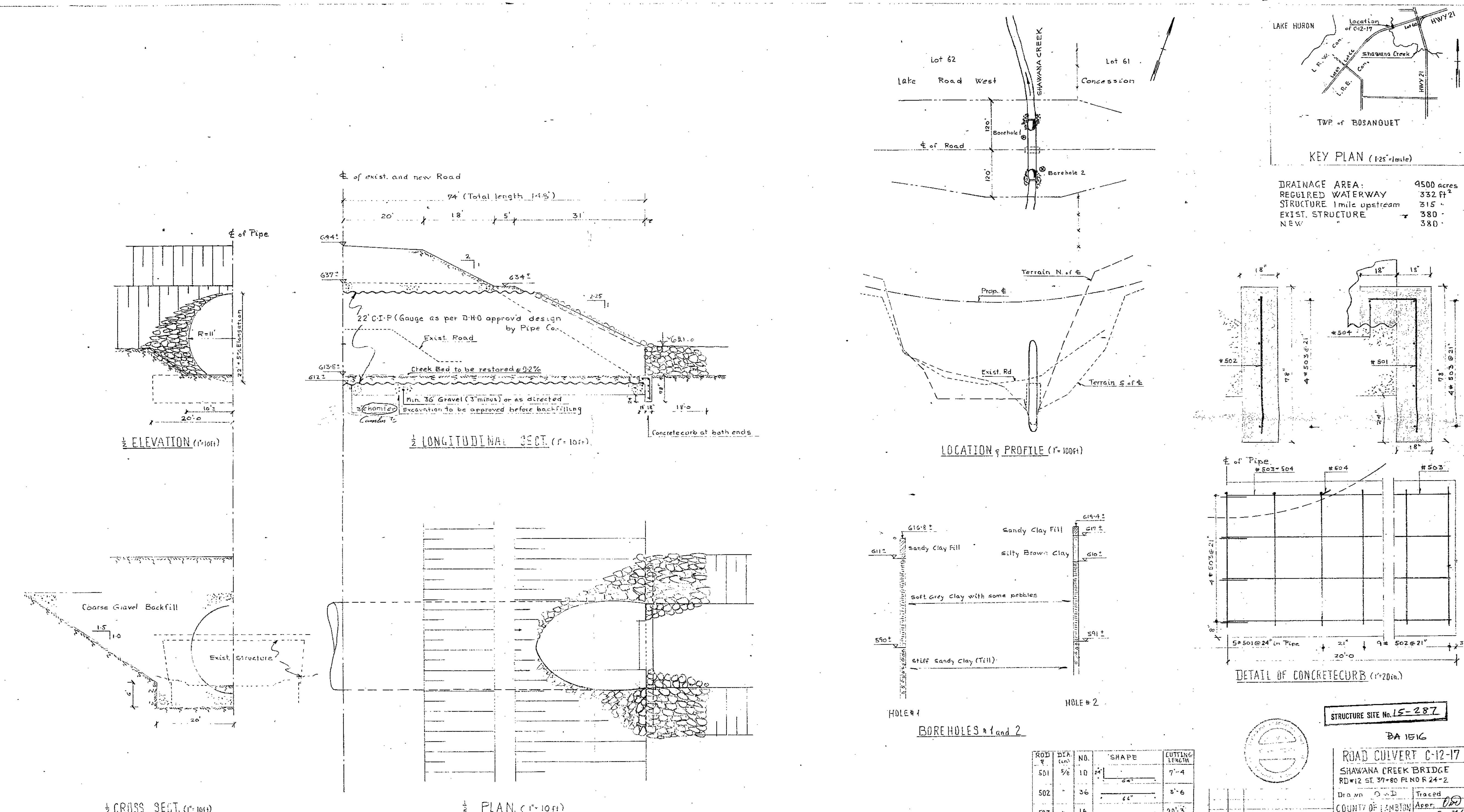
#62-F-272M
C-12-17 CULVERT
SHASHAWANDAH
CREEK
BOSANQUET
TWP



ROD #	DIA. (in)	NO.	SHAPE	CUTTING LENGTH
501	5/8	10	24" / 64"	7'-4
502	"	36	66"	5'-6
503	"	16	243"	20'-3
504	"	2	4" / 96" / 4"	4'-10

REV	DATE	BY	REMARKS

STRUCTURE SITE No. 15-287

BA 1516

ROAD CULVERT C-12-17

SHAWANA CREEK BRIDGE

RD #12 ST. 37+80 PL NO R 24-2

Drawn D.v.D	Traced
COUNTY OF LAMBTON ENGINEERING DEPT	Appr. OvD
	Date: Sep. 24/62
	Scale: as noted
County Eng. O. van Deurs	PL.NO. B-40

cc: Foundations Office (Rm. 110)

Mr. K. L. Kleinsteiber,
Municipal Bridge Liaison Engr.,
Bridge Division.

Attn: Mr. G.C.E. Burkhardt.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.
October 25, 1962.

Soils Report by E. M. Peto Assoc., Ltd.
County of Lambton,
Proposed Pipe Culvert,
Township of Bosanquet,
Lot 62, Rd. between L.F.E. & L.R.W. Cons.,
Structure Site #15-287.
Bridge Office Ref. #BA 1516.

We have reviewed the above report and hereby submit our comments for your consideration:

The foundation investigation has revealed the presence of a soft clay layer approximately 21 ft. thick. The soft layer is followed by a very stiff sandy clay layer with pebbles (till). Overlying the soft layer is sandy clay fill on the west side, and stiff brown silty clay and the sandy clay fill on the east side of the creek. It appears that no such materials overlie the soft clay layer in the creek, itself.

Due to the presence of such a soft clay layer, two main problems arise: The stability of the embankment and the magnitude of differential settlements along the culvert axis.

To resolve these two problems, the shear strength as well as the compressibility characteristics of the soft clay, have to be reliably determined. In evaluating the results of unconfined compression tests, the consultant has discarded all the low values on the ground of probable sample disturbance. An average of the higher values of 400 p.s.f., was then used in the stability analysis. The results of the analysis show that the fill is unstable. A 20-ft. berm increases the factor of safety only to one.

The shear strength is the governing factor in the analysis. Unfortunately, this is also the item of uncertainty in the report. Based on the "factual" information as contained therein, we have therefore, to take them as "reliable" values on which to base our judgment.

It would appear to us that a logical conclusion based on the results of the field and laboratory investigation and the subsequent analyses, would be the recommendation of a berm which would assure the stability of the structure.

cont'd. 2 ...

Mr. K. L. Kleinsteiber,
Municipal Bridge Liaison Engr.
Attn: Mr. G.C.E. Burkhardt. October 25, 1962.

Additional investigation could have been proposed only to enable a more precise dimensioning of the berm, i.e., to establish its limits.

However, the consultant is of the opinion that an embankment failure is not likely to occur, and in view of the minor importance of the road and the cost of measures to increase the embankment stability, he thinks that the risk of having a failure is warranted. The consultant bases the above statement on the uncertainties involved in the stability analysis and its consequent limited reliability, and also the probable presence of a stiff upper crust.

To predict accurately the stability of the embankment, the consultant requests additional data among which also includes additional information regarding the shear strength of the subsoil and the presence and thickness of any stiff crust.

The choice of either taking the risk of having a failure or having additional investigation carried out, is left now to the client.

We can hardly agree with such a choice. It is our opinion that however limited or extensive an investigation is, it should produce results that are reliable. Factual information should be beyond doubt. Therefore, we think that the two boreholes should have revealed, and the subsequent lab. investigation accurately established the type and properties of different soil layers. From the presented information - and if this information can be considered reliable - there is no upper crust in B.H. #1. It also appears reasonable to assume that no crust is present in the creek bed. This would then represent the most unfavourable condition and therefore the basis for the stability analysis.

With respect to the consultant's reference on the reliability of the stability analysis, there is ample evidence both in the literature and from our experience, that such analyses have produced reasonably accurate results provided that the relevant soil properties are reliably determined. However, it seems that the consultant has little faith in his results.

