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DUNVEGAN LANDSLIDE

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PLATES

FIGURES

Acknowledgements

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DUNVEGAN HILL

Interim Report No. 1

1. Introduction

All weather transportation in the Peace River country of Alberta has proved to be an outstanding problem since the days of the voyageurs. This country, the most northerly productive land in Canada, yields large crops. In the early development of Alberta the Peace River country and Dunvegan crossing played very important roles and persist in importance today.

The crossing of the Peace River at Dunvegan was originally made by canoe. As the population density increased, transportation capable of greater capacity and with a more predictable schedule became necessary and a ferry system was initiated by the Provincial Government. Today, this is inadequate because of its limited capacity, and due to the fact that in certain spring and fall months it is not possible to negotiate the crossing at all. The obvious solution to this was an all weather bridge which is under construction at present.

The topography of the Peace River crossing is very rugged, consisting of steep banks some 700 feet in height. Because of this topography the approach roads to this bridge pose a serious problem. In solving the problem of maintaining gentle grades in the approach to the bridge, the Dunvegan Creek valley offered a natural route. Certain large fills were required along sections of the road. As the predominant valley forming agency along these banks is landslide and slump the result of construction over this terrain was uncertain.

During the fall construction period along the South approach to the

Dunvegan crossing of the Peace River, significant movements of the fill about one mile south from the river were observed. Previous experience of the Department of Highways in similar topography led to the conclusion that a major earth movement was imminent. The Alberta Research Council in a co-operative effort with the Department of Highways Construction Branch undertook the investigation of this problem.

It was initially planned that the investigation would yield data by which the existing stability conditions could be assessed and possible means of stabilizing the slope logically arrived at. This, however, proved impossible as a landslide of major proportions occurred shortly after spring construction was begun. Treating the problem as a large scale field shear test, it was impossible to define, with reasonable accuracy, the slide mechanism. On the basis of this knowledge speculation as to stabilizing methods is possible.

A number of uncertainties enter the problem due to the geological history and mineralogic nature of the soils. Since the problem of shear strength of these clays has been investigated in a conventional manner, it is not surprising that no startling developments have been made. Nevertheless, the accumulated data has been found to agree with existing hypothesis and this is in itself significant.

Laboratory strength tests have been found in complete agreement with observed field information and stability analysis. The results of these analyses indicate an infinite slope type of failure and that the stable slope angle of these hills can be as low as ten degrees. The results emphasize that the best available method of predicting stability is by use of effective stress analysis assuming an effective angle of internal friction of about 20° and zero effective cohesion. These analyses do not account for the effects of long time leaching and base exchange which are fundamental to the problem of slope stabilization.

In the light of this investigation it is reasonable to assume that the conventional methods of analysis, although indispensable, are not sufficient to adequately explain the phenomena of loss in strength in detailed form. It has become increasingly apparent to the writer that basic research into the mineralogical and physico-chemical properties of these soils is required. The importance of this type of research has been demonstrated by Lambe, Warkentin, Grim and other prominent investigators. Certain procedures have already been established in this connection. The application of this form of research to our problems is felt to be of importance.

The stabilization of these overconsolidated soils has always presented a serious problem. The application of electro-osmotic methods has been thought of as one possible means. Research could be profitably directed towards this. Applied to preconsolidated materials, electro-osmotic stabilization is a virgin and challenging field. There is sufficient evidence to indicate that electro-osmosis would aid materially in stabilizing the Dunvegan slopes.

The following sections of this report will cover, in more detail, various aspects of the problem. It is concluded that the work carried out is of significance, but a great deal of research into detailed aspects is necessary before the general problem of stability of pre consolidated clay slopes can be satisfactorily solved.

2. Description of Site

Previous to the landslide, the area affected lay in Dunvegan Valley about one mile upstream from the Peace River. At the area under consideration (Fig. 10), the proposed highway was to cross a series of old landslides. Design considerations necessitated this crossing on fill to maintain consistent grades. The final depth of fill was to be about seventy feet. In spite of previous troublesome experience gained by both the Department of Highways and the Northern Alberta Railway in similar conditions, geometric design considerations of the proposed highway favoured this location.

The Dunvegan Creek valley stretches from its origin about five miles north-east of Spirit River in a north-easterly direction to its mouth at the Peace River. The creek itself carries run-off in the spring, but during the summer months has very little flow, this flow resulting mainly from subsurface seepage.

At the site of the landslide the valley is some six or seven hundred feet deep and has mainly landslide and slump topography. Indications of recent landslides are readily found and take the form of escarpments and cracks, on which vegetation has not had time to redevelop. Landslides are a natural erosional agent along these valley walls as recorded by Rutherford[1]. It is of interest that the first record of landslides in this general area was made some 150 years ago by an early settler at Dunvegan[2].

The site of the recent landslide was inspected during the early part of May, 1959 by the writer and other Research Council staff, previous to the slide.

The landslide occurred during the period May 19 to 23, 1959, and was first evidenced by slight cracks forming in the area below the filling operations.

In an attempt to drain water the Department of Highways began construction of a large trench near the base of the slope. The major movements of the landslide occurred when the above-mentioned trench was partially completed. It is of interest that quicking conditions were encountered in the trench when it reached a depth of about twelve feet.

The precise boundaries of the slide as determined by a transit and stadia survey are shown on Fig. 13. Figs. 1 - 5 are photographs showing the nature of the topography after the slide.

Examination of the slide area showed that there had been a seventy foot vertical drop of the grade. The crown of the slide was at the top of the fill against the natural ground. The ground below this was extremely broken and rugged as evidenced by the photographs.

At the north flank of the slide there was a vertical displacement of some thirty feet. Along the south flank the ground is again broken and cracked. Between the flanks there is an extensive network of cracks averaging about three feet wide add twenty feet deep.

The lower end of the slide area fans out into Dunvegan Creek. The creek was dammed to a depth of about thirty feet by material thrust into it from the landslide. The total material moved has been estimated to be between four and six million cubic yards.

As a result of these movements, construction was stopped and to date has not been recommenced. This has given ample opportunity to study the movements that took place and install instrumentation in adjacent areas for the purpose of determining possible future stability.

Topographically, the area drains from the south along Dunvegan Creek and there is evidence to indicate that subsurface seepage would be in the same direction. It has been a continuing disappointment to the writer that the opportunity of investigating this matter has not been taken to date.

3. Geology of Area

The geology of the area has been adequately described by Rutherford[1]. The materials forming the valley banks at Dunvegan are fine-grained, sedimentary clay shales, formed by the consolidation of clays and silts. These shales have a fissile, laminated or thinly stratified structure as well as argillaceous composition.

Geological evidence indicates that post depositional glaciation has occurred here. The overburden pressure caused by huge masses of the glaciers has been the main shale forming process. The action of consolidation due to this glaciation and subsequent retreat has resulted in a highly preconsolidated clay or what is commonly referred to as overconsolidated clay shale. Problems encountered with this material have been described in detail by Hardy[3]. One of the main processes responsible for the unusual nature of these soils is that it is now in a state of "rebound". Implications of this state are also described by Hardy.

Mineralogically, it has been suggested that these clays contain a certain percentage of the mineral montmorillonite, which has unusual water absorbing capacities. It is possible that this is the case, however, there is no definite quantitative data on the Dunvegan soils pertaining to clay mineral content or chemical composition. It is felt that such information would be a very significant contribution towards the final solution to this problem.

Fossils recovered from test borings were classified and found to be Cretaceous in age.

The Dunvegan valley is in a youthful stage of development and the main erosional agent now responsible for its formation is landslide and slump.

It has been observed that the strata dip in a north-westerly direction. That is, the strata dip predominantly towards the Dunvegan Creek and Peace River valley bottom. Under such conditions experience has shown that weak bedding contacts may cause slides if the dip is flatter than the valley walls, or if construction cuts or fills have slopes steeper than the dip[4]. This problem is accentuated with the build up of hydrostatic pressure along the bedding planes. In addition, these clay shales introduce problems of rebound and deterioration due to weathering.

The outward or streamward movements common to most valley walls are the result of weathering and creep. These movements are further accentuated in these preconsolidated slopes by elastic and consolidation rebound, oversteepening of slopes and surcharge loads. The manifestations of these movements are the opening of joints and development of shear planes for considerable distances back of the normal zones of creep. Subsequent to this, the problem becomes further complicated by the possibility of seepage and hydrostatic pressure build up in the joints and fissures so opened. The existance of these conditions is clearly indicated by the photographs (Figs. 6 to 9) of samples taken from one of the test holes.

Because of the multiplicity of conditions and unknown factors of these slopes the precise mathematical analysis of slope stability has been unreliable in predicting stability. Stability analysis has been used mainly as a tool in assessing the slide mechanism, and results of such computations are invaluable in suggesting correction measures. The case under consideration in this report has been well documented and the stability analysis clearly indicates the conditions in the slide area when failure occurred.

A wealth of information pertaining to the behaviour of over-consolidated clays and of slopes of laminated clays has been gained from considering the geological aspects of the problem. It is felt that such examination should be carried out as part of exploratory programs in determining the suitability of difficult locations for highway construction.

4. Description of Borings and Laboratory Tests

In total about eighty test holes have been advanced in the area by means of power-driven exploration equipment. The test holes of major significance are shown on the plan of the slide area (Fig. 13). The test hole logs corresponding th these test holes are shown on plates 1 to 30 inclusive.

These borings were advanced into soft sandstone about ten feet and indicate the overlying material to be overconsolidated stiff laminated clay shales of medium to high plasticity, containing slicken-side and fissures. This material is interbedded with thin silt layers of low plasticity and seams of fine to medium sand varying in thickness from about one-half inch to two feet. The materials show inclination of strata (Figs. 6 - 9), and indicate previous geologic distrubance resulting in fissures. It is interesting to note that evidence of previous disturbance can be found at depths of 180'.

Moisture content profiles were obtained from these holes, and classification tests were carried out at frequent intervals by the Department of Highways soils laboratory. Strength test and consolidation data were obtained on several specimens only at the University of Alberta.

Only the test hole data pertaining directly to the stability investigation are enclosed herewith. All other test hole data is on file and may be obtained from the Department of Highways or Alberta Research Council.

The amount of strength and consolidation data available at present is disappointing. It is hoped that the subsequent winter and 1960 program will show a greater accumulation of such data. The choice of samples for this purpose will, by necessity have to be very selective. Because of the heterogeniety of the subsurface conditions, random strength tests would not contribute significantly to the data. Strength testing should be confined to specific materials at depths where instability originated, i.e. along the surface of failure. Triaxial compression tests have been carried out on some of the material from the foot of the slide and the results are shown on Fig. 11.

There was a distinctive lack of manpower and time available for futher testing during the 1959 summer period.

Other laboratory tests include consolidation tests and density tests, the results of which are not included in this report.

An interesting and pertinent aspect of laboratory testing could be opened in the field of electro-osmotic stabilization. It has been suggested by Dr. R. M. Hardy in a personal discussion that beneficial results might be obtained by use of electro-osmosis. Although the masses of soil involved in such slides would not allow economical use of electro-osmosis, the possibility of the joints and fissures being selective to an electric current exists. Experience has shown that seams of water-bearing material are generally the cause of landslide. These seams can be silt or sand or even fissures with water percolating through them. As such seams or joints and fissures would have a lower electric resistivity than the boundary soil masses, electric current would be channeled to these small areas. The possibility of a favourable base exchange phenomena in the soils of these seams or adjacent to the fissures coincident with the application of electric current exists, and there is no doubt that there would be some drying effects.

The selectivity of such seams could be readily determined in the laboratory and based on this a pilot electro-osmosis test might be run in the field next year.

Other laboratory tests could be initiated in order to determine the mineralogical composition of these soils.

It must be recognized that when highly heterogeneous soil conditions exist, laboratory strength testing may be of limited value in assessing the overall behaviour of the soil masses.

It is interesting to note the triaxial specimen tested (Photo 9). A seam of silty-fine sand will be observed along the plane of failure of the specimen. The importance of this is that, by luck, both samples tested displayed this characteristic and this parallels exactly the conditions prevailing in the field at the time of failure. It will be noted on Fig. 14 the surface of failure passed through semi-continuous layers of material identical in plasticity characteristics with that found in the lab specimens. One of the specimens tested gave an unusually high strength (Fig. 12). Upon slicing the subsequent examination of this specimen, no indication of such a plane of weakness was observed. This would appear to demonstrate the effects of thin seams of silty sand on the strength of a soil mass.

The effective angle of internal friction found for the first two samples was 20 1/2°, which checks perfectly with observed field conditions and computations, whereas the specimen in which no faults existed, displays a possible angle of internal friction of 30°. The latter angle is highly inconsistent with the probable field conditions.

5. Field Tests and Instrumentation

Field tests and instrumentation is conveniently subdivided in pre-slide (A) and post-slide (B) observations.

A. Pre-slide Instrumentation consisted of open cased holes for the purpose of water level and hole inclination observations. These holes were cased with either 3/4" I.D. plastic tubing or 2" I.D. galvanized iron pipe. The holes in which these pipes were placed are indicated in Table I. The information obtained from this instrumentation is test hole water levels and the depths at which significant pre-slide movements were observed.

It must be recognized that care is required in interpreting these data, and that they have been used in identifying significant trends only.

In the ground water observations it can be stated that this form of instrumentation was not sensitive to the volumes of water available. Open holes sometimes require lengthy periods of time to show reliable results as pointed out by Hvorsley[5]. This is apparent by the ground water levels in holes drilled in the summer and fall of 1958 which displayed a ground water table near the ground surface as compared to those drilled during the spring and summer of 1959, which showed no consistent trend either up or down. In future operations, this type of instrumentation would be of more value if a type of piezometer of sensitivity consistent with the permeability of the soils involved were used.

The use of these same cased holes as probe wells was, however, of greater reliability and by using 1/2" O.D. coupled rods as a probe, the depths at which sliding occurred was found on line 1208+04 (Fig. 14).

B. Post slide instrumentation consists mainly of Casagrande type piezometers. To date, four of these piezometers have been placed above the slide area along the proposed location of the new highway centerline. These four have shown no piezometric heads at all. This is consistent with all the test hole data obtained from exploration in this area to date.

One Casagrande type piezometer has been placed in the slide area, the results of which were not available when this report was completed.

A full description of these piezometers is given in Appendix C, page 43. Other possible forms of piezometers are also given in this Appendix.

Electric Resistivity measurements were made adjacent to test holes, with the object of identifying moist seams. These tests showed no consistent trend and were, therefore, abandoned. The analysis of these results however, will be of value in determining soil condictivity for purposes of possible electro-osmotic treatment.

More reliable information in the initial phases of an investigation of this type could be obtained by the immediate installation of Casagrande Piezometers and perhaps tiltmeters. Several forms of tiltmeter are presently available and their application in areas some time in advance of construction would yield valuable information on subsurface movements.

6. Ground Water Conditions

The area referred to herein is that shown on Map 1, adjacent to the landslide of May 1959. It is evident from this map and the contour Map 2, that the most pronounced drainage to the site of the landslide is from the south-west along Dunvegan Creek.

Exploration of this area during June 8 to 12 revealed the following ground water conditions.

North of the landslide, somewhat upstream from the mouth of Dunvegan Creek on the east side, there are two freely flowing springs. These springs run from an outcrop of pebble conglomerate which forms an almost vertical bank of Dunvegan Creek at this point. The location of this area is easily identified on aerial photographs because of its marked relief.

Examination of test wells along line 1193 directly adjacent to the primary slide area on the south revealed the water conditions as outlined in Table I. The distances from the proposed highway center line to these holes were obtained from existing survey stakes while the elevations were obtained from a line of trigonometric levels run for the purpose of obtaining the ground cross-section of line 1193. It is evident from these data that from distances 592 to 1245 feet left of center line the ground water table is, for all practical purposes, at the ground surface. In contrast no other observation wells established in the primary slide area showed this condition. This conflict may be explained by the fact that all observation wells on line 1193 were established during the late summer of 1958 while most other wells were established in the spring of 1959. Experience has shown that considerable time is often required before open test holes reflect the true ground water conditions.

The wells along line 1193 yield, in all likelihood, the most reliable data. The test wells along lines in the primary slide were destroyed by the landslide.

There has been an element of doubt that the water in the test holes along line 1193 actually displays the true piezometric levels, or if it is actually from the surface run-off filling the holes. There are several strong arguments in favour of the former condition as follows:

(i) Most of these holes are cased and either backfilled or sloughed in, and in most cases in such a topographic position that water cannot easily flow into the hole.

(ii) These holes were drilled in the summer and fall of 1958 allowing sufficient time for ground piezometric levels to develop properly.

(iii) They are south of the prime slide area so that they were neither destroyed by the slide nor had the seepage water been cut off due to the slide.

(iv) They are in the probable line of moisture movement.

(v) These holes yield the ground water condition necessary for an infinite slope failure in a soil having the shear parameters as measured in the University ofAlberta laboratory.

It is more than mere coincidence that these conditions exist. However, in order to confirm the water levels, it has been urged that better instrumentation be installed. As this has not been done to date, it could be included in the next summer program.

In the Alberta Geological Report No. 21 by R. L. Rutherford, it is shown that the only successful dug wells in the adjacent country are at the town of Spirit River. The topography of the area is such that drainage is from Spirit River north-east to the slide area as evidenced by the accompanying contour map extracted from the above-mentioned report.

The evidence appears conclusive that the source of troublesome water occurs from a south-west direction along the valley of Dunvegan Creek.

It is possible that the water may be carried by a pervious strata, or that this general ground water table is near the ground surface. Evidence gathered from the borings favours the former condition. As no further exploration has been carried out to date, south of the slide area, the exact conditions are at present inconclusive. Water emerging from the spring north of the slide appears to have its origin from an easterly direction. Since the slide did not affect the flow of water here it might be concluded that its origin is not from the south. There is, however, the possibility that this water is coming vertically upward through a fault from some underlying water bearing zone.

Recent test holes and the Casagrande piezometers directly above the slide area have yielded no indication of ground water or piezometric head. This appears to preclude the possibility of a source of water from an easterly direction. If such is the case then the extreme eastern limit of the aquifer has been established.

In order to establish the path of the ground water and a possible method of drainage it is advisable to trace the water from its origin by means of exploratory boring. It is urged that drilling south of the slide be commenced early next spring.

7. Mechanism of Landslide

There has probably been no better documented landslide in Alberta than the Dunvegan slide. Field observations and measurements before the slide, and laboratory test results have allowed a comprehensive analysis to be made. It is interesting to find that all the necessary information to establish the slide mechanism has been obtained.

Based on field observations of probe wells and measurements of the actual landslide, the surface of failure has been established beyond reasonable doubt. This surface takes the form of a long radius arc extending from the crown of the slide, reaching depths up to eighty feet, then breaking out at the toe about eight hundred and seventy feet downhill.

The most comprehensive and reliable data was gathered from test holes along line 1208 (Fig. 14). Examination of these test holes shows that the surface of failure passes through and/or along successive seams of saturated, silty find sand. It is significant that at the location of the proposed drainage trench a toe was first evident. The bottom of this trench was in a quicking condition when the major slide movements took place.

Analysis of these conditions leads to some interesting conclusions.

Laboratory test results (Figs. 11 - 12) yield an effective angle of internal friction as determined from triaxial consolidated quick tests of 20 1/2 degrees. Theoretical stability analysis (Figs. 14 - 15) based on the method of slices and infinite slope analysis show a factor of safety of unity for the critical piezometric condition of the ground water under sufficient pressure to produce a head as high as the ground surface. These piezometric conditions were observed adjacent to the south of the slide on line 1193 (Fig. 15).

It can be concluded that the governing material was the above-mentioned layers of silty sand and that piezometric pressures were in all likelihood built up in these seams.

At the location of the proposdd drainage trench, the surface of failure is found to be about twenty four feet deep. Quicking conditions were observed when the trench reached a depth of twelve feet. Analysis of this shows that a piezometric head at the original ground surface would be required to cause a quicking condition.

Fig. 11 shows the data obtained from triaxial compression tests. Fig. 12 shows the relationship between the effective angle of internal friction required for a factor of safety of unity and the piezometric head with respect to the surface of failure for the same conditions. This relationship was obtained from results of the method of slices analysis (Fig. 14). It is significant that with the piezometric head at the ground surface the effective angle of internal friction required for stability is 20 1/2 degrees, the same as that measured from laboratory tests. Combined with the fact that such ground water conditions have been observed in the area the results are conclusive evidence of an infinite slope slide. Examination of the slide surface relative to the pre-slide cross-section shows that the slide surface is of great length compared to the depth of slide and that the radius of slide is in the order of one hundred times the maximum depth of slide, further fulfilling conditions of an infinite slope slide. It is of further interest that the results of the method of slices analysis and the infinite slope analysis show no significant difference in results.

Knowledge of the slide mechanism is useful in assessing possible means of stabilization. However, at the depths involved, there are very few effective means of stabilization available.

8. Corrective Measures

As previously mentioned in this report, the direction of ground water movements and the corresponding piezometric levels have not yet been conclusively determined. However, there is strong evidence indicating a north-easterly flow of ground water along Dunvegan Valley. It must be recognized that even though the quantities of flow may be extremely small, piezometric pressure is independent of the quantity of flow. The implication of this is that normal methods of drainage may be ineffective in actually reducing pressures.

Because of the depths under consideration, most types of drainage are impractical and the most obvious answer to this problem is the use of electro-osmotic methods. Some of the mechanisms of what might be expected have been discussed earlier. In addition, it is pointed out that ion exchange and drying of the mass of moist soil at present in existence below the slide area is possible. However, the problem of keeping it dry may not be possible without diverting the still unknown source of water feeding the area.

Indications are that the above-mentioned water flows from the south-west along Dunvegan Creek. If this is the case then the possibility of diverting this flow upstream of the slide exists.

Apart from diverting the original source of water, permanent stabilization cannot be hoped for. The other obvious measure is to re-locate the highway so that it will avoid the troublesome ground completely. This has been done to a degree by moving the center line of the highway uphill and into cut. However, it is pointed out that in so doing, lateral support of the natural slope must be removed. In addition, a great deal of lateral support was removed as a result of a landslide.

Removal of such support is conducive to subsequent outward movements, weathering of material as yet not affected and resulting deterioration of strength. Moreover, in removing overburden, the problem of consolidation rebound exists. Providing the quantities moved are not large, a road so constructed may be stable for some time, but experience gained by the Department of National Defence along the Alaska Highway has shown that the problem is not permanently solved.

It is again emphasized that electro-osmotic procedures hold some hope, combined with exploration and drainage.

9. Discussion

The detailed investigation of the Dunvegan landslide has yielded positive results in identifying the slide mechanism. Because of this, it is possible to take logical corrective measures.

Several interesting aspects have presented themselves as a result of this investigation. Firstly, is that not enough attention is being given to geological details in preliminary location work. It has become apparent that minor geological details may govern the behaviour of valley slopes. A thorough investigation of these details would at least indicate probable problems to be encountered. Secondly, in this case it appears that the slide took place along and/or through seams of silt and sand. If this is the case then rebound of adjacent clay materials may not be the governing criteria in these problems. On the other hand, softening of these materials may be the cause of creep which stains adjacent soil to its critical strain and causing failure. Further consideration of these details is required. In either case, high piezometric heads must exist and so the problem resolves itself to troublesome ground water conditions.

Detailed attention must be given to ground water survey in highway location. It has become vital that the direction of movement, hydraulic gradient, and quantities of flow are known. Moreover, it is necessary to know the chemical analysis of the water, the mineralogic nature of the soils, and the reciprocal base exchange capacity of the water and soil phases. This is important because it is now known that a base exchange in the soil can alter its strength characteristics greatly.

Other lanslide problems in the Peace River area offer a side latitude for future work. Considerable data on several landslides is available from the Department of Highways. It is not expected that these landslides would display identical characteristics to the Dunvegan slide. However, it may well be the case that similarities exist. An interesting program is offered in the classification of these various slides. This would allow a better assessment of stabilizing measures to be made. A program such as this would by necessity be experimental, and if undertaken, it should be recognized that various corrective measures must be attempted, some perhaps, without success.

It has been the observation of the writer that the Department of Highways Laboratories are not geared for proper study of landslide problems. It is, of course, the object of the Research Council to handle these problems. However, the writer respectfully submits the following remarks:

1. Special triaxial compression tests and other laboratory tests are often required. It has been found that time and facilities are not presently available to do these tests. For research purposes it is desirable to have equipment which is strain controlled for compression tests. In order to achieve continuity in a program, a full-time laboratory technician, skilled in triaxial work would be desirable.

2. More attention should be paid to the mineralogical nature of the soils. Facilities are available at the University for this purpose.

