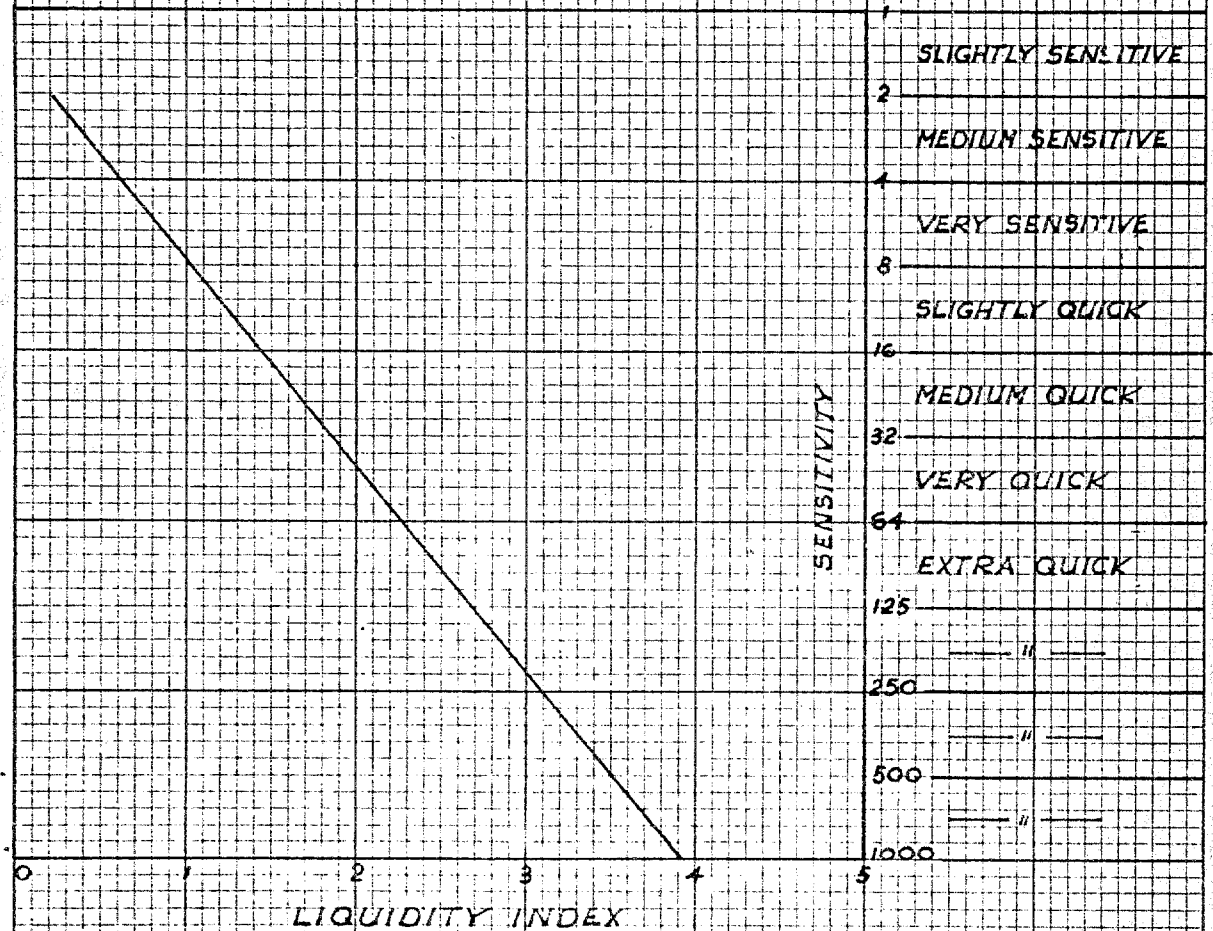


FIG. NO. 11

JOB. NO. 67-F98
a.m.peto assoc. l.t.d.

CLASSIFICATION



RELATION BETWEEN SENSITIVITY
AND LIQUIDITY INDEX
(AFTER BJERRUM)

FIG. #12

JOB NO. 67-F98
e.m. peto associates ltd.

Toronto 19, Ontario



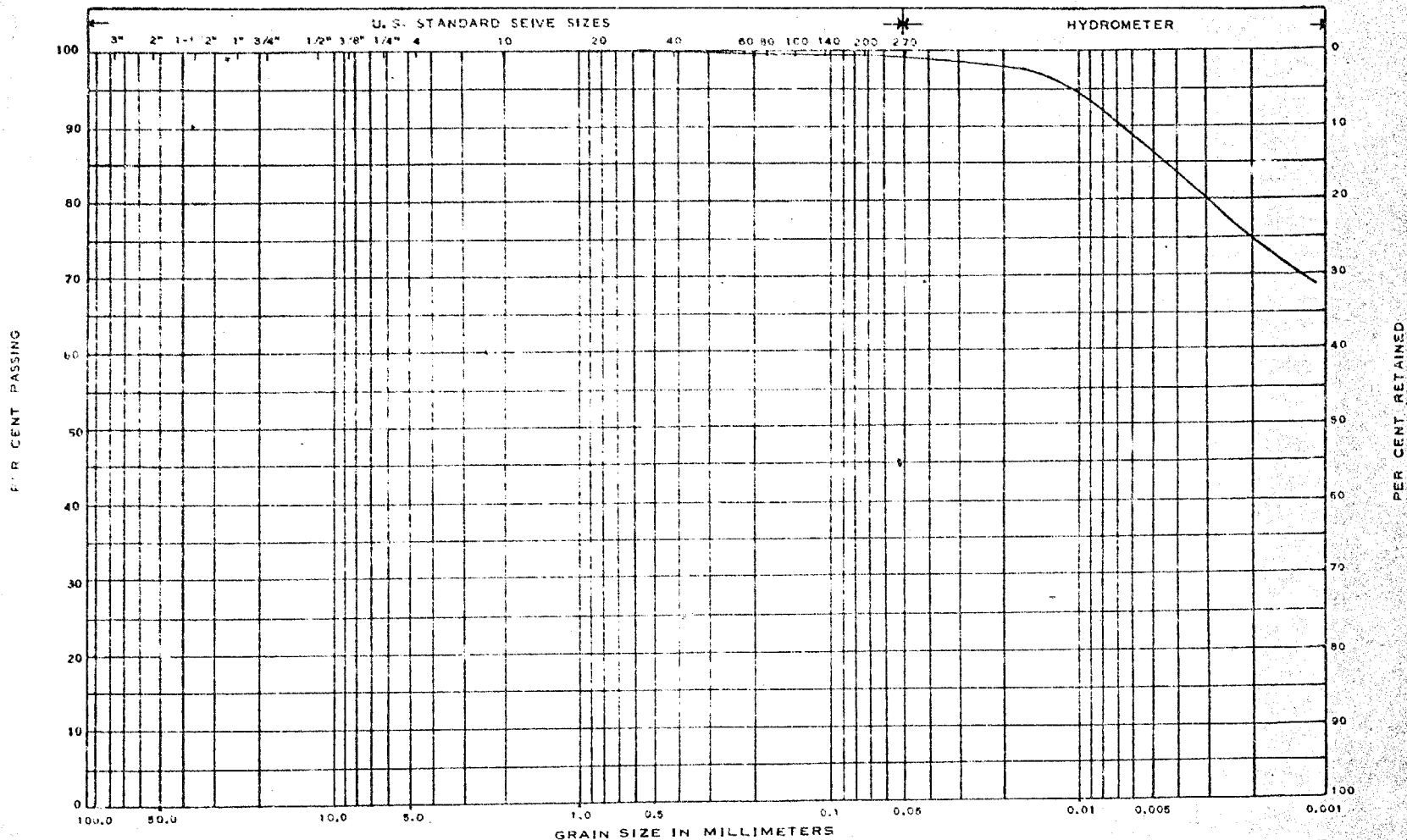
MASS. INST. OF TECH. CLASSIFICATION

MASS. INST. OF TECHNOLOGY
 JOB NAME Heron Road, Ottawa JOB NO. 67 F 98 HOLE NO. 1 SAMPLE NO. 1
 DEPTH 39'-41' ELEVATION _____ REMARKS Silty clay

GRAIN SIZE DISTRIBUTION

e. m. peto associates ltd.

Toronto 19, Ontario



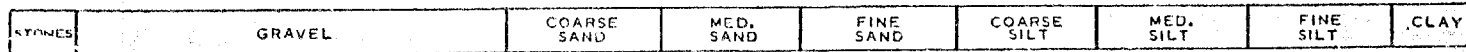
STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
--------	--------	-------------	-----------	-----------	-------------	-----------	-----------	------

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Heron Road, Ottawa JOB NO. 67 F 98 HOLE NO. 2 SAMPLE NO. 5
 DEPTH 8'6"-10'0" ELEVATION REMARKS Silty Clay

GRAIN SIZE DISTRIBUTION

Toronto 19, Ontario



JOB NAME Heron Road, Ottawa JOB NO. 67 F 98 HOLE NO. 2 SAMPLE NO. 8
DEPTH 13'-15' ELEVATION _____ REMARKS Very silty clay

GRAIN SIZE DISTRIBUTION

Toronto 19, Ontario



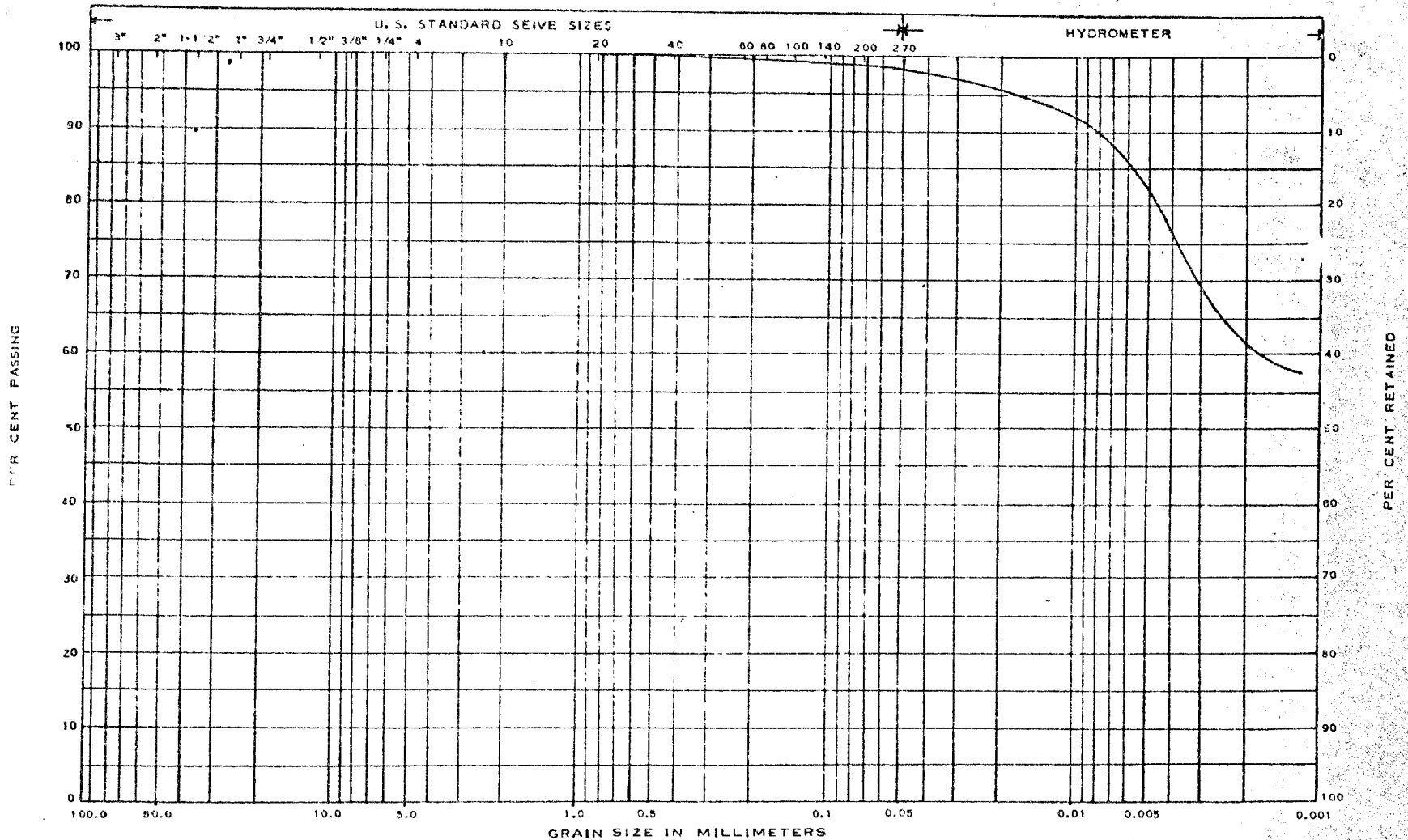
JOB NAME Heron Road, Ottawa JOB NO. 67 F 98 HOLE NO. 2 SAMPLE NO. 9

DEPTH	ELEVATION	REMARKS
23'-25'		Very silty clay

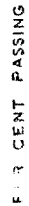
GRAIN SIZE DISTRIBUTION

e. m. peto associates ltd.