We are also perplexed by the statement that the differential settlement between the centre of the embankment and its toes would be 1" since the calculated settlement at the centre of the embankment is 7" and the settlement at the toe, though not reported, must be small.

AGStermac

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/McdeF
cc: Foundations Office
Gen. Files.

BA 1516

E. M. PETO ASSOCIATES LIMITED

1287 Caledonia Road,
Toronto 19, Ontario.

RUssell 9 - 1126.

Our Job Number 62131

September 11th, 1962.

Mr. O. van Deurs, P. Eng.,
Lambton County Engineer,
County Buildings,
Sarnia, Ontario.

62-F-272M

Dear Sir,

C-12-17 Culvert Shashawandah Creek
Soil Site Investigation.

We have pleasure in presenting four copies of our Report No. 62131 on the above site investigation.

As was earlier indicated, the most critical consideration in the design of the crossing will be the stability of the embankment with respect to a gravitational slip. However, any such slip is unlikely to involve the proposed culvert, the bearing capacity and stability of which can be considered as satisfactory, if the precautions suggested in the report are followed.

"Continued"

With respect to the danger of embankment slip failure, results of the present site investigation can only be regarded as preliminary, giving an approximate indication of the risks involved in the construction of the embankment. A thorough investigation of stability could only be based on additional information, which would have to include a detailed survey of the critical cross section of the embankment as well as additional probes along this section, in order to determine the possible existence of a stiff crust of subsoil, the presence of which, if proved, would considerably lessen the risk of a shear failure. A more thorough investigation of the shear strength of the soft clay than was possible in the present two test holes would also be advisable.

However, in view of the minor importance of the road and in view of the small likelihood that any failure could affect the culvert itself, you may not consider it economical to perform the additional work. Instead, there may be a good case for proceeding with the construction as planned, while bearing in mind the risks involved. Should any indication of tendency for slips be observed, remedial measures could be taken during and after construction. A number of steps aimed at improving the stability of the embankments are suggested.

We consider the report to be comprehensive under the circumstances, but we would be most pleased to answer any queries connected with this work, or to provide additional assistance, if required.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

E. M. Peto, P. Eng.

RK/ap

THE COUNTY OF LAMBTON

SOIL SITE INVESTIGATION

C-12-17 CULVERT SHASHAWANDAH

E. M. PETO ASSOCIATES LTD.

1287 Caledonia Road,
Toronto 19, Ontario.

TABLE OF CONTENTS

A. INTRODUCTION

The work described in this report was authorized verbally by Mr. O. van Deurs and confirmed by the return of our work order, signed.

A new culvert is to be constructed on County Road 12, near Ravenswood, Ontario, where this road crosses the Shawana Creek. A site investigation was required to determine the subsoil conditions.

The culvert, of corrugated metal, is to have a diameter of 22 ft, and the invert is to be located at the elevation 612.0. The base is to consist of a compacted gravel cushion.

The culvert is to be covered with embankment fill to a heightof 12 ft above the crown of the pipe. The roadway is to be 40 ft wide at the crown inclusive of shoulders, and the embankment is designed for side slopes of 2.5 to 1.

B. GENERAL INFORMATION

1. Two test holes were performed at the site, in the positions shown on the enclosed site plan. The positions were indicated in the field to our drilling foreman by the County Engineer, who later supplied a sketch showing their locations and ground elevations.

Test hole 1 was terminated at a depth of 45 ft, after proving 18 ft of a very stiff till.

Test hole 2 was taken down to a depth of 31.5 ft, which was sufficient to prove the presence of the very stiff till at a comparable elevation as in test hole 1.

2. The field work was performed between July 24th and 26th, 1962, by our drilling rig unit #6. Our standard drilling and sampling procedures were followed, as outlined in the enclosed Appendix "A".

3. Details of the soil conditions encountered in the test holes are described on the appended borehole logs, which contain also results of in-situ standard penetration tests, moisture content measurements and values of cohesive strength of clay.

A simplified subsoil profile, in the form of a section through the test holes, was prepared and is shown on the enclosed drawing.

B. GENERAL INFORMATION - Cont'd

4. The following tests were performed in our soil mechanics laboratory:

Moisture content determinations
Atterberg Limits tests
Grain size distribution
Unconfined compression tests with volumetric analysis
Consolidation tests
Undrained triaxial tests for the determination of
Modulus of linear deformation (Young's Modulus)

Results of the above tests are included in Appendix "B".

C. SITE nand GEOLOGY

The site of the new culvert is located in the Bosanquet Township on County Road No. 12 and approximately 0.5 mile west of the junction of this road with Highway 21. The culvert is to carry Road No. 12 over the Shashawandah Creek (or Shawana Creek), which flows in a northerly direction. At the time of the site investigation, water in the creek was stationary, the flow being blocked by some recently deposited fill a short distance north of the existing bridge. Normal water level is believed to be near elevation 613.5.

C. SITE and GEOLOGY - Cont'd

Road No. 12 is a gravel road.

The valley which contains the creek is some 800 ft wide, with the invert up to 40 ft below the level of the surrounding terrain. The existing bridge deck is at the elevation 625.5. The terrain in the vicinity of the bridge has been regraded recently, with the placement of fill on both sides of the bridge and with alteration of valley slopes.

Geologically, the area is located within the St. Clair Clay Plain, where glacial processes have deposited a mantle of till over a shale bedrock. A very stiff sandy clay till was encountered below the elevation 590, and was overlain by a considerably more plastic soft clay with some grits and pebbles. The subsoil conditions are described in the following chapter.

D. SOIL CONDITIONS

Details of the soil conditions found in the test holes are shown on the enclosed borehole logs. A simplified subsoil profile has been plotted on the enclosed drawing, in the form of a section through the test holes.

As far as can be judged from the two borings, the subsoil below elevation 611 is horizontally uniform at the site. Above this level, the soil consisted of a stiff crust of clay in test hole 2, overlain by some fill, while in test hole 1 the stiff crust was absent and the subsoil above the elevation 611 consisted entirely of fill.

The main types of subsoil, in the order of depth, can be defined as follows:

1. Topsoil and fill
2. Stiff brown silty clay with pebbles
3. Soft grey clay with some pebbles
4. Very stiff clay till.

Each of the above soil types will now be described in turn.

1. Topsoil and fill

The subsoil immediately below the existing grade in both test holes was found to consist of a variable fill, composed mainly of a sandy clay with gravel and organic matter. The fill was brown or mottled brown and of firm consistency. It extended to a depth of approximately 5 ft below the existing grade in test hole 1 and to 2.3 ft in test hole 2.

D. SOIL CONDITIONS - Cont'd

The fill material can be reused in the reconstructed embankment, but where visual examination indicates that it contains much organic matter, or is very soft and wet or otherwise inferior, it should be rejected. The top foot of the subsoil, which can be expected to contain a considerable proportion of plant roots and decayed vegetation, should preferably be scraped off under the embankments, as it may be a seat of additional long-term settlements.

2. Stiff brown silty clay with pebbles.

A stiff, brown silty clay with pebbles was found in test hole 2 between the depths of 2.3 ft and 8.5 ft below the existing grade (elevations 617.1 to 610.9).

In test hole 1, this material was absent.

The brown clay forms the desiccated crust of the underlying grey silty clay stratum. It is considerably drier and stronger; standard penetration test results of 21 and 14 were recorded in this layers, as compared with an average of 5 at a greater depth.

Where it is present, the stiff brown crust of clay will act as a raft and distribute embankment loads over a wider area, so that the stability will be less critical. However, the horizontal extent of occurrence of this crust is unknown.

D. SOIL CONIDITIONS - Cont'd

3. Soft grey clay with pebbles.

This deposit was encountered at a depth of 5 ft in test hole 1 and 8.5 in test hole 2, and extended to a depth of 27 ft and 28.5 ft respectively. The average elevations of the top of this stratum was 611 while the bottom was at 590, so that the thickness is approximately 21 ft. The soft clay rests on a stratum of very stiff sandy clay till, which has a very high bearing capacity.