3. In order to obtain information from the field at a research level, it would be desirable that the Research Council have a direct command of field observation. The degree of cooperation between the Highways

Department has been exceptionally good. However, the information sought is often of a different nature and it is felt that if the Research Council were to attach the problem without external influence, we could more effectively deal with its fundamental nature.

10. Recommendations for a Continuing Research Program

1. Greater strength data is required. It is therefore recommended that an investigation of strength characteristics be carried out. This may require the acquisition of strength testing equipment. There is evidence that this phase of research would be very useful.

   The best method of approach to this problem would be to commence research from a working hypothesis. There is evidence (Fig. 12) that an extension of Casagrandes working hypothesis for the strength relationship of normally consolidated saturated soils. could be made. The writer proposes that the strength line found by Casagrande would be found to the left of the field rebound curve for preconsolidated materials. With this as the hypothesis useful research could be carried out. Along with this there appears to be an adquate physico-chemical background to explain the loss in soil strength and it supports the above hypothesis.

2. Becauuse strength characteristics are affected by the mineralogic nature of the soil it is recommended that an investigation into this appect of the problem be undertaken.

3. The possibility of stabilizing the Dunvegan Hill by electro-osmotic means exists. It is therefore suggested that a pilot test to establish the possibilities of this means be undertaken.

4. It is suggested that a detailed study of all slides in the Peace River area or those in valley slopes be undertaken with the object of discovering consistencies in order that the general problem can be better defined and stabilatory means investigated.

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APPENDIX A

DUNVEGAN HILL - STABILITY ANALYSIS

1. Method of Slices

Width of slice = 50'

Average unit wt. of soil mass = 128#/1^3

Slice No.	Shear Vector	Total Normal Stress Vector	Effective Norm. Vector
1	- 5	23	
2	0	35	
3	0	45	
4	6	57	
5	9	61	
6	10	71	
7	21	75	
8	13	83	
9	4	86	
10	10	82	
11	12	75	
12	14	80	
13	15	86	
14	18	93	
15	17	85	
16	23	90	
17	31	91	
18	50	30	

$\bar{v} = 248$ $\qquad \bar{P}_t = 1248$

Av'g. depth $= \frac{1248}{18} = 69.1'$

Case (1)

Average Piezometric Level at Ground Surface

$$\text{Effective normal stress} = \frac{1248(128-62.5)}{128} = 638$$

$$\tan \emptyset' \text{ reg.} = \frac{248}{638} = .256 = 21.2°$$

$$\text{Avge. hydrostatic head} = \frac{610}{18} \times \frac{128}{62.5} = 69.3'$$

Case (2)

Assume Av'g. Piezometric level (hp) at surface of failure (S.F.) (i.e. = 0)

$$p^1 \text{ reg.} = \tan^{-1} \frac{248}{1248} = 11.2°$$

Case (3)

Assume hp - 20' above S.F.

$$\frac{20}{69.3} \times 610 = 176 \qquad \emptyset'\text{reg.} = \tan^{-1}\left(\frac{248}{1072}\right) = 13°$$

Case (4)

Various hp levels w.r.t. S.F.

hp 40' above S.F.

$$\frac{40}{69.3} \times 610 =$$

Hp above S.F.	$\bar{\bar{n}}$	$\bar{p} = (1248 - \bar{n})$	$\emptyset' = \tan^{-1}\left(\frac{\bar{\bar{v}}}{\bar{p}}\right)$	
40	353	895	.277	15.5°
60	529	719	.345	19.0°
80	705	543	.457	24.6°
100	882	366		34.1°

2. Infinite Slope Analysis

a.) Average depth of soil over area $=$ 69.1' (h)

b.) Effective angle of internal friction from laboratory consolidated quick triaxial compression tests with pore pressure measurements $=$ 20 1/2° (ϕ')

c.) Average slope angle taken from crossection $=$ 10 1/2° (i)

d.) Hydrostatic head $=$ w

then for equilibrium

$$h \cos i \sin i = (h - u) \cos^2 i \tan\phi'$$

$$\frac{h \cos i \sin i}{\cos^2 i \tan \phi'} = (h - u)$$

$$h - u = h\frac{\tan i}{\tan \phi'}$$

$$u = h(1 - \frac{\tan i}{\tan \phi'})$$

$$u = 69.1 \times 128\ (1 - \frac{.185}{.373}) = 4470\#/\text{sq. ft.}$$

In terms of head in feet of water

$$\frac{4470}{62.5} = 71.5$$

This shows that the average piezometric head required to cause instability is 71.5 feet above surface of failure.

APPENDIX B

A. Observation Wells

The requirements of an observation for the purposes required in this investigation are as follows:

1. Water from a wide range in depth must be made available to a standpipe.

2. Sources must not be blocked by mud either produced from drilling or used supplementary to drilling where loss in circulation occurs.

3. The well must be capable of being pumped in order to check the amounts occurring.

These requirements may be met by use of the type of well shown in Figure. 1. This well would consist of drilling a 12 inch cased hole. In drilling it would be preferable for the casing to be pushed ahead of the drill in order to prevent blocking pervious seams by wet drilling. After advancing the hole to the required depth, the bottom is filled about 2 feet deep with clean fine gravel. An inner smaller diameter casing is then installed concentric to the large casing. The annular ring left between the two casings isthen filled with clean gravel and the outer casing then withdrawn. The only requirement of the inner casing is that it be large enough to pump when so required.

Although this form of well is expensive, it is felt that they will yield the information required.

For long-term and detailed observations in less permeable soils for ground water levels and pressures the type of observation well shown in Fig. 1 should be used. A well point is attached to the lower end of the standpipe and is surrounded by a filter of well graded gravel or sand. The hole must be carefully cleaned and filled with clear water before the standpipe with its well point is installed. Extreme care must be taken in obtaining tight joints in the standpipe. It is suggested that a 2" diameter casing be used. This will facilitate use of the ohmeter type depth gauge.

The Annular space around the standpipe must be carefully back-filled and the lower part adequately sealed. For the diameter of the pipe used the seal between the hole and casing may be confined to the lower part of the casing as shown. The seal should consist of compacted clay and bentonite.

Table I

Piezometric Heads in Test Wells - Line 1193

Distance Left of CL	Elevation Top Hole	Depth to Water in Hole - ft.
480		36.0
592		66.0
800		2.0
950		4.5
1150		4.0
1245		3.0
1350		Dry

Table II

DUNVEGAN HILL - PROBE WELL LEVELS

Depth indicates depth to which 1/2" O.D. rod may be thrust into various test holes. Total depth of test hole is shown on test hole logs and cross sections.

Section 1205 and 1208

Hole No. / Date	25	1s	2s	6s	7p	14p	4p	8p
May 1	92,27		97	109	64	68 1/2,12	86	
6				109	54,35	68 1/2,25		
8			97	110	64	54	82	65
11		18	98		54,35	Blocked with	86	73,55
12					54,35	pump rod	87,80	74,60
13	68		97 1/2/65	109/85	54,35		87,80	74,60
14								
15								
16	67.5	14.5	63	83.5	54	--	22	73
18	67.8	4.9	63.2	83.4	53.5	55.3	22.8	73.6

Note: casing was installed initially to bottom of test holes -
p - plastic casing
s - galv. iron pipe.

Section 1211+00

Hole No.	11	23p	24p	13p	27p	28p	12Ap
May 1	85	82	7	69	75	68	
6							
8		82	7	61	76	68	71
11							
12		82	73,65	58 1/2	76 1/2	68 1/2	72
13	85						
14							
15							
16	84.5	82	42.5	58.5	75.5	78.2	71.5
18	84.5	81.8	42.2	58.6	76.3	68.9	71.8

Eyewitness Account of the Slide at Dunvegan

Thursday May 21. A large crack had opened up on the grade during the night. It was about 2 feet wide and one side had dropped about 2 feet below the other. It extended from about 1204 + 00 to 1212 + 00 on the grade, and then ran off down hill each way. It was about 30 feet deep at its deepest point. In probing the test holes there was only one large change. That was at Hole 14A located at 760 ft. left of CL on 1208 + 04. The depth of this hole changed from approximately 95 ft. to 55 ft. The casing in this hole was heavy galvanized pipe so extreme movement would be necessary to cut it off. In nearby Hole 14 located at 763 ft. left of CL on 1208 + 04 the depth changed from 55 ft. to 29 ft. This change was not considered as significant because the hole contained plastic tubing which can be cut off much easier and with much less movement. Another significant change was noted at Hole 24 located at 640 ft. left of CL on 1211 + 00. The plastic tubing in this hole was protruding 1.5 feet above the ground before the slide. At 9.00 a.m. Thursday this pipe had apparently been pulled into the ground and protruded only about .1 feet. At 12.00 the pipe had been pulled farther into the ground, but was still visible about .5 feet below ground level. At 1.00 p.m. it was not visible at all. During this time there was no significant change in the depth of the hole. The depths of the other holes remained unchanged.

There were readings taken across the cracks throughout the day and at one point, about 1208 + 00 on the grade, the rate of movement was recorded as .3 feet per hour, vertically.

The slide "toed out" on a line approximately parallel to the road and about 825 ft. left of CL. There was no visible movement below this toe. The toe was located approximately down the centreline of a 120 ft. wide drainage ditch.

The ditch had been excavated to an average depth of 12 ft. on Wednesday May 20. The material excavated was piled down hill from the ditch. This excavation is believed to have triggered the slide. Roughly, the slide was between lines 1201 + 00 and 1215 + 00. By 5.00 p.m. Thursday the top of the slide had dropped about 7 feet below the grade.

Friday, May 22. The top of the slide had dropped about 70 feet. The toe of the slide had completely filled the uphill half of the drainage ditch. There was sliding all the way to the creek another 700 feet. The creek was cut off and there was visible and audible movement in the creek bed. On a straight line of stakes at 1205 + 50 from 900 to 1500 ft. left of CL there was lateral movement of 3 feet each way from the straight line.

There were no more depth or water level readings taken on the test holes. On Thursday and Friday snapping of roots and trees could be heard plainly where the cracks went through bush.

Saturday, May 23. At the top, the slide had dropped a maximum of 100 feet. In this same area the horizontal displacement was about 120 feet. At the creek, the horizontal movement was approximately 60 feet. The creek bed was completely filled to a depth of about 20 feet. Long lateral cracks appeared parallel to the centreline of the road.

Bruce Bryson,

Materials Inspector,
Department of Highways.

Eyewitness Account of the Slide at Dunvegan

Stakes were placed on Wednesday, 20th May, 1959, for the construction of a 40 foot deep ditch which was to have been constructed from approximately Hole 14 to Hole 8 on the Station Line of 1208 + 04.

Material (dirt) was hauled out and work had commenced.

About 11 p.m. on the night of Thursday, 21st May, 1959, cracks started to show on the upper fill of the proposed new highway for the curve between Stations 1205 + 50 to beyond the 1211 + 00 line.

The ditch had already been dug to 7 feet when work was suspended by the contractor, but however the whole of the hill extending right down to a stream which ran at the bottom had begun to mo ve.

The movement was most apparent.

The upper and lower fills just completely slid. The movement stopped momentarily but reoccurred in 24 hour cycles. Deep crevises appeared and the whole stablised structure was completely shattered.

An interesting occurrence was noted by inspection on Wednesday, 27th May, 1959, that the upper and lower fills had unbalanced themselves to join at an angle of 45°.

The whole area of the hill which had been reasonably steep was now much flatter and had slid down to block off the stream at the foot of the hill and come to rest being stopped by the buffering effect of the hill on the opposite side.

The toe of the slide being at the points at where the ditch was to have bben constructed on inspection afterwards was found to have saturated sandy and silty material.

The shock wave sent out caused the hill to slide beyond the 1193 station line, trees etc. being all carried away.

The vastness of the tragedy can only be visualised by photographs however.

John B. Owles,
Materials Inspector,
Department of Highways.

Eyewitness Account of the Slide at Dunvegan

On Thursday May 21, the first crack appeared in the grade situated on the slide. This crack extended from Station 1205 to approximately Station 1213 and curved around the northern perimeter to about half way to the creek. The crack looked to be about 20 feet deep, and was 5 feet wide at the most. Test holes drilled up to that time had showed no radical change in depth or water level, however, on Thursday morning changes occurred in Holes 14, 14A, 24. At Hole 14 (640 ft. left CL at 1211 +00) the plastic pipe had been several feet above ground, and was now several feet below -- evidently it had been sucked under. The rod reading changed from 42.0 ft. to 39.8 ft. The pipe (galvanized) in Hole 14A (760 ft. left CL at 1208 + 04) was heard to snap while 14 was being tested, and the depth was found to have changed from 93.8 ft. to 53.8 ft. The water level changed from 21.6 ft. to 42.5 ft. The changes in Hole 14 (760 feet left CL at 1208 + 04) were: depth - 55.2 to 29.5 ft.; water level - same as before. Water levels in Hole 6 (509 feet left CL at 1208 + 04) changed 16.3 feet and in Hole 12A (370 ft. left CL at 1211 + 00) they changed 3.8 feet. No changes were recorded on 1205 + 50 line. There was visible sliding around the cracks that day and they widened considerably. The drainage ditch dug on Wednesday the 20th proved to be the toe of the slide and a slight raise along the whole ditch at approximately 880 ft. at CL was noticed. Towards the end of the day small surface cracks appeared over the whole surface of the slide.

The slide moved considerably during the night, and by Friday morning the difference in height on either side of the crack on the grade was about 50 feet. On Thursday we had been down to the creek and it had been evident that everything below the toe was solid. On Friday however, the area below the toe of the original slide had covered the creek and was visibly moving toward the other bank. Great cracks appeared over the whole surface that day, and considerable effort had to be made in travelling through the slide to avoid them. The road perpendicular to the grade and running between 200 ft. and 300 ft. left CL between 1208 and 1211 was now level whereas it had been at least a 3 to 1 slope before. The galvanized pipe in Hole 6 (509 ft. left CL at 1208 + 04) that had been two feet above ground on Thursday, was now ten feet above ground, and bent at an angle of 45°. All road entrances to the slide were now blocked off by cracks also.

By Saturday morning the whole area was completely covered by cracks and holes. The difference in height on the grade now must have been at least 100 ft. Where the grade had slid, there was a smooth clay surface that looked like it had been the side of the hill before the road was built. Pieces of the grade still clung in chunks to the hill, but they were widely isolated. While we were cross-sectioning from 300 ft. left CL to CL between 1205 and 1214 great chunks of the grade had to be crossed. There was no level ground, and all these chunks were separated by cracks up to 50 ft. deep. Cracks also appeared south of 1205 and below the grade, but there was no apparent sliding there. By now the whole creek bed was covered right up to the other bank and the creek had begun damming up. This seemed to be the last day of movement, as no great differences have been noticed since.

Ross Norstrom,
Materials Inspector,
Department of Highways.

APPENDIX C

PIEZOMETERS FOR PORE PRESSURE MEASUREMENTS IN CLAY

by

A. Casagrande

(Revision prepared in February 1958)

Table of Contents

I. Introduction

The first edition of these instructions was issued in mimeographed form in July, 1946. Since then many modifications were tried by various organizations and by the writer for the purpose of simplifying the rather cumbersome and time-consuming details of such installations. From such experimentation and extensive installations several improvements have developed which are incorporated in the present instructions; particularly the us e of shielded microphone cable, originally introduced by the Waterways Experimental Station, instead of two parallel stranded wires. The essential details of the design and installation of this type of piezometer has remained unchanged.

For countries where the metric system is used, most dimensions can be rounded off in a reasonable manner in centimeters. E.g., instead of specifying pebbles of approximately one-half inch diameter, one should specify approximately one cm. diameter; instead of a 3 in. thick layer, one should specify an 8 cm. thick layer; etc. However, unless the exact available sized of tubing are known when marketed in the metric system, it will be better to retain those dimensions in inches.

II. Equipment and Materials Required for Installation of Piezometers

(1) For the riser pipe use Saran or Polyethylene plastic tubing, 3/8 in. I.D. and 1/2 in. O.D., in lengths corresponding to the depths at which the porous points will be installed. Such tubing is delivered in rolls and should be unrolled from a spool to prevent kinks. It should be unrolled several days before it is installed and pulled straight frequently. In cold weather one should siphon hot water through it while it

is being pulled straight. In freezing weather Saran tubing becomes brittle and fractures easily, and under such conditions it is preferable to use Polyethylene tubing.

Splicing of the plastic tubing should be avoided if at all possible. Even a minute amount of leakage at a joint may cause misleading information.

(2) For the porous "point", use a ceramic tube, one or two feet long, having 1.5 in. O.D. and 1.0 in. I.D. E.g, Norton porous tubes, either fine or medium grade, are satisfactory. Their commercial length is 2 ft., and either a full length or a half length may be used.

(3) One Neoprene or rubber stopper tapered from slightly larger to slightly smaller than one inch in diameter. E.G., Cenco 18155, No. 5 Neoprene stopper.

(4) One Neoprene or rubber bushing consisting of a 5 in. long piece of soft pressure tubing, 1.0 in. O.D. and 3/8 in. I.D., or 17/16 in. O.D. and 7/16 in. I.D.

(5) A tamping hammer, as shown in Fig. 3 made of a 3 ft. length of seamless steel tubing, 1 5/8 in. O.D. and 5/8 in. I.D., and provided with a flat tamping face 3/16 in. smaller in diameter than the inside diameter of the casing. At the upper end a loop of 1/8 in. diameter, galvanized airplane cable should be firmly attached to the hammer and to a grooved ring, as shown in Fig. 3. The inside surface of the hammer should be smooth and all edges that could touch the plastic tubing should be rounded. This hammer-cable assembly is used for the following purposes:

(a) To tamp the bentonite layers and, thereby, assure a water-tight seal between the casing and the plastic tubing.

(b) To centre the plastic tubing while the bentonite seal is being tamped into place.

(c) To measure depths at various stages of the installation.

(6) Galvanized performed airplane cable, 1/8 in. diameter of sufficient length to permit installation of deepest points. At one end there should be securely fastened a snap-type, swivel hook, e.g., of the type used on a leash for a large dog. For measuring depths, the cable should be marked in five foot intervals, on strips of white tape wrapped around the cable, starting with zero length at the bottom surface of the hammer.

(7) A tripod and sheave for operating the tamping hammer.

(8) Washboring equipment.

(9) Thoroughly washed sand between No. 20 and No. 40 mesh in grain size; or Ottawa Standard sand, if available.

(10) Bentonite balls about 1/2 in. diameter, which are formed at a water content somewhat above the plastic limit but below the sticky limit (i.e. at a putty-like consistency), rolled in a talcum powder to prevent sticking, and stored in glass jars to protect them from drying. (American Colloid Co., Volclay KWK No. 33, is easy to use.) When bentonite is not available, fat clay may be substituted.

(11) Rounded pebbles of about 1/2 in. diameter.

III. Assembly of Porous Point and Plastic Tubing

One end of the porous tube should be plugged with the neoprene or rubber stopper, and the plastic tubing should be attached to the other end by means of the Neoprene or rubber bushing. The necessary length of the plastic tubing will be governed by the depth of the point below the elevation where the observer will stand. To connect the plastic tubing to the porous tube one should proceed in the following steps:

(1) Insert the end of the plastic tubing about one inch into the Neoprene or rubber bushing, using water as lubricant.

(2) Push the bushing into the porous tube with a twisting motion for a length of at least 3 in.

(3) With a twisting motion force the plastic tubing further into the bushing until the end is approximately flush with the end of the bushing (prior to this operation place a mark on the plastic tube at a distance from its end equal to the length of the bushing). It requires considerable effort to force the plastic tubing into the bushing. This can be facilitated by the use of the following method:

> Wrap the end of the plastic tube in cloth for protection and then clamp it in a vise. Then turn the porous tube with the bushing and force it over the plastic tube until the required penetration is reached.

If properly installed, it is impossible for one man to pull this joint apart by hand.

IV. Installation of Porous Point and of Seals

A cased hole should be advanced to the elevation of the bottom of the "pervious space" that will contain the porous tube embedded in sand. See Fig. 2. Two-inch diameter pipe is recommended for the casing.

The bottom 10 ft. long section of the casing should be in one piece, without joints orcouplings, and it should not be provided with a drive shoe or coupling at its lower end. For this section of casing one may use a No. 18 to No. 20 gage, 2 in. thin-wall steel tube, with a standard 2 in. pipe coupling welded to one end in order to permit extending the casing in the ordinary manner. In locations where pore water has a corrosive effect on metal, the outside and inside of this bottom 10 ft. section of the casing should be protected by a corrosion-proof coating.

While advancing the casing for the lowest 10 ft. no washing should be done below the bottom of the casing, to ensure a tight contact between the outside of the casing and the surrounding soil. However, the soil inside the casing should be removed during driving, to prevent excessive disturbance of the soil by displacement.

After the casing is advanced to the desired depth, the installation of the porous point and of the seals should proceed in the following steps:

(1) Clean the casing thoroughly by washing. Replace water in the casing with clear water, by reversing the flow of the pump and using the jet pipe as intake, with its lower end held a few inches above the bottom of the casing. Continue this operation, keeping the casing full of clean water, until the return water becomes clear.

(2) Pull up casing 2 ft. preferably by jacking. A slip weight may be used, if necessary.

(3) Pour thoroughly saturated sand slowly into the casing, to fill the bottom 2 ft. of the open hole. The required volume should be computed and controlled. Then measure elevation of top of sand by lowering tamping hammer.

(4) Fill the porous tube and the attached plastic tubing with clean water and close the end of the plastic tubing with a stopper. Then lower the porous point a short distance into the casing, remove the stopper, and keeping the end of the plastic tubing above the level of the top of the casing, continue to lower the porous tube intil it rests on the sand. Thread the hammer over the plastic tubing, and while applying a constant tension to the top of the plastic tubing equal to the weight of the plastic tubing, but at least 5 lb. lower the hammer until it rests on the porous tube, tap the top of the porous tube lightly with the hammer; then read the depth on the cable and withdraw the hammer. During this and the subsequent operations check to see that the plastic tubing is not disturbed. For this purpose a piece of tape should be wrapped around the plastic tubing at the elevation of the top of the casing and while tension is applied to the tubing the exact elevation should be marked on the tape.

(5) Maintaining a constant tension on the plastic tubing, pull up the casing so that its bottom will be one foot above the top of the porous tube. Simultaneously, pour saturated sand slowly into the casing, of such predetermined volume that it will fill the space around and above the porous tube and approximately 2.5 ft. above the bottom of the casing, as shown in Fig. 2.

(6) Drop into the casing a volume of pebbles to form a 3/4 in. to 1.0 in. thick layer. Then lower the hammer and, keeping a constant tension on the tube, apply 10 blows with a drop of 6 in.

(7) Then place the bentonite seal consisting of five layers of bentonite balls, each one placed and compacted as follows, while simultaneously maintaining a constant tension on the tube:

(a) Lower the water 3 in. below the top of the casing.

(b) Drop bentonite balls individually into the casing until the water rises to the top of the casing.

(c) Drop into the casing an amount of pebbles which will form a layer approximately 3/4 in. thick.

(d) Lower the hammer, applying tension to the plastic tubing, and then apply 20 blows by raising the hammer about 6 in. and dropping it freely.

(e) Whenever the hammer does not move freely, it should be removed immediately and carefully cleaned. In particular, after each of the five layers is tamped, clean the hammer carefully before using it again. Also, shake the plastic tubing slightly in order to loosen material that may be sticking to the tube.

(8) Pour into the casing a sufficient volume of sand to fill casing approximately over a height of 2 ft. cover with a thin layer of pebbles and compact with 20 tamps.

(9) Repeat the placement of five compacted bentonite layers as described in step (7).

(10) Repeat Step (8).

(11) Fill remainder of casing with any available soil or leave open if desired.

(12) To maintain the plastic tubing permanently in a taut condition, provide its upper end with the following support:

(a) Place a bearing plate on top of the casing, with a hole in the centre which is slightly larger than the tubing and through which the tubing protrudes. While applying tension to the tubing, place cloth padding around and apply a hose clamp just above the bearing plate. The plastic tube should be cut off eight inches above the bearing plate, or higher if desired.

(13) Surround the top of the casing and tubing with a section of larger pipe and with a cover for protection.

V. Equipment for and Performance of Measurements

(1) When the water level is below the top of the plastic tubing, an electric sounding device is used, consisting of a shielded microphone cable as sounding wire and an ohmmeter to which the upper end of the sounding wire is connected.

Shielded microphone cable No. 1248 of Alpha Wire Corporation (50 Howard Street, New York, N.Y.) was found satisfactory. It is helpful to ballast the lower foot by means of several two-inch long sections of the same copper tubing used for making the contact point shown in Fig. 4, and allowing approximately oneinch between these sections. Their outer edges should be rounded and they must be over the wire before making up the contact point. It is desirable to paint these sections with insulating paint.