Toronto 19, Ontario



Toronto 19, Ontario



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
--------	--------	-------------	-----------	-----------	-------------	-----------	-----------	------

MASS. INST. OF TECH. CLASSIFICATION

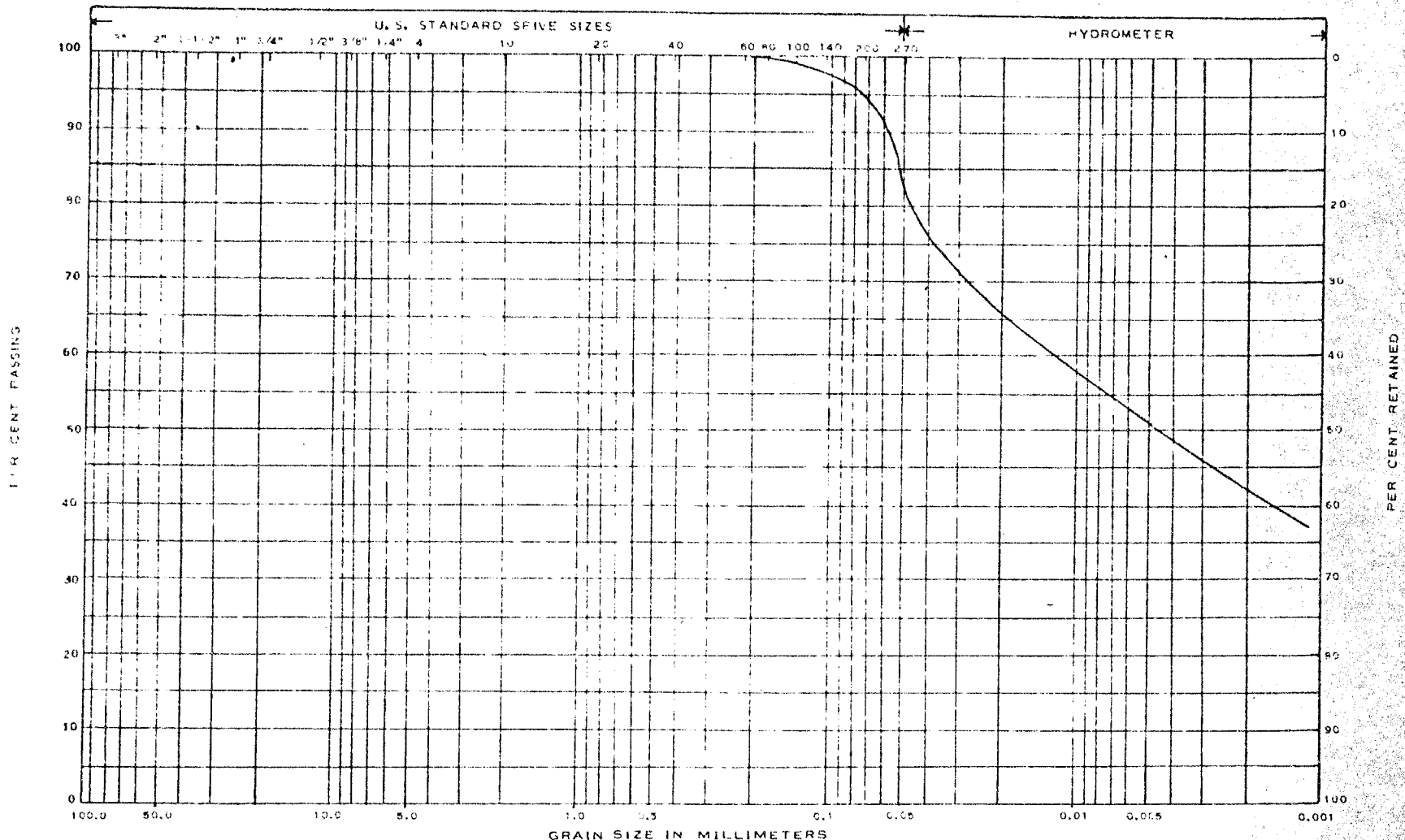
JOB NAME Heron Road, Ottawa JOB NO. 67 F 98 HOLE NO. 2 SAMPLE NO. 12

DEPTH 47'-49' ELEVATION _____ REMARKS Very silty clay with a trace of sand

GRAIN SIZE DISTRIBUTION

e: m. peto associates ltd.

Toronto 19, Ontario



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
--------	--------	-------------	-----------	-----------	-------------	-----------	-----------	------

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Heron Road, Ottawa JOB NO. 67 F 98 HOLE NO. 2 SAMPLE NO. 13
 DEPTH 55'-57' ELEVATION _____ REMARKS Very silty clay with trace of sand

GRAIN SIZE DISTRIBUTION

Toronto 19, Ontario



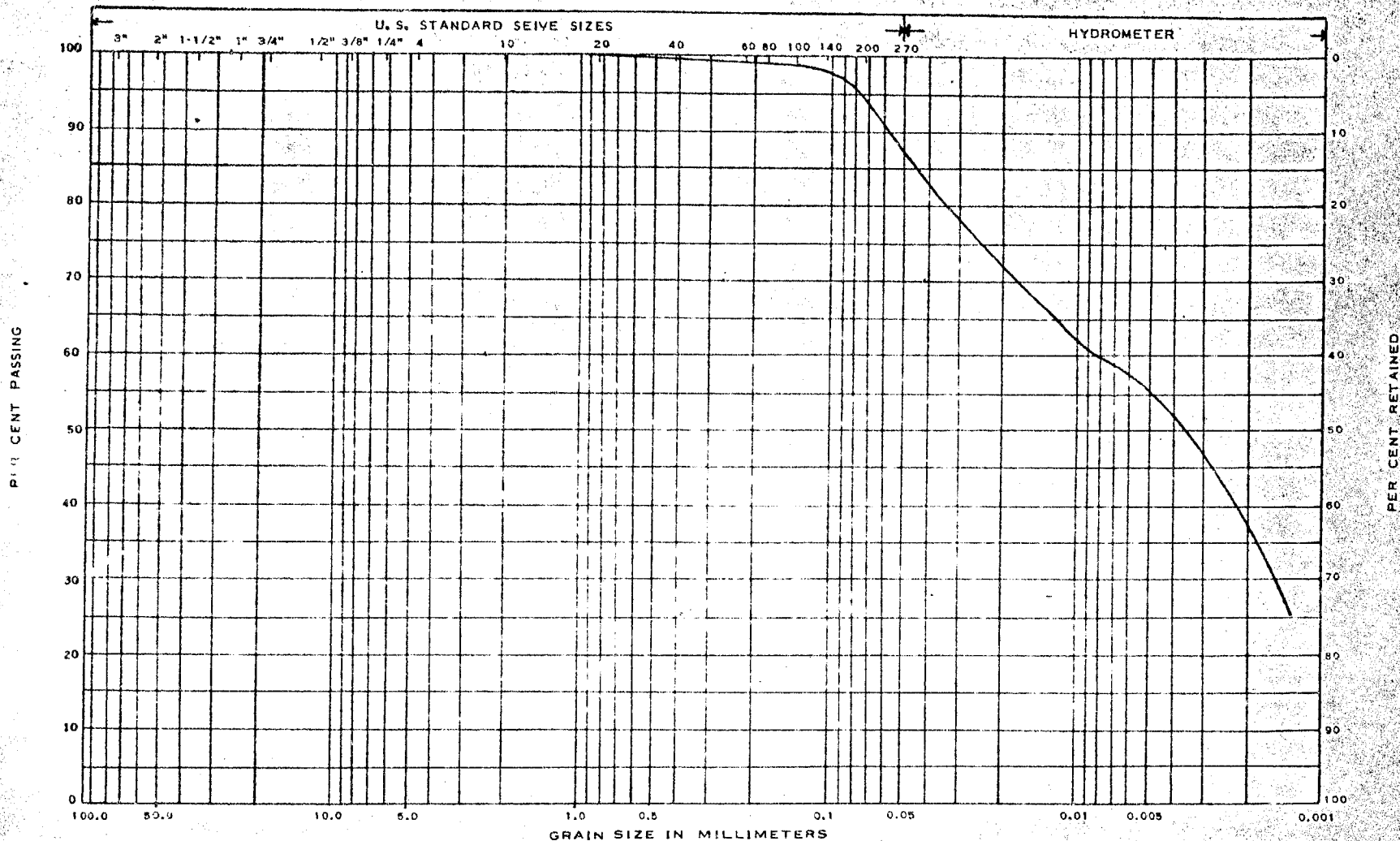
MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Heron Road, Ottawa JOB NO. 67 F 98 HOLE NO. 2 SAMPLE NO. 17
DEPTH 87'-89' ELEVATION _____ REMARKS Silty Clay

GRAIN SIZE DISTRIBUTION

e. m. peto associates ltd.

Toronto 19, Ontario



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE S. T	CLAY
--------	--------	-------------	-----------	-----------	-------------	-----------	-----------	------

MASS. INST. OF TECH. CLASSIFICATION

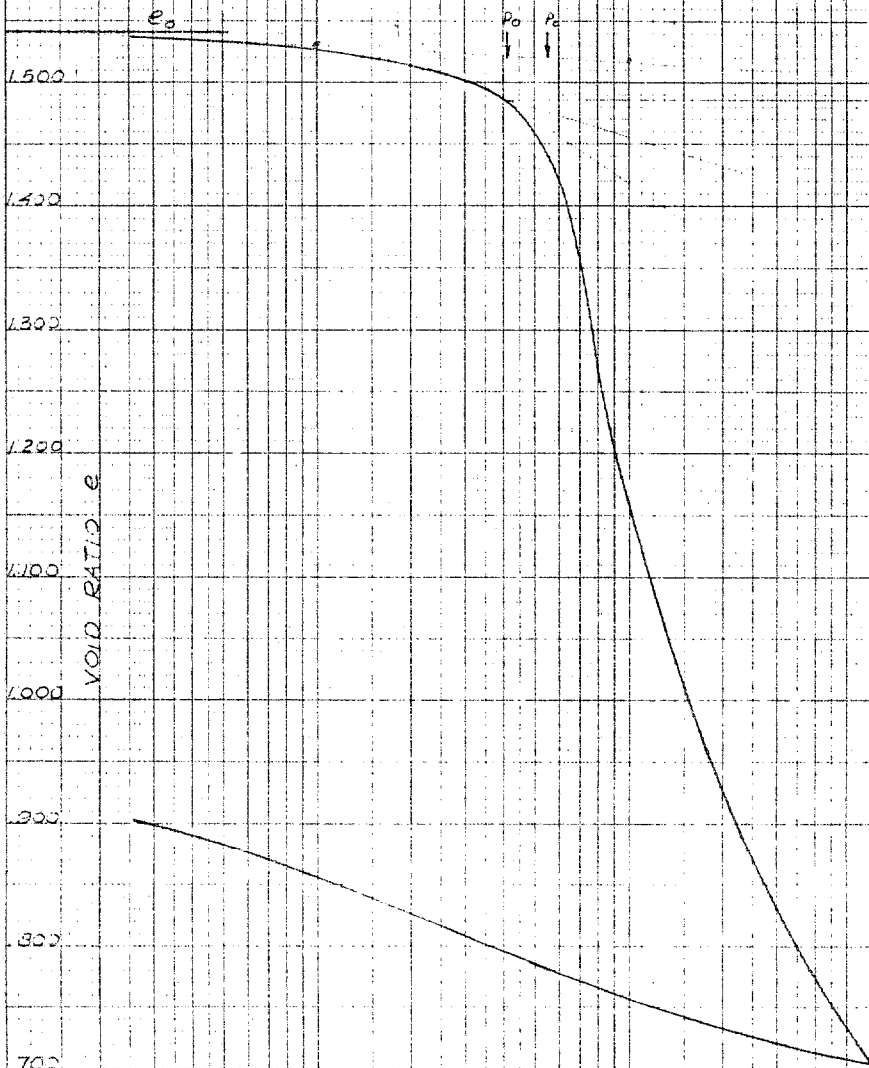
JOB NAME Heron Rd., Ottawa JOB NO. 67F98 HOLE NO. 2 SAMPLE NO. 14

DEPTH 67'-69' ELEVATION _____ REMARKS _____

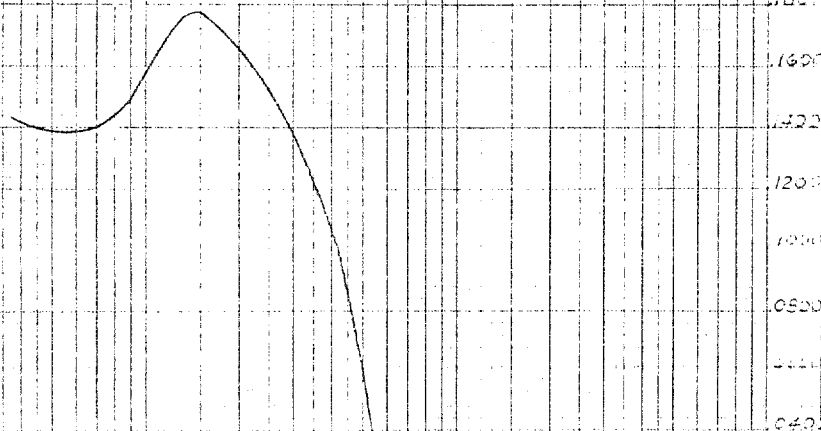
GRAIN SIZE DISTRIBUTION

CONSOLIDATION TEST

$C_c = 0.013$

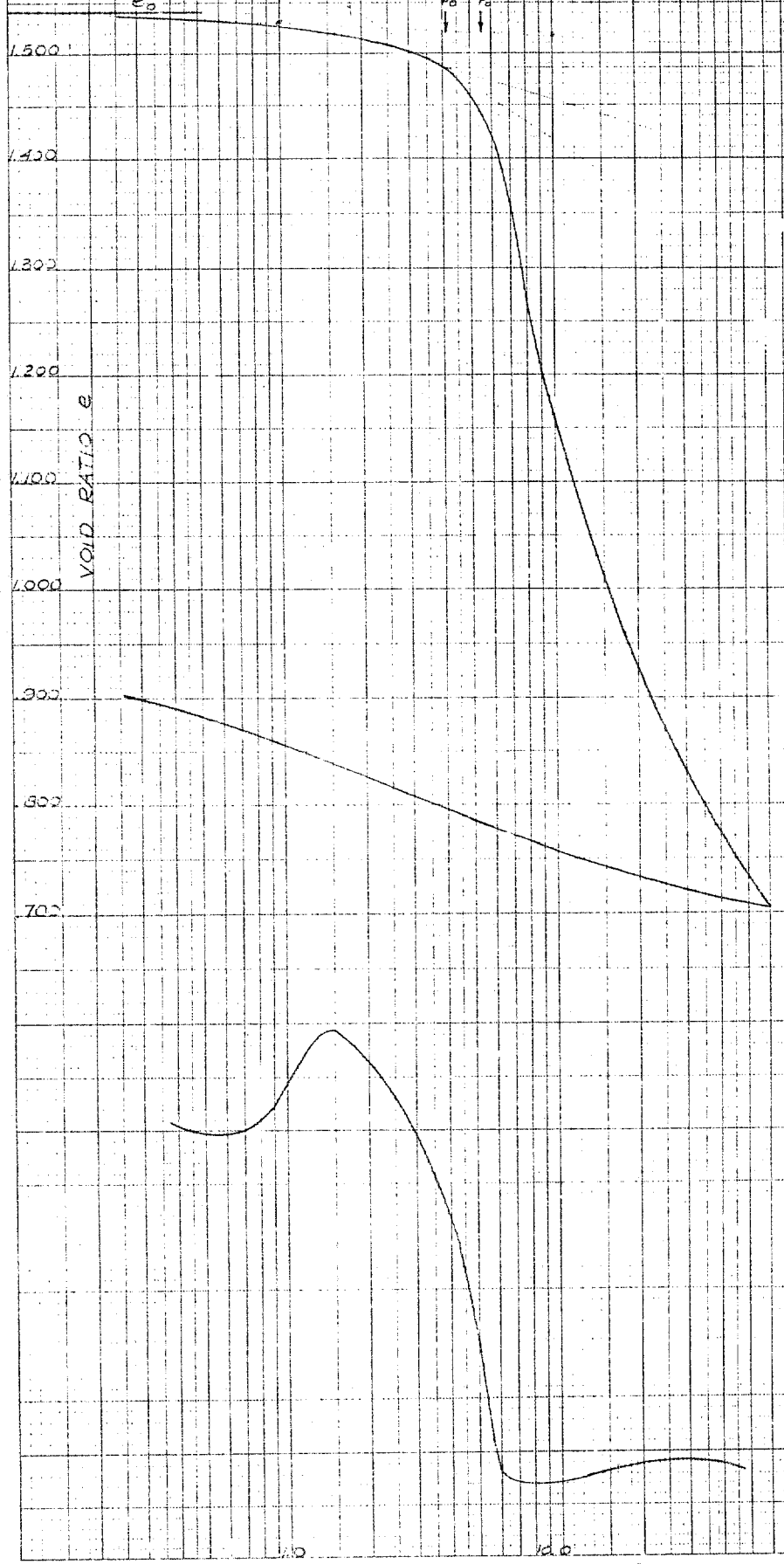


HOLE # 1
 SA # 11
 DEPTH 39'-41"
 $\gamma_{sat} = 120$
 $w_L = 53.1\%$
 $w_p = 35.0\%$
 $\gamma_{DL} = 66.3$ PCF
 $\gamma_{DT} = 63.4$ PCF