Because of the critical character of the soft clay deposit, its geotechnical properties were studied in some detail, and were as follows:

a) Composition and Plasticity

A grain size distribution curve, performed on a typical sample of the material from a depth of 22 ft in test hole 2 is included on Fig. 1, Appendix B. It shows a 55% clay content and a 7% fraction of sand; the remainder consists of mainly fine to medium silt. Some grits were present in samples of the clay, and a layered structure was observed.

Atterberg limits tests were performed on three typical samples;

The Liquid Limit was between 39% and 42%, and the Plastic Limit was 18%; the Plasticity Index was between 21 and 24.

b) Consistency and shear strength

The deposit was generally very soft, standard penetration test results ranging from 3 to 7 blows per foot, with an average of about 5 blows per foot.

Moisture content was found to vary from 25.0% to 39.0%. Compared to the Atterberg limits, the natural moisture content is found to be in the higher parts of the plastic range of the material and to approach the Liquid Limit, corresponding to a soft to very soft consistency.

The shear strength of the clay under undrained conditions, ("apparent cohesion") was measured by seven unconfined compression tests. The undrained shear strength, taken as one half of the unconfined compressive strength, was found to vary from 160 to 440 lb/sq.ft. However, the lower values are probably caused by sampling disturbance and it is considered that the average cohesive strength can be taken as 400 lb/sq.ft.

The bottom layers of this material, below the depth of 21 ft in test hole 1 and about 25 ft in test hole 2, were found to be somewhat stronger, and a potential slip plane is likely to pass above this level.

D. SOIL CONDITIONS - Cont'd

c) Compressibility

Two oedometer consolidation tests were performed on undisturbed samples of the soft clay and the results are presented on Figs. 2a and 2b, in the form of void ratio-log pressure curves. The coefficients of volume change and consolidation are included on the above figures in tabular form, for the various load stages of the tests.

The results of the tests indicate that the compressibility of the material is high, so that the layer is subject to relatively large settlement under the weight of the new embankment.

Modulus of linear deformation, E (Young's Modulus) was determined on two undisturbed samples of the material, from the average gradient of stress-strain curves during repeated loading and unloading cycles in triaxial compression. The average value of E was 52 ton/sq. ft.

This value is used in determination of the "elastic" or short-term settlement of this stratum under applied loads.

4. Very stiff clay till

A very stiff stratum was reached at a depth of 27 ft in test hole 1 and 28.5 ft in test hole 2, corresponding to an average elevation of 590. It was proved to a depth of 45 ft in test hole 1, i.e. to 18 ft below its surface.

D. SOIL CONDITIONS - Con''d

This material, of grey colour and consisting of a well graded mixture of silt, clay and sand with some pebbles, is very strong, and will safely withstand any stresses transmitted to it from the overlying deposits.

Standard penetration test results in the till ranged from 24 to 68 blows per foot.

An Atterberg Limits test gave the following results:

Liquid Limit: 22%; Plastic Limit: 13%; Plasticity Index: 9.

The moisture content of the till varied from 10.4% to 18.2%, but most of the results were between 12% and 14%. The natural moisture content, except in the top layers, was thus at or below the Plastic Limit, corresponding to a very stiff consistency.

In stability analysis, it can be assumed that any potential slip plains will be located above the surface of this stratum. In settlement calculations, insignificant error will be involved if the stiff till stratum is assumed incompressible.

E. WATER CONDITIONS

Slow seepage of ground water was only reported at a depth of 3 ft 8 in. in test hole 1, in the fill material. It reflects water level in the creek, which can be assumed to control the water table at the site. The water level in the stream was near the elevation 613.5.

The soft clay and the stiff sandy clay till which underlies the backfill, are of relatively very low permeability and all noticeable seepage of ground water was cut off as soon as borehole casing penetrated into the virgin subsoil. No seepage seams were encountered in either of the test holes below the elevation 612.

F. CONCLUSIONS and RECOMMENDATIONS

(a) Culvert Foundation

1. The invert of the 22 ft diameter, corrugated metal culvert is to be located at the elevation 612.0. According to the results of the two test holes, at this level the culvert would be resting practically on the surface of stratum of soft clay with some pebbles, which extended to the average elevation 590, where it was followed by a very stiff, sandy clay till. The average undrained shear strength of the soft clay stratum can be taken as 400 lb/sq.ft; however, this value is based on a relatively small number of tests performed on samples from the two test holes, and it is not certain to what extent this average result is representative of the strength of the clay immediately under the proposed culvert.

Also, although the surface of the soft clay in the two test holes was near the elevation 611 to 612, it is possible that at the proposed location of the culvert the soft clay has been eroded by the stream to a greater depth.

2. Assuming that the shear strength of the clay, as determined in the two test holes, can be taken as representative of the material below the invert of the culvert, there appears to be no danger of a bearing capacity failure below the culvert, the weight of which is estimated at 500 lb/lin.ft. The bearing capacity below the full height of the embankment adjacent to the pipe will be considerably more critical and will be discussed later.

F. CONCLUSIONS and RECOMMENDATIONS

3. The culvert should be laid on a cushion of well compacted granular material. The purpose of the cushion, in view of the sufficient bearing capacity of the subsoil, is not so much to spread out the culvert loads over a wider area, as to ensure a good, uniform bearing for the culvert, which will allow a uniform settlement.

The thickness of the granular cushion under the pipe invert will depend on the material which exists at the present stream bed; from results of test holes located at some distance from the stream it is not possible to judge what materials are present in the surficial layers below the pipe invert, and this information can only be obtained by additional probes or by visual examination of excavation below the pipe.

If one interpolates results of the two test holes, the soft clay stratum would appear to begin near the elevation611. However, it is possible that thismaterial below the existing stream has been eroded by scouring to a greater depth and replaced with materials carried by the stream, which may consist of sand and gravel, but also organic matter and soft mud.

It is recommended that the bedding for the culvert be prepared by excavating the possible variable stream deposits under the pipe invert until the soft clay stratum is reached. The depth of excavation

F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

below the culvert invert should preferably be uniform throughout the length of the culvert. It is particularly important that all organic material or soft mud be removed, and also it is advisable to remove any boulders.

The excavated trench should then be filled up to the required level of pipe invert with well compacted granular material, placed in layers 4 to 6 inches thick. If, as is probable, the granular cushion will rest on top of the soft clay, compaction should only begin after an initial layer at least one foot thick has been laid on top of the soft clay; this precaution is intended to ensure against disturbance of the soft clay by the compacting operation.

During the excavation, it is important not to disturb the formation grade of the soft clay at the bottom of the excavation. Excavating machinery should operate from outside the trench and the excavated grade should be protected against the inflow of water. The flow in the creek should be diverted, or cut off from the excavation by sheeting in order to ensure a dry bottom. In the event that stream-deposited granular layers exist below the stream bed, the water seepage through such pervious layers can be cut off by driving sheet a short distance below the lower limits of such layers, and into the relatively impervious soft clay.

F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

The thickness of the granular cushion below the culvert invert will depend on the necessary depths of excavation to remove unsatisfactory subsoil, as indicated above. In the event that the excavation discloses the presence of surface of the soft clay stratum near the invert of the culvert, a three foot minimum thickness of granular cushion (i.e. down to elevation 609) would probably be sufficient to provide a uniform bedding.

The width of the cushion should be equal to at least one and a half times the pipe diameter, though a width equal to twice the pipe diameter would be preferable and would provide a good base for the proper compaction of the backfill on the sides of the pipe.

4. The preparation of base for the pipe and compaction of the fill around the pipe should be performed in accordance with the customary requirements of good practice. The main of these requirements are:

The lower quater of the circumference of the culvert should be firmly supported. Thus the compacted granular cushion should extend to a minimum height of 3 to 4 ft above the invert level. The bedding should then be carefully shaped to accurately fit the pipe. Best results for bedding corrugated metal structures are obtained by preparing a flat surface and carefully tamping the fill under the haunches.

F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

It is advisable to seal the pervious granular fill under the pipe at the ends of the structure against the inflow of water. This can be done by bedding the ends in well-tamped clay or by including a head wall or end section.