The end of the cable which measures the contact with the water surface inside the plastic tubing, should be made as shown in Fig. 4. The shield is folded back and soldered to a section of copper tubing 1/4 in. O.D. and 3/16 in. I.D. Soldering can be facilitated by drilling a few holes through the tube. The outside of the copper tubing should be painted with an insulating paint, except the bottom contact surface. The space in the end of the tube should be filled with sealing wax or other hard sealing compound, which will serve as an electric insulator. When using this device it is helpful to grease the surface of the sealing wax.

Any small ohmmeter which covers ranges up to 1,000,000 ohms may be used. (A convenient pocket-size instrument is the TRIPLETT MIGHTY MITE Volt-Ohmmmeter Model 310-VOM, manufactured by Triplett Electrical Instrument Company, Bluffton, Ohio; it has a range from zero to 20 million ohms; list price $33.80). The best range for a clear definition of the contact with the water surface is readily determined by trial. While lowering the sounding wire, the operator should differentiate between an occasional slight movement of the hand on the ohmmeter which is due to scraping of moisture from the sides of the plastic tubing, and the sharp kick of the hand when the water level is reached and which can be repeated several times for an accurate reading of the depth. To facilitate reading the depth, 5 ft. intervals should be marked on pieces of plastic electricians tape wrapped around the sounding wire.

(2) When the piezometric head is above the top of the standpipe, the readings can be made either with a reliable Bourdon gage attached firmly to the standpipe or a mercury manometer.

There are various ways to attach a Bourdon gage. A simple attachment is shown inFig. 5. However, it is desirable to add a vertical standpipe, in the form of an up-side-down Y, with that standpipe closed off with a gate valve. Gas bubbles that form in the porous point and plastic tubing will then collect below the gate valve. When the amount of gas begins to show in the plastic tubing, then it should be allowed to escape. When attaching the gage, all tubes must be filled with water and all connections must be made watertight. The Bourdon gage should be mounted securely on a suitable board, so that the elevation of the gage is not higher than the top of the plastic tubing.

A simple and entirely satisfactory mercury manometer can be improvised in thefollowing manner using a 10 ft. long piece of 1/8 in. O.D. Saran tubing one end of which is pushed through a tight fitting hole of a No. 00 rubber stopper.

(a) Put mercury and water into a dish as shown in Fig. 7(a).

(b) Immerse one end of the Saran tube in the water, and by sucking at the other end, fill the tube with water.

(c) Lower the end of the tube into mercury and by sucking at the other end fill approximately a three ft. length of the tube with mercury.

(d) Mount the Saran tubing on a suitable board in the form of a U with an additional downward leg, as shown in Fig 7(b), so that each of these three legs is approximately 3 ft. in length. Prior to final mounting, slowly lower the top end until most of the water in that leg has spilled out.

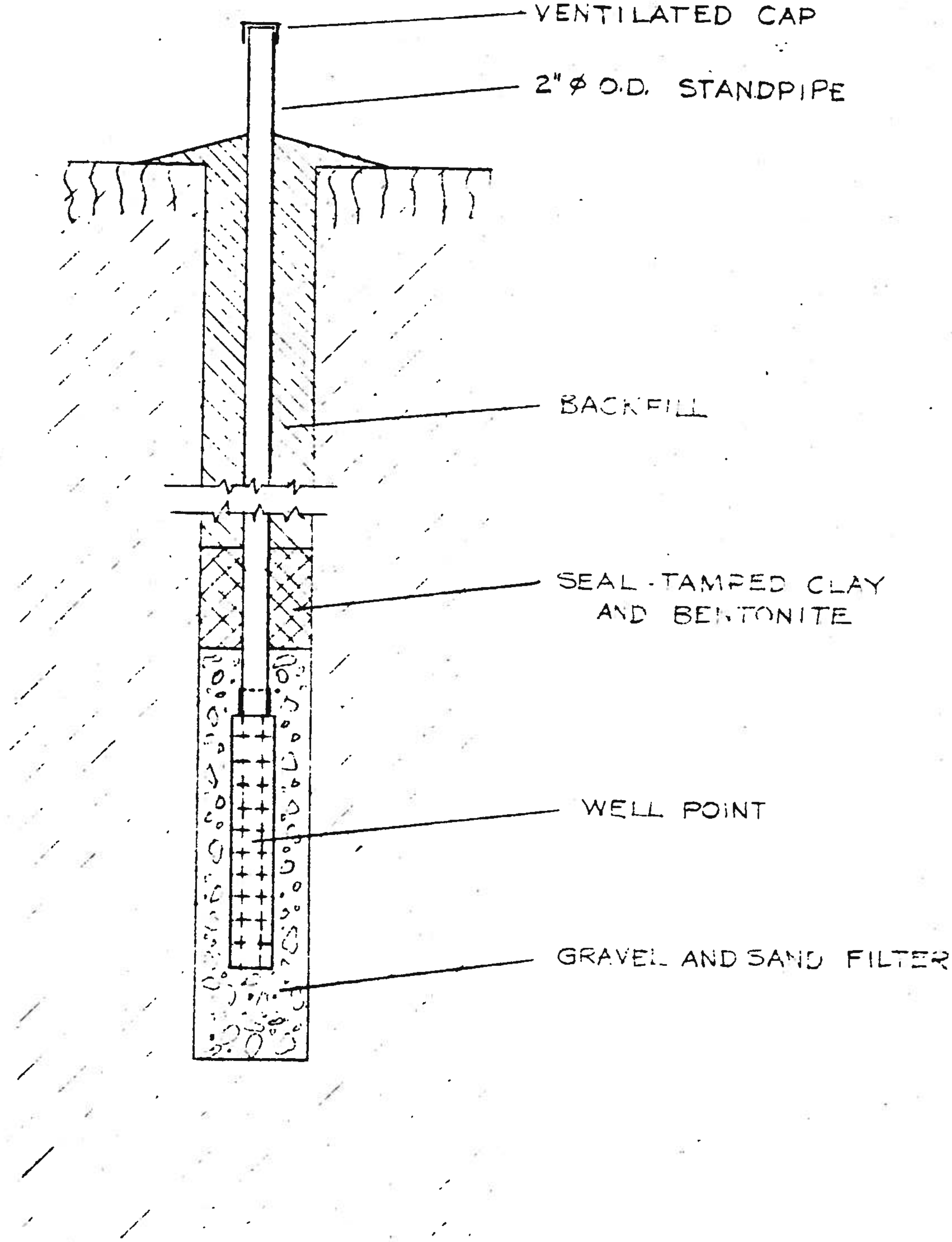
(e) Force the stopper into the top of the plastic tubing with the following precaution to assure that no air is trapped; Fill the plastic tubing with water so as to form a convex menisous, and then move the manometer tubing slightly to cause a drop of water to hang from the bottom end.

The piezometric head above the lower leg of the mercury column, in cm., is equal to the difference in elevation between the two mercury surfaces, in cm., multiplied by the specific gravity of mercury (*).

(3) Irrespective of the type of equipment used for measuring the piezometric heads, it is important to protect the top of the piezometer pipe, and any instruments above the ground, against direct rays of the sun. Furthermore, to minimize errors due to temperature variations, the readings should preferably be made at the same hour early in the morning.

(*) May be estimated from the following values:

13.60 for 30°F or -2°C
13.55 for 65°F or 18°C
13.50 for 103°F or 39°C

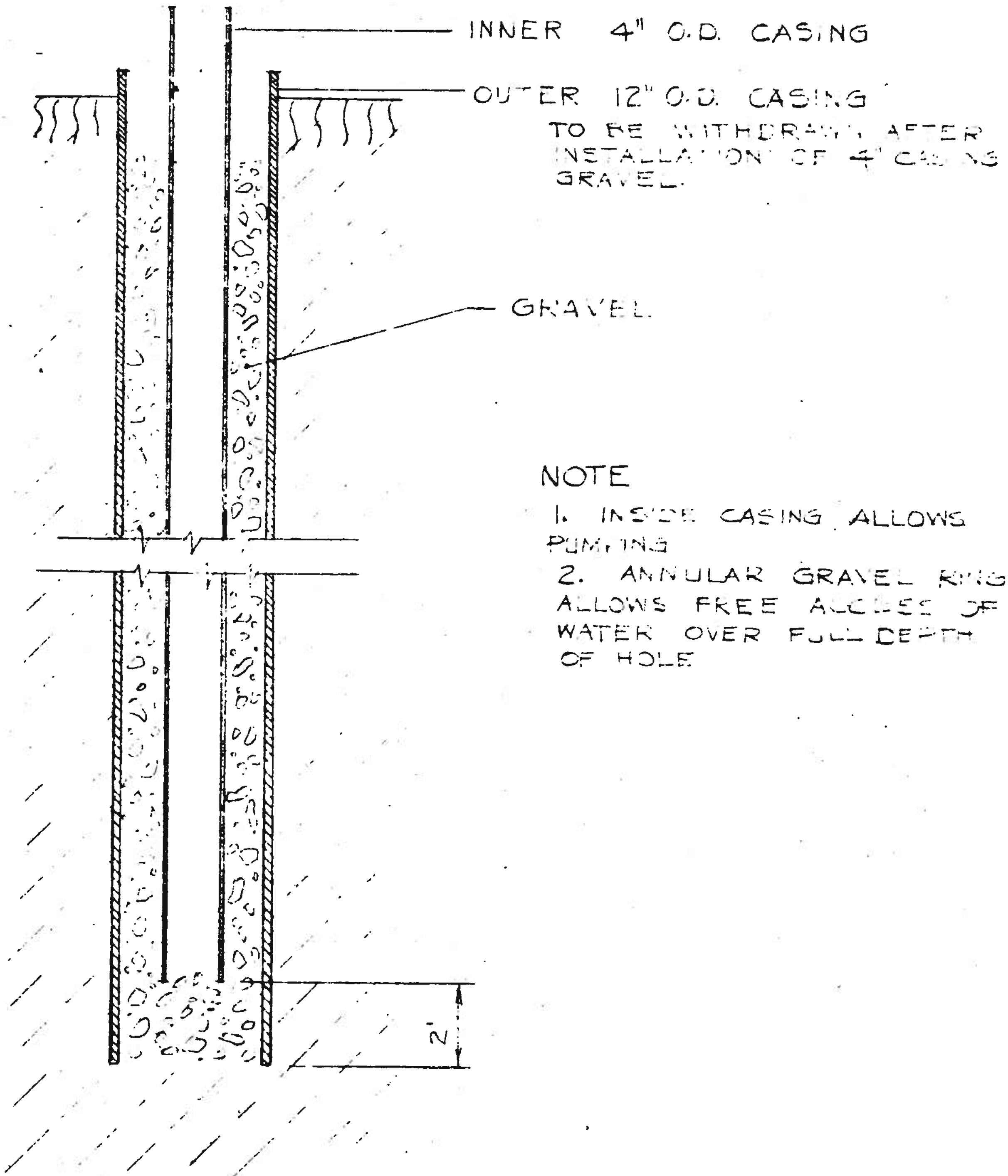


ALBERTA RESEARCH COUNCIL

OBSERVATION WELL

JUNE 18/59

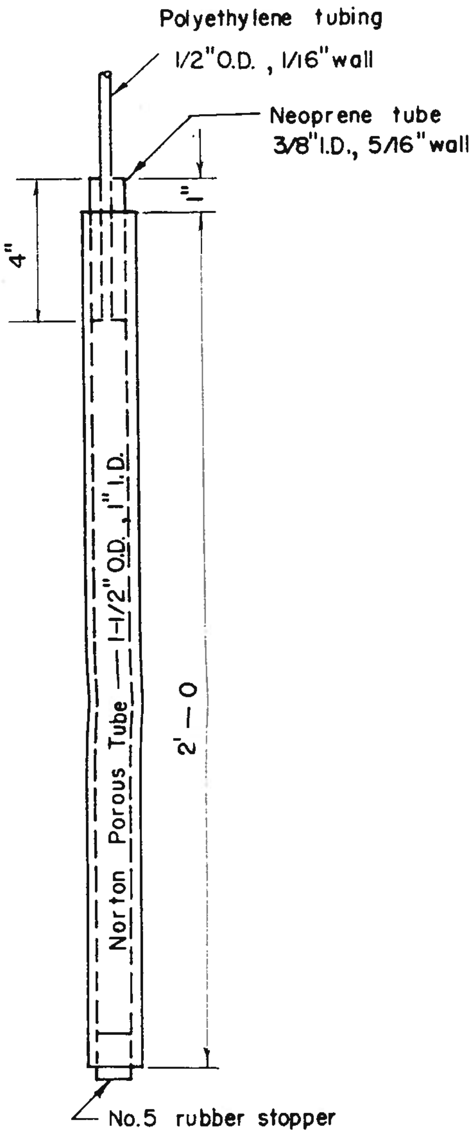
FIG. 1.



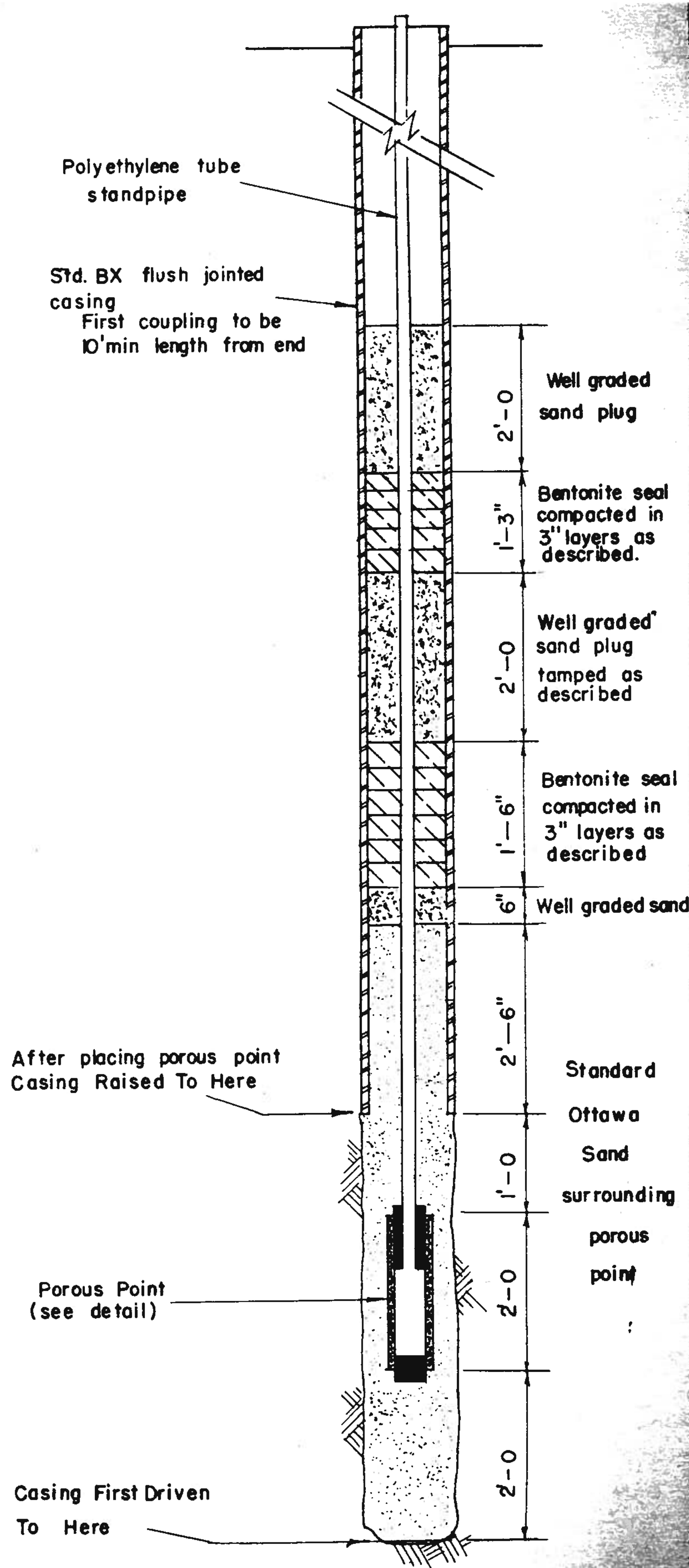
ALBERTA RESEARCH COUNCIL

OBSERVATION & PUMPING WELL

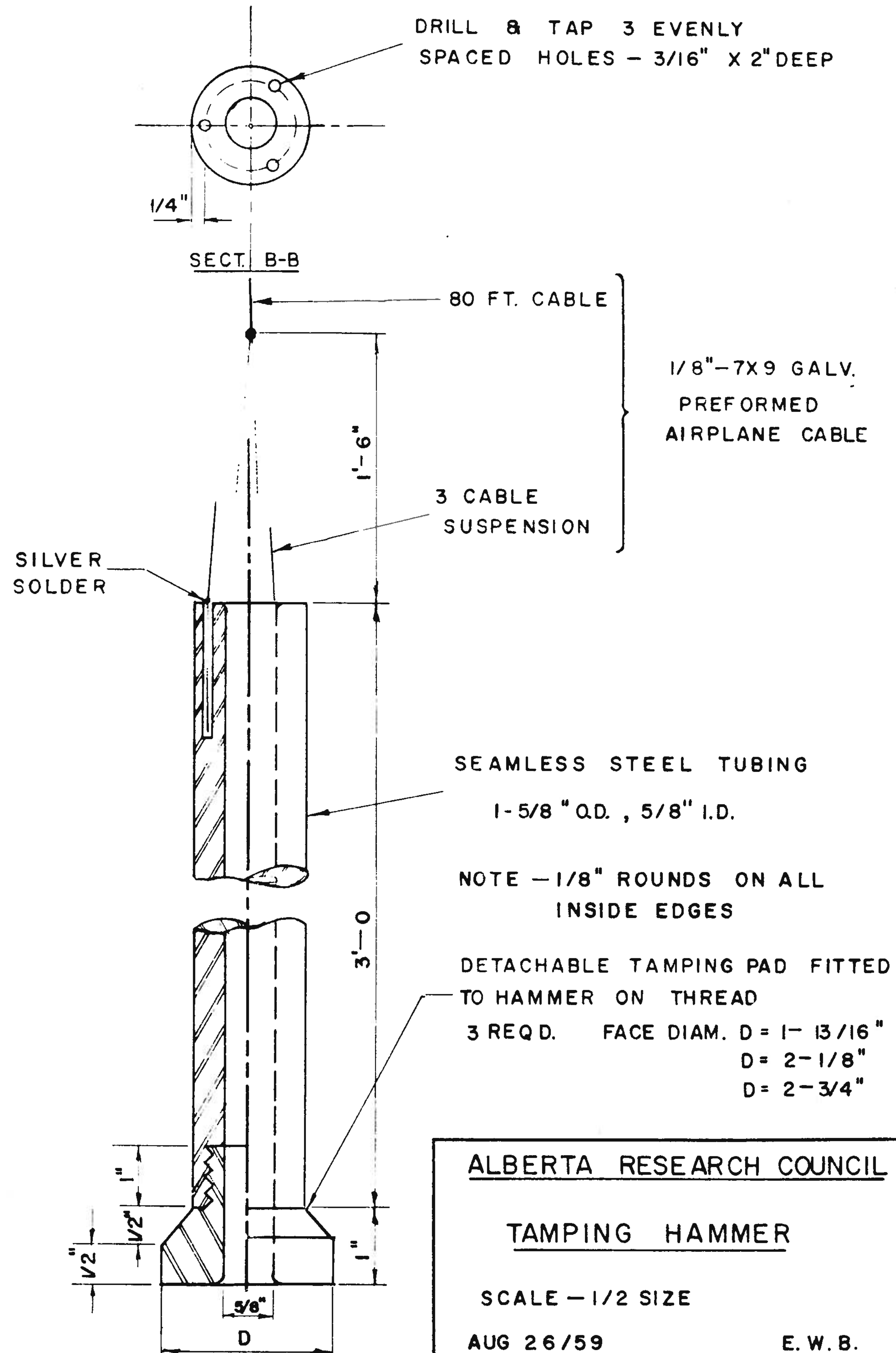
JUNE 20/59 E.W.B



POINT ASSEMBLY
3" = 1'-0



ALBERTA RESEARCH COUNCIL
CASAGRANDE TYPE
PIEZOMETER
PORE PRESSURE INSTALLATION
AUG.26/59
E.W.B.



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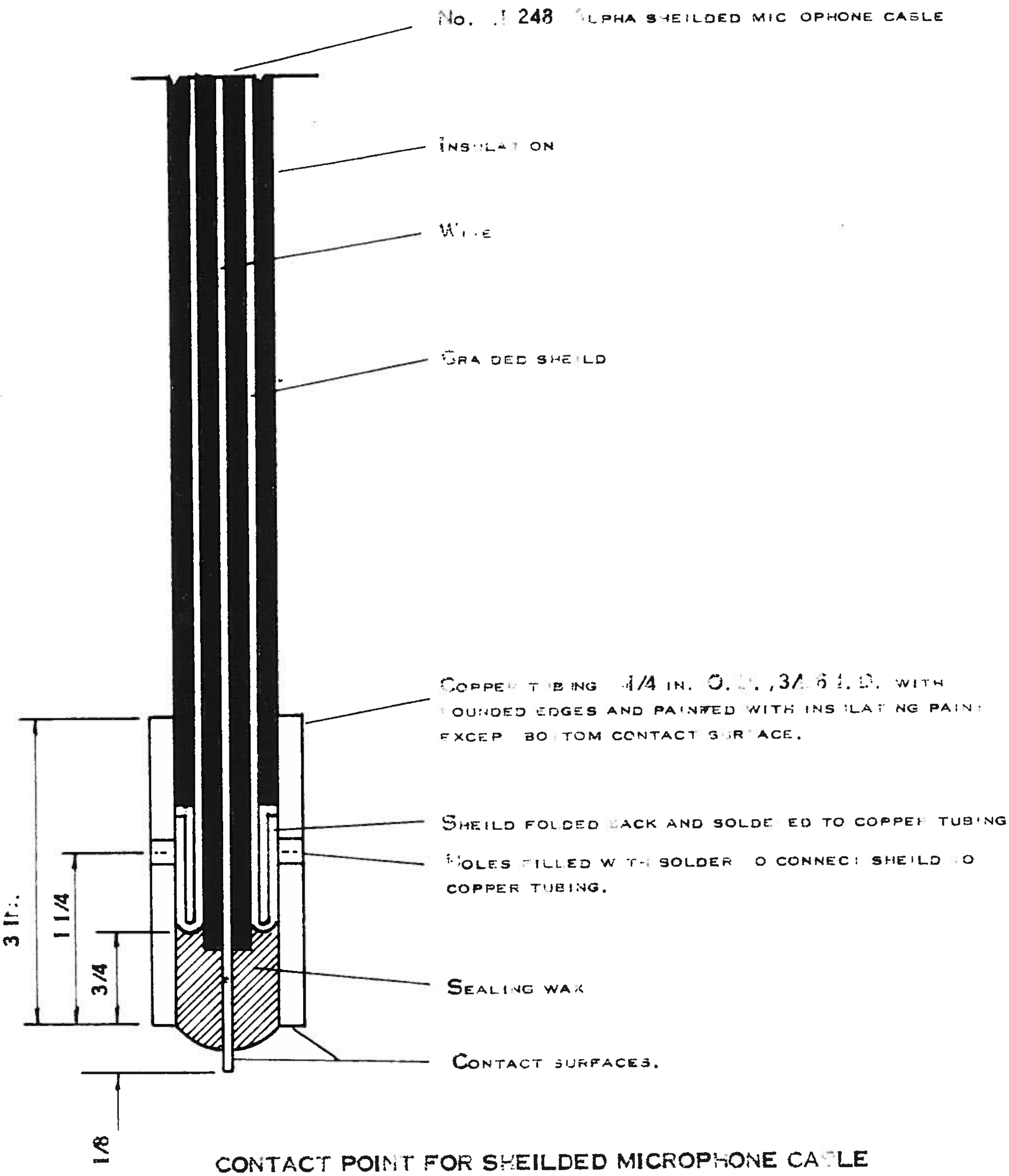
TAMPING HAMMER

SCALE - 1/2 SIZE

AUG 26/59 E. W. B.