SA # 11

Ccr = 0.013



HOLE # 1
SA. # 11
DEPTH 39'-41"

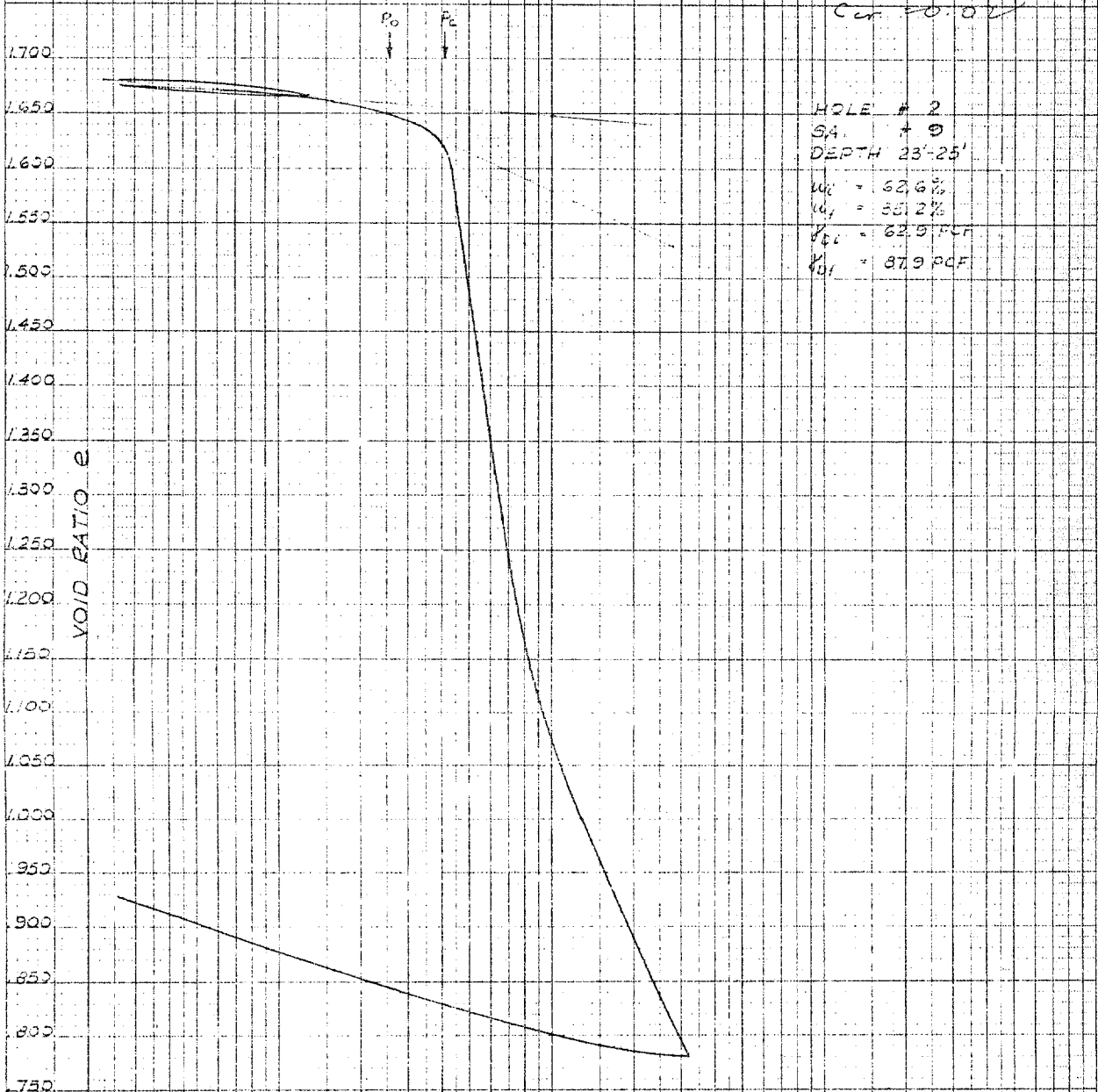
SA 190
 $w_s = 53.1\%$
 $w_f = 35.0\%$
 $V_{sc} = 66.3 \text{ PCF}$
 $V_{sf} = 63.4 \text{ PCF}$

1800
1600
1400
1200
1000
800
600
400
200
0000

Sq. ft.

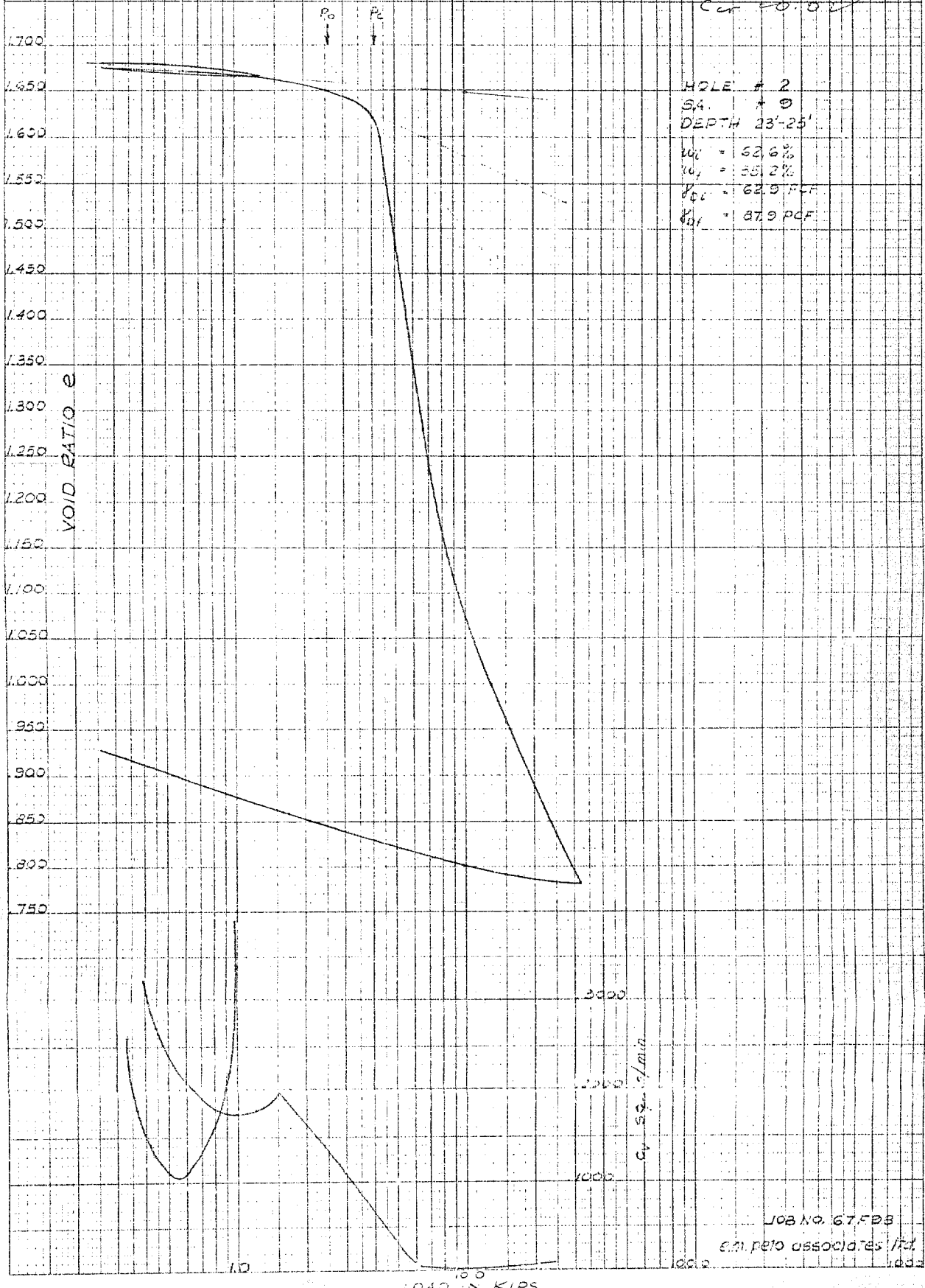
JOB NO. 67F98
empco associates, inc.

CONSOLIDATION TEST



CONSOLIDATION TEST

Cur 10.02



HOLE # 2
 SA. + 0
 DEPTH 23'-25'
 $w_L = 62.6\%$
 $w_p = 55.2\%$
 $\gamma_{sat} = 62.9 \text{ PCF}$
 $\gamma_{air} = 87.9 \text{ PCF}$

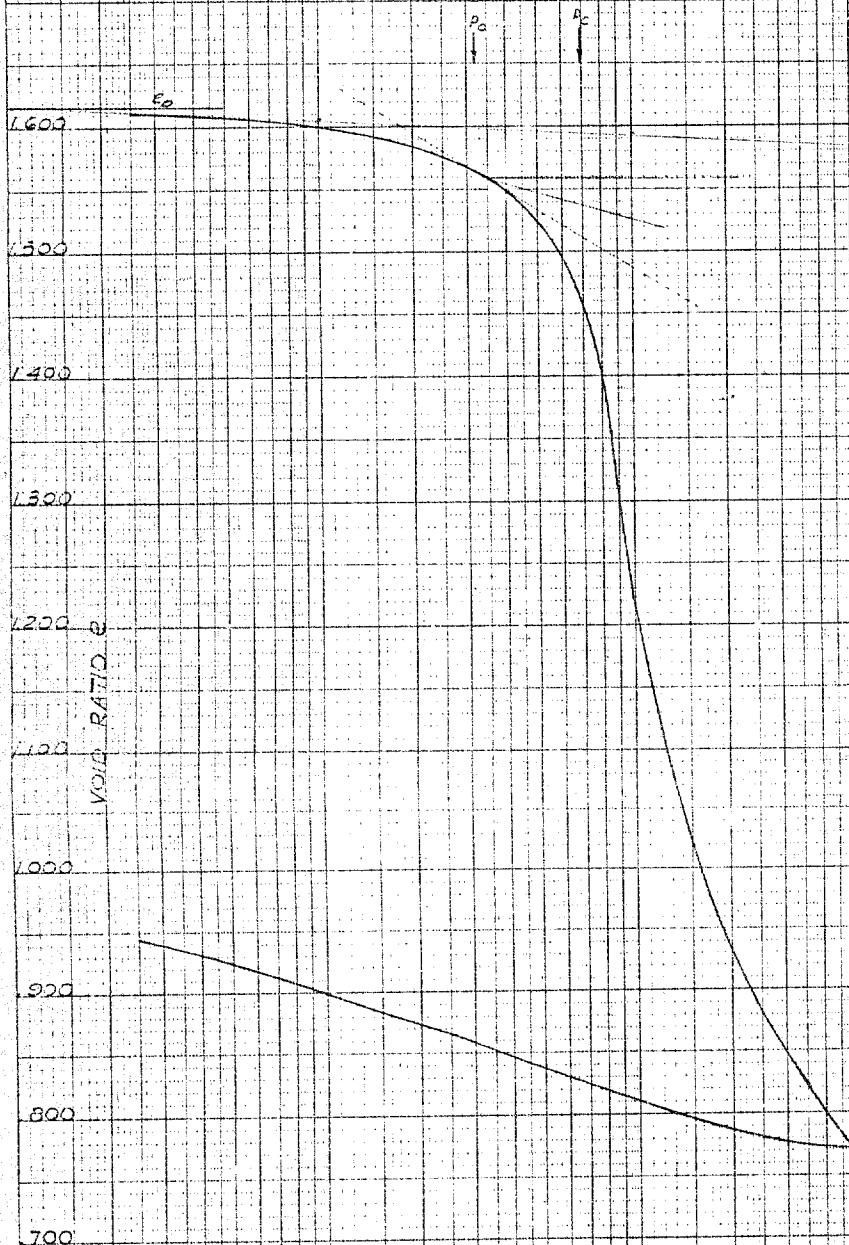
JOB NO. 67.FDB

emp. reo associates /rl

LOAD IN KIPS

CONSOLIDATION TEST

$C_r = 0.013$



HOLE # 2
SA. A. 10
DEPTH 31'-33'

3 X 190

$w_L = 60.7\%$

$w_p = 36.2\%$

$\gamma_{OL} = 64.7 \text{ PCF}$

$\gamma_{Of} = 86.8 \text{ PCF}$

0.7000

0.8000

0.9000

1.0000

1.1000

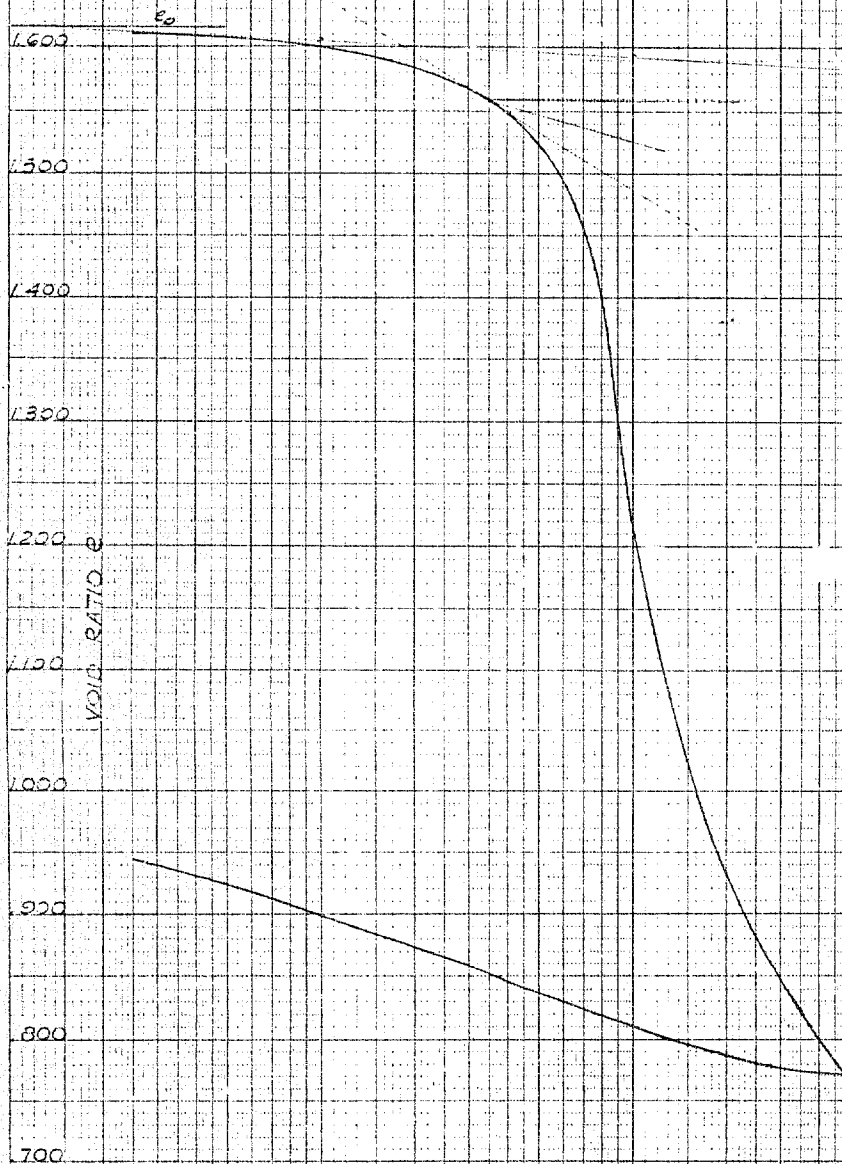
Cv 4.5 in/min

CONSOLIDATION TEST

$C_r = 0.013$

p_0

p_c



HOLE # 2

SA. # 10

DEPTH 3'-33"