The corrugated metal structure, being flexible, will bend under embankment loads and deflect out horizontally, thus building up side support. In order to allow sufficient mobilization of lateral resistance, the backfill around the pipe should consist of good quality, drainable material, free of rocks and organic matter. The fill under and around the structure should be placed in layers not exceeding 6 inches at a time to permit thorough compaction. The height of fill on both sides of the culvert should be built up simultaneously.

The fill above the crown of the culvert should be of equally good quality and standard of compaction as on the sides of the structure.

5. Because of the presence of an approximately 21 ft thick layer of soft clay below the elevation 611, theculvert and the embankments will be subject to settlements. The settlement of the culvert will be caused not only by the weight of the structure and of the superimposed fill, but also by the stressing of the subsoil under the pipe by the adjacent embankment. Also, a downward force on the walls of the culvert and on the perimeter of the fill projecting above the crown of the culvert, caused by the settlement of the adjacent embankment, will cause some additional

F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

settlement. This frictional or "drag" force will depend on the type of backfilling and the relative timing of construction of the embankment and culvert, and is therefore very difficult to estimate.

The upper limit of the likely settlement can be taken as that due to a pressure corresponding to the full height of embankment adjacent to the culvert. In actual fact, however, due to the empty space inside the pipe, the actual settlement of the culvert will be less, and the following figures should only be treated as a guide to the maximum loss of head clearance that may occur due to pipe settlement.

Under the above assumptions, the settlement of the culvert invert would be as follows:

Ultimate settlement of centre of culvert: 7.4 inches.

The settlement will be reduced by roughly one-twentieth of the above value for each foot thickness of compacted sand cushion below the assumed elevation of top of soft clay of 611. For a sand cushion extending to the elevation 608, the theoretical settlement would thus be 6.3 inches.

F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

The above estimate of settlement assumes that the culvert is placed on a well compacted granular cushion, in accordance with the above recommendations. Additional settlement would occur if any soft mud or organic deposits were retained under the pipe. Also, uneven settlements may take place if materials of variable density are retained close to the invert of the culvert. The danger of this also will be lessened by the provision of a suitable granular cushion.

Assuming a uniform bedding of the pipe, and also assuming it to be fully flexible, the differential settlement of the culvert invert between the centre of the embankment and its toes would be of the order of one inch. This will be caused by the progressively decreasing pressure on the pipe in the direction of the embankment toes. 2

b) Stability and Settlement of Embankment

The stability of the embankment with respect to a gravitational slip was investigated, assuming a typical section through the embankment, as illustrated on the enclosed sketch. (page 20).

The section, which was considered a typically critical one, was obtained from Client's Plan No. R24-2, though no actual cross sections of the proposed embankment were available. Also, the subsoil profile at the critical section was assumed to be identical to the section through the two test holes. Because of the above assumptions, the accuracy of the

F. **CONCLUSIONS and RECOMMENDATIONS** - Cont'd

analysis is limited, and it should be considered only as an approximate guide to the safety of the embankment.

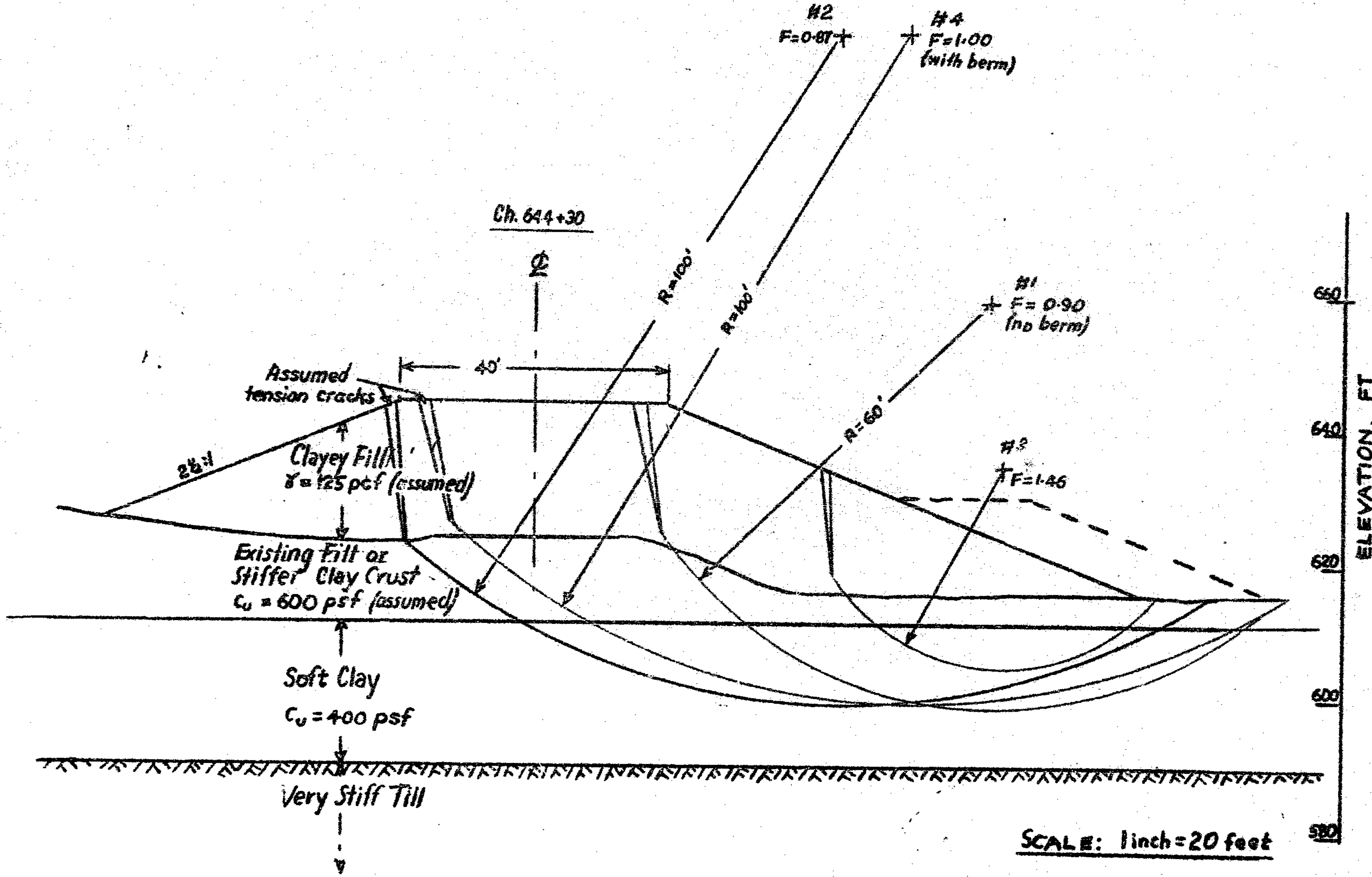
The following conclusions can be drawn from the stability analysis.

1. If the critical conditions of shear strengths and soil configuration, assumed in the calculations and indicated on the sketch, are valid, then there is a distinct danger of failure of the embankment. A factor of safety of 0.87 was obtained (slip circle #2).

2. A twenty foot wide berm, introduced at mid-height of the embankment slope, was found to raise the factor of safety from 0.87 to 1.00. Thus, the increase in factor of safety due to introduction of the berm is not very large. For a substantial increase in safety factor, a considerably wider berm would be necessary, or a series of smaller berms, which would flatten the effective average slope of the embankment.

Provision of berms, which, designed on the basis of the above stability analysis, would ensure a substantial increase in the factor of safety of the embankment, would involve a considerable quantity of additional fill and a large width of acquired property. Because of the uncertainties involved in the stability analysis and its consequent limited reliability, such more expensive measures cannot be justifiably recommended on the basis of the presently available information.

ILLUSTRATION OF STABILITY ANALYSIS



F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

3. In the stability analysis, the most critical conditions based on results of the two test holes were considered. Only a small increase of strength in the top, drier portion of the topsoil was assumed, and the shear strength of the embankment material was neglected completely, assuming a tension crack penetrating to the virgin soil.

In fact, the contribution to resistance to shear from the stiffer crust of the subsoil may be considerably higher than assumed. Also, some resistance to shear in the embankments may be mobilized, particularly in view of the limited longitudinal extent of the critical section of the embankment as compared to its width.