CONTACT POINT FOR SHEILDED MICROPHONE CA LE

ENLARGED --NOT TO SCALE

FIG. 4

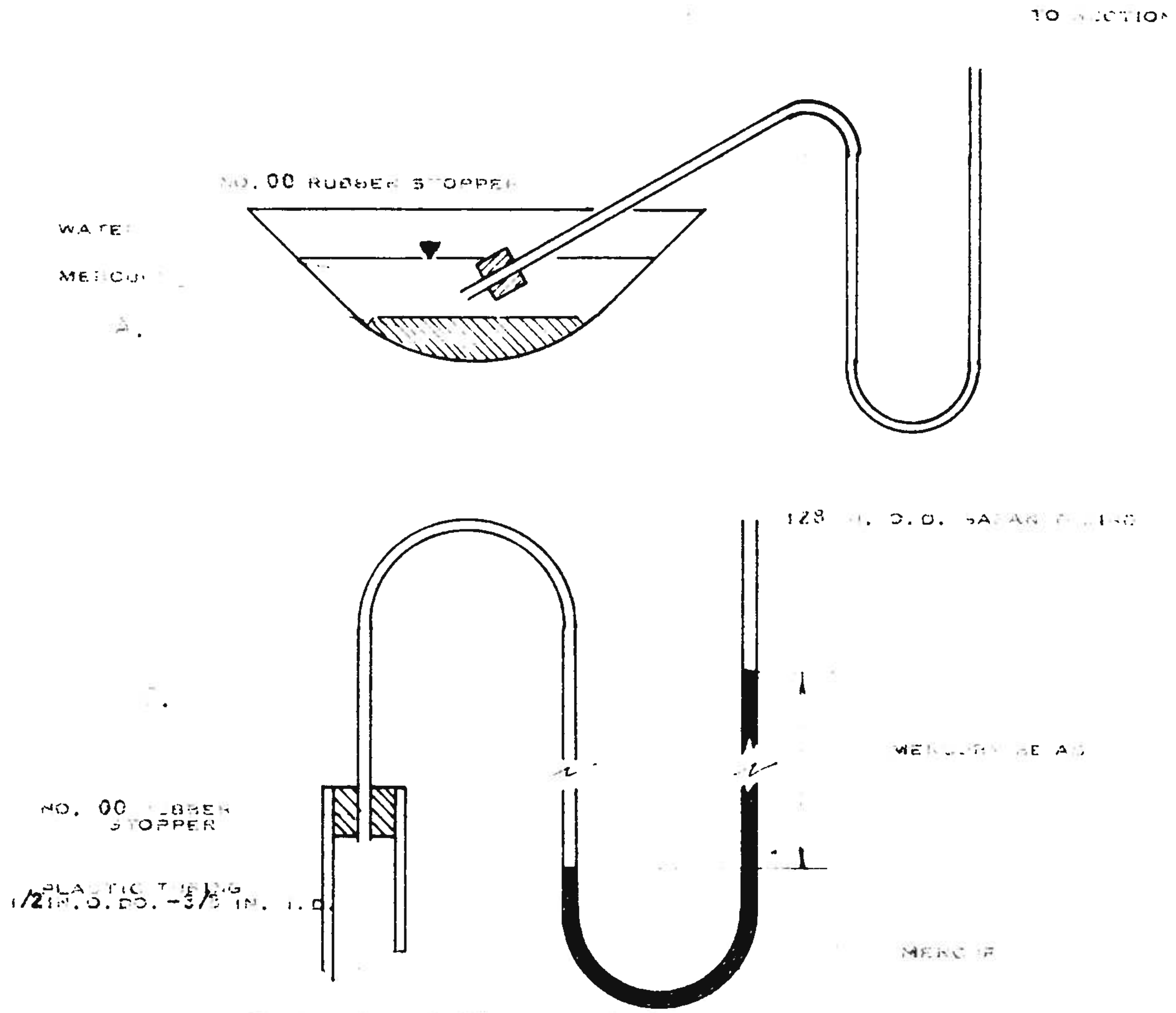


FIG. 7 MERCURY MANOMETER

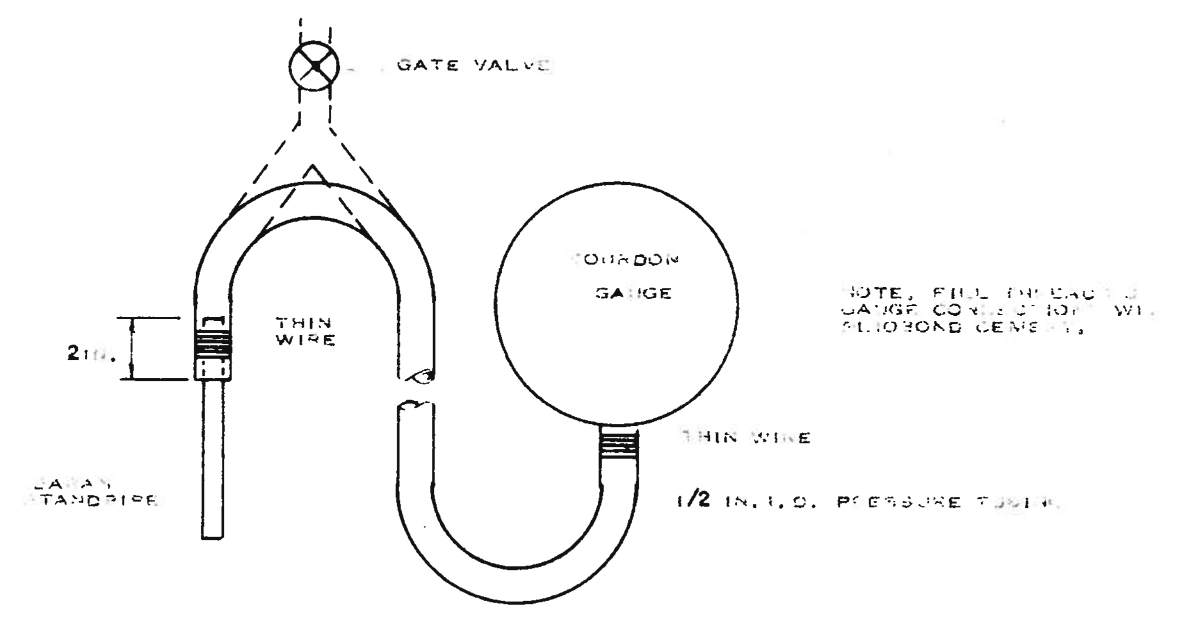


FIG. 8 BOURDON TUBE

FIGURES

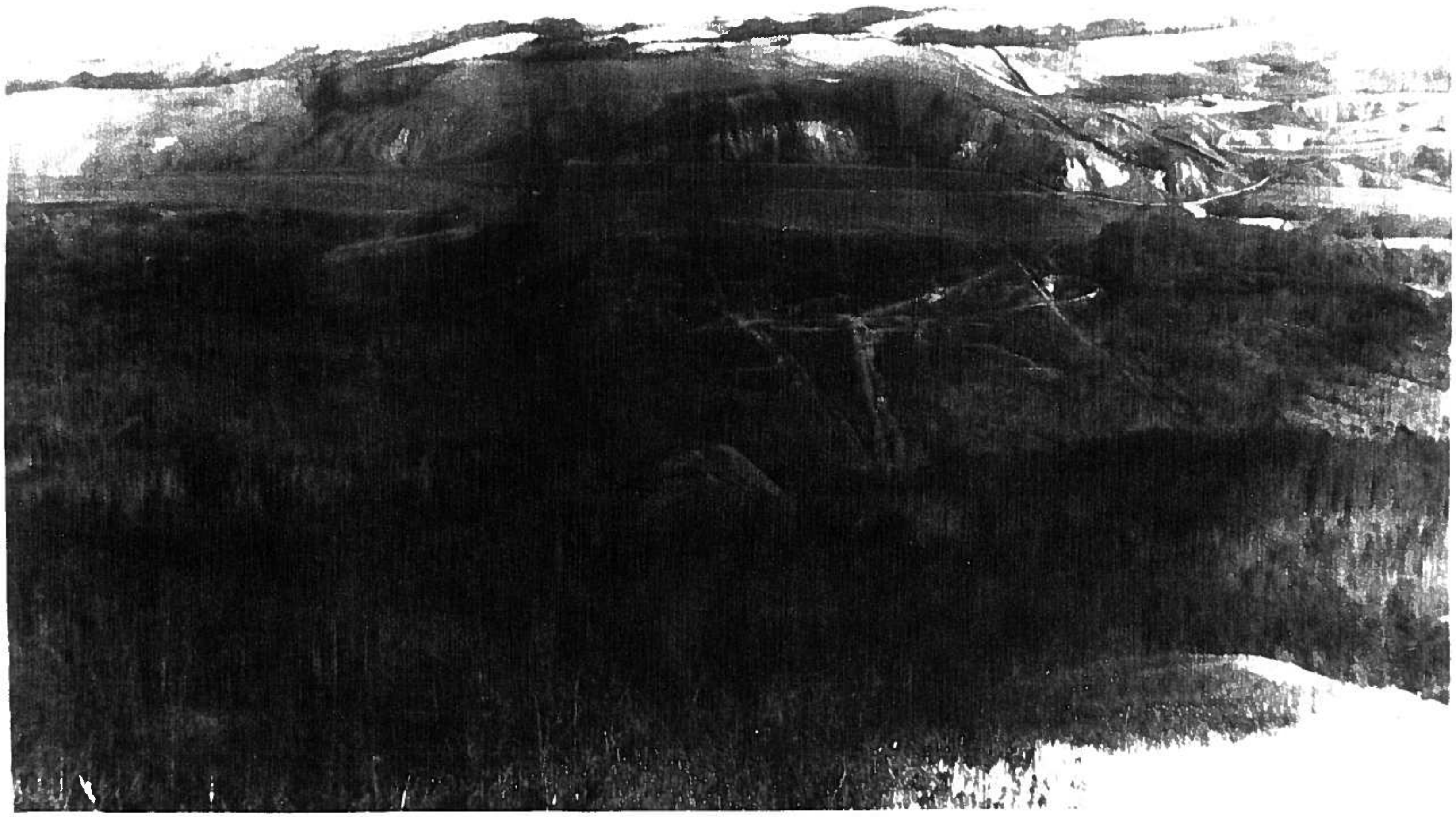


Fig. 1
View looking east at slide area
before slide of May 1959.
Note cut control lines.

Fig. 2
(Same as Fig. I but closer)

Fig. 3
View of landslide after slide of May 1959.
Note broken line shows boundary of slide.

Fig. 4
Looking south into slide escarpment.
Man standing on original highway grade.

Fig. 5
Looking south over slide.
Material shown was fill for highway grade.

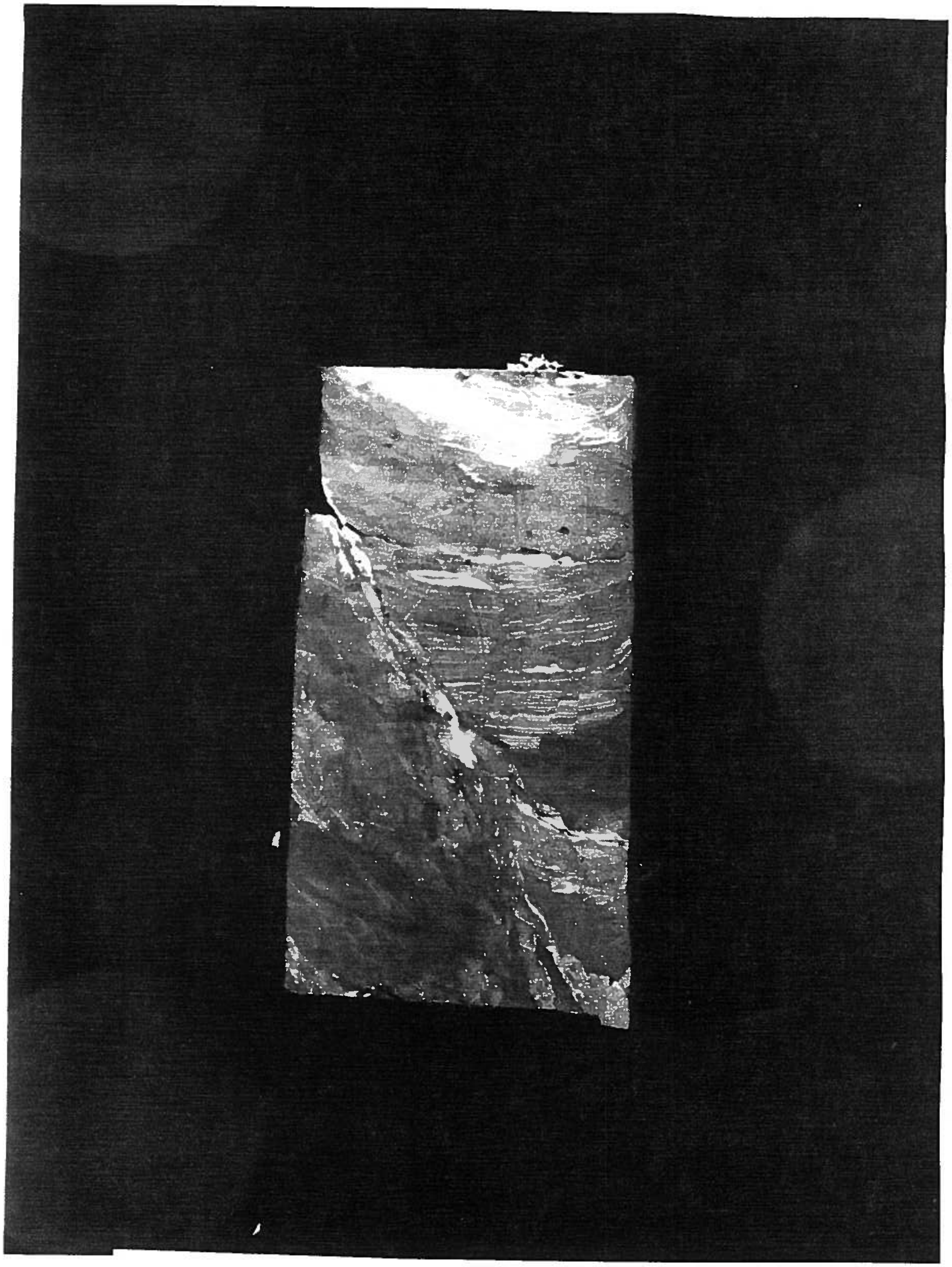


Fig. 6
Failed sample from Triaxial compression test.
Note the plane of failure
is in a pre-existing silt seam.

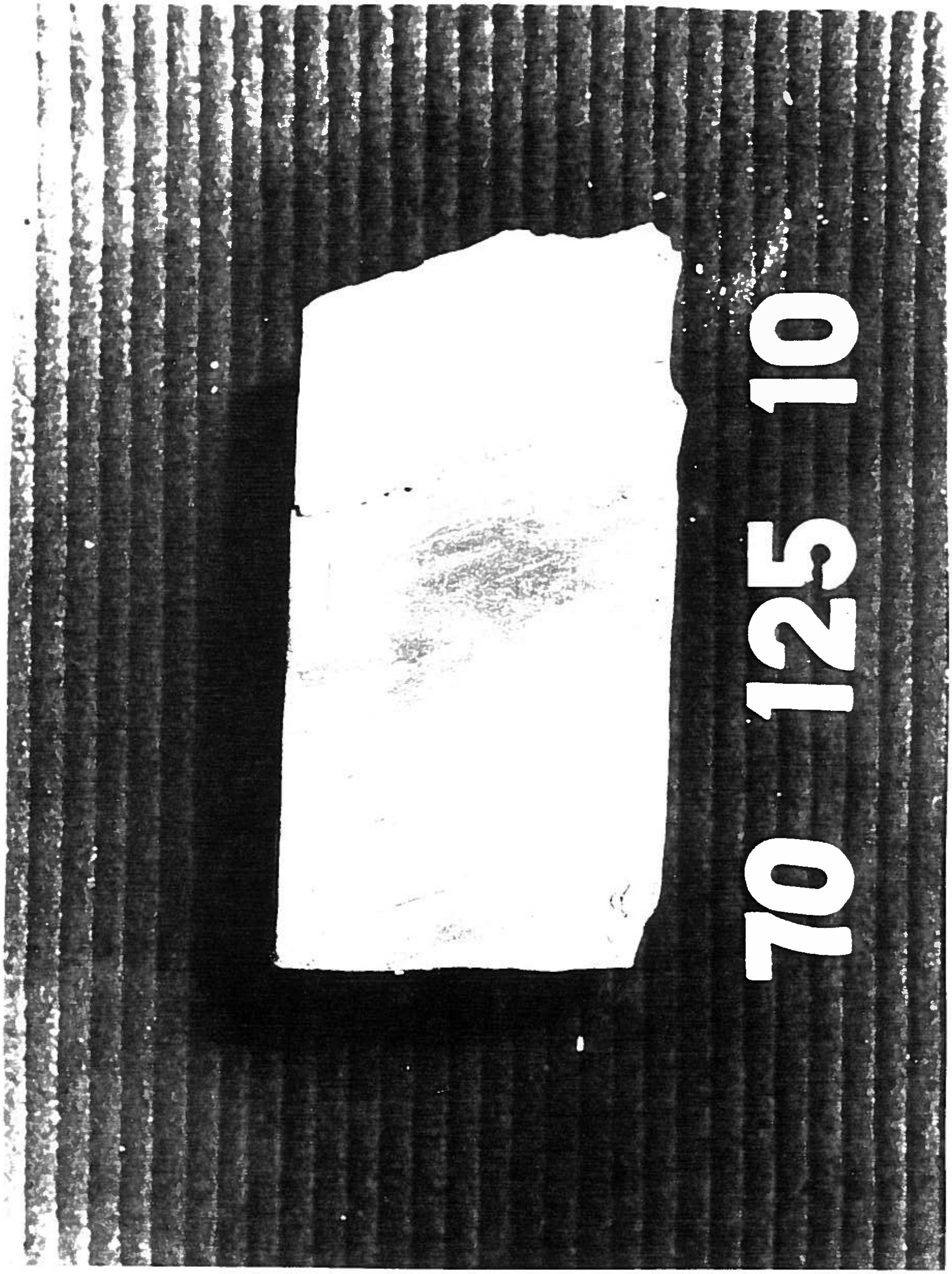


Fig. 7
Sliced sample from
Test Hole No. 70 depth 125 ft.
Note dip of strata and slickensides.

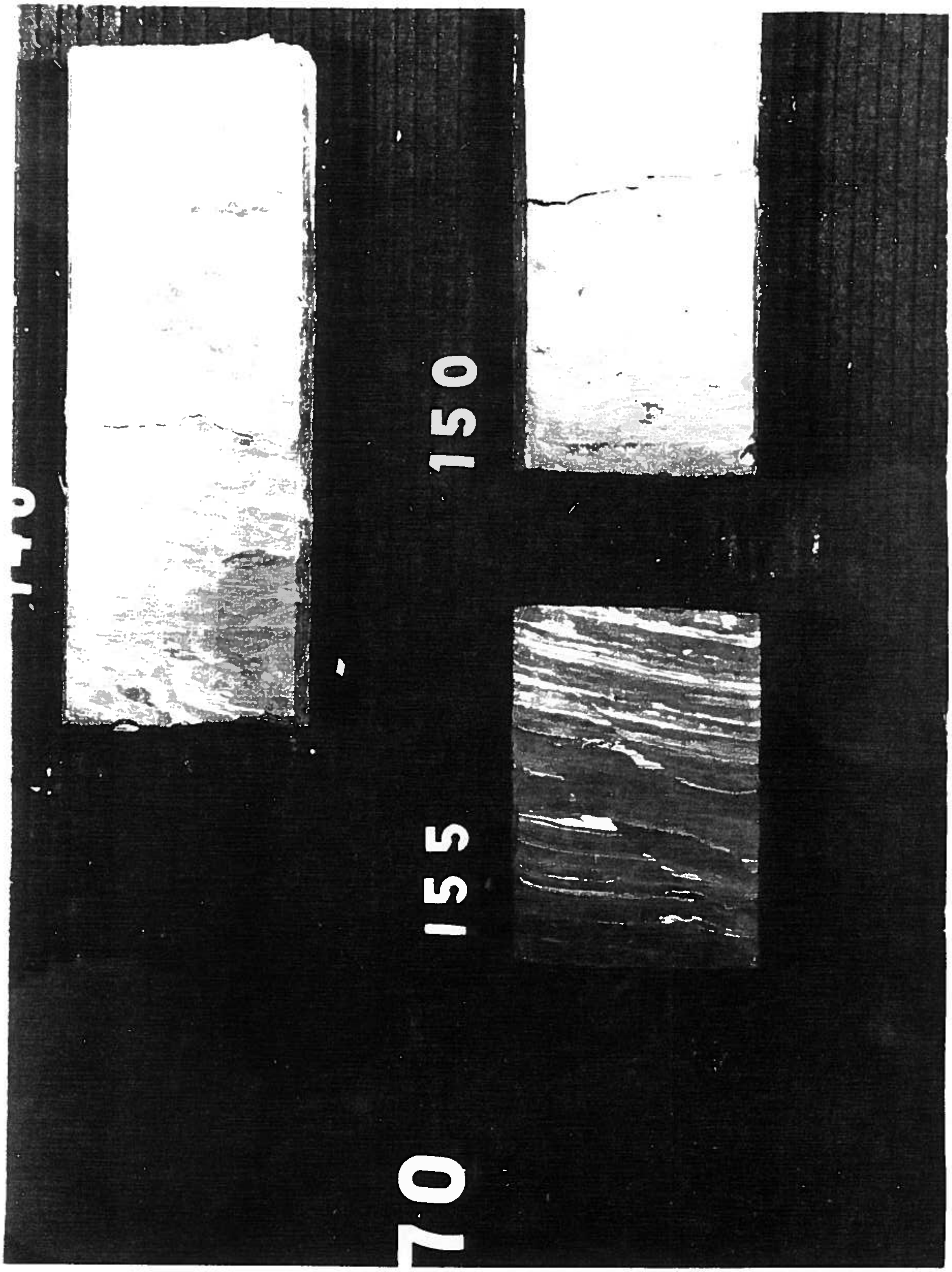


Fig. 8
Sliced samples from
Test Hole 70 Depth 140-155 ft.
Note laminations and slickensides.