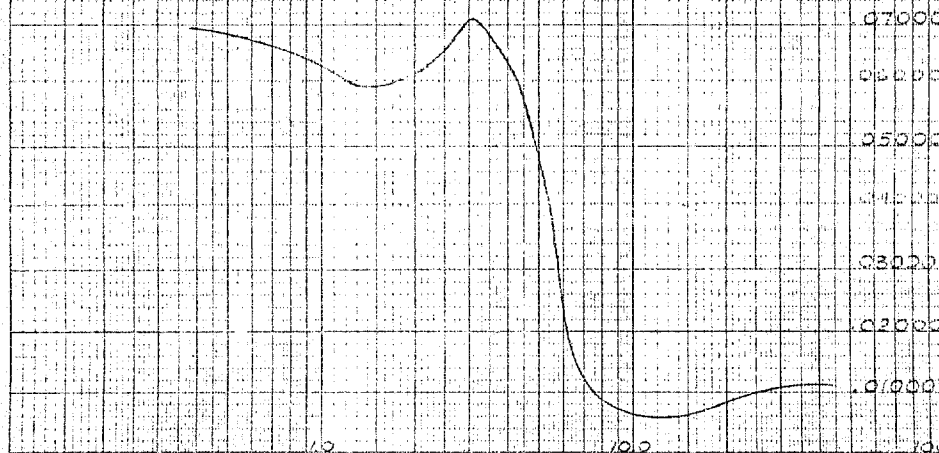
3 X 190

$w_L = 60.7\%$

$w_p = 36.2\%$

$\gamma_{so} = 64.7 \text{ PCF}$

$\gamma_{bf} = 86.8 \text{ PCF}$



0.7000

0.6000

0.5000

0.4000

0.3000

0.2000

0.1000

$C_v = 3.2 \text{ cm/yr}$

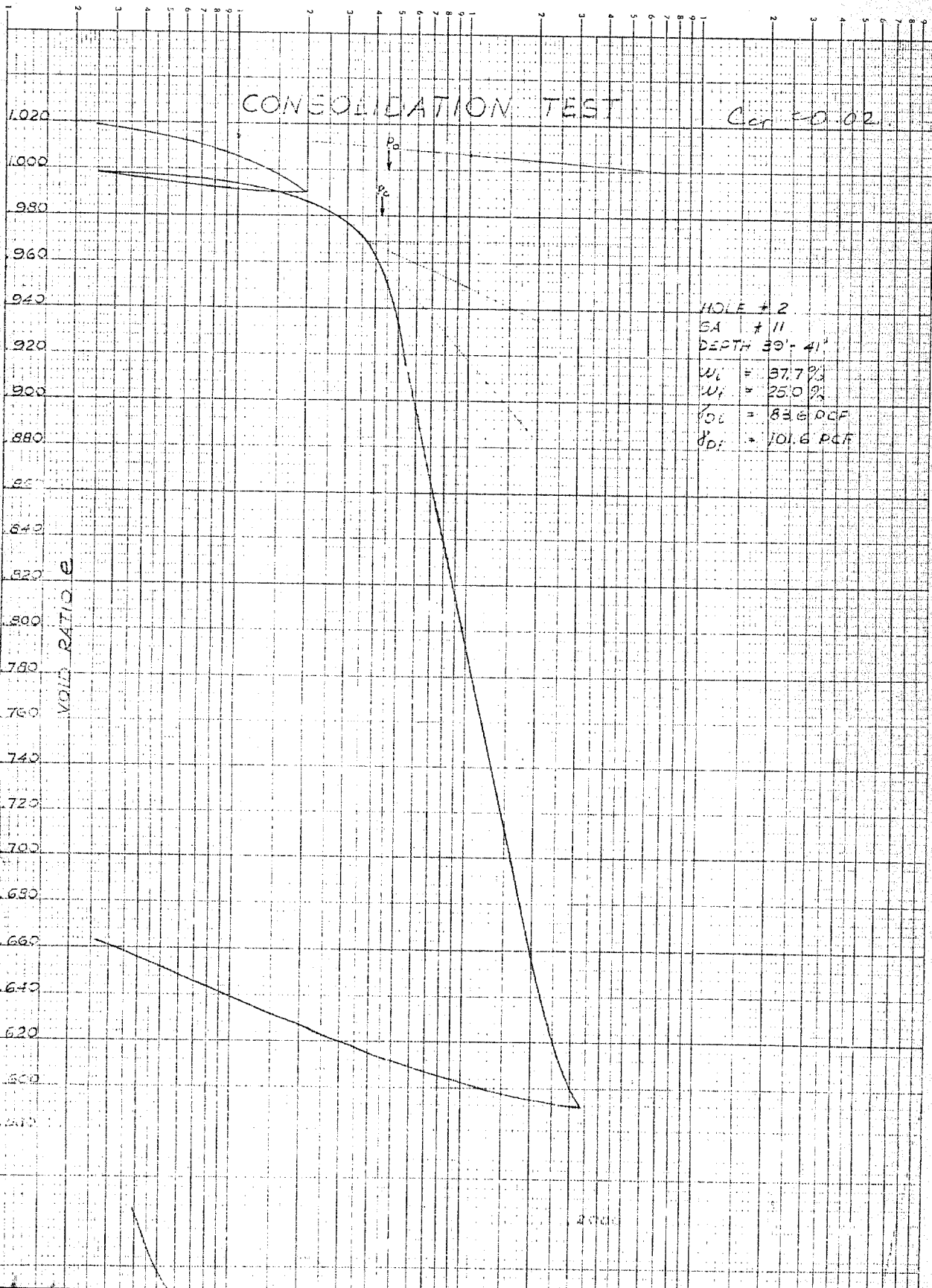
JOB NO. 67A93

Em. Pac. Associates, Inc.

LOAD IN KIIPS

CONSOLIDATION TEST

$C_{cr} = 0.02$



HOLE # 2
SA # 11
DEPTH 39' - 41'

$W_L = 37.7\%$
 $W_P = 25.0\%$
 $\gamma_{OL} = 83.6 \text{ PCF}$
 $\gamma_{DL} = 101.6 \text{ PCF}$

VOID RATIO e

CONSOLIDATION TEST

$C_{cr} = 0.02$

1020

1000

980

960

940

920

900

880

860

840

820

800

780

760

740

720

700

680

660

640

620

600

580

560

540

VOID RATIO e

p_0
 p_0

HOLE # 2

SA # 11

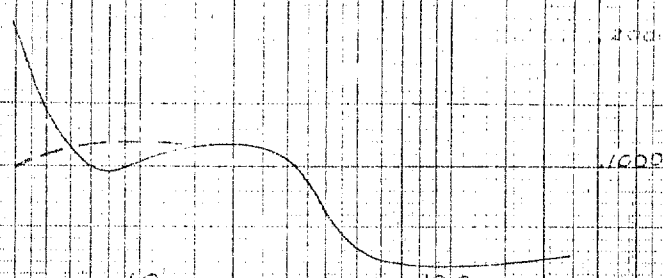
DEPTH 39'-41"

$w_L = 97.7\%$

$w_i = 25.0\%$

$\gamma_{DL} = 83.6 \text{ PCF}$

$\gamma_{DL} = 101.6 \text{ PCF}$

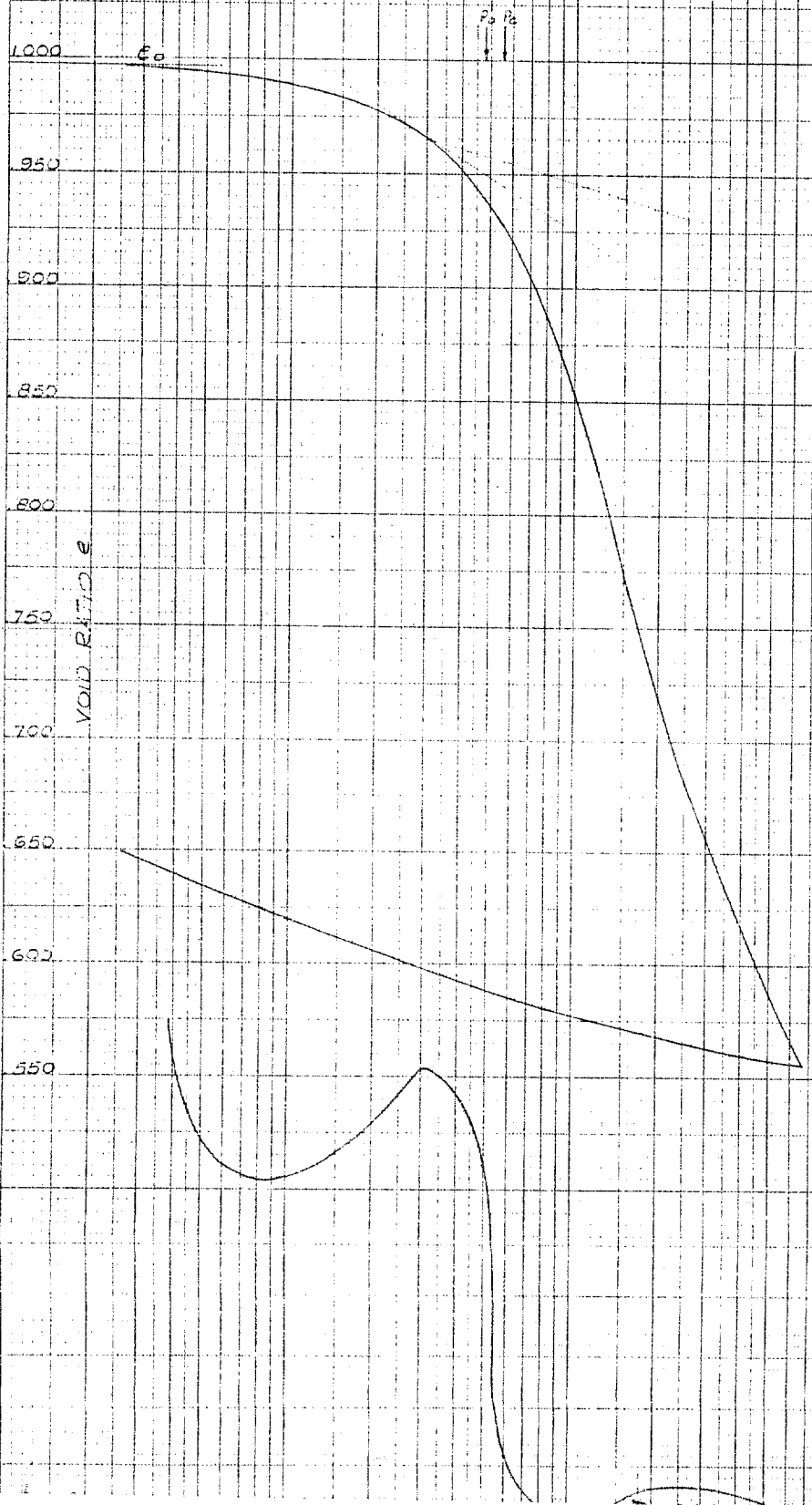


LOAD IN KIIPS

JDS # 67799

AM. BOLD ASSOCIATES, INC.