An allowance in the stability calculation for a higher shearing resistance in the desiccated portion of the clay and in the embankment would greatly increase the factor of safety. A stiff layer, crossed by a potential slip line and even only a few feet thick would greatly increase the factor of safety and bring it to over unity. Such a stiff crust was present in test hole 2 but was absent in test hole 1. On the basis if the results of the test holes, we did not feel justified in making assumptions regarding the occurrence of such a stiff layer at the critical section; instead, we have restricted the stability analysis to the most critical likely conditions, but we include the above reasons why in actual fact the problem may be less acute than indicated.

F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

4. The shear strength of the soft clay will increase with time, due to the consolidation of the material under the weight of the new embankment. Consequently, the factor of safety will increase in due course. A risk of a slope failure will be decreased if the embankment is built up gradually over a period of several weeks, rather than in one rapid operation.

5. In view of the above conclusions, although the stability analysis, based on the most critical visualized conditions, indicates a possible failure of the embankment, it appears that, in actual fact, the factor of safety may be higher than calculated, and that the work can be successfully completed without a failure. However, without additional information about the site conditions at the critical section it is not possible to offer an accurate estimate of the factor of safety.

The stability of the embankment could be reliably predicted only with the help of the following additional data:

(i) Detailed cross section of the site, with elevations of existing and final grade at a potentially critical section of the embankment. The profile should extend to 150 ft on either side of the centre line of the embankment.

F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

(ii) Additional information on subsoil conditions, obtained from a line of probes, performed at 20 to 30 ft intervals along the critical section. The purpose of the probes would be to determine the thickness of any stiff crust of subsoil, which is of critical importance in stability analysis.

(iii) Additional information regarding the shear strength of the subsoil, obtained by unconfined compression tests and in situ vane tests performed in the probes. At least one of the probes should be carried out from the existing embankment, to indicate whether subsoil below this additional overburden is more consolidated and stronger than under the low areas; this could serve as a guide as to the likely increase in strength due to consolidation of the clay below the new embankment.

(iv) Information on the shear strength of the compacted, new embankment material.

6. Settlement of the centre of embankment, under the maximum height of new fill assumed as 20 ft, was estimated at 7.4 inches. The settlement will be caused by the compression of the 21 ft thick layer of soft clay, existing between the elevations 611 and 590, and does not include any settlement within the new embankment itself.

F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

Additional settlement could occur in any organic layer that may be retained under the embankments. It is recommended to remove all organic matter and topsoil to a minimum depth of one foot below the existing grade, or as may be considered necessary after examination of the scraped grade. The excavated material should be rejected and not reused in the reconstructed embankment.

7. The consolidation of the soft clay would be accelerated, and the stability of the embankment would be improved, by including a granular filter layer on top of the excavated grade and below the compacted embankment material in the critical section of the embankment, and particularly where the removal of organic topsoil will uncover the soft grey clay. Presence of such a granular blanket, which should be 6 inches thick, would allow a more rapid drainage and consolidation of the subsoil, by decreasing the length of the drainage path.

Report prepared by:

R. Kulesza, P. Eng.

E. M. PETO ASSOCIATES LTD.

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

RK/ap

APPENDIX "A"

STANDARD PROCEDURE

The field investigation work is carried out by means of a skid mounted diamond drill rig.

Standard sampling procedures are followed. Casing is driven and cleaned, either by augers, tubes or by wash water.

Samples are recovered ahead of the casing at frequent intervals, with either a 2 inch or 3 inch O.D. split barrel sampling tube, Shelby tube, or split barrel sampling tube fitted with brass liners and special sharp cutting nose.

The standard penetration test results are recorded when sampling with the regular 2 inch O.D. split barrel sampler, these being the number of blows of a 140 pound hammer falling 30 inches, required to drive the sampling tube a distance of one foot into undisturbed soil.

The Dutch Cone probe test is made by driving the drill rods into the ground with a 2 inch dia. x 60^{o} cone tip. The number of 4200 inch pound blows per foot of penetration are recorded, as in the standard penetration test.

Where required, "in situ" shear strength tests are made ahead of the casing, using Modified Acker vane test equipment.

Disturbed samples are visually classified in the field, sealed in sample jars, and are re-examined, and tested as necessary, in the soils laboratory. Undisturbed samples are returned to the laboratory for later examination and testing as required.

The test holes are bailed (or pumped out) during the work as necessary, at the end of the day, and on completion. Subsequent water level readings are taken for the duration of the field work. Water pressure readings are recorded when Artesian water conditions are encountered. Moisture content samples are recovered at frequent intervals to assist in the soil classification and the interpretation of water table results.

Borehole logs are prepared giving details of the soil description and condition as recorded in the field. These logs form the basis of the soil profile, which indicates the general stratigraphy assumed to exist between the boreholes as represented by the borehole logs.

The boreholes are normally set out by the Field Engineer, who also records the ground elevations referred to a temporary bench mark or known reference point. If the client has been responsible for setting out the boreholes and recording their ground elevations this is stated in the preamble to the report.

A plan is drawn up from drawings supplied by the Client or his representatives, showing the locations of the boreholes and the T.B.M. where applicable.

Normally, the standard penetration blows and the natural moisture contents are plotted against elevation as a graph, and these graphs form part of the appendices, together with laboratory test result details, ground water readings and other soil characteristics which can be best illustrated in graphical form.

APPENDIX "B"

SOIL TEST RESULTS

Job No. 62131

ATTERBERG LIMIT TEST RESULTS

BH/SA. No.	Depth Ft.	Liquid Limit %	Plastic Limit %	Plasticity Index	Natural Water Content %
1/5	11	39.2	18.0	21.2	{28.1 32.6
1/10	21	41.5	18.0	23.5	36.0
1/14	31	22.0	13.2	8.8	13.3
2/8	17.5	43.1	19.5	23.6	28.6