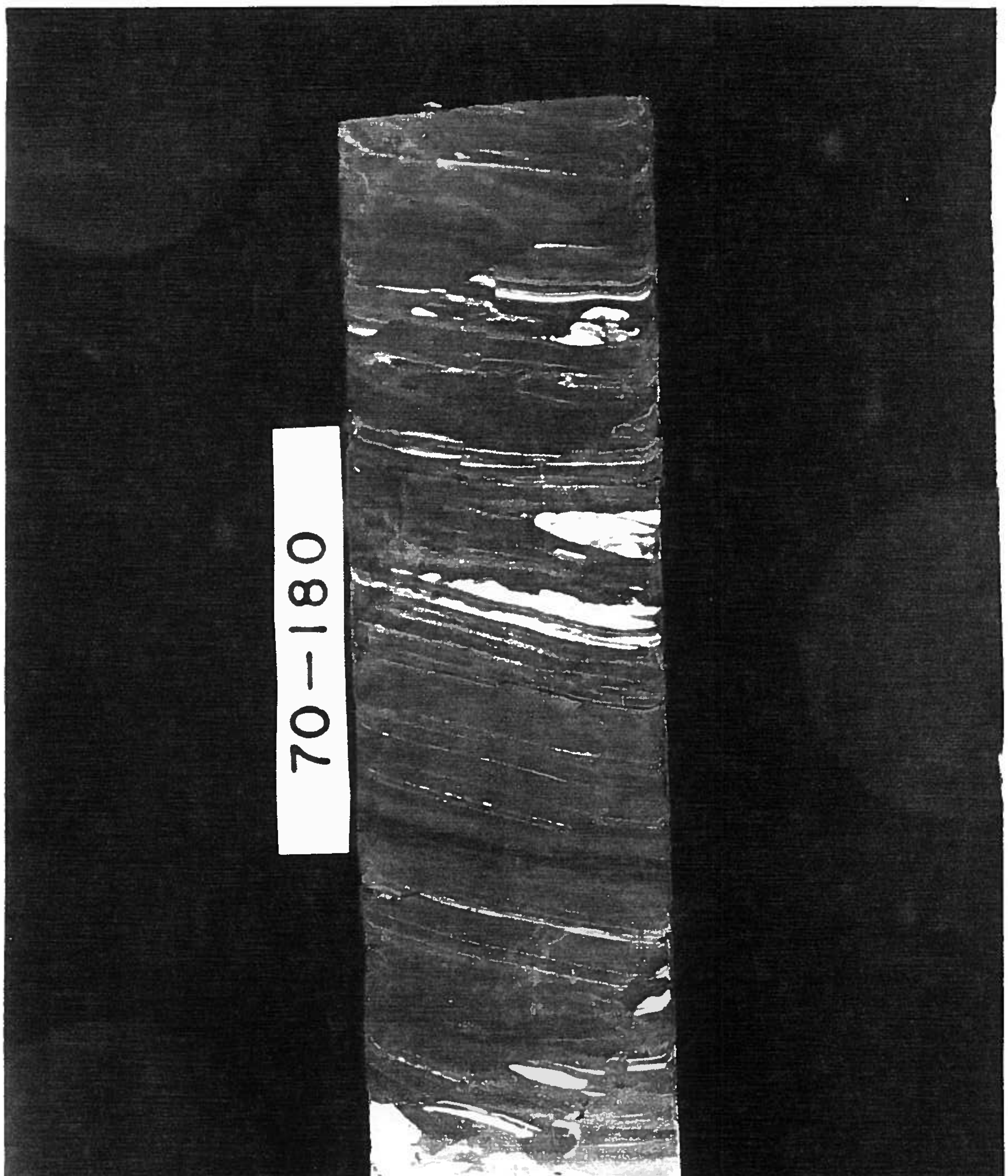
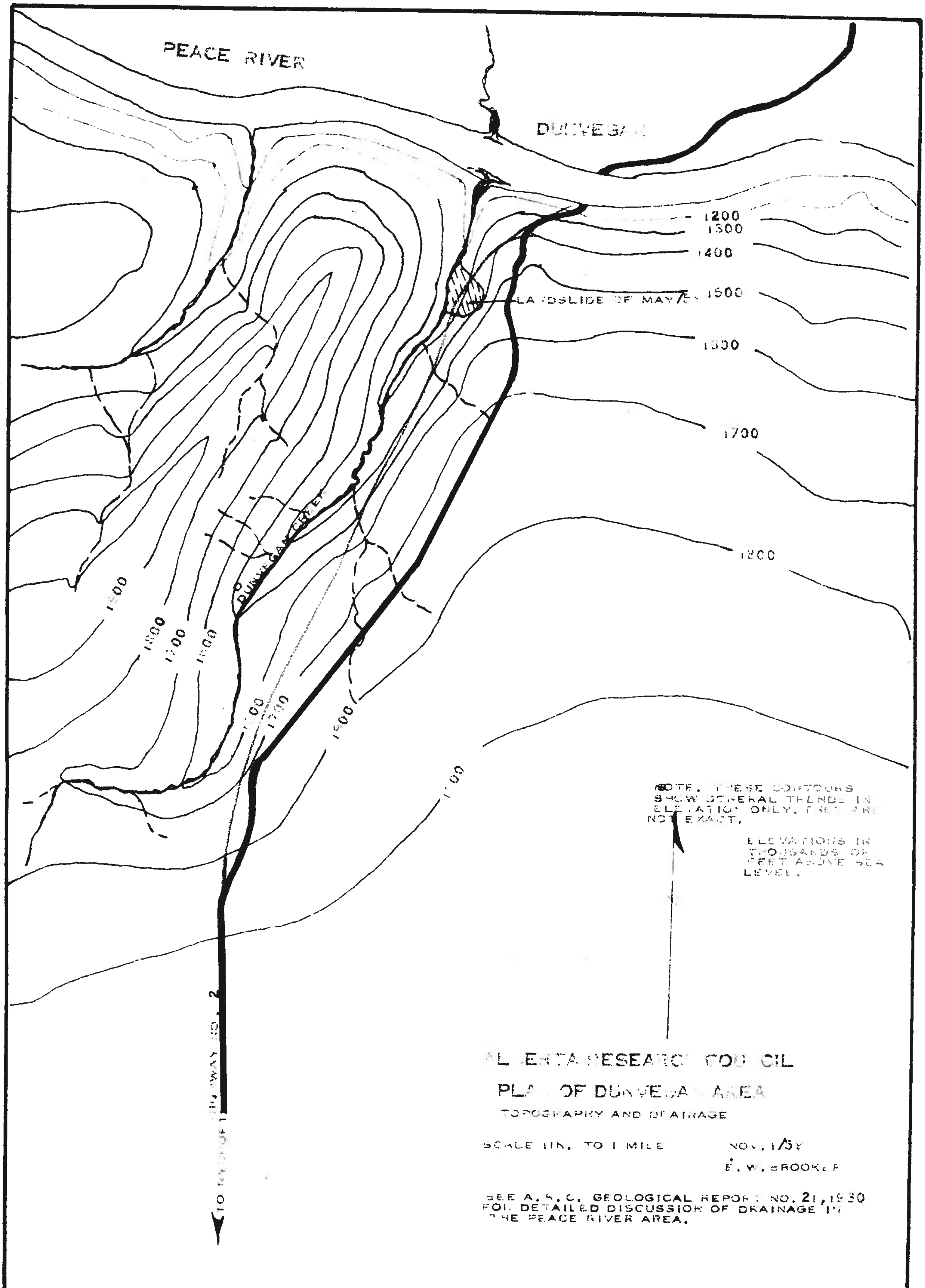


Fig. 9
Sliced samples from
Test Hole 70 depth 180 ft.
Note dip of laminations and
slickensides even at this depth.

PLATES



PEACE RIVER
1200
1300
1400
1500
1700
1800
TOPOGRAPHY AND DRAINAGE
SCALE 1IN. TO 1 MILE
SEE A. R. C. GEOLOGICAL REPORT NO. 21, 1930
FOR DETAILED DISCUSSION OF DRAINAGE IN
THE PEACE RIVER AREA.

Fig. 10

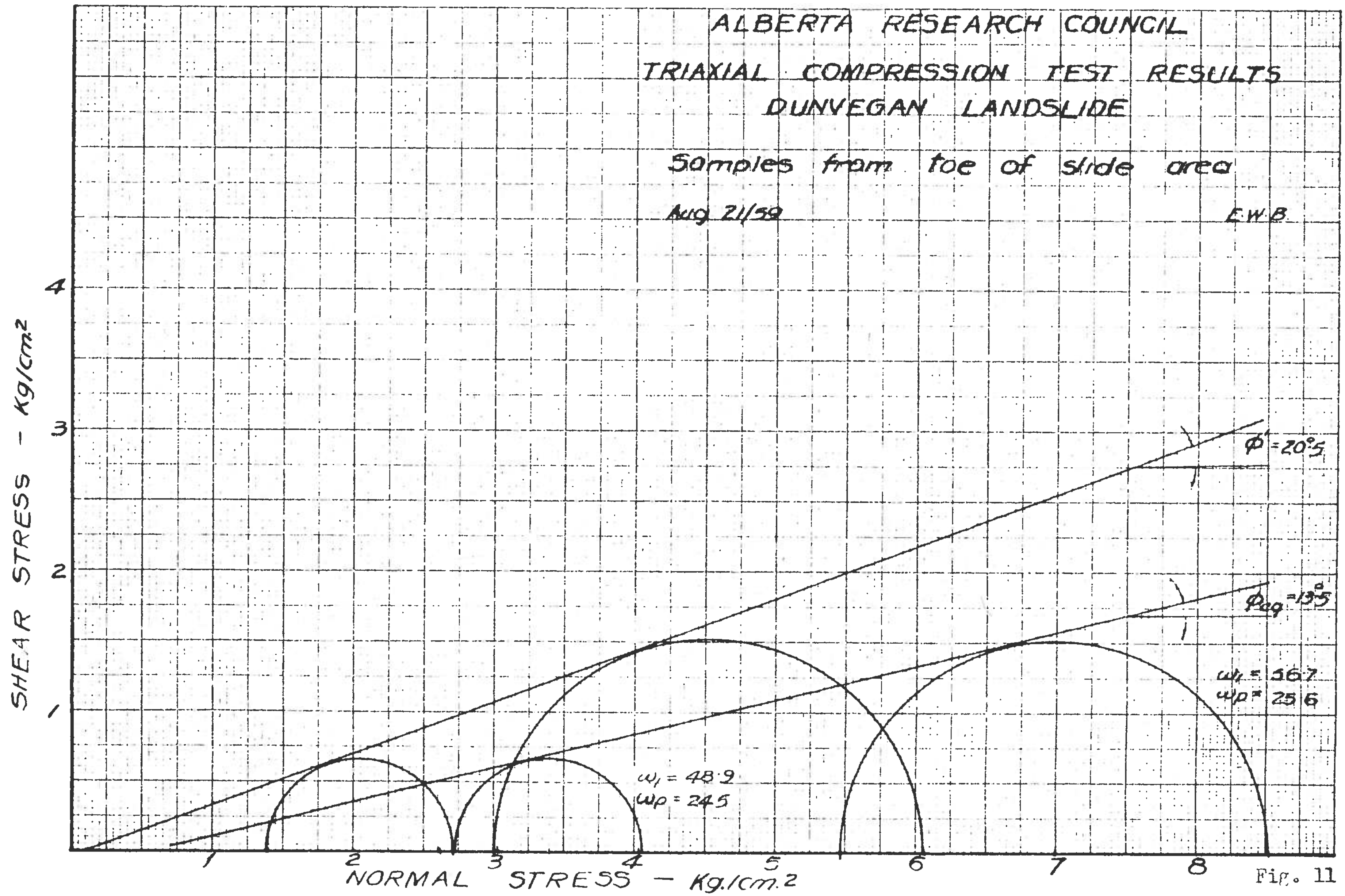
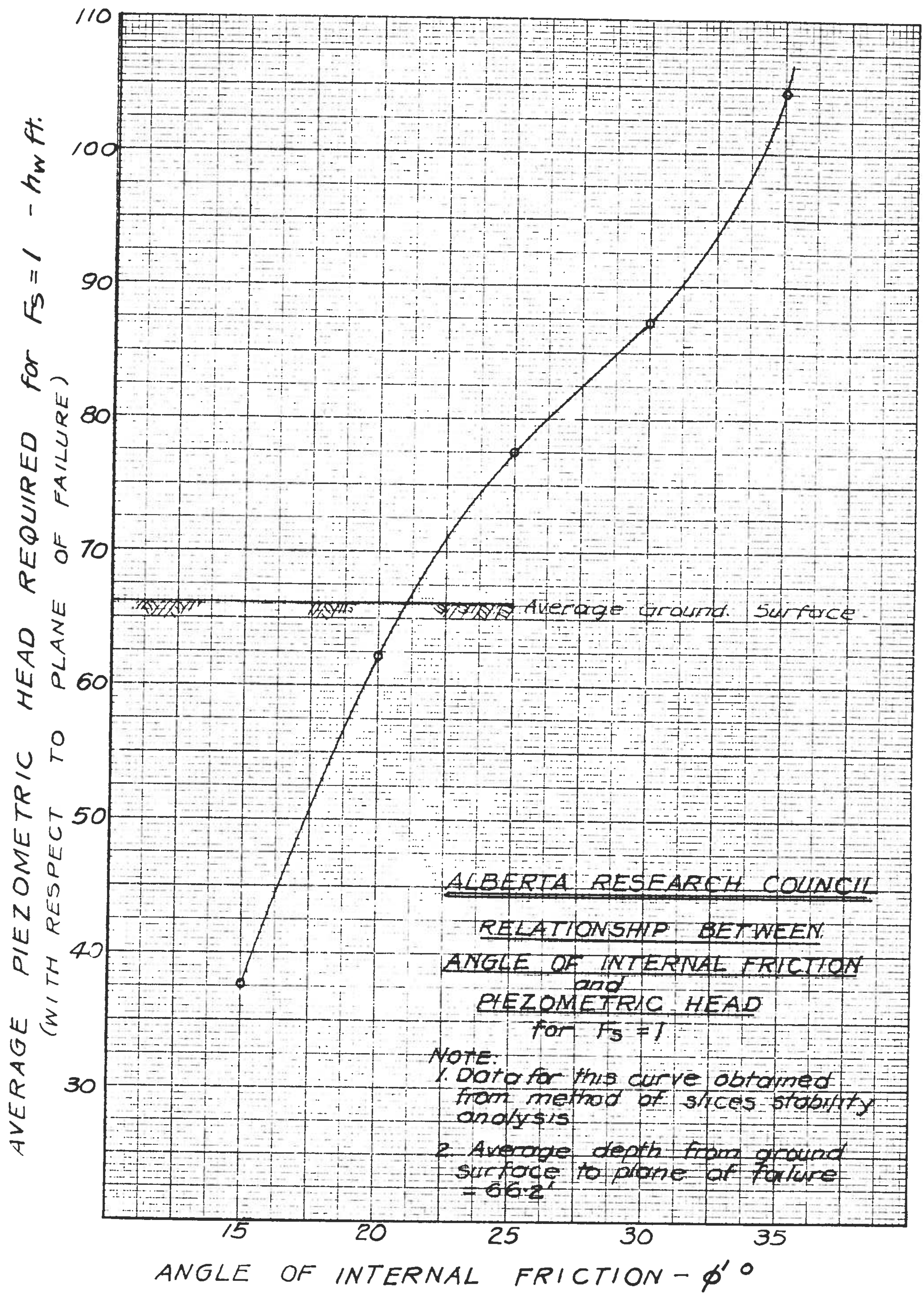

ALBERTA RESEARCH COUNCIL
TRIAXIAL COMPRESSION TEST RESULTS
DUNVEGAN LANDSLIDE
Samples from toe of slide area
Aug 21/59
E.W.B.
SHEAR STRESS - Kg/cm.2
NORMAL STRESS - Kg./cm.2
$\phi' = 20.5°$
$\phi_{cq} = 13.5°$
$\omega_l = 56.7$
$\omega_p = 25.6$
$\omega_l = 48.9$
$\omega_p = 24.5$
1
2
3
4
5
6
7
8

Fig. 11



AVERAGE PIEZOMETRIC HEAD REQUIRED for Fs=1 - hw ft.
(WITH RESPECT TO PLANE OF FAILURE)
110
100
90
80
70
60
50
40
30
15
20
25
30
35
ANGLE OF INTERNAL FRICTION - φ'°
Average ground Surface
ALBERTA RESEARCH COUNCIL
RELATIONSHIP BETWEEN
ANGLE OF INTERNAL FRICTION
and
PIEZOMETRIC HEAD
for Fs = 1
NOTE:
1. Data for this curve obtained from method of slices stability analysis
2. Average depth from ground surface to plane of failure = 66.2'

Fig. 12

R.C.A. 5

SUMMARY OF SAMPLING & LABORATORY TESTS

RESEARCH COUNCIL OF ALBERTA
EDMONTON — ALBERTA

PROJECT: DUNVEGAN LANDSLIDE -1959

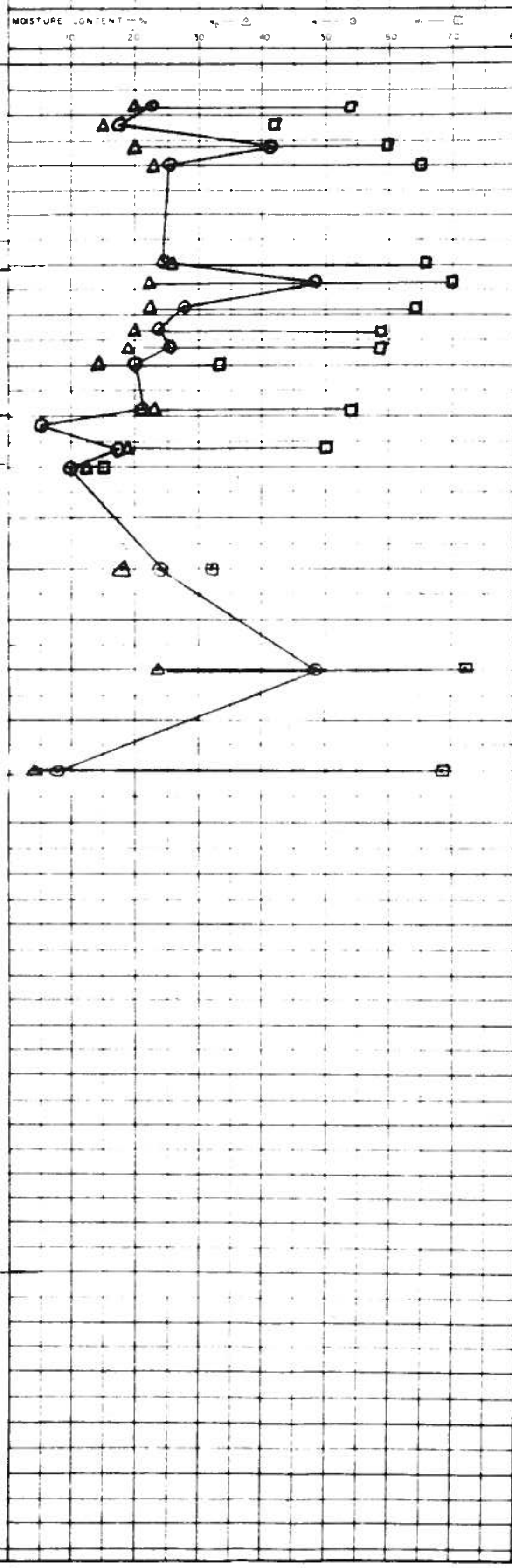
DWN.	CK'D	JOB NO	DATE	HOLE	PLATE
E.W.B			NOV 10/59	1	1

SOIL PROFILE

ELEV. DEPTH FEET	SOIL DESCRIPTION (DATUM)
0–17	FILL -clay
17–20	SAND -WET
20–35	CLAY
35–39	FINE WET SAND
39–70	CLAY - blue med to high plast
70	END HOLE
120	END HOLE

Depth scale (feet): 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80, 85, 90, 95, 100, 105, 110, 115, 120

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL SAMPLES

OTHER TESTS	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY % DEPTH FEET

PLATE 1

℄ PROPOSED HIGHWAY

GROUND DISTURBANCE DATA

LINE 1211

DIST. WEST OF C.L.	CRACK WIDTH	CRACK DEPTH
0	1	15
ESCARP		26
200	1	6
224	1	8
235	1	10
245	1-5	12
265	4	14
275	6	20
405	4	13
420	24	24
440	5	10
800	0-5	5
1155	15	8
1250	0-5	4-5

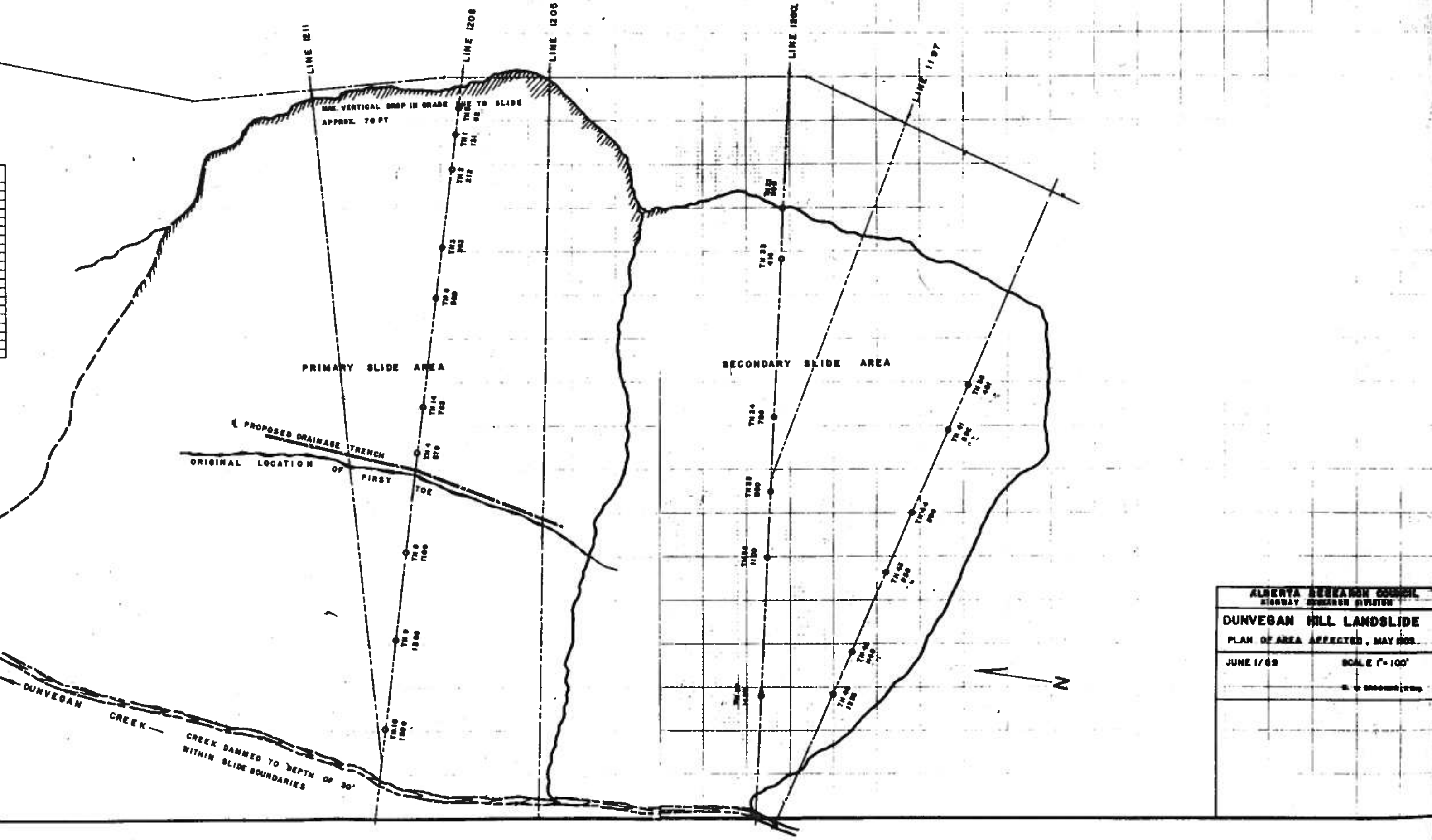
LINE 1205

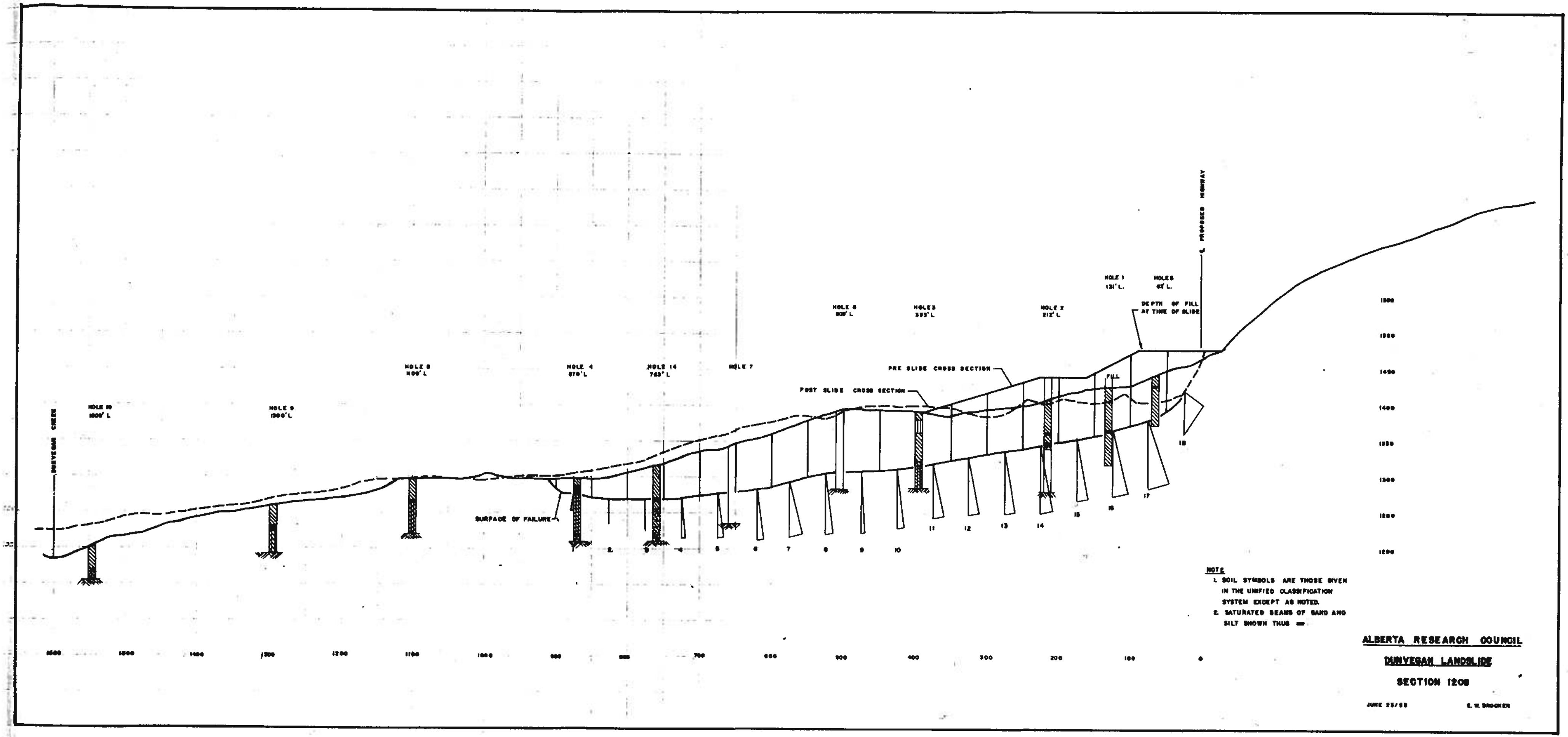
ESCARP	150	100
120	3	15
200	40	27
325	0-5	8
300	2	10
400	5	25
410	1	17
415	2	15
420	5	17
440	5	13
475	4	12
495	5	16
1050	7	7
1080	1-5	6
1130	0-5	3-5
1465	1	26
1480	1	4
1630	1	4