CONSOLIDATION TEST



HOLE # 2
SA # 12
DEPTH 47'-48'
 $w_L = 36.1\%$
 $w_f = 25.2\%$
 $\gamma_{dl} = 84.6 \text{ PCF}$
 $\gamma_{ut} = 102.5 \text{ PCF}$

1100

1000

0.300

0.250

0.200

0.150

0.500

0.400

0.300

Vertical Axis

CONSOLIDATION TEST

P_0 P_2

1000

e_0

950

900

850

800

750

700

650

600

550

VOID RATIO e

HOLE # 2
 SA # 12
 DEPTH 47'-48'
 w_L = 36.1%
 w_f = 25.2%
 ρ_{01} = 84.6 PCF
 ρ_{0f} = 102.5 PCF

1100

1000

0900

0800

0700

0600

0500

0400

0300

0200

0100

DEPTH

JOE NO. 67F88
 EMPLOYED 08/01/57

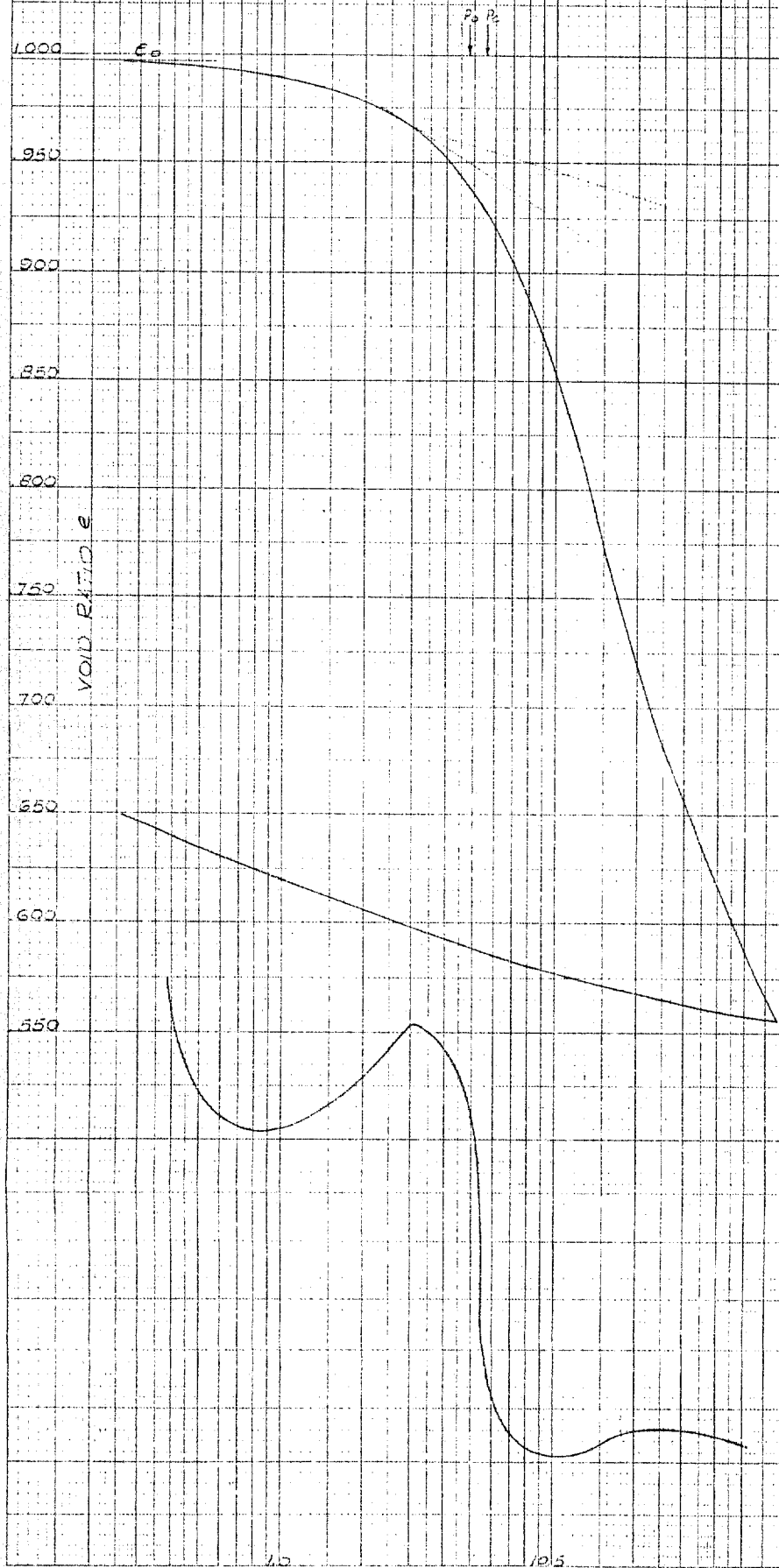
10

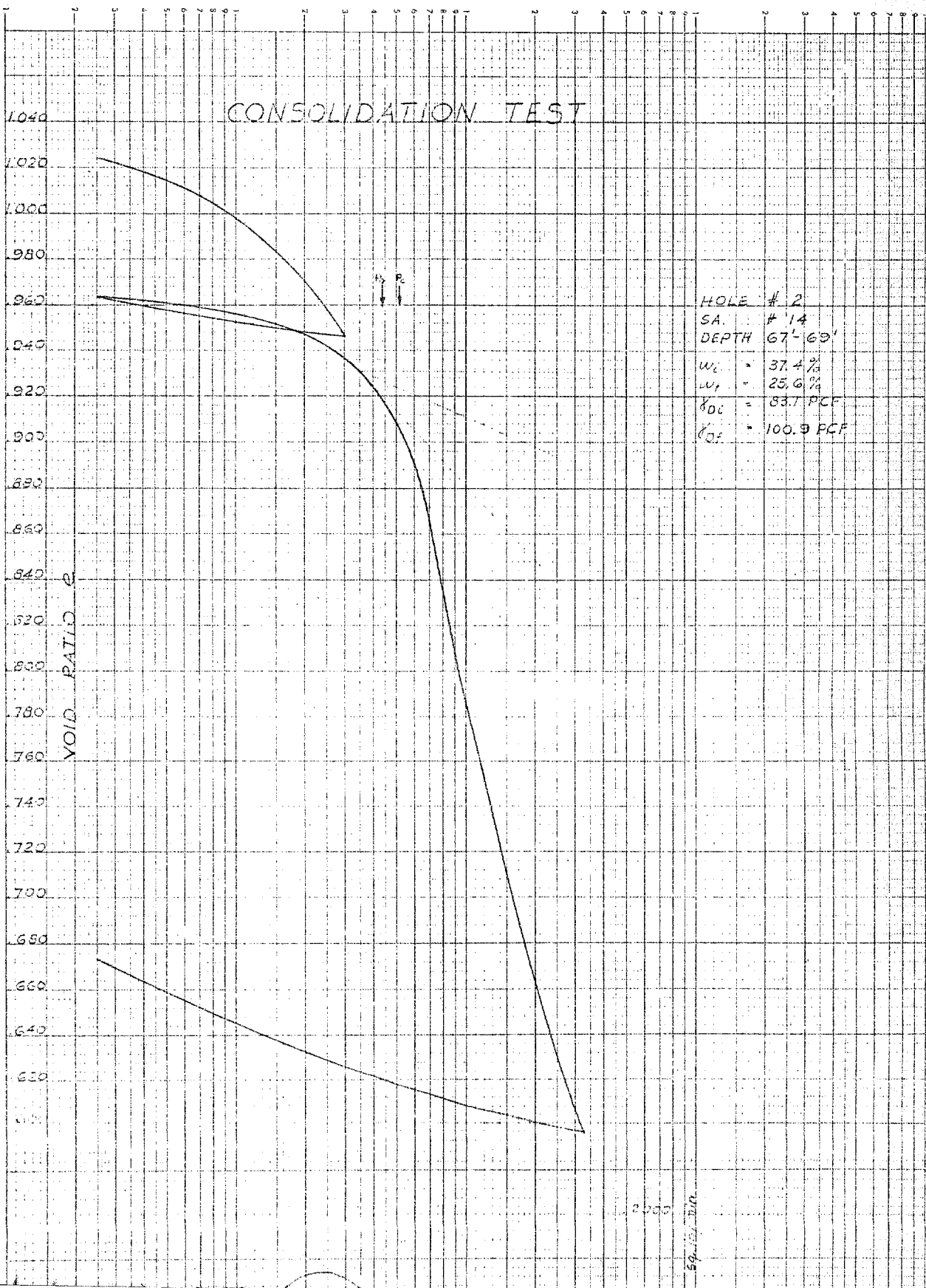
100

1000

10000

LOAD IN KIPS





CONSOLIDATION TEST

1040

1020

1000

980

960

940

920

900

880

860

840

820

800

780

760

740

720

700

680

660

640

620

600

VOID RATIO e

H_0 P_c

HOLE # 2

SA # 14

DEPTH 67'-63'

w_L = 37.4%

w_f = 25.6%

γ_{DL} = 83.1 PCF

γ_{DL} = 100.9 PCF

2000

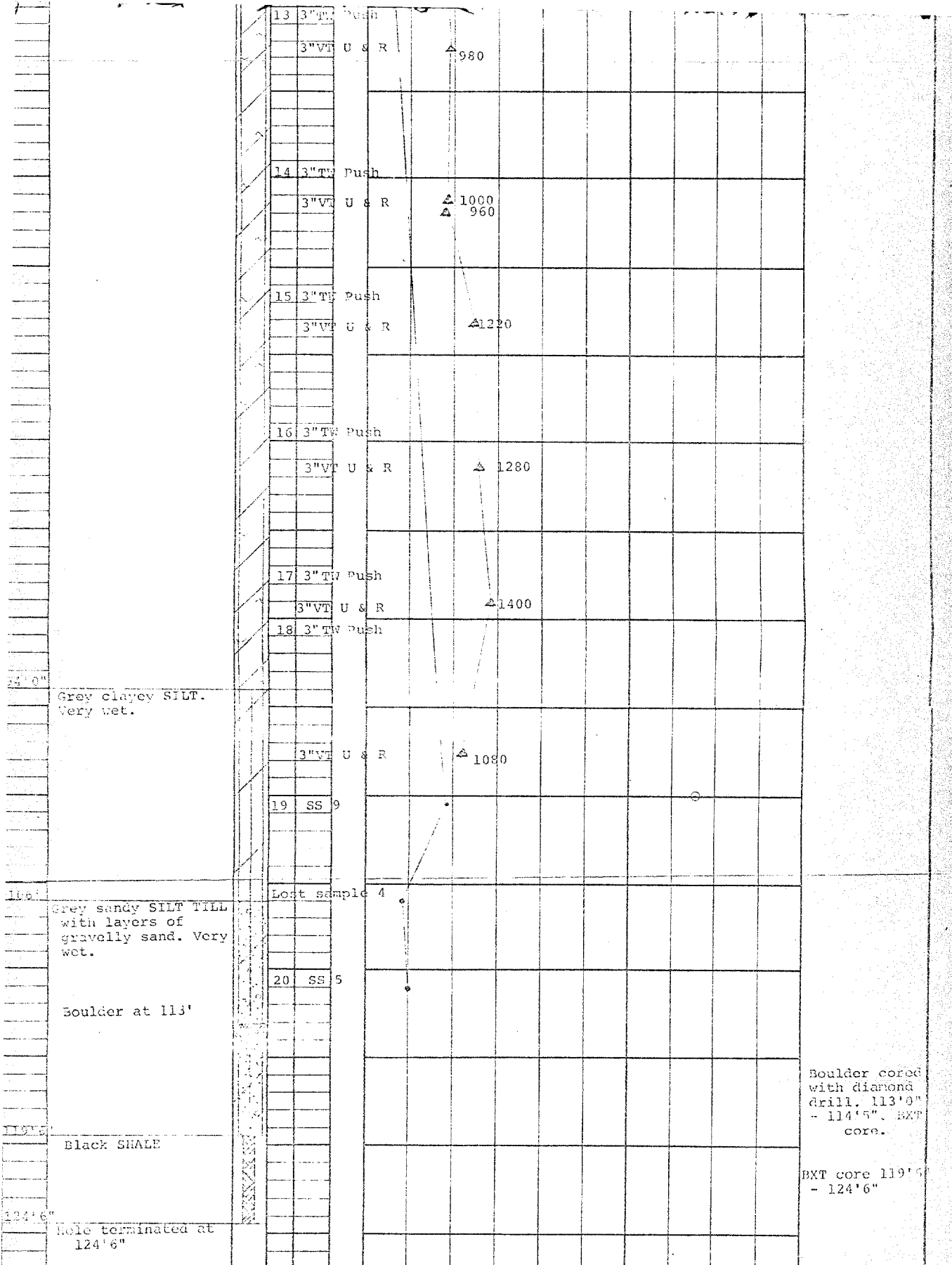
1000

CV 59.121 MIN

JOB NO. 67F98

EM. PERO ASSOCIATES, INC.

LOAD IN KIPS



e.m.peto associates ltd.

Consulting soil engineers

RECORD OF BOREHOLE NO. 2

JCB NO. 67 F98

JOB NAME Heron Road and C.P.R. Grade Separation

TECHNICIAN E.S.

BORING DATE Apr. 25-May 1/67 CLIENT City of Ottawa c/o M.M. Dillon & Co. Ltd.

ENGINEER A. N. S. E.

GROUND ELEV. 246.2

BOREHOLE TYPE 3 1/2" auger with "H" casing to 95'. Then BA to 130.8'

TYPED BY J.M.

[illegible]

851.

After sa. 17
"H" casing to
95'. Much
harder pushing
with hydraulic

12 3"PS Push

990

3"VT U & R

13 3"PS Push

3"VT U & R

1210

14 3"PS Push

3"VT U & R

1330

15 3"PS Push

3"VT U & R

1260

Some shell content.

16 3"PS Push

3"VT U & R

1720

Stiffer

Grey clayey silt with
some sand content.

17 3"PS Push

3"VT U & R

2030

After taking
104'6" sample
enlarged hole
with BX casing
shoe to advance
BX casing beyond
boulder at 97'.
Before taking
sa. 19 advanced
BX to 102'6".
After taking
sa. 19 advanced
BX to 108'.

93'0"

Sandy SILT TILL with
gravel pockets and
seams.
Cored boulder 97'3"
to 99'6".

Cored boulder from
101' to 102'.

Material has graded
to light grey fine
to coarse silty sand
till.

18 3"PS 16

19 5S 78

20 5S 28

21 5S 27

Artesian
Head

*Drove stone.

Some shell content.

15 3"PS Push
3"VT U & R

1260

16 3"PS Push
3"VT U & R

1720

Stiffer

Grey clayey silt with
some sand content.

17 3"PS Push
3"VT U & P

2030

After taking
104'6" sample
enlarged hole
with BX casing
shoe to advance
BX casing beyond
boulder at 97'.
Before taking
sa. 19 advanced
BX to 102'6".
After taking
sa. 19 advanced
BX to 108'.

93'0"

Sandy SILT TILL with
gravel pockets and
seams.
Cored boulder 97'3"
to 99'6".

18 3"PS
18 BS 16

Cored boulder from
101' to 102'.

Material has graded
to light grey fine
to coarse silty sand
till.

19 SS 78

78

20 SS 28

21 SS 27

Artesian
Head

22 SS 171 *

171

rove stone.

124'0"

Shale pieces and
boulders.

127'0"

Black weathered
clayey SILT TILL
Water bearing under
artesian pressure

130'0"

Black SHALE intact.

When tapped
this stratum
releases water
to 119'. Bailer
3 times, re-
turned always
to 119'.
5' of core
recovered.