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Toronto 19, Ontario

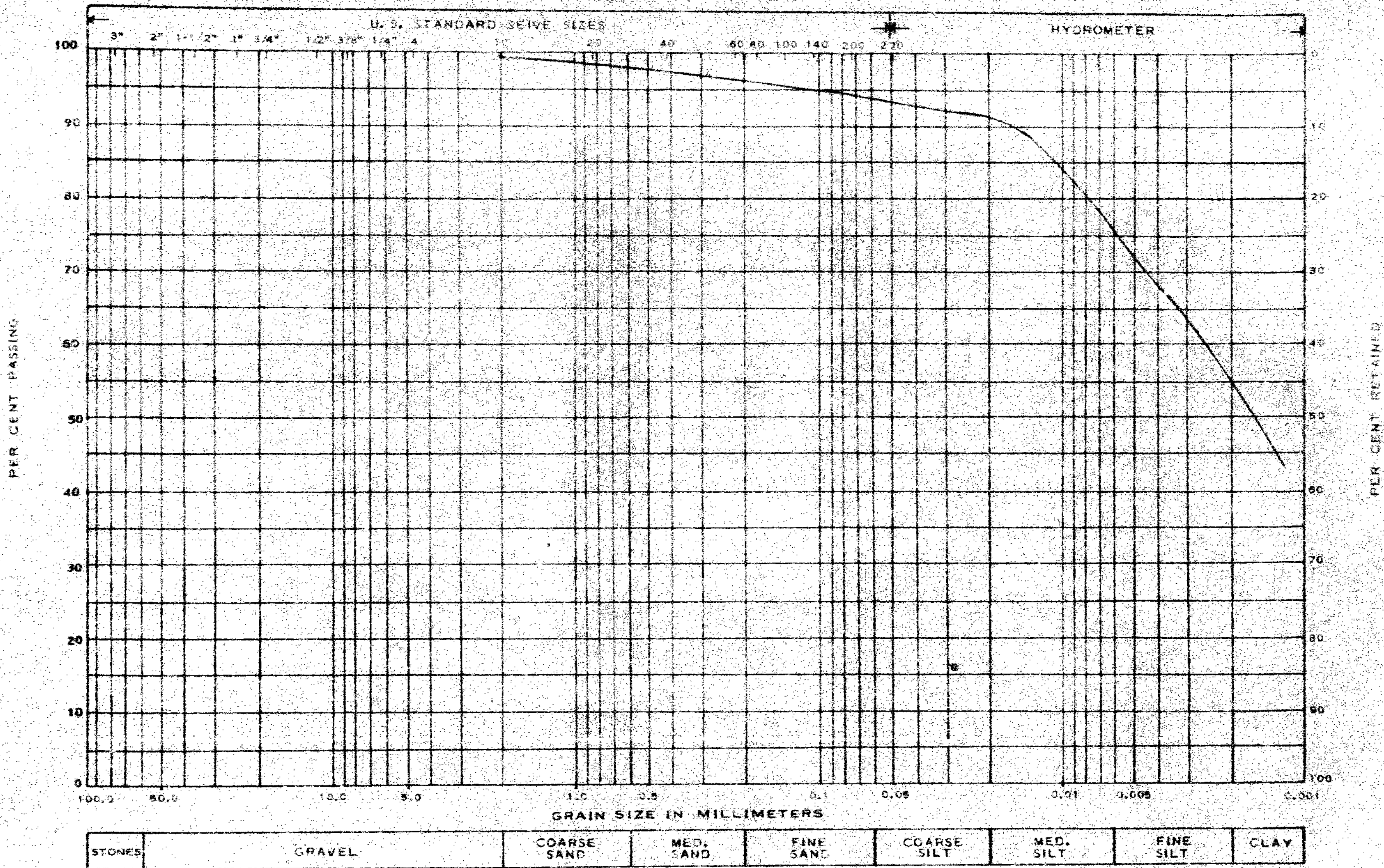


Fig. 1

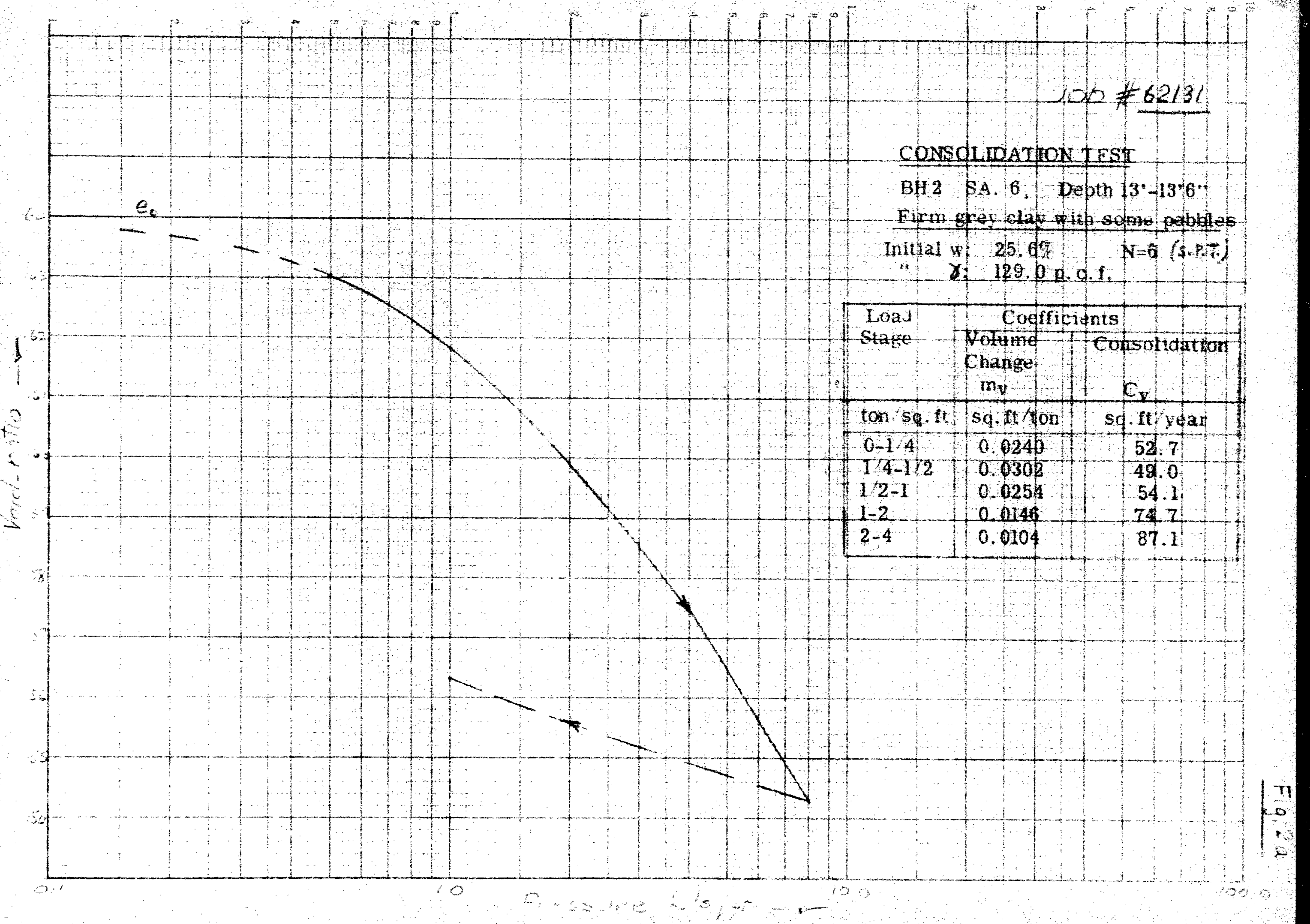
JOB NAME Culvert C-12-17 JOB NO. 62131 HOLE NO. 2 SAMPLE NO. 10

DEPTH 22' ELEVATION ______ REMARKS Very soft grey silty clay

GRAIN SIZE DISTRIBUTION

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

JOB # 62131



CONSOLIDATION TEST

BH 2 SA. 6, Depth 13'-13'6"

Firm grey clay with some pebbles

Initial w: 25.6% N=6 (S.P.T.)

" γ: 129.0 p.c.f.

Load Stage	Coefficients	
	Volume Change m_v	Consolidation C_v
ton/sq.ft	sq.ft/ton	sq.ft/year
0-1/4	0.0240	52.7
1/4-1/2	0.0302	49.0
1/2-1	0.0254	54.1
1-2	0.0146	74.7
2-4	0.0104	87.1

Fig. 2a

JOB # 63131

CONSOLIDATION TEST

BH 2 SA. 9, Depth 21'-21'6"

Very soft grey clay with some pebbles

Initial w: 34.4% N=5 (S.P.T.)

" γ: 117.3 p.c.f.

Load Stage	Coefficients	
	Volume Change m_v	Consolidation C_v
ton/sq.ft	sq.ft/ton	sq.ft/year
0-1/4	0.0230	37.2
1/4-1/2	0.0164	30.0
1/2-1	0.0370	67.7
1-2	0.0342	62.6
2-4	0.0304	55.6