LINE 1208

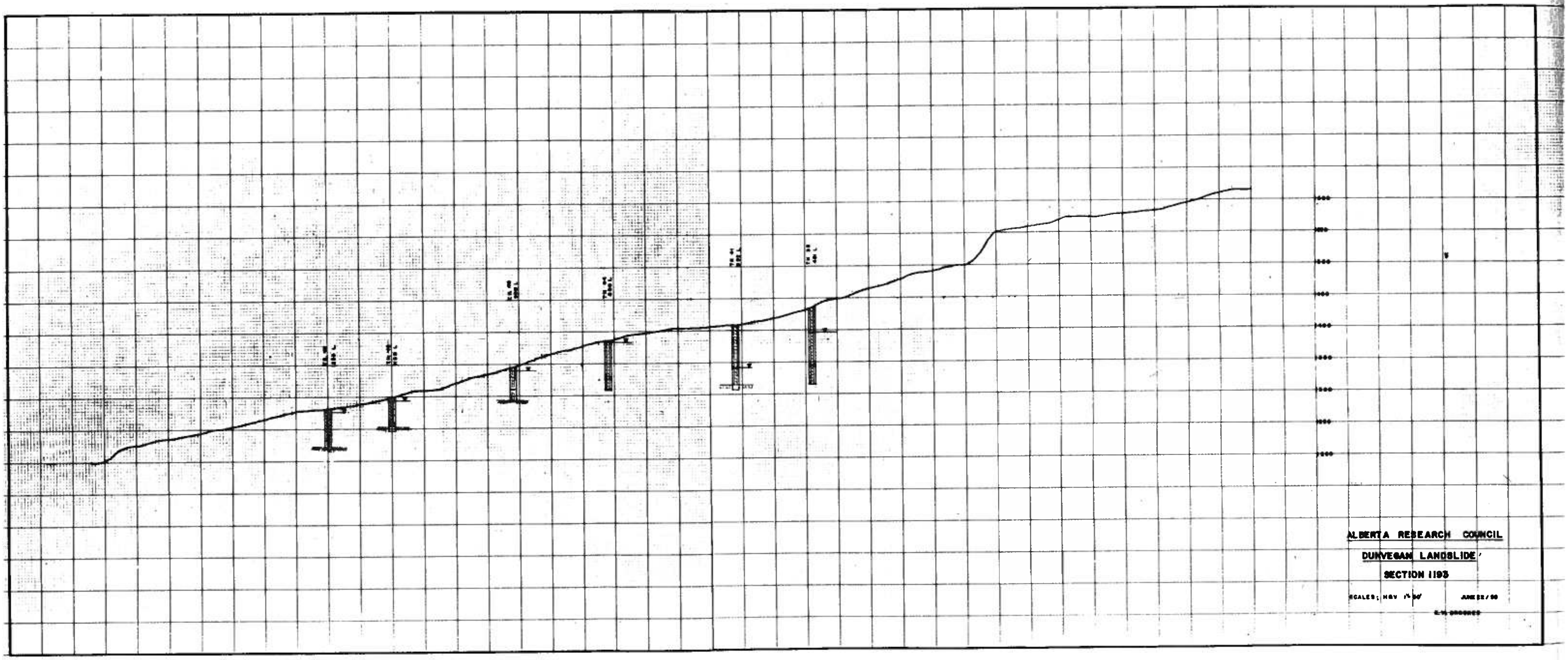
DIST. WEST OF C.L.	CRACK WIDTH	CRACK DEPTH
0	20	30
200	0-5	5
210	0-5	6
335	5	14
355	6	14
400	1	8
470	2	9
490	1	7
500	3	10
520	1	6
590	2	4-5
620	1	4
680	3	8
750	3	7
770	0-5	3
1210	3	5
1220	1	4-5
1230	1	5
1560	0-5	8
1650	20	6

NOTE
1. CRACK DEPTHS MEASURED BY MEANS OF A WEIGHTED CLOTH TAPE.
2. ORIGINAL CRACK DEPTH NOT OBTAINED DUE TO SLOUGHING OF WALLS.





PRE SLIDE CROSS SECTION
POST SLIDE CROSS SECTION
SURFACE OF FAILURE
DEPTH OF FILL AT TIME OF SLIDE
FILL
PROPOSED HIGHWAY
NOTE
1. SOIL SYMBOLS ARE THOSE GIVEN IN THE UNIFIED CLASSIFICATION SYSTEM EXCEPT AS NOTED.
2. SATURATED SEAMS OF SAND AND SILT SHOWN THUS
ALBERTA RESEARCH COUNCIL
DUNVEGAN LANDSLIDE
SECTION 1209


ALBERTA RESEARCH COUNCIL
DUNVEGAN LANDSLIDE
SECTION 1193

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

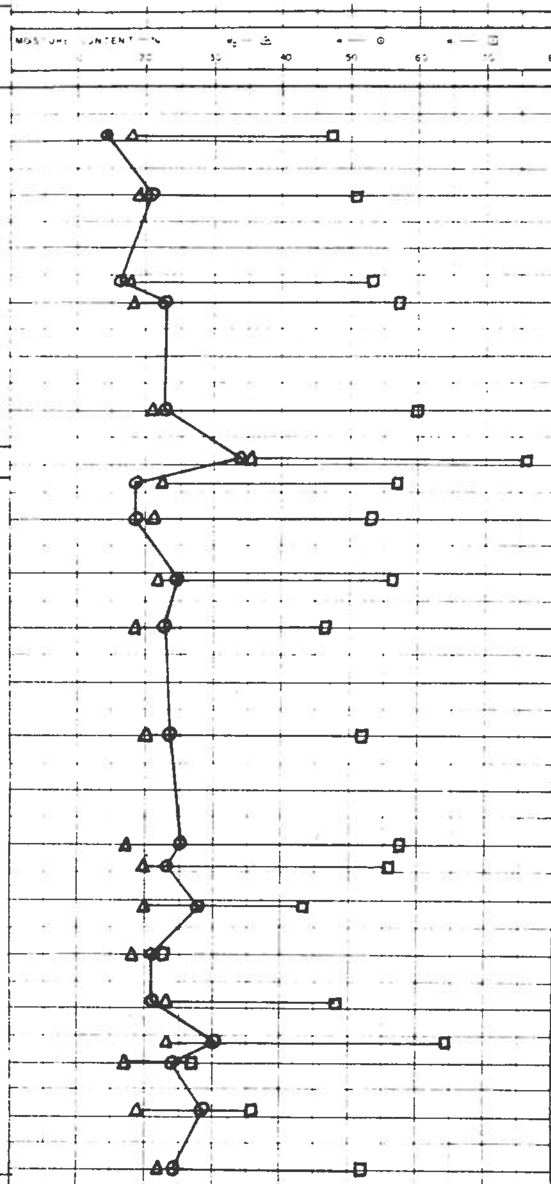
SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE – 1959

DWN. E. W. B | CHKD. | JOB NO. | DATE NOV 9, 1959 | HOLE 2 | PLATE 2

ELEV. DEPTH FEET	SOIL PROFILE — SOIL DESCRIPTION
	DATUM: DRILLED OCT 17/58
	GND ELEV. 1445
	FILL
5	Compacted clay
10	
15	
20	
25	
30	
35	ORGANIC SOIL
	CLAY
40	brown, firm, moist
45	
50	
55	
60	
65	
70	
75	silt pockets
80	
	saturated sandy silt
85	
90	
95	
	[illegible] clay
100	END HOLE

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL SAMPLES

OTHER TESTS	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY % DEPTH FEET

PLATE 2

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE -1959

DWN: E. W. B | CKD: | JOB NO: | DATE: NOV 10, 1959 | HOLE: 3 | PLATE: 3

ELEV DEPTH FEET	SOIL PROFILE — SOIL DESCRIPTION (DATUM: STA [illegible] +34)	MOISTURE CONDITIONS & ATTERBERG LIMITS	OTHER TESTS	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY % DEPTH FEET
	CLAY brown						
5	dry [illegible] plastic						
10							
15							
20	Saturated silty fine sand						
25							
30							
35							
40	CLAY						
45							
50							
55							
60							
65							
70							
75	saturated silty fine sand						
80							
85							
90							
95							
100	Sand						
105	SANDSTONE						
110	END HOLE						

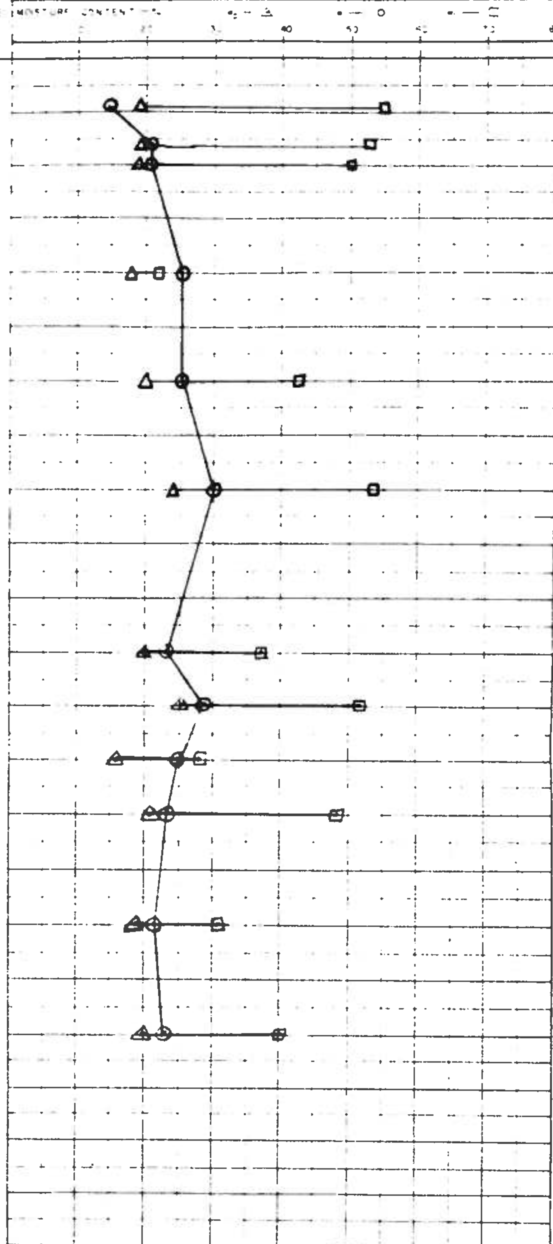


PLATE 3

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE - 1959

DWN: E. W. B. | CK'D: | JOB No.: | DATE: NOV 10/59 | HOLE: 4 | PLATE: 4

SOIL PROFILE		MOISTURE CONDITIONS & ATTERBERG LIMITS	SOIL SAMPLES				
ELEV / DEPTH FEET	SOIL DESCRIPTION — DATUM: LOCATION - 870' L ℄, DRILLED OCT 27/59	MOISTURE CONTENT — %, w_p — △, w — ⊙, w_L — ☐ (scale 20 – 80)	OTHER TESTS	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY % / DEPTH FEET
5	CLAY light brown med plasticity						
10							
15	silt and sand lens						
20							
25							
30							
35	pockets of sand						
40							
45							
50							
55							
60	saturated sand and silt						
65							
70							
75							
80							
85							
90	END HOLE						

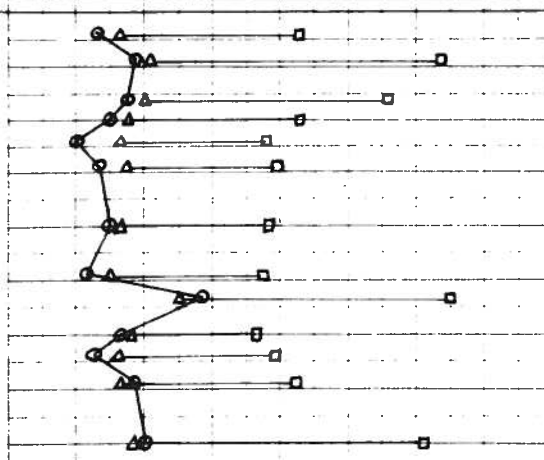


PLATE 4

SUMMARY OF SAMPLING & LABORATORY TESTS

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

PROJECT: DUNVEGAN LANDSLIDE - 1959

DWN	E. W. B	DATE	NOV 10/59	HOLE	5	PLATE	5

ELEV. DEPTH FEET	SOIL PROFILE — SOIL DESCRIPTION	MOISTURE CONDITIONS & ATTERBERG LIMITS	OTHER TESTS	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY % DEPTH-FEET
	DATUM 62' L ℄ DRILLED OCT 22/58	MOISTURE CONTENT % — w_P Δ, w ○, w_L □ — 10 20 30 40 50 60 70 80					
	CLAY						
5							
10							
15	wet sand						
20							
25							
30							
35							
	silt						
40							
45							
50	sand and silt lens						
55							
60							
65							
70	— END HOLE —						

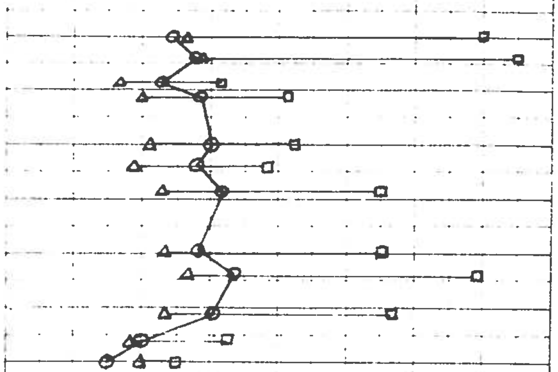


PLATE 5

R.C.A. 3

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE - 1959

DWN	CKD	JOB NO	DATE	HOLE	PLATE
E.W.B.			NOV 10/59	O	6

SOIL PROFILE — SOIL DESCRIPTION

ELEV / DEPTH FEET

DATUM: LOCATION 500'L ℄ DRILLED MAY 13/58

SOIL SYMBOL

MOISTURE CONDITIONS & ATTERBERG LIMITS — MOISTURE CONTENT – % — w_p – △ — ○ — □ — 0 10 20 30 40 50 60 70 80

SOIL SAMPLES — OTHER TESTS — CONDITION — TYPE — PENETRATION RESISTANCE — RECOVERY % DEPTH FEET

DEPTH (FEET)	SOIL DESCRIPTION
	FILL
5	
10	
15	ORGANIC
20	CLAY
25	
30	
35	
40	
45	
50	
55	
60	
65	silt lens
70	
75	
80	
85	
90	silt lens, sand lens
95	
100	
105	
110	SANDSTONE
115	
120	END HOLE

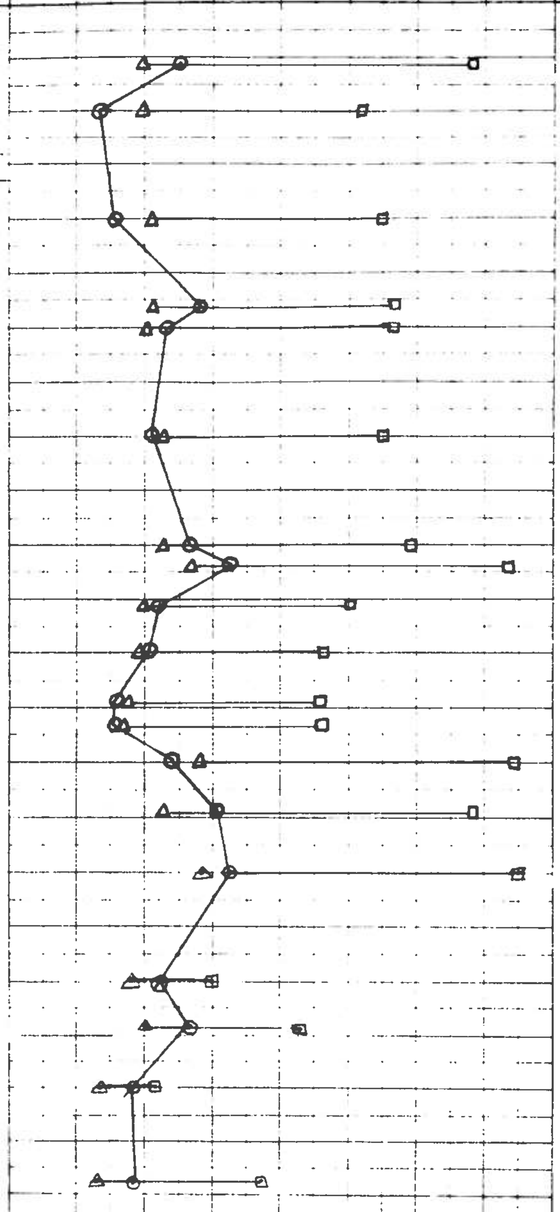


PLATE 6

RCA-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE -1959

DWN. E.W.B | DATE NOV. 10/59 | HOLE 7 | PLATE 7

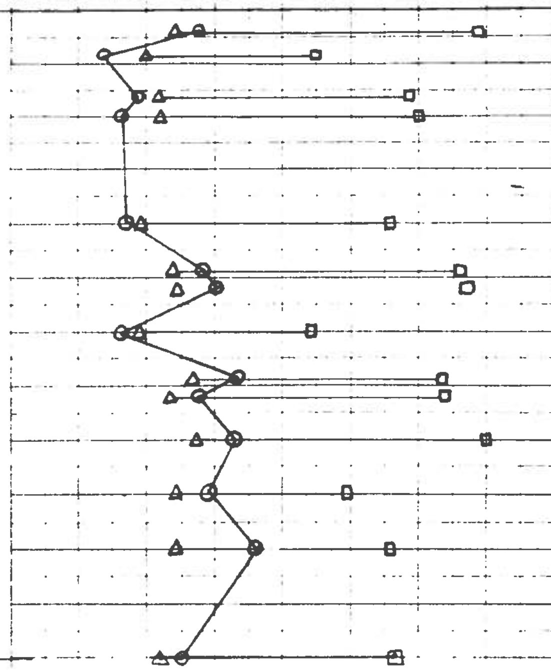
SOIL PROFILE — SOIL DESCRIPTION

DATUM: LOCATION — 650 L ℄

ELEV. DEPTH FEET	SOIL DESCRIPTION
	CLAY
5	
10	
15	
20	
25	
30	sand lers
35	
40	
45	
50	
55	
	SANDSTONE
60	END OF HOLE

MOISTURE CONDITIONS & ATTERBERG LIMITS

MOISTURE CONTENT % — W_P — △ — W — ○ — W_L — ☐



SOIL SAMPLES

OTHER TESTS | CONDITION | TYPE | PENETRATION RESISTANCE | RECOVERY % DEPTH-FEET

PLATE 7

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN HILL - 1953

DWN: E.W.B | JOB NO: 2-S-1 | DATE: NOV 3/53 | HOLE: 8 | PLATE: 8

SOIL PROFILE — SOIL DESCRIPTION

LOCATION - 1400' L ℄

DRILLED - OCT 23/53

MOISTURE CONDITIONS & ATTERBERG LIMITS

SOIL SAMPLES — OTHER TESTS

DEPTH FEET	SOIL DESCRIPTION
	CLAY
5	
10	
15	
20	
25	SAND LENS light brown
30	
35	
40	CLAY
45	SILT
50	
55	
60	SILTY SAND - very wet
65	
70	SANDY ZONE
75	END OF HOLE

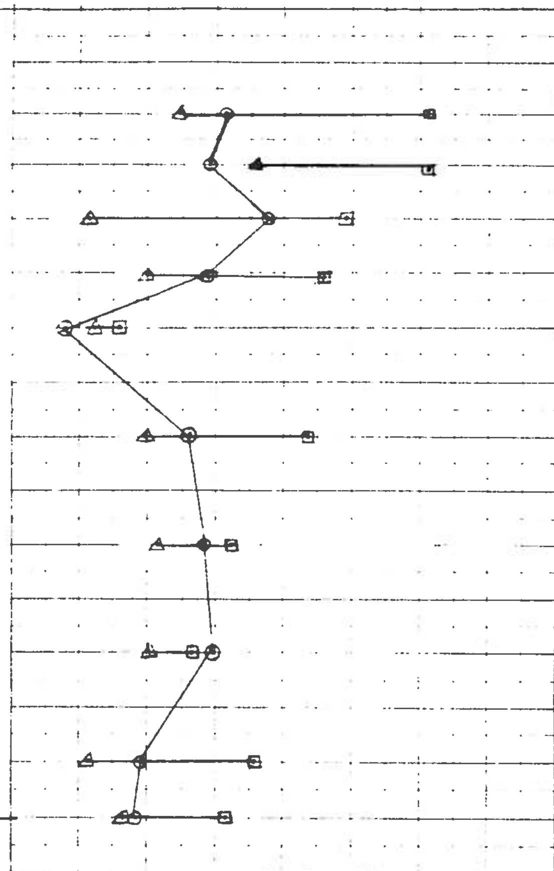


PLATE 8

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE - 1959

DWN	CH'D	JOB NO	DATE	HOLE	PLATE
E. W. B.			NOV 9, 1959	9	9

SOIL PROFILE — SOIL DESCRIPTION

ELEV. / DEPTH — FEET

DATUM: LOCATION ...