Terminated hole
at 135'6"

Blue, grey
silty CLAY
Black organic specks

Some shell content

93'0"

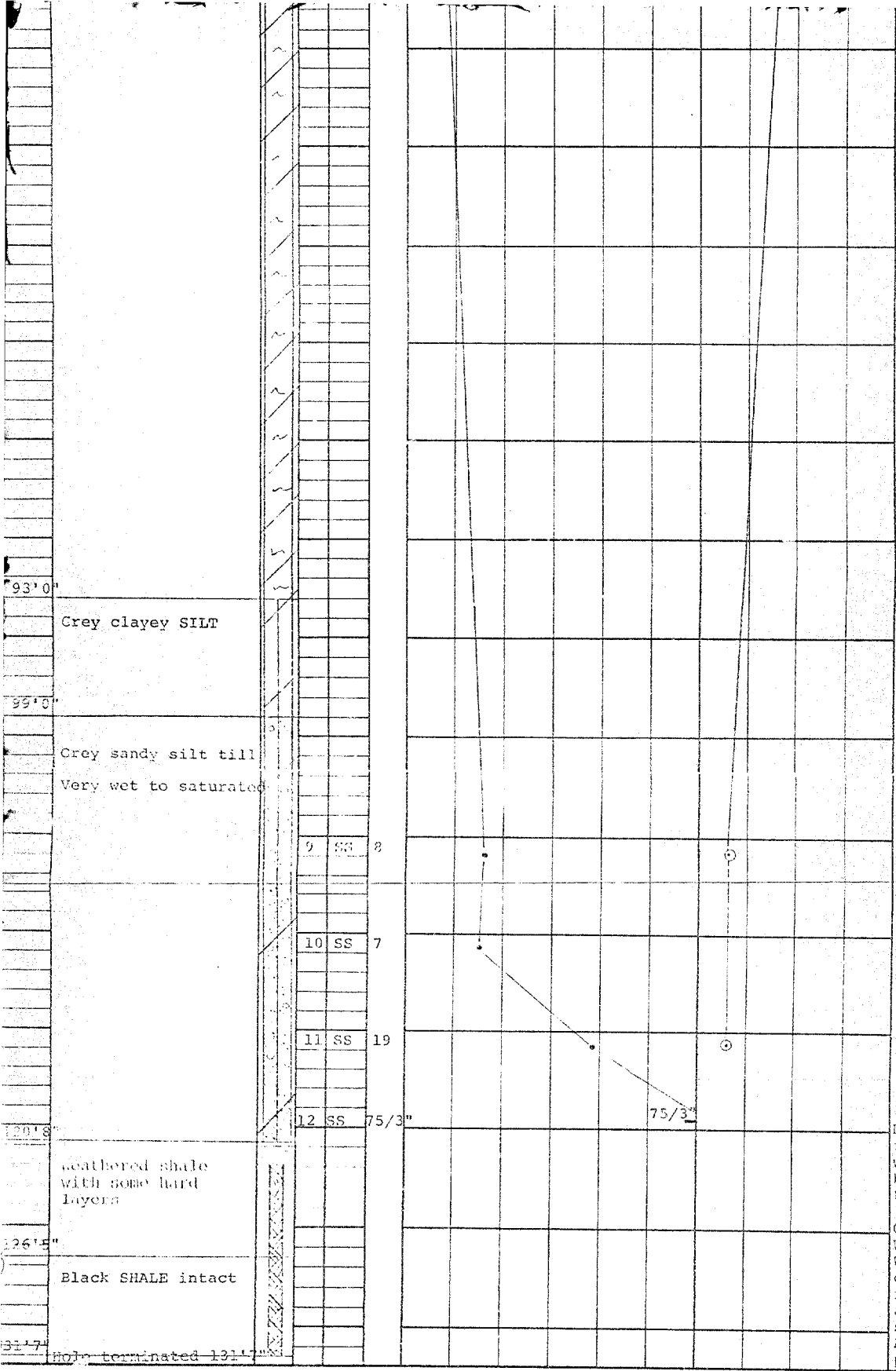
Grey clayey SILT

89'0"

Grey sandy silt till
Very wet to saturated

9 SS 2

10 SS 7



Diamond drilled
from 121'10"
with BXT core
barrel.
Full recovery
126'6" to 131'
Casing at 120'
Water at 19'2"
When pulling
casing water
reached 3' to
above ground
surface.
Artesian or gas
pressure between
123' & 126'
suspected.

Blue-grey very silty
CLAY

Black organic specks

87'0"

Grey clayey SILT

84'5"

Grey sandy silt till
Very wet.

Pockets and seams
of gravelly sand.

Becoming black

9 SS 6

10 SS 10

11 SS 24

12 SS 71

71 -

Boulder at
119'6"
More small
boulders to
122'

137'0"

Grey clayey SILT

134'0"

Grey sandy silt till
Very wet.

9 SS 6

10 SS 10

11 SS 24

Pockets and seams
of gravelly sand.

Becoming black

12 SS 71

71 -

Grading to silty
sand.

133'0"

Black SHALE intact.

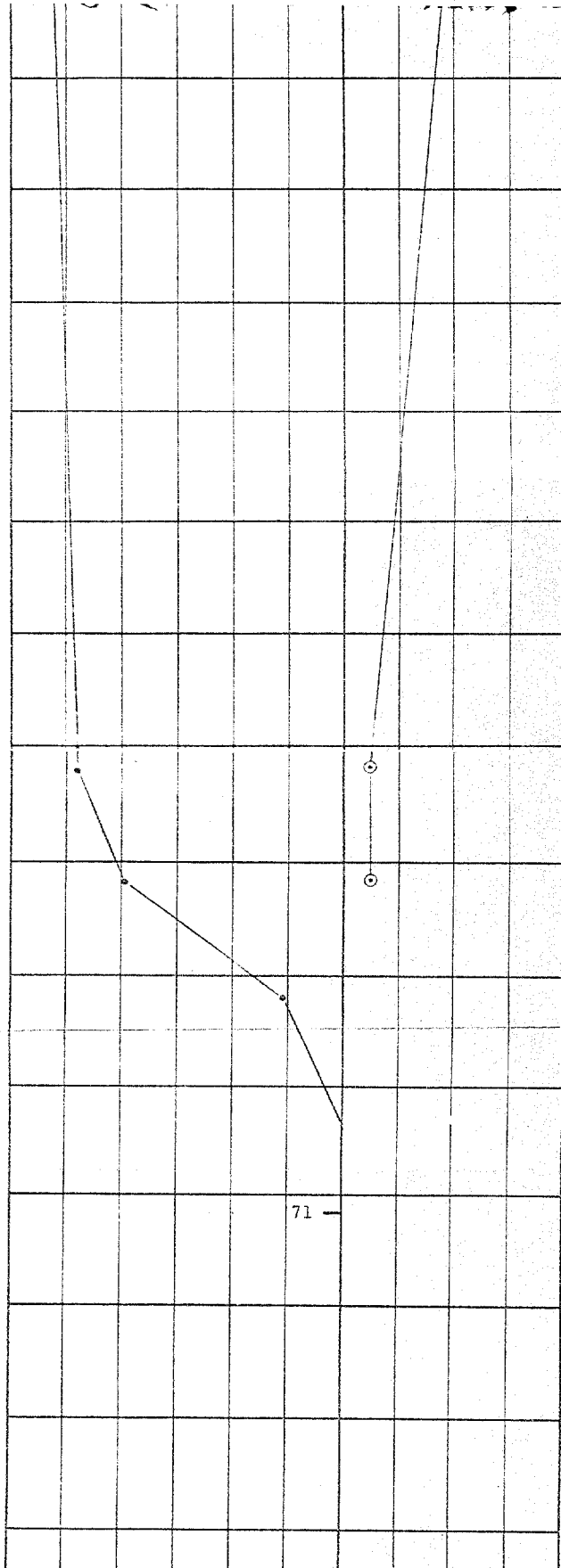
137'0"

Borehole terminated
at 137'0"

Boulder at
119'6"
More small
boulders to
122'

Casing to 125'
Water at 13'0"

Cored AXT
133'-137'
Water at 13'0"
on completion



e.m. peto associates ltd.

RECORD OF BOREHOLE NO. 5

Consulting soil engineers

JOB NO. 67 F98

JOB NAME Huron Road & C.P.R. Grade Separation

TECHNICIAN E.S.

BORING DATE May 3, 4/67

CLIENT City of Ottawa c/o N.M. Dillon & Co. Ltd.

ENGINEER A.N.S.B.

GROUND ELEV. 252.5

BOREHOLE TYPE 3 1/2" Auger and "H" casing to 90'

TYPED BY J.W.

SOIL PROFILE

SAMPLES

DYNAMIC CONE PENETRATION
BLOWS/FOOT
STANDARD PENETRATION TEST
BLOWS/FOOTLIQUID LIMIT _____ W_L
PLASTIC LIMIT _____ W_P
WATER CONTENT _____ W

REMARKS

DEPTH
ELEV.

DESCRIPTION

LEGEND

NUMBER

TYPE

BLOWS/FOOT

5 10 15 20 25

Vane SHEAR STRENGTH C_u LB/SQ. FT ▲

300 1000 1500

W_P W W_L
WATER CONTENT %
20 40 60

2'0"

Dark brown stony
silty clay FILL

1

SS

14

Brown grey silty CLAY
STIFF, about plastic
limit.

2

3"TW

Push

3

SS

7

1/2" seam of fine sand
becoming wetter than
plastic limit.

4

SS

5

3'6"

Grey very silty CLAY.
Much wetter than
plastic limit.
Very soft.

5

3"TW

Push

6

SS

2

7

SS

-1

/

3"TW

Push

8

3"PS

Push

3"VT

U & R

▲ 640

27'0"

Blue - grey silty
CLAY, black organic
specks, some shell
content.

9

AS

3"VT

U & R

▲ 1580*

3"VT

U & R

* Seems high,
but was
checked/

3"VT

U & R

▲ 740

After taking
vane at 30'
installed "H"
casing to 45'After taking
vane test at
45' installed
"H" casing to
60'

After taking
vane test at
45' installed
"H" casing to
60'

11 AS

3"VT U & R

▲ 730

After taking
vane test,
installed
casing, to 75'

12 AS

3"VT U & R

▲ 940

After Vane
test, "H"
casing to 90'

13 AS

3"VT U & R

▲ 1620

Light grey clayey
SILT. Much wetter
than plastic limit.

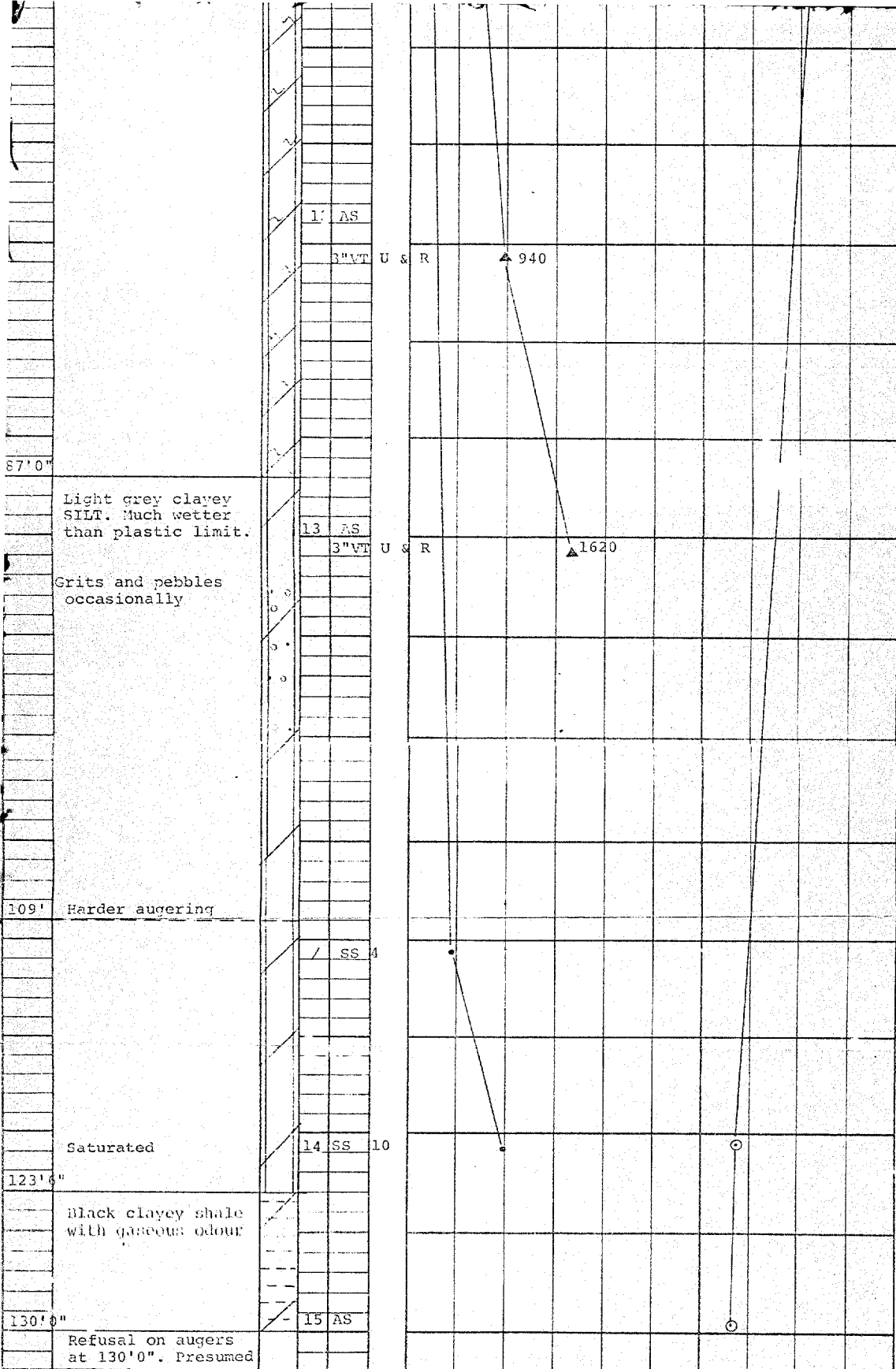
Grits and pebbles
occasionally

87'0"

109' Harder augering

SS 4

W.L. on com-
pletion of
hole 5:20 pm



After Vane test, "H" casing to 90'

W.L. on completion of hole 5:00 pm May 4th -16' 0"
Next morning W.T. 2' 2"

JOB NO. 67 P98 JOB NAME Heron & C.P.R. Grade Separation, TECHNICIAN F.S.
 BORING DATE May 12/67 CLIENT City of Ottawa c/o M.M. Dillon Ltd., ENGINEER A.N.S.B.
 GROUND ELEV 251.6 BOREHOLE TYPE 3 1/2" Auger hole uncased TYPED BY J.W.

[illegible]

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Consulting soil engineers

RECORD OF BOREHOLE NO. 7

JOB NO. 67 F98	JOB NAME Heron Road & C.P.R. Grade Separation	TECHNICIAN E.S.
BORING DATE May 11/67	CLIENT City of Ottawa c/o M.M. Dillon Ltd.	ENGINEER A.N.S.B.
GROUND ELEV. 254.6	BOREHOLE TYPE 3 1/2" Auger & "H" casing to 10'	TYPED BY J.B.

SOIL PROFILE		SAMPLES		DYNAMIC CONE PENETRATION BLOWS/FOOT STANDARD PENETRATION TEST BLOWS/FOOT		LIQUID LIMIT _____ W _L PLASTIC LIMIT _____ W _P WATER CONTENT _____ W		REMARKS
DEPTH ELEV.	DESCRIPTION	LEGEND	NUMBER	TYPE	5 10 15 20 25	Vane SHEAR STRENGTH C _u LB/SQ. FT ▲	W _p W W _L WATER CONTENT %	
0'0"						500 1000 1500	20 40 60	
9'5"	Mixed brown and grey sandy to clayey FILL with stone and cinder content. Very moist, loose to compact.		1	SS	10			
12'6"	Grey brown silty CLAY. Wetter than plastic limit. Stiff to firm.		2	SS	14			
			3	3"TW	Push			
			4	SS	8			
			5	SS	4			
			6	SS	3			
15'9"	Grey silty CLAY. Much wetter than plastic limit. Soft.		7	3"TW	Push	1380		
	Hole terminated at 15'9"		8	VT	Y&R			

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RECORD OF BOREHOLE NO. 8

Consulting soil engineers

JOB NO. 67 F98

JOB NAME Heron Road & C.F.R. Grade Separation

TECHNICIAN E.S.