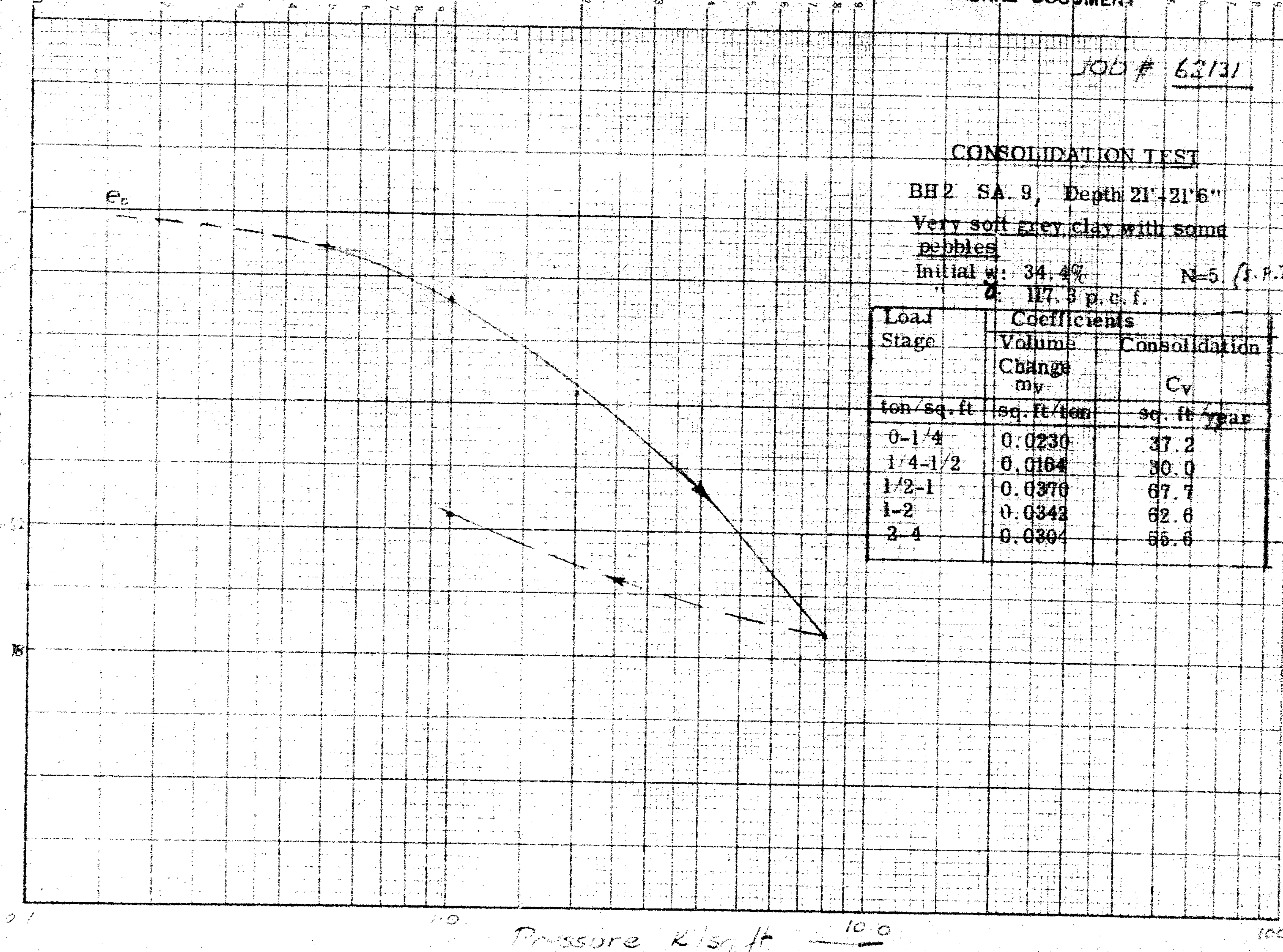


Fig. 2b

E. M. PETO ASSOCIATES LTD.

UNCONFINED COMPRESSION TEST DATA SHEET Job No. 62131

Borehole Number	Sample Number	Depth feet	Nat. M. C.	Wet Density p.c.f.	Dry Density p.c.f.	Void Ratio, e	u/c Shear Strength p.s.f.
1	4A	9'-9'6"	31.7	121	92	0.87	340
1	4B	9'6"-10'	28.1	126	98	0.77	420
1	7A	13'6"-14'	33.7	122	91	0.93	195
1	7B	14'6"-15'	32.9	121	91	0.91	260
1	11A	22'-22'6"	35.8	115	85	0.98	160
1	11B	22'6"-23'	37.2	116	84	1.04	440
1	9	17'-17'6"	34.2	115	85	0.96	340
1	13	27'-28'6"	26.8	125	99	0.71	458
2	12	27'6"-28'	30.3	126	97	0.74	777

UNDRAINED TRIAXIAL COMPRESSION TESTS FOR DETERMINATION OF MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)

B.H./SA No.	Depth ft	Water Content %	Bulk Density p.c.f.	Dry Density p.c.f.	Void Ratio 3 e	Undrained shear strength C_u p.s.f.	Young's Modulus ton/sq.ft	$\frac{E}{C_u}$	Cell Pressure p.s.i.
1/9	17.5	35.3	117.0	86.5	0.95	530	54,3	205	15
2/8	17.5	27.5	121.5	95.8	0.78	850	50.2	118	15

Note:- E was obtained from average slope of stress-strain curve loops in repeated loading cycles.

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SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Culvert C - 12 - 17 | Job No. 62131 | Borehole No. 1

Client The County of Lambton | Casing BX | Boring Date July 24 & 25th, 1962.

Elevation 616.8 | Compiled By J. F. G. | Checked By A. P.

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- A.S. AUGER SAMPLE
- C.S. CASING SAMPLE
- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- M. MOIST
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL
- W.T.P.L. WETTER THAN PLASTIC LIMIT
- D.T.P.L. DRIER THAN PLASTIC LIMIT
- A.P.L. ABOUT PLASTIC LIMIT

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	[illegible]	WATER LEVELS & REMARKS
Ground Surface			0'0"						
Sandy fill-organic	Brown				1	CS			Dry
Sandy clay with gravel, fill	Mottled brown	Firm	2'8"		2	SS	7	16.6	W.T.P.L.
			5'0"						
Silty clay odd grit	Grey	Soft to firm			3	SS	4	25.0	W.T.P.L.
As above	Grey	Soft to firm			4	SS	4	28.9	W.T.P.L. C_V = 340 / 420
			10'0"		4A	2"SL	Tapped	28.0	Sensitive
As above	Grey	V. soft			5	SS	5	28.1 / 32.6	W.T.P.L.
As above	Grey	V. soft			6	SS	3	32.2 / 39.0	W.T.P.L. C_V = 195
			15'0"		7	2"SL	Pushed	32.9	260
As above	Grey	V. soft			8	SS	3	26.8	W.T.P.L.
					9	2"SL	Pushed	35.3	
As above	Grey	Soft	21'0"		10	SS	4	36.0	W.T.P.L. stiffens at 21'0"
					11	2"SL	Pushed	38.8 / 37.2	C_V = 160 / 440
As above with silty lenses	Grey	Soft	27'0"		12	SS	5	27.0	W.T.P.L.
					13	2"SL	Tapped	26.8 / 14.3	C_V = 458
			30'0"						
Very silty & sandyclay till sand seams & pebbles	Grey	V. stiff			14	SS	24	13.3	W.T.P.L.
			35'0"						
As above or clayey silt Till	Grey	V. stiff			15	SS	68	12.9	D.T.P.L.
			40'0"						
Clayey silt till	Grey	V. stiff			16	SS	55	16.3	D.T.P.L.
As above	Grey	V. stiff	45'0"		17	SS	59	18.2 / 14.0	D.T.P.L.

BORING TERMINATED AT 45'-0"

Water Conditions:- Water only in upper 5 feet and it reflects the river level (W.L. = 3'8"). After driving casing, cut water off.

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SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Culvert C - 12 - 17 — Job No. 62131 — Borehole No. 2

Client The County of Lambton — Casing 4" — Boring Date July 25 - 26th, 1962.

Elevation 619.4 — Compiled By J. F. G. — Checked By A. P.

