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DUNVEGAN LANDSLIDE

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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The crossing of the Peace River at Dunvegan was originally made by canoe.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 possible 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).

5. 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 condiactivity 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 southwest 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