BORING DATE May 11/67

CLIENT City of Ottawa c/o M.M. Dillon

ENGINEER A.N.S.B.

GROUND ELEV. 255.6

BOREHOLE TYPE 3 1/2" Flight Auger and "H" casing to 20'

TYPED BY J.W.

SOIL PROFILE		SAMPLES		DYNAMIC CONE PENETRATION BLOWS/FOOT STANDARD PENETRATION TEST BLOWS/FOOT				LIQUID LIMIT _____ WL PLASTIC LIMIT _____ WP WATER CONTENT _____ W			REMARKS
DEPTH ELEV.	DESCRIPTION	LEGEND	NUMBER	TYPE	BLOWS/FOOT	Vane SHEAR STRENGTH C _u LB/SQ. FT. ▲				W _p W W _L WATER CONTENT % 20 40 60	
0'0"	Grassed topsoil					500	1000	1500	2000		
0'0"	Brown silty clay FILL with stone and brick pieces.										
	Grey brown silty CLAY. Wetter than plastic limit. Stiff becoming soft and much wetter than plastic limit.		1	SS	15						W.T. 3'4"
			2	3"TW	Push						
			3	SS	11						
			4	3"TW	Push						
			5	SS	3						Minor seepage at 9'1"
			6	SS	3						After 11'6" sa. hole dry.
13'6"											
	Grey silty CLAY. Much wetter than plastic limit. Very soft.		7	3"TW	Push						After 13'6" sa. W.L. 10'4"
				3"VW	U&R		890				After 5 min. '0"
			8	3"TW	Push						Then installed "H" casing to 15'.
				3"VW	U&R		1070				After 17' tube installed Casing to 20'
			9	3"TW	Push						Before pulling casing no water in borehole.
				3"VW	U&R		1380				After pulling casing W.L. 22'0"
24'3"	Hole terminated at 24'3"										Final W.L. reading taken May 12th 3'4"

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Consulting soil engineers

RECORD OF BOREHOLE NO. 9

JOB NO. 67 F98

JOB NAME Heron Road & C.P.R. Grade Separation

TECHNICIAN E.S.

BORING DATE May 10/67

CLIENT City of Ottawa c/o H.M. Dillon & Co. Ltd.

ENGINEER A.N.S.B.

GROUND ELEV. 254.9

BOREHOLE TYPE 3 1/2" Auger & "H" casing to 29'6"

TYPED BY J. W.

SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION BLOWS/FOOT STANDARD PENETRATION TEST BLOWS/FOOT		LIQUID LIMIT _____ W _L PLASTIC LIMIT _____ W _P WATER CONTENT _____ W		REMARKS
DEPTH ELEV.	DESCRIPTION	LEGEND	NUMBER	TYPE	BLOWS/FOOT	STRENGTH C _u LB/SQ. FT ▲		W _P W W _L WATER CONTENT %		
						5 10 15 20 25		20 40 60		
0'0"	Topsoil									
0'3"	Grey brown silty CLAY									
	Wetter than plastic limit, stiff.		1	SS	15					
			2	3"TW	Push					
			3	SS	11					
			4	SS	7					
			5	3"TW	Push					
10'6"			6	SS	2					
	Grey silty CLAY		7	SS	-1					
	Much wetter than plastic limit.		8	3"TW	Push					
	Very soft.			3"VT	U					
				3"VT	U&R					
			9	3"TW	Push					
				3"VT	U&R					
	Becoming very silty		10	3"TW	Push					
				3"VT	U&R					
33'3"										
	Hole terminated at 33'3"									

W.T. 5'8"

Minor seepage at 10'±

After 11'6" W.L. 8'0"

After 5 min. W.L. 7'0"

After taking 13' Shelby installed "H" casing to 16'0"

After 23'6" sa. installed casing to 29'6"

Hole dry before pulling casing.

After pulling casing W.L. 27'7"

Final water level established May 12 5'8"

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RECORD OF BOREHOLE NO. 10

Consulting soil engineers

JOB NO. 67 P98

JOB NAME Heron Road & C.P.R. Grade Separation

TECHNICIAN E.S.

BORING DATE May 5/67

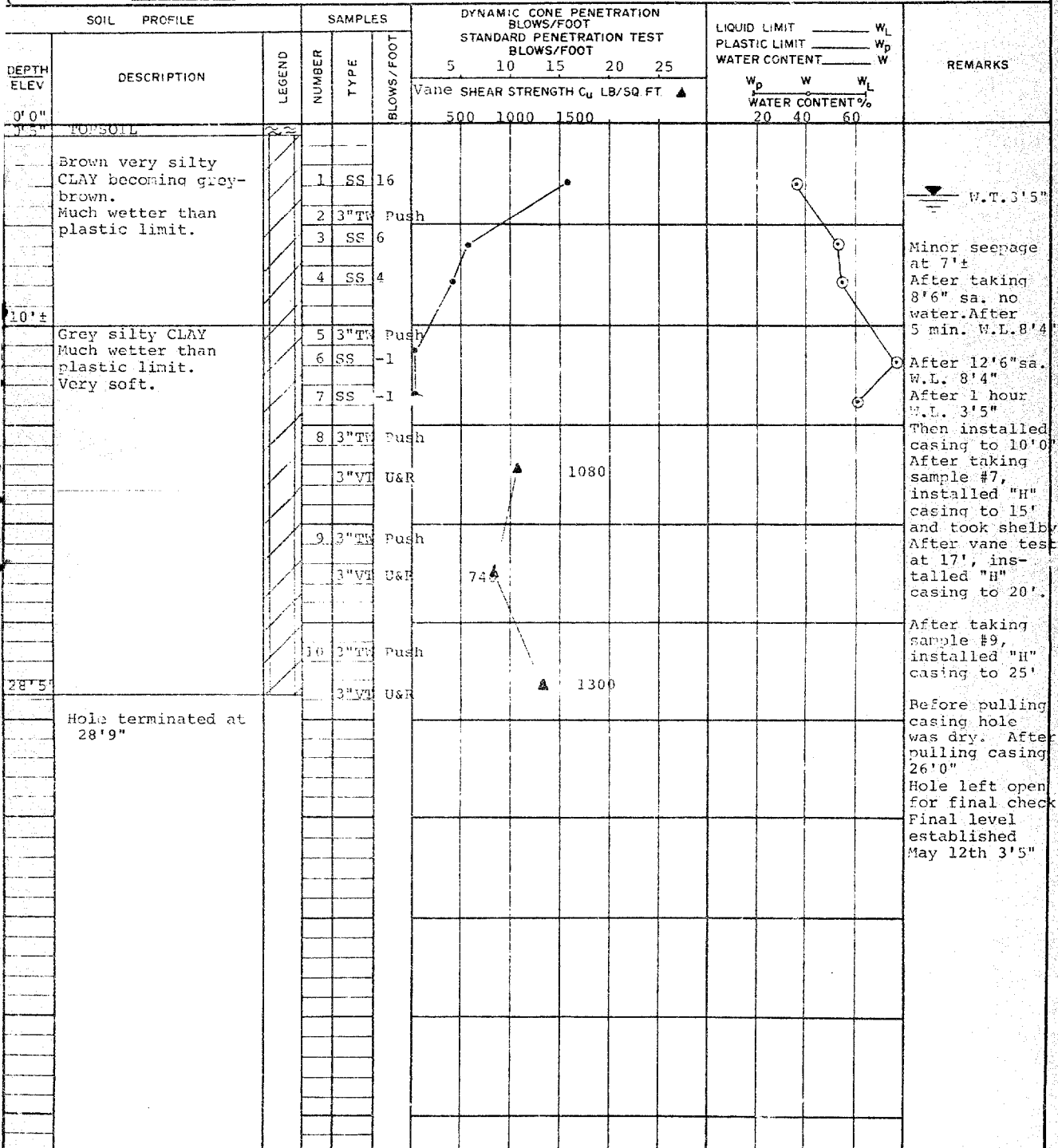
CLIENT City of Ottawa c/o M.L. Dillon & Co.Ltd.

ENGINEER A.N.S.B.

GROUND ELEV. 252.9

BOREHOLE TYPE 3 1/2" Auger & "H" casing to 25'

TYPED BY J.V.



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RECORD OF BOREHOLE NO. 11

Consulting soil engineers

JOB NO. 67 P98

JOB NAME Heron Road & C.P.R. Grade Separation

TECHNICIAN E.S.

BORING DATE May 5 & 6/67 CLIENT City of Ottawa c/o M.M. Dillon & Co. Ltd.

ENGINEER A.N.S.B.

GROUND ELEV. 252.7

BOREHOLE TYPE 20' "H" casing and 3 1/2" auger

TYPED BY J.W.

DEPTH ELEV.	SOIL PROFILE DESCRIPTION	LEGEND	SAMPLES		BLOWS/FOOT	DYNAMIC CONE PENETRATION BLOWS/FOOT STANDARD PENETRATION TEST BLOWS/FOOT					LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			REMARKS
			NUMBER	TYPE		5	10	15	20	25	W_p	W	W_L	
0'0"						Vane SHEAR STRENGTH C_u LB/SQ. FT. ▲					WATER CONTENT %			
0'6"						500	1000	1500			20	40	60	
0'6"	TO SOIL													
	Brown to grey silty CLAY. Much wetter than plastic limit. Firm.		1	SS	10									
			2	3"TW	Push									
			3	SS	7									
			4	SS	4									
9'±			5	3"TW	Push									
	Grey silty CLAY. Much wetter than plastic limit. Very soft.		6	SS	2									
			7	SS	<1									
			8	3"TW	Push									
				3"VT	U&R									
			9	3"TW	Push									
				3"VT	U&R									
			10	3"TW	Push									
24'3"	Testhole terminated at 24'3"			3"VT	U&R									

Seepage at 7'±
After 11'6"sa.
W.L. 7'0"
After 11'6"sa.
installed
casing to 10'0"
Before taking
vane test at
15' installed
"H" casing to
15'
W.L. next morn-
ing in casing
to 7'0"

After sample
#9, casing to
20'.

No other see-
page other
than the 7'
level noted in
above hole.
After pulling
casing muck
in bottom of
hole.
Hole left open
for final
check.

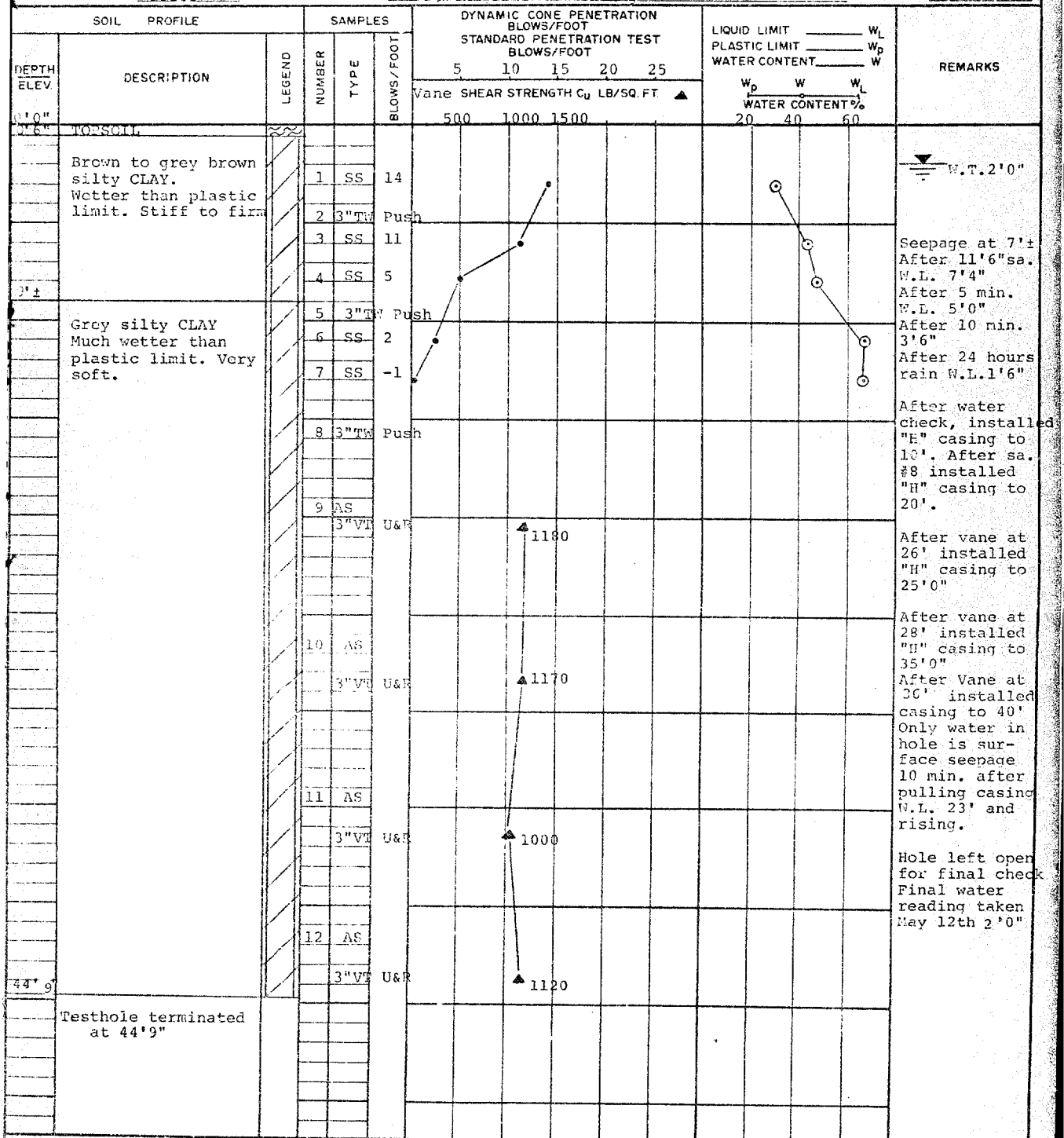
Note final
level could not
be established
because of
surface water
in area on
May 12th water
collected to
7" below sur-
face.

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RECORD OF BOREHOLE NO. 12

Consulting soil engineers

JOB NO. 67 F98 JOB NAME Heron Road & C.P.R. Grade Separation TECHNICIAN E.S.
 BORING DATE May 8&10/67 CLIENT City of Ottawa, c/o M.M. Dillon & Co. Ltd. ENGINEER A.N.S.B.
 GROUND ELEV. 253.5 BOREHOLE TYPE 3 1/2" Auger & "H" casing TYPED BY J.W.