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- A.S. AUGER SAMPLE
- C.S. CASING SAMPLE
- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- M. MOIST
- W.L. WATER LEVEL IN CASING
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- W.T.P.L. WETTER THAN PLASTIC LIMIT
- D.T.P.L. DRIER THAN PLASTIC LIMIT
- A.P.L. ABOUT PLASTIC LIMIT

Soil Description	Colour	Density or Consistency	Dep[illegible]	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	[illegible]	Water Levels & Remarks
Ground Surface			0'-0"						
Sandy fill	Brown				1	CS			Dry
Organic sandy clay	Brown	Firm	2'-3"		2	SS	7	24.5 23.4	W.T.P.L.
			5'-0"						
Silty clay-grits & pebbles	Brown	V. stiff			3	SS	21	15.7	About P.L.
Silty clay layered	Grey	Stiff to v. stiff	8'-6"		4	SS	14	22.8	W.T.P.L.
			10'0"						
As above	Grey	Firm			5	SS	7	26.0	W.T.P.L.
					6	3"SL	Tapped	29.7 25.6	
			15'0"						
Silty clay	Grey	V. soft			7	SS	5	30.7	W.T.P.L.
					8	2"SL	Pushed	28.6	
			20'0"					32.4	
					9	3"SL	Tapped	34.4	
Silty clay slightly varved	Grey (reddish tinge)	V. soft			10	SS	5	16.3	Much W.T.P.L.
			25'-0"						
As above	Grey (reddish tinge)	Soft			11	SS	7	33.3 29.6	Much W.T.P.L.
			28'6"		12	2"SL	Tapped	30.3 31.6	C_u = 777 p.s.f. w = 30.3%
		V. stiff	31'6"		13	SS	25	10.4 13.3	D.T.P.L.
Clayey silt and sand till	Grey								

BORING TERMINATED AT 31'-6"

Note:- No ground water noted in hole on July 26, 1962.

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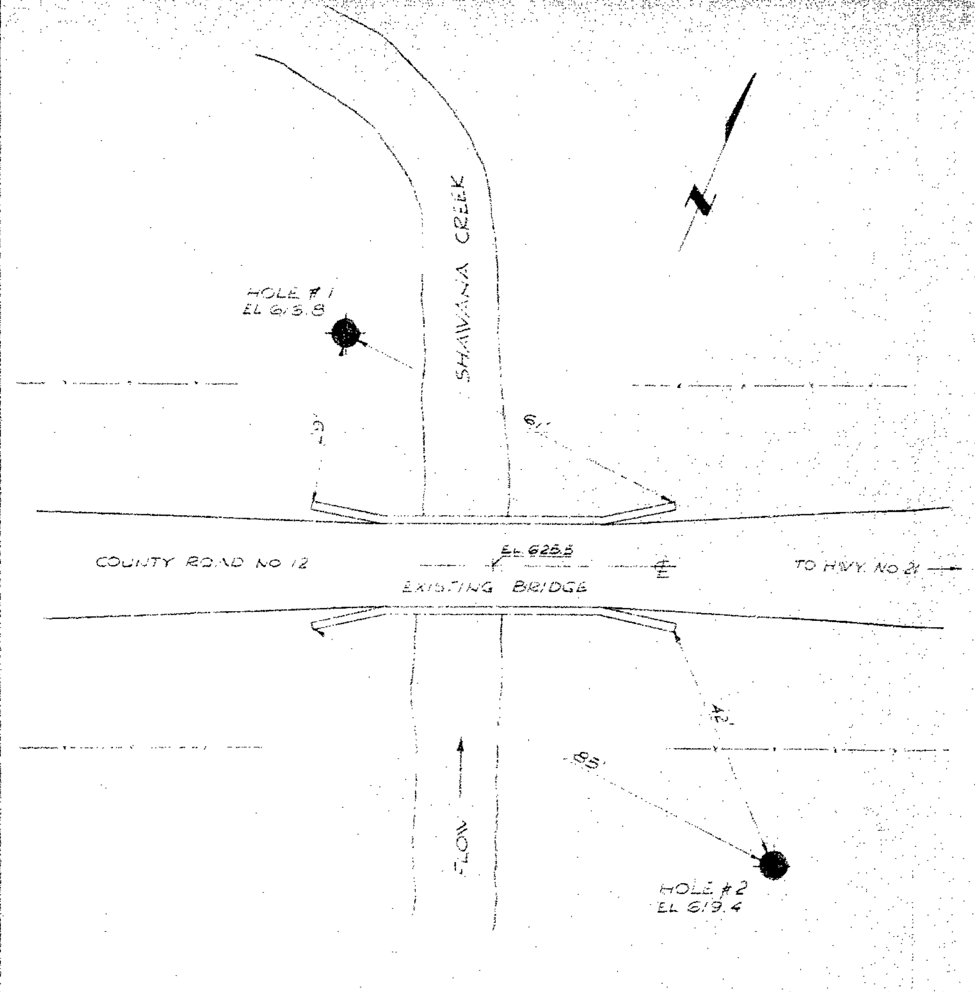
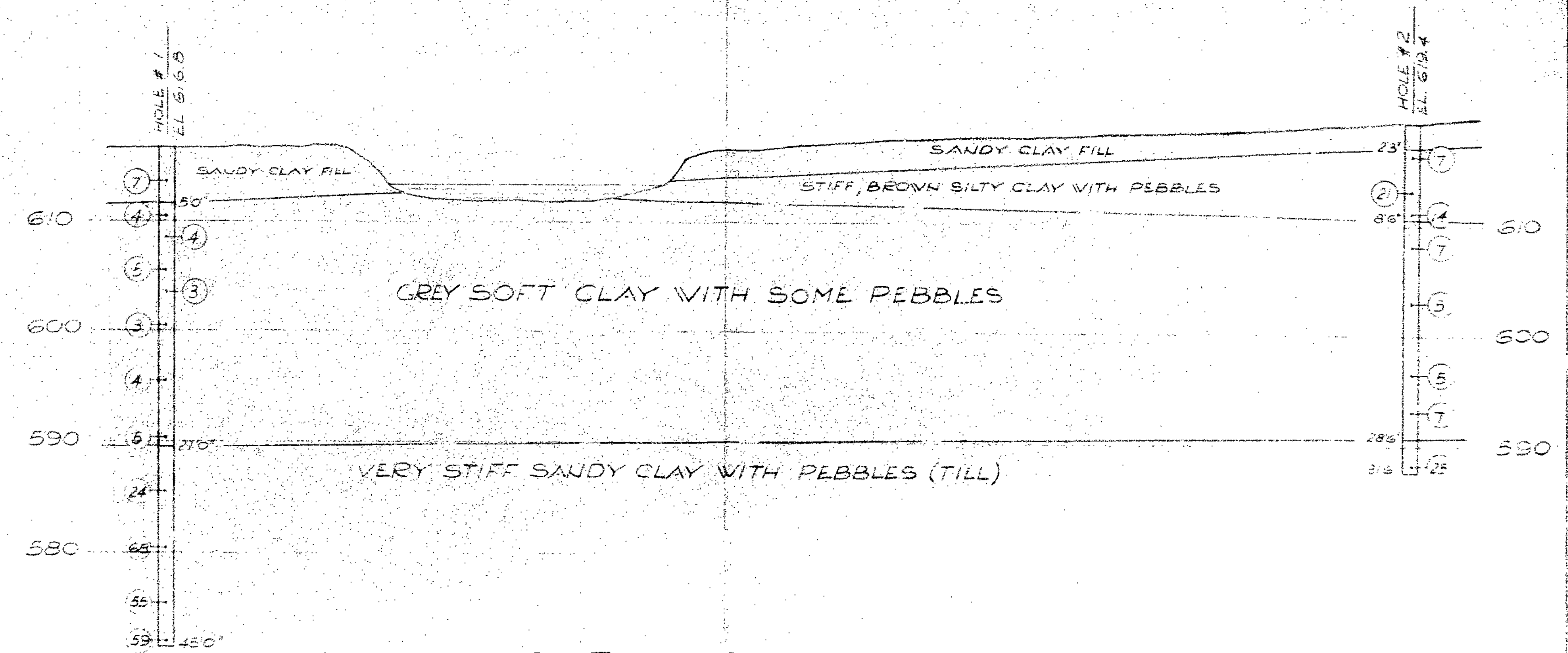


SITE PLAN

SCALE: 20' TO 1" (APPROX)



SECTION THROUGH HOLES 1 & 2

SCALE: 10' TO 1" (NATURAL)

LEGEND

BOREHOLE

4 BLOWS/FOOT S.P.T.

NOTE:

SEE BOREHOLE LOGS FOR COMPLETE SOIL DATA.

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

E. M. PETO
PROVINCE OF ONTARIO

THE COUNTY OF LAMBTON

SHAWANA CULVERT (3-12-17)

PREPARED BY
e.m peto associates ltd.

JOB NO	SCALE	DATE	DWN. BY
62131	AS NOTED	AUG. 1962	K. K.