DRILLED MAY 29, 1959

MOISTURE CONDITIONS & ATTERBERG LIMITS

MOISTURE CONTENT — %

SOIL SAMPLES — OTHER TESTS — CONDITION — TYPE — PENETRATION RESISTANCE — RECOVERY % / DEPTH-FEET

DEPTH (FEET)	SOIL DESCRIPTION
	CLAY — grey black
5	
10	
15	... silt ...
20	
25	
30	
35	silt ...
40	
45	
50	
55	
60	... pockets.
65	
70	
75	
80	
85	END HOLE

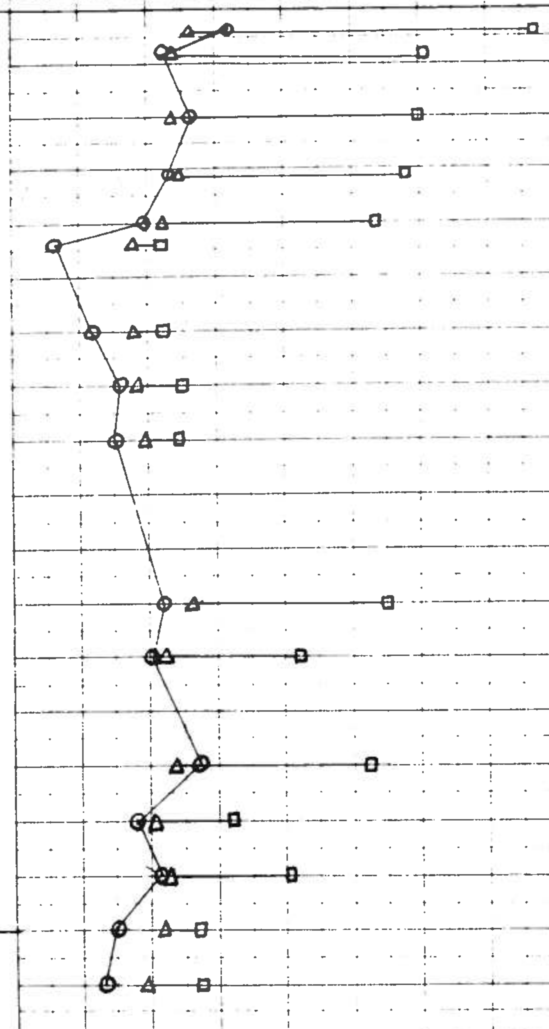


PLATE 9

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE

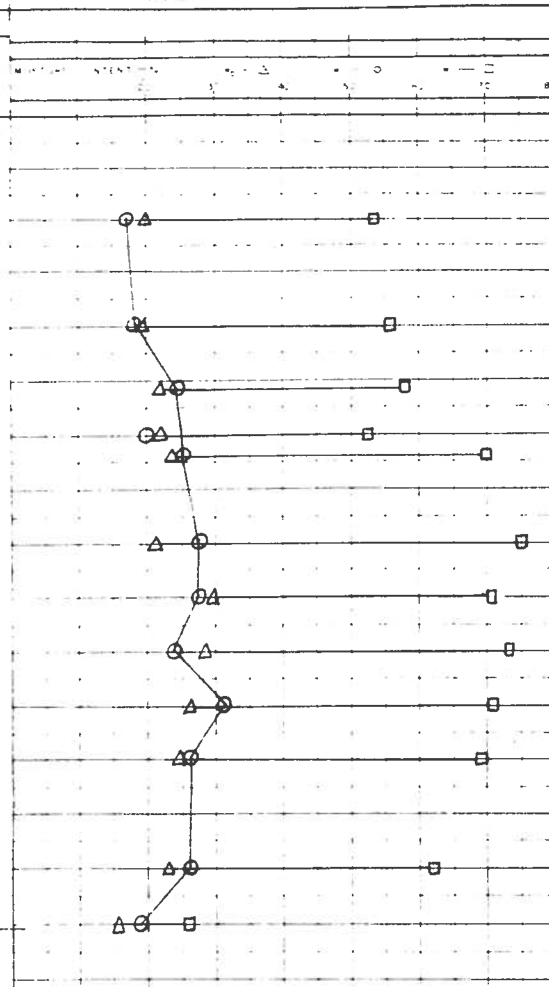
DWN: E.W.B | DATE: NOV 9/59 | HOLE: 10 | PLATE: 10

SOIL PROFILE — SOIL DESCRIPTION

LOCATION - STA 1208 100'E.
DRILLED MAY 22, 55

DEPTH	SOIL DESCRIPTION
	CLAY - brown
5	
10	
15	
20	
25	
30	
35	
40	gravel
45	
50	SILT
55	
60	
65	
70	SILT
75	END HOLE SANDSTONE

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL SAMPLES

OTHER TESTS	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY % DEPTH FEET

PLATE 10

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT DUNVEGAN LANDSLIDE - 1959

DWN E.W.B.   CK   JOB NO.   DATE NOV 9, 1959   HOLE 14   PLATE 11

ELEV. DEPTH FEET	SOIL PROFILE — SOIL DESCRIPTION	MOISTURE CONDITIONS & ATTERBERG LIMITS	SOIL SAMPLES — OTHER TESTS
	DATUM LOCATION - 763' L. E. DRILLED OCT 3, 1959	MOISTURE CONTENT % 20 30 40 50 60 70 80	
	CLAY - brown		
5			
10	silty sand		
15			
20			
25	silt lens		
30	sand pockets		
35			
40			
45			
50			
55			
60			
65	gravel		
70			
75			
80			
85			
90			
95	gravel		
100	SANDSTONE		
105	END HOLE		

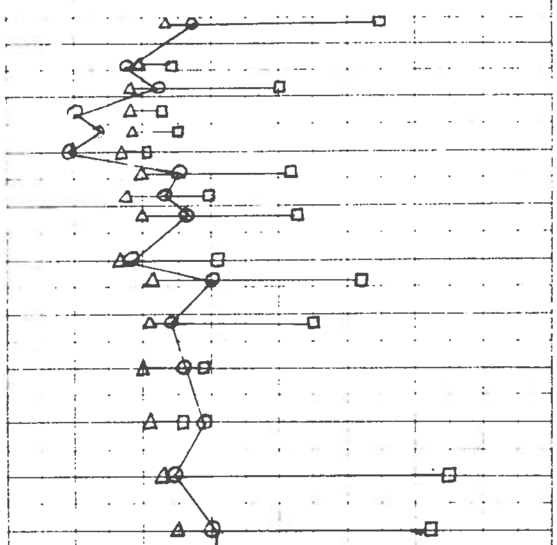


PLATE 11

R.C.A.-57

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE – 1959

DWN	CK'D	JOB N.	DATE	HOLE	PLATE
E. W. B			NOV 9/59	3 [illegible]	12

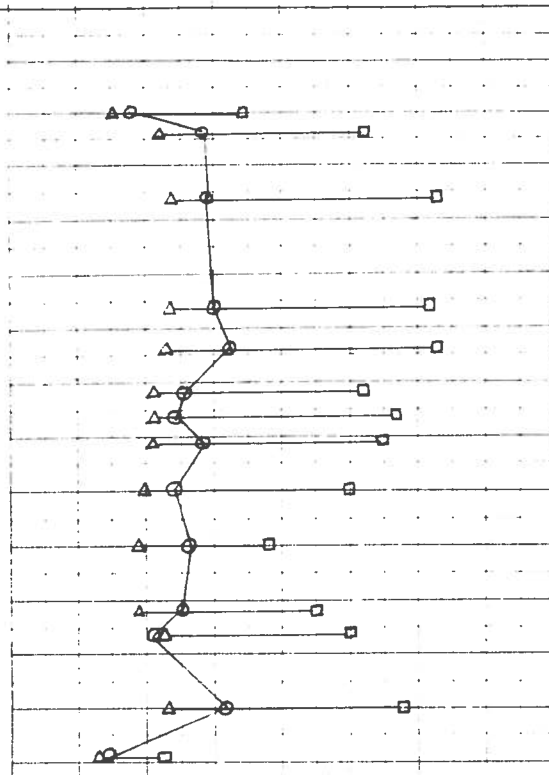
SOIL PROFILE — SOIL DESCRIPTION

DATUM: LOCATION – 300' L ℄
DRILLED NOV. [illegible]/58

ELEV. DEPTH FEET

Depth (ft)	Soil description
	CLAY – brown
5	
10	
15	
20	
25	
30	
35	
40	Silt lens
45	
50	
55	
60	
65	[illegible]
70	
75	
80	[illegible] SILT – [illegible] [illegible] grey
85	
90	
95	
100	
105	
110	
115	Silty, [illegible]
120	
125	
130	END HOLE

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL SAMPLES — OTHER TESTS

PLATE 12

R.C.A.-3

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE -1959

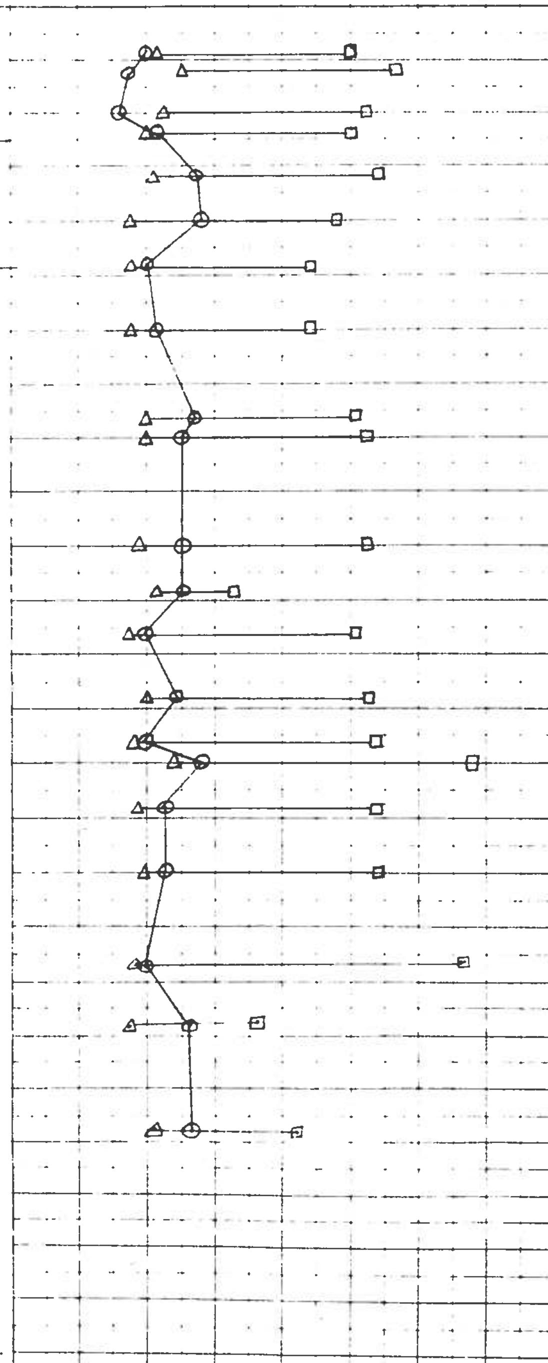
DWN				PLATE
E.W.B		NOV 9 1959	33	13

SOIL PROFILE — SOIL DESCRIPTION

LOCATION 4 6 - E
DRILLED NOV [illegible]/59

DEPTH	SOIL DESCRIPTION
	CLAY - grey brown
5	
	sand [illegible] seams
10	
	CLAY
15	silty
20	[illegible]
25	CLAY
30	
	silt laminations
35	
40	
45	
50	
55	silt [illegible]
60	
65	
70	
	[illegible] lam [illegible]
75	
80	
85	
90	
95	
100	
105	END TEST DATA
110	
115	
	very fine grey [illegible] sand
120	
125	END HOLE

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL SAMPLES: OTHER TESTS | [illegible] | TYPE | PENETRATION RESISTANCE | RECOVERY % DEPTH FEET

PLATE 13

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT [illegible] LANDSLIDE – 1959

DWN E. W. B. | JOB No. | DATE [illegible] 9, 1959 | HOLE 34 | PLATE 14

ELEV. / DEPTH FEET	SOIL PROFILE — SOIL DESCRIPTION	MOISTURE CONDITIONS & ATTERBERG LIMITS	SOIL SAMPLES — OTHER TESTS
	DATUM LOCATION [illegible] 735 DRILLED [illegible]	MOISTURE CONTENT %	
	CLAY – light brown		
5			
10	silt laminations		
15	sand pockets		
20			
25			
30			
35			
40			
45			
50			
55			
60			
65	silt laminations		
70			
75	END TEST DATA		
80			
85			
90			
95			
100			
105	HARD SAND		
110	END HOLE		

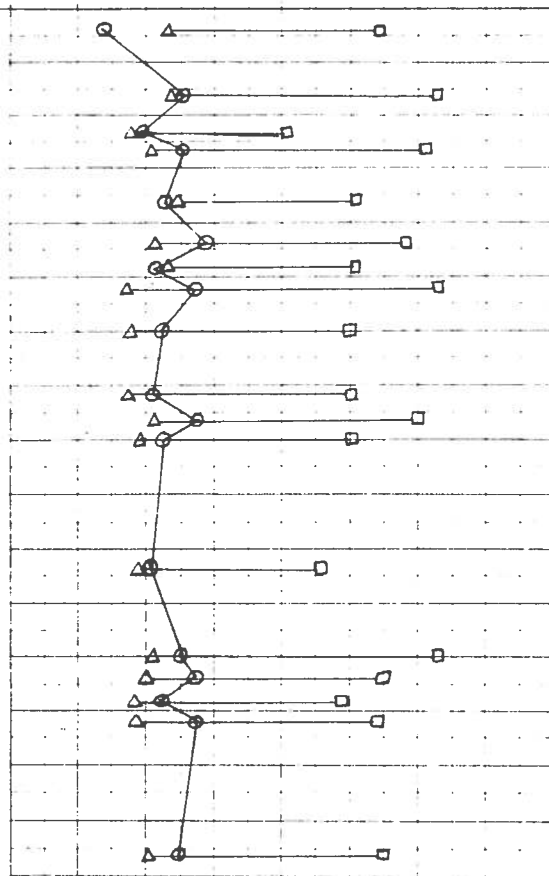


PLATE 14

RCA-3

SUMMARY OF SAMPLING & LABORATORY TESTS

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

PROJECT: DUNVEGAN LANDSLIDE 1959

DWN	CKD	JOB NO	DATE	HOLE	PLATE
E. W. B			NOV 9, 59	35	15

SOIL PROFILE — SOIL DESCRIPTION — MOISTURE CONDITIONS & ATTERBERG LIMITS — SOIL SAMPLES — OTHER TESTS

DATUM: LOCATION — 950' L℄

ELEV / DEPTH FEET	SOIL DESCRIPTION
5	CLAY light bwn low to med plast. dark bwn silt pockets
10	
15	SILT low to med plast
20	CLAY — grey bwn low to med plast. silt and sand lens
25	
30	
35	
40	[illegible] silt pockets
45	
50	
55	SAND [illegible]
60	SANDSTONE
65	END HOLE

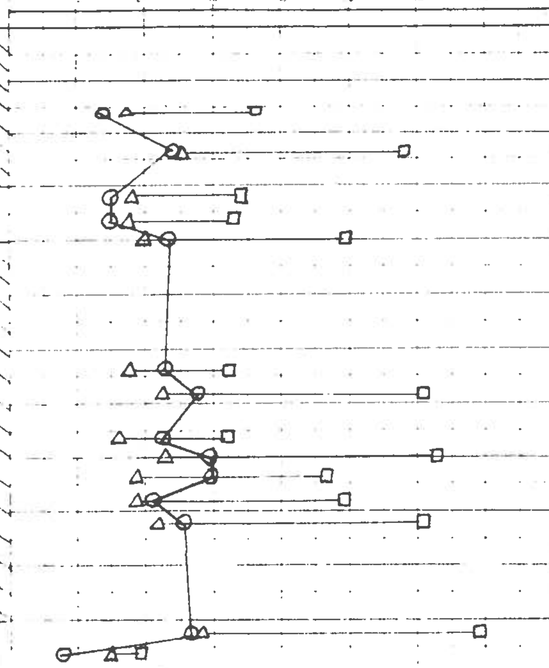


PLATE 15

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE, 1959

DWN: E.N.B. | DATE: NOV 9/59 | HOLE: 36 | PLATE: 16

SOIL PROFILE
SOIL DESCRIPTION
LOCATION — 110' L ℄
DRILLED NOV 27/59

MOISTURE CONDITIONS & ATTERBERG LIMITS

SOIL SAMPLES: OTHER TESTS | CONDITION | TYPE | PENETRATION RESISTANCE | RECOVERY %

Depth	Soil description
	CLAY, silty, brown
5	med. to high plast.
10	grey silt lens
15	sand pockets
20	SAND, dense, damp fine
25	
30	SANDSTONE, hard
	— END HOLE —

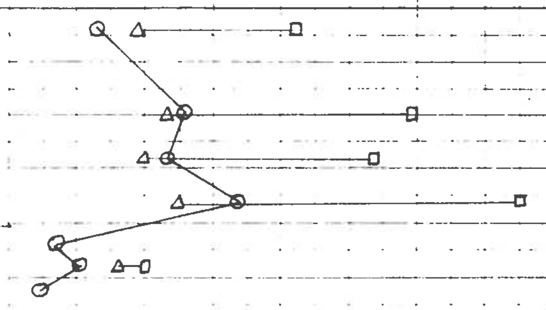


PLATE 16

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE 1959

DWN	E. W. B.	CKD		HOLE NO		DATE	NOV 9 / 1959		33	PLATE	17

ELEV. DEPTH FEET	SOIL PROFILE — SOIL DESCRIPTION	MOISTURE CONDITIONS & ATTERBERG LIMITS	SOIL SAMPLES — OTHER TESTS
	LOCATION — 481' L ₵ DRILLED NOV 28, 59		
5	CLAY - brown, grey med to high plast yellow sand lens		
10			
15			
20	dry silt layer.		
25	CLAY, grey, silty med to high plast.		
30			
35	Sand lens		
40			
45			
50	brown silt pocket		
55			
60	dry sand pocket		
65	CLAY - silty grey		
70	laminated sand and silt		
75			
80			
85			
90			
95			
100			
105			
110			
115	dense hard clay		
120	END HOLE		

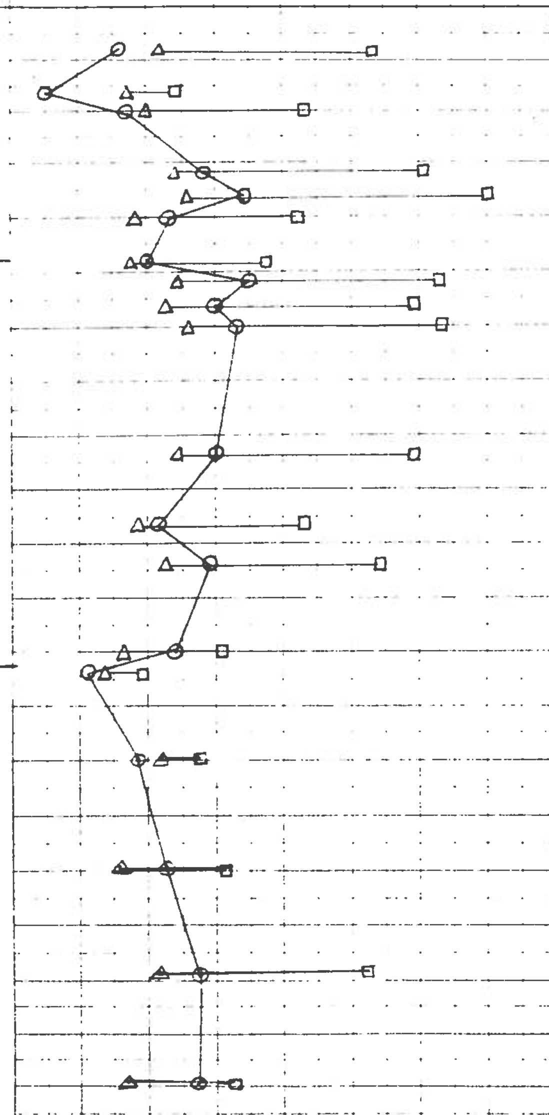


PLATE 17

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE 1959

DWN: E.W.B | DATE: NOV 9/59 | PLATE: 18

SOIL PROFILE — SOIL DESCRIPTION

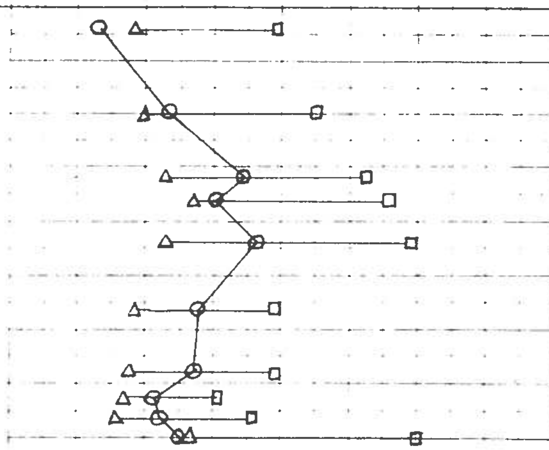
[illegible] 93+35

[illegible] 958

MOISTURE CONDITIONS & ATTERBERG LIMITS

SOIL SAMPLES — OTHER TESTS

Depth	Soil description
5	CLAY - yellow silty [illegible]
10	
15	[illegible] silt [illegible]
20	
25	
30	
35	[illegible]
40	
45	
50	
55	silt [illegible] [illegible]
60	
65	
70	
75	
80	
85	
90	
95	SANDSTONE
100	END HOLE



END TEST DATA

PLATE 18

R.C.A.

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE 1959

E.W.B. — NOV. 3, 1959 — 42 — PLATE 19

SOIL PROFILE	MOISTURE CONDITIONS & ATTERBERG LIMITS	SOIL SAMPLES
SOIL DESCRIPTION		OTHER TESTS

Depth (feet)	Soil description
	CLAY Brown grey
5	[illegible]
10	
15	
20	[illegible]
25	
30	
35	
40	SATURATED SAND
45	SANDSTONE
50	
	END TEST HOLE @ 51'

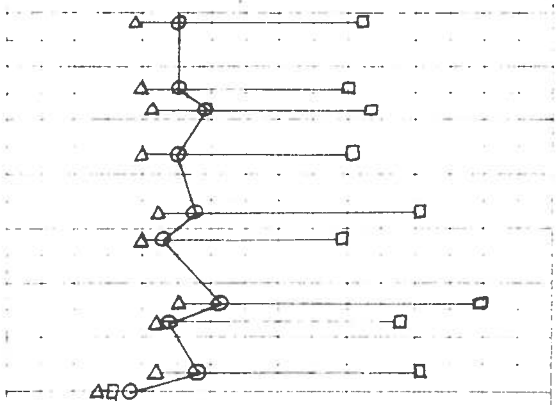


PLATE 19

RESEARCH COUNCIL OF ALBERTA EDMONTON — ALBERTA	SUMMARY OF SAMPLING & LABORATORY TESTS
	PROJECT DUNVEGAN LANDSLIDE -1959

E.W.B. | 43 | PLATE 20

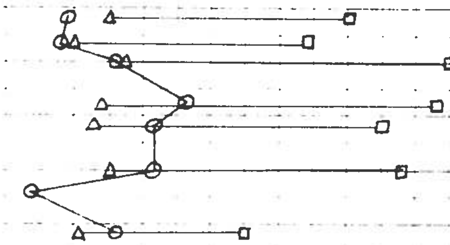
SOIL PROFILE SOIL DESCRIPTION	MOISTURE CONDITIONS & ATTERBERG LIMITS	SOIL SAMPLES OTHER TESTS
CLAY [illegible] 5 [illegible] [illegible]		
[illegible]		
35 [illegible]		
50 SANDSTONE AND [illegible] 54' [illegible]		

PLATE 20

R.C.A. 5

SUMMARY OF SAMPLING & LABORATORY TESTS

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

PROJECT ... LANDSLIDE - 1959

HOLE E.W. 8 | DATE NOV 9 1959 | PLATE 21

SOIL PROFILE — SOIL DESCRIPTION | MOISTURE CONDITIONS & ATTERBERG LIMITS | SOIL SAMPLES — OTHER TESTS

LOCATION - 800' L E
DRILLED DEC 2, 58

Depth	Soil description
	CLAY brownish grey
5	
	med. to high plast.
10	
15	yellow ... lens
20	
25	
30	
35	silty ... streaks
40	
45	
50	
55	10 ... sand silty
60	
65	... clay
70	
75	END HOLE 75'

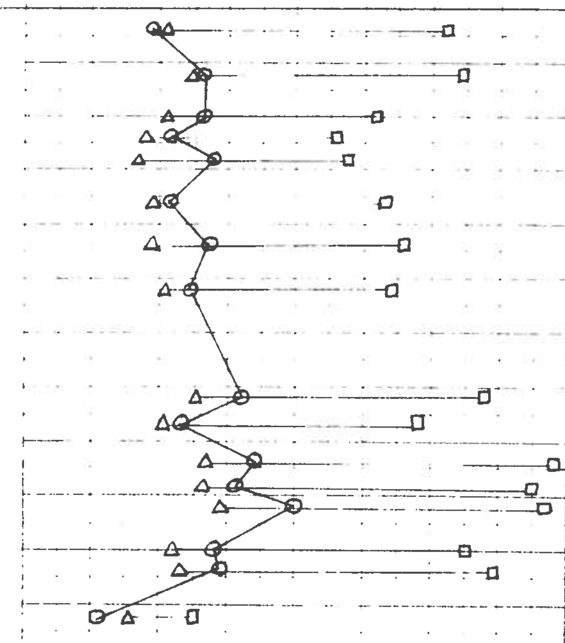


PLATE 21

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: DUNVEGAN LANDSLIDE - MAY 1959

DATE: NOVEMBER 3/1959 | HOLE: 65 | PLATE: 22

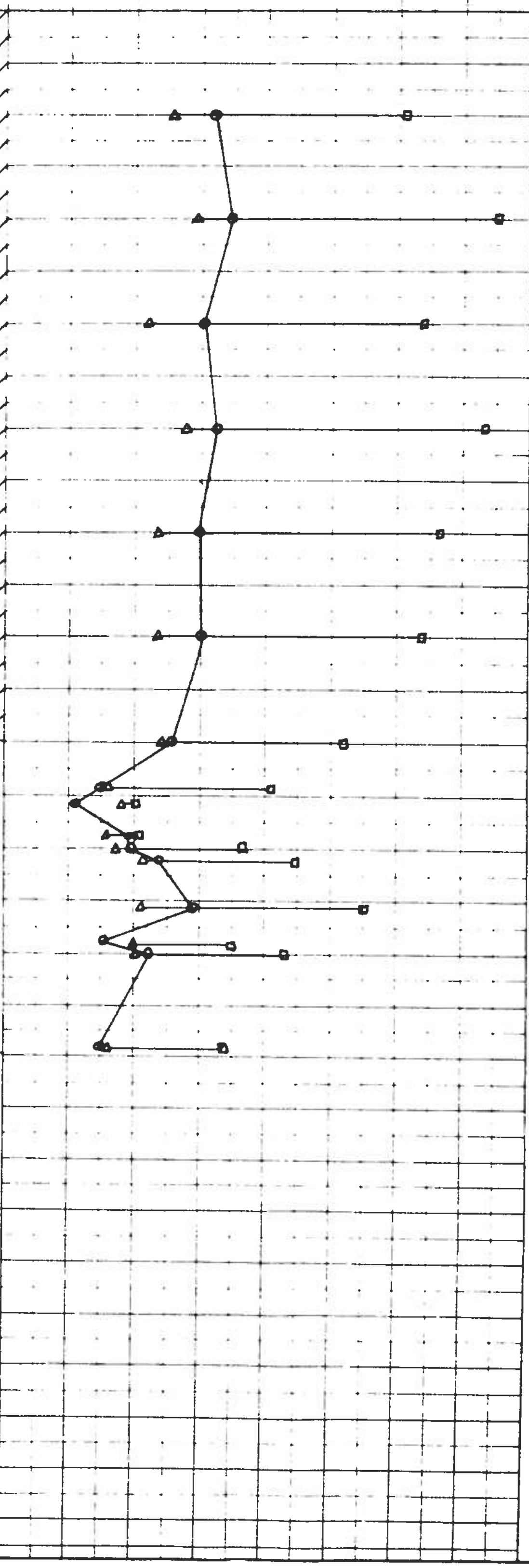
SOIL PROFILE

SOIL DESCRIPTION

ELEVATION OF GROUND AT TOP OF HOLE 1600 FEET ABOVE SEA LEVEL. LOCATION 355'R. ORIG. AND 1209 + 50

DEPTH	SOIL DESCRIPTION
0	'A' HORIZON
10	CLAY - INORGANIC, HIGH PLASTIC, STIFF.
70	MEDIUM PLASTIC
72	SILT - INORGANIC, SANDY, LOW PLASTIC
88	CLAYEY, MOIST
90	SAND LENS.
100	END TEST DATA.
110	FIRM GREY SHALE AND SANDSTONES
120	TO END
130	OF HOLE.
160	END OF HOLE