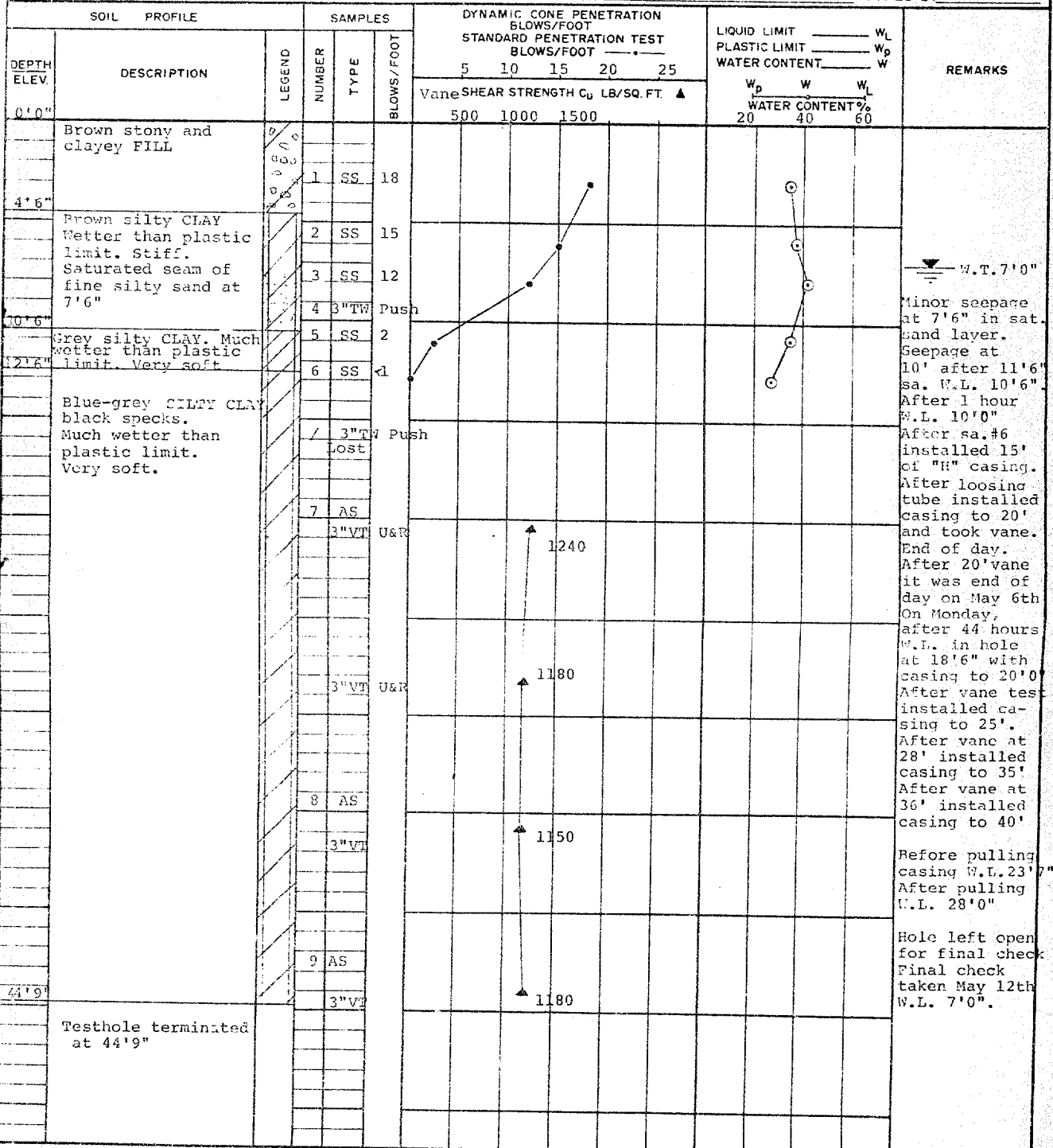


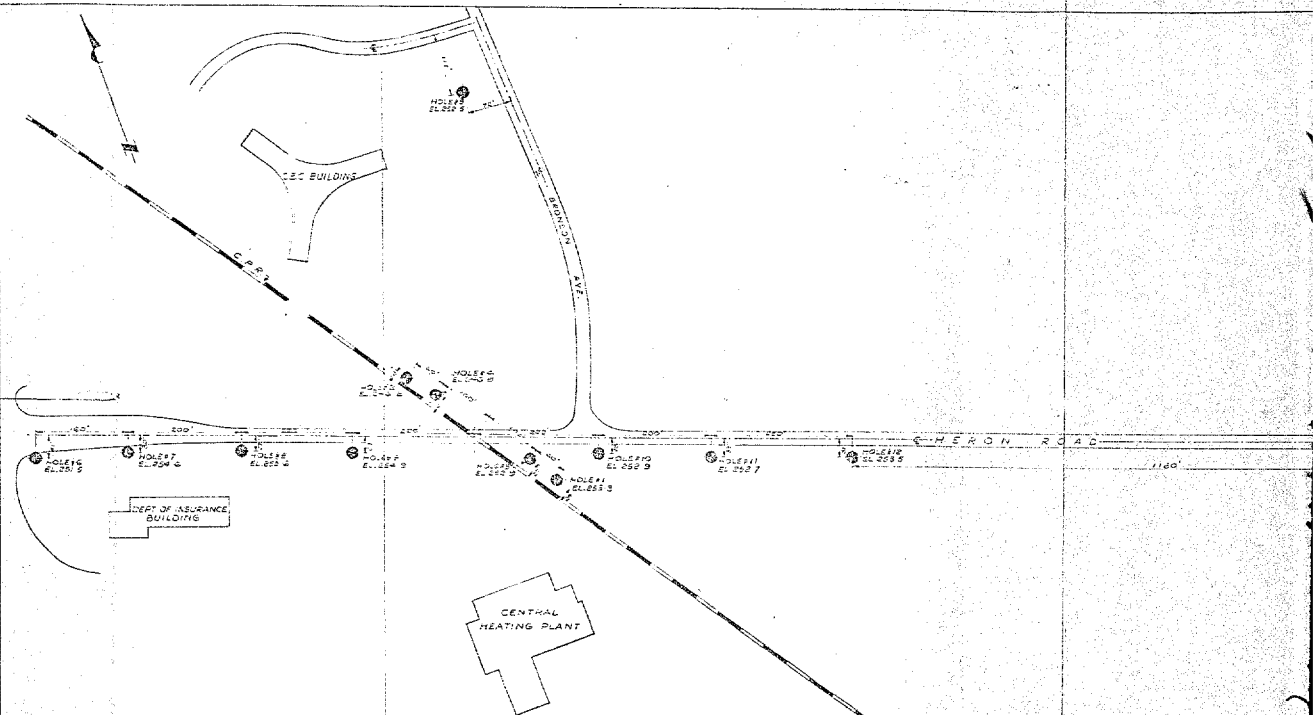
e.m. peto associates ltd.

RECORD OF BOREHOLE NO. 13

Consulting soil engineers

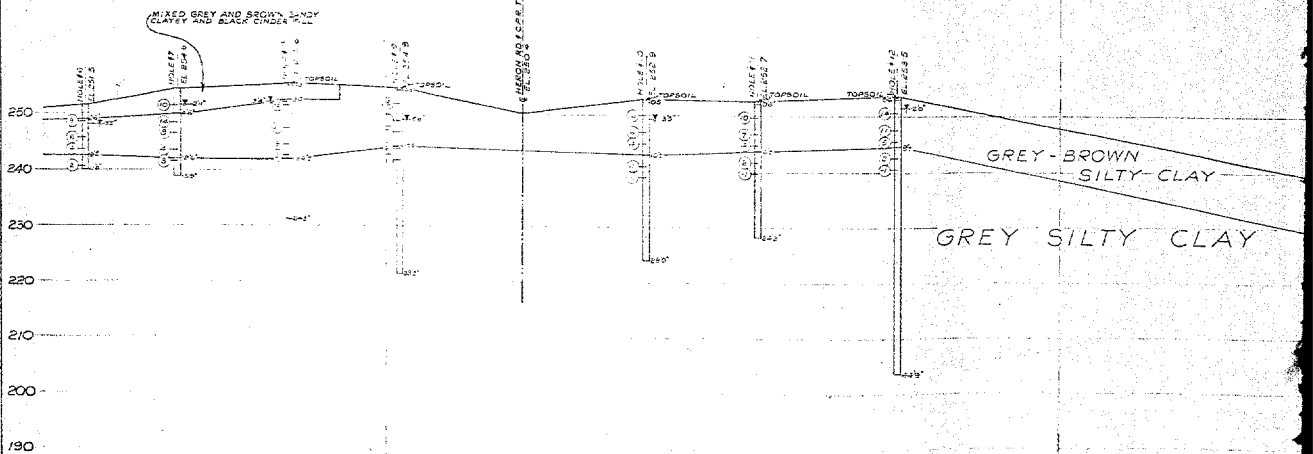
JOB NO. 67 F98 JOB NAME Heron Road & C.P.R. Grade Separation TECHNICIAN E.S.
 BORING DATE May 6 & 8/67 CLIENT City of Ottawa, c/o M.M. Dillon & Co. Ltd. ENGINEER A.N.S.B.
 GROUND ELEV. 230.9 BOREHOLE TYPE 3½" & "H" casing TYPED BY J.W.





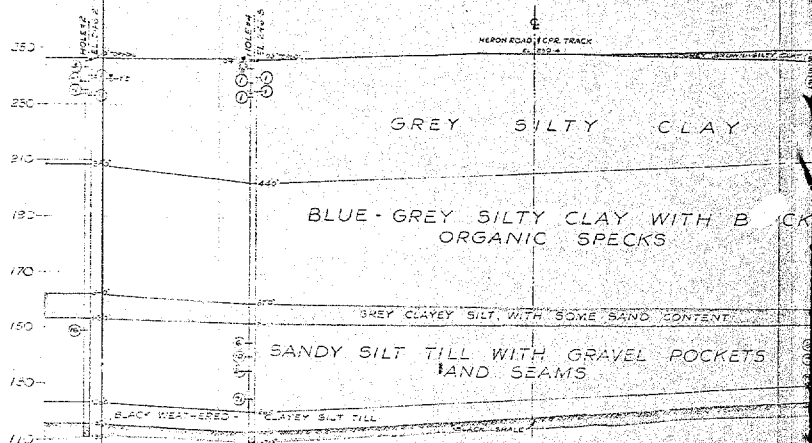
SITE PLAN

SCALE: 100' TO 1"



SECTION THROUGH HOLES 6, 7, 8, 9, 10, 11, 12 & 13

SCALE: HOR. 100' TO 1", VERT. 10' TO 1"

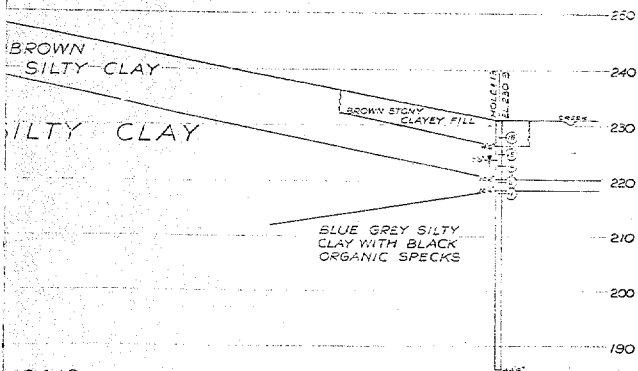


SECTION THROUGH HOLES 2, 4, 3 & 1
SCALE 30' TO 1" (NATURAL)

LEGEND

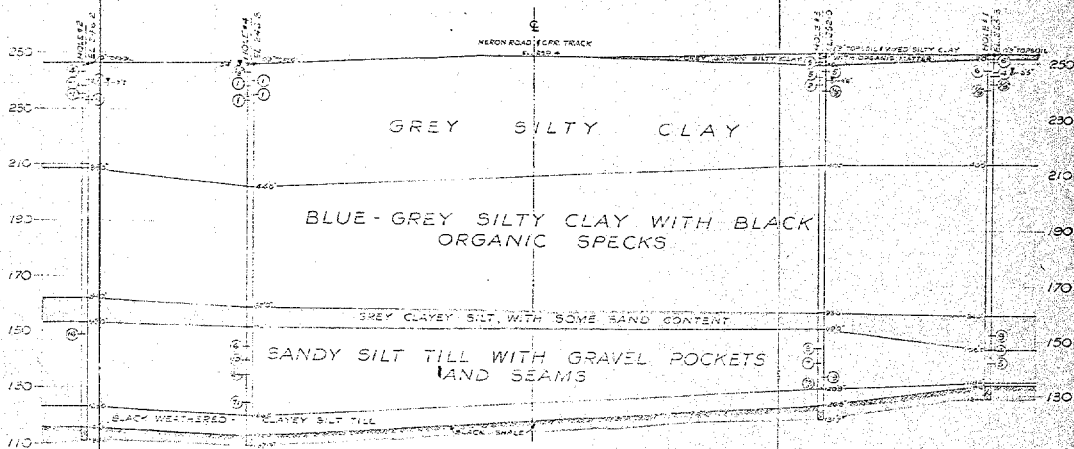
- BOREHOLE
- BLOWS/FT
- WATER LEVEL
- NO COMPLETE SOIL
- SEE BOREHOLE COMPLETE SOIL

NOTE: The actual well description is from data obtained only. The inferred geological evidence and these shown between b



12&13

CPE	
56. M. T. D.	
HERON GRAD	
e. m.	
JOE NO.	D
67-F55	JUL



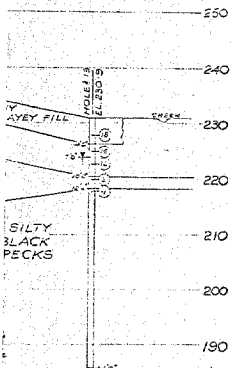
SECTION THROUGH HOLES 2, 4, 3 & 1
SCALE 10' TO 1" (NATURAL)

LEGEND

- ⊕ BOREHOLE
- ⊙ BLOWS/FT.
- WATER LEVEL

NOTE
SEE BOREHOLE LOGS FOR
COMPLETE SOIL DETAILS

NOTE: The actual soil stratification has been verified
from data obtained at the borings. However, the
only the inferred portions shown are based on
geological evidence and these may vary from
those shown between borings.



3165-117
GEOTECH N.L.

CITY OF OTTAWA
To: M. M. DILLON LTD., CONSULTING ENGINEERS

HERON RD. AND C.P.R.
GRADE SEPARATION

PREPARED BY:
E.M. PETO ASSOCIATES LTD.

JOB NO. 67F93	DATE: JULY 1987	DWN. BY: K	CHECKED BY: JHSC
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