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL SAMPLES

OTHER TESTS | CONDITION | TYPE | PENETRATION RESISTANCE | RECOVERY % DEPTH FEET

PLATE 22

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: **DUNVEGAN LANDSLIDE - MAY 1959**

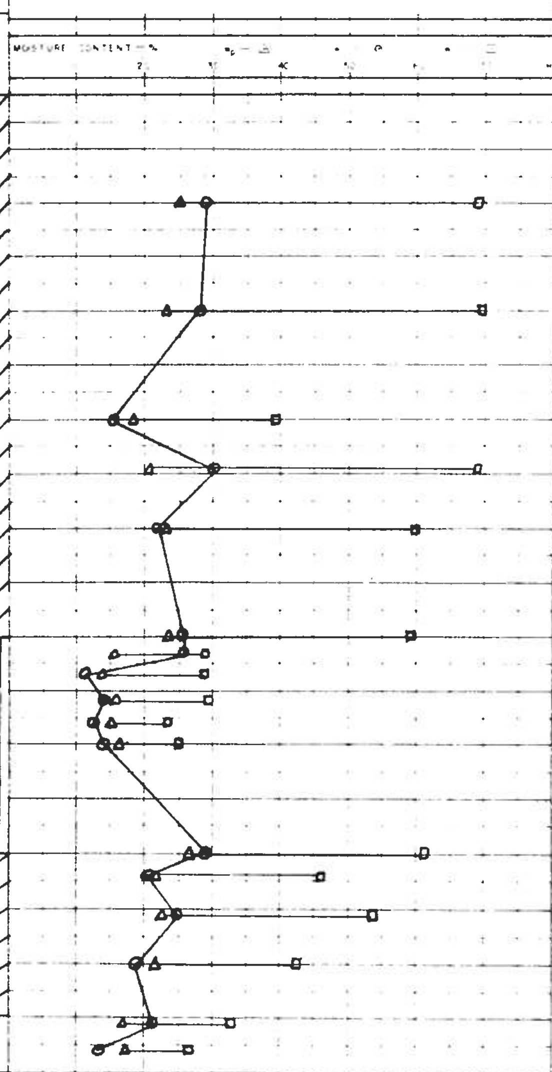
DATE: NOVEMBER 3/1959 — HOLE: 67 — PLATE: 23

SOIL PROFILE — SOIL DESCRIPTION

ELEVATION OF GROUND AT TOP OF HOLE 1587 FEET ABOVE SEA LEVEL. LOCATION 1212 + 50

DEPTH FEET	SOIL DESCRIPTION
0	
5	CLAY — INORGANIC, HIGH PLASTICITY, FIRM CONSISTENCY
30	SILTY - MEDIUM PLASTICITY
50	SILT - INORGANIC, SANDY - DRY, LOW PLASTIC, LIGHT BROWN, SANDY.
70	CLAY - LOW TO MEDIUM PLASTIC FIRM - SILT AND FINE SAND POCKETS
85	SILT - LOW-MEDIUM PLASTIC, SANDY
90	END OF TEST DATA

MOISTURE CONDITIONS & ATTERBERG LIMITS



SOIL SAMPLES — OTHER TESTS

PLATE 23

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT

DUNVEGAN LANDSLIDE - MAY 1959

DWN		NOVEMBER 3/1959	HOLE 70	PLATE 24

ELEV. DEPTH FEET	SOIL PROFILE — SOIL DESCRIPTION	MOISTURE CONDITIONS & ATTERBERG LIMITS	SOIL SAMPLES — OTHER TESTS	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY % DEPTH FEET
0	DATUM ELEVATION OF GROUND TOP OF HOLE 1606 FEET ABOVE SEA LEVEL.						
	CLAY - INORGANIC HIGH PLASTIC						
5							
	BWN. - OK.						
10							
15							
20							
25							
30							
35							
40							
45							
50							
55	50 FEET						
60	SILT LENS.						
65							
70	SILT - INORGANIC - MED. -						
75	LOW PLASTIC.						
	DRY.						
80	SANDY						
	END TEST DATA.						
85							
90							

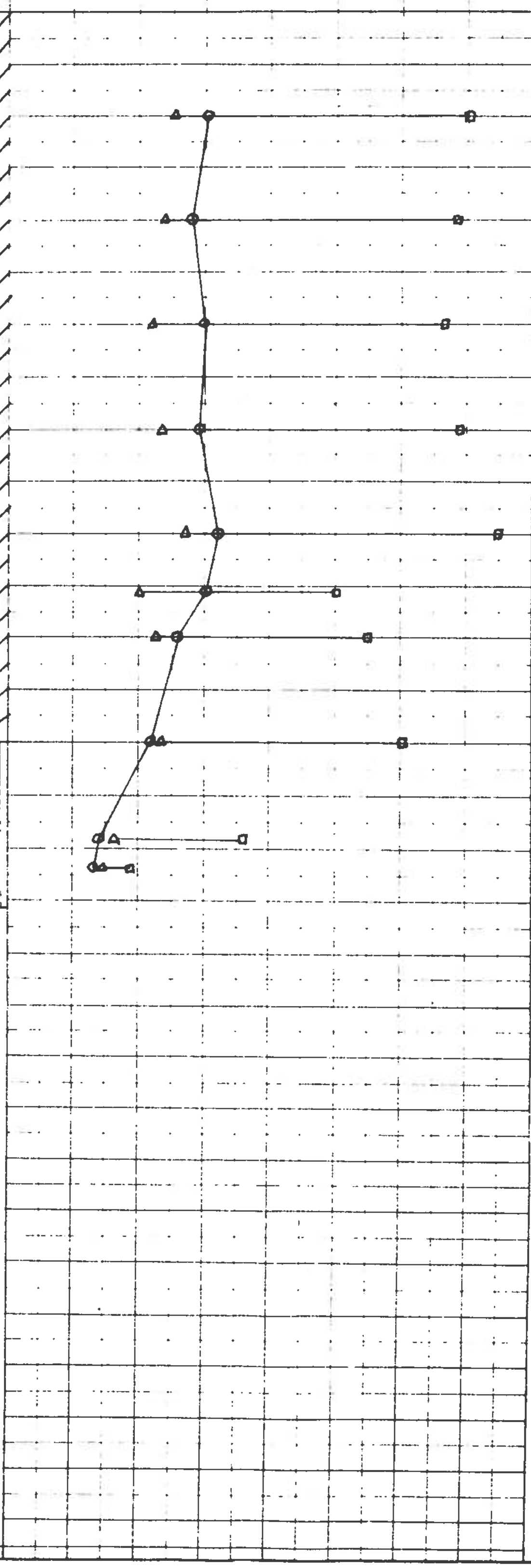


PLATE 24

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT: **DUNVEGAN LANDSLIDE - MAY 1959**

DATE: NOVEMBER 3/1959 | HOLE: 76 I | PLATE: 27

SOIL PROFILE — SOIL DESCRIPTION

ELEVATION OF GROUND AT TOP OF HOLE 1583 FEET ABOVE SEA LEVEL. LOCATION 1197 + 00

MOISTURE CONDITIONS & ATTERBERG LIMITS

SOIL SAMPLES — OTHER TESTS

ELEV. / DEPTH — FEET: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80, 85, 90, 95, 100, 105, 110, 115, 120, 125, 130, 135, 140, 145

CLAY - INORGANIC, HIGH PLASTIC (0–90 ft)

SILT - INORGANIC, SANDY, DRY, LOW PLASTIC (from 90 ft)

MORE PLASTIC WITH DEPTH (120 ft)

MOIST SILT [FIELD LOG] (130 ft)

CLAYEY (145 ft)

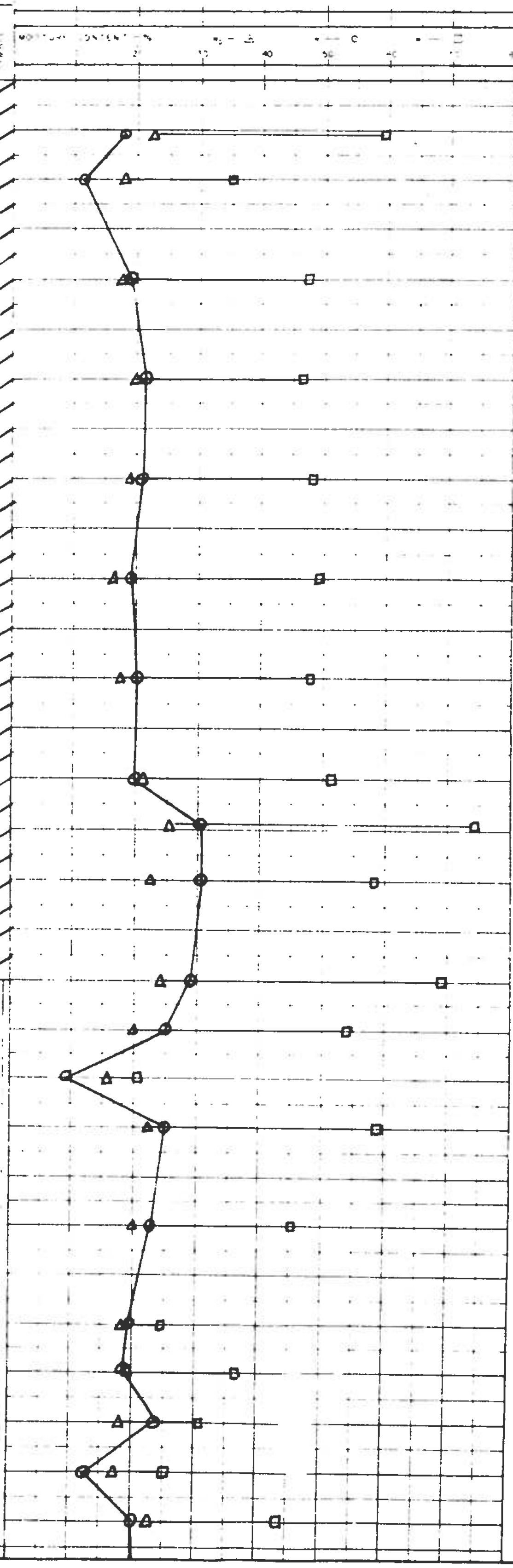


PLATE 27

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT

DUNVEGAN LANDSLIDE – MAY 1959

DATE NOVEMBER 3/59 | HOLE 72 | PLATE 25

SOIL PROFILE — SOIL DESCRIPTION

ELEVATION OF GROUND AT TOP OF HOLE 1601 FEET ABOVE SEA LEVEL. LOCATION 1203 – 50

MOISTURE CONDITIONS & ATTERBERG LIMITS

SOIL SAMPLES — OTHER TESTS

DEPTH FEET	SOIL DESCRIPTION
0	CLAY – INORGANIC, HIGH PLASTIC, STIFF
90	SILT LENS MOIST
100	SOFT CLAY – VERY MOIST
110	SILT – INORGANIC, LOW TO MEDIUM PLASTIC, DRY.
145	

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80, 85, 90, 95, 100, 105, 110, 115, 120, 125, 130, 135, 140, 145

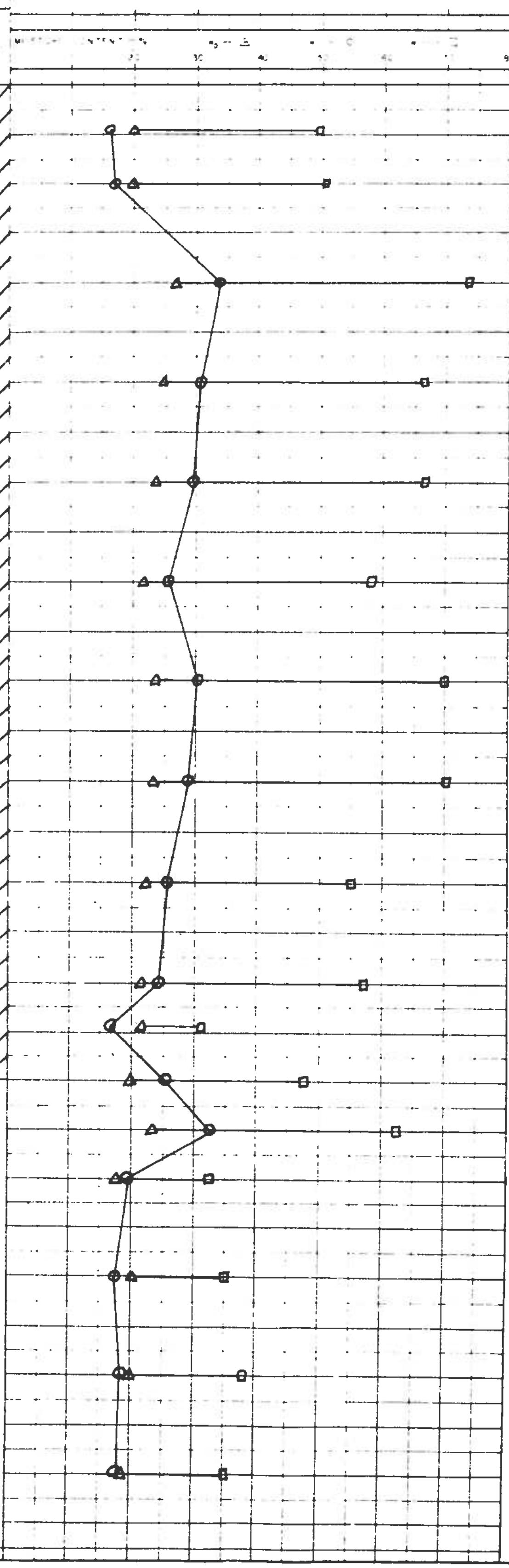


PLATE 25

R.C.A. 5

SUMMARY OF SAMPLING & LABORATORY TESTS

RESEARCH COUNCIL OF ALBERTA
EDMONTON — ALBERTA

PROJECT: DUNVEGAN LANDSLIDE - MAY 1959

DWN	CK'D	JOB NO.	DATE	NO.	PLATE
			NOVEMBER 3/1959	74	26

SOIL PROFILE		MOISTURE CONDITIONS & ATTERBERG LIMITS	SOIL SAMPLES
ELEV. DEPTH FEET	SOIL DESCRIPTION	MOISTURE CONTENT	OTHER TESTS
DATUM	ELEVATION OF GROUND AT TOP OF HOLE 1575 FEET ABOVE SEA LEVEL. LOCATION 1200 + 50		
0	CLAY - INORGANIC,		
5	HIGH PLASTIC.		
10			
15			
20			
25			
30			
35			
40			
45			
50			
55			
60			
65			
70			
75			
80			
85			
90	SILT - INORGANIC,		
95	LOW-MEDIUM PLASTIC.		
100			
105	SAND LENS		
110			
115	MOIST.		
120			

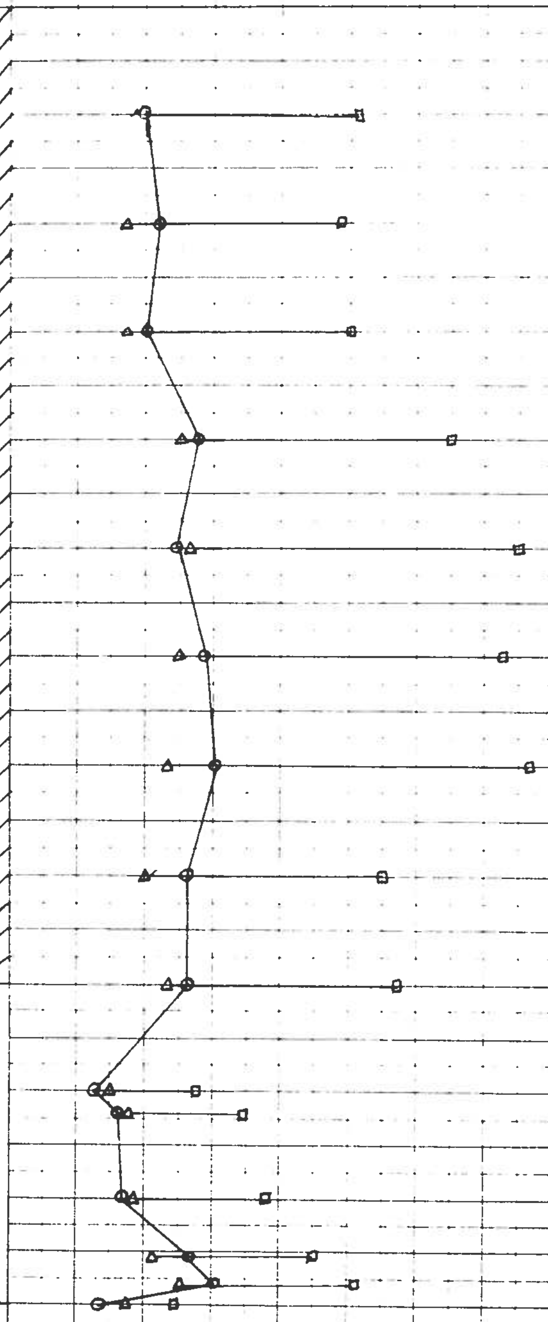


PLATE 26

R.C.A.-5

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT

DUNVEGAN LANDSLIDE - MAY 1959

DATE NOVEMBER 3/1959 | 76 2 | PLATE 28

SOIL PROFILE		MOISTURE CONDITIONS & ATTERBERG LIMITS	SOIL SAMPLES				
ELEV. DEPTH FEET	SOIL DESCRIPTION ELEVATION OF GROUND AT TOP OF HOLE 1583 FEET ABOVE SEA LEVEL. LOCATION 1197 + 00	MOISTURE CONTENT	OTHER TESTS	CONDITION	TYPE	PENETRATION RESISTANCE	RECOVERY % DEPTH FEET
150							
155							
160	CLAY						
165							
170							
175							
180							
185							
190							
195							

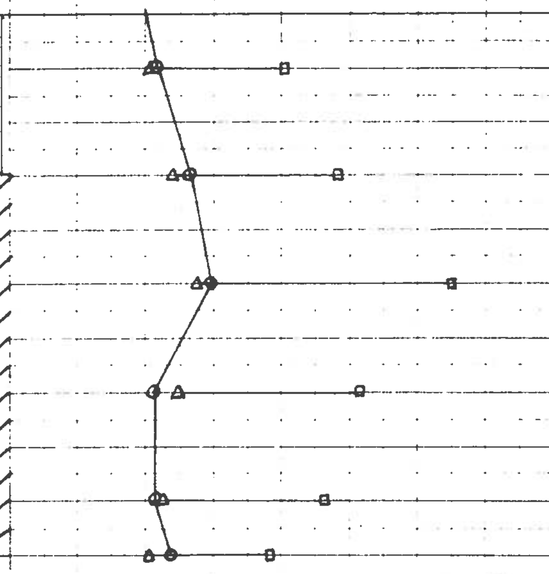


PLATE 28

RESEARCH COUNCIL OF ALBERTA

EDMONTON — ALBERTA

SUMMARY OF SAMPLING & LABORATORY TESTS

PROJECT

DUNVEGAN LANDSLIDE - MAY 1959

DATE NOVEMBER 3/1959 | 78 | PLATE 29

SOIL PROFILES

SOIL DESCRIPTION

ELEVATION OF GROUND AT TOP OF HOLE 1598.5 FEET ABOVE SEA LEVEL. LOCATION 1194+00

MOISTURE CONDITIONS & ATTERBERG LIMITS

SOIL SAMPLES

OTHER TESTS

0 CLAY - INORGANIC, HIGH PLASTIC, DRY.

20 SILTY

70 SILT AND SAND LENS

100 SILT - SANDY

END TEST DATA

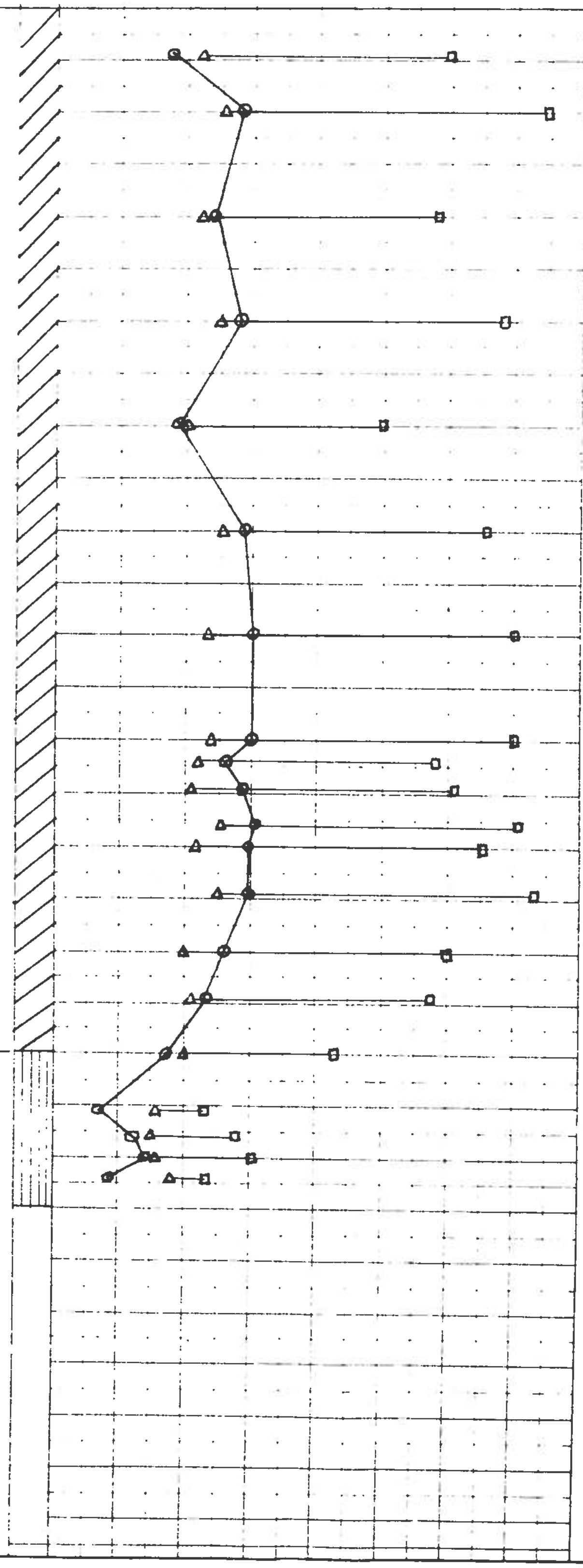


PLATE 29

RESEARCH COUNCIL OF ALBERTA EDMONTON — ALBERTA	SUMMARY OF SAMPLING & LABORATORY TESTS PROJECT DUNVEGAN LANDSLIDE MAY 1959

NOVEMBER 3/1959	78 2	PLATE	30

	SOIL PROFILE SOIL DESCRIPTION *ELEVATION OF GROUND AT TOP OF HOLE 1598.5 FEET ABOVE SEA LEVEL. LOCATION 1194-00	MOISTURE CONDITIONS & ATTERBERG LIMITS	SOIL SAMPLES OTHER TESTS
150	SOFT LAYER [FIELD LOG]		
155			
160			

PLATE 30

Instructions for Riley's

Publication: Dunveyan Landslide

Details:	No. of pages:	74	$8\frac{1}{2}$" x 11"
	No. of maps, figures:	4	$10\frac{1}{2}$" x [illegible]
		30	$14\frac{3}{4}$" x [illegible]