# The Peace River Highway Bridge—A Failure in Soft Shales

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This paper deals with the failure of the Peace River highway bridge and the stability of slopes in soft shales formed by heavy overconsolidation of alluvially deposited clays under the weight of glaciers. Geologically these are classed as soft rocks, but in many instances no cementation of the particles has occurred. The only physical alteration to the original clays has been compression due to the consolidating pressure from glaciers during the Pleistocene period.

These shales are presently in a state of rebound and on release of overburden pressure, along with access to water, they may revert to soft clays, frequently having high swelling characteristics. The mineralogical constitution of the clay minerals does not appear to be the predominant factor governing the swelling characteristics. Rather, these appear to vary with the physical-chemical environment of the pore water and adsorbed moisture films.

Such soils are of wide occurrence in the northwestern portion of the North American continent. The paper deals with the performance of such materials in engineering construction in northwestern Canada, and points out the deficiencies of conventional methods of analysis for predicting their behavior. The swelling mechanism and its significance are discussed, and suggested improvements in concepts for analyzing the stability of slopes in such materials are presented.

• OVER EXTENSIVE AREAS of the North American continent lying in a strip several hundred miles wide to the east of the Rocky Mountains there occur shales in which the major diagenetic process of formation has been overburden pressure. These shales now exist at overburden pressures much reduced from the maximum that has occurred in their geological history. They are in a state of rebound and under certain environments tend to revert to clays. They are frequently described as "clay shales."

This paper deals with engineering experience, particularly in regard to slope stability in highway construction and bridge approaches, with such clay shales occurring in northern Alberta, northeastern British Columbia, and the western portion of the Yukon Territory in Canada. They are of Cretaceous age and occur interbedded with coal seams, silt, sand, siltstones and sandstones having a wide variation in quality of cementing media. Many of them have an appreciable organic content. It has been estimated that they have been subjected to the weight of at least 1,500 ft of sediments and ice in addition to their present overburden, during the various periods of glaciation since their original deposition. They now frequently lie under a shallow overburden of recent deposits. The clay shales extend to depths of hundreds of feet, and the major rivers in the area are incised to depths of as much as 800 ft in these deposits.

Several cases of spectacular slope failures have occurred for which extensive soil investigations were subsequently performed, and for which fairly comprehensive data

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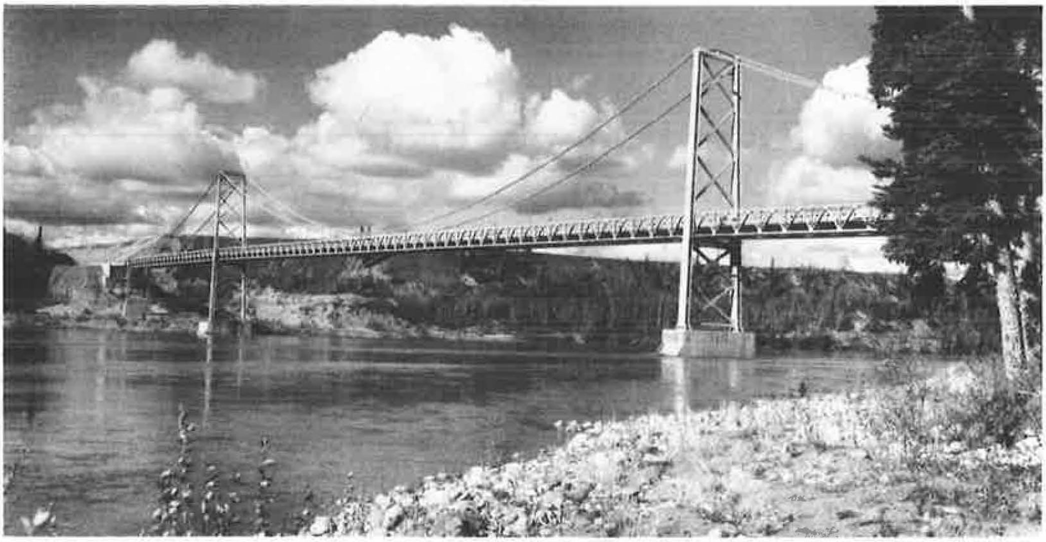

Figure 1. The Peace River highway bridge as originally constructed.

concerning the subsoil conditions have become available. This paper deals specifically with one of these failures which involved a major bridge collapse and which has not previously been reported in detail in terms of these data. A comparison is also made with the results of two other case histories involving slides in these soil types.

The particular case described in detail is a slide which developed at the north end of the Peace River highway bridge at Taylor, British Columbia. It resulted in the collapse of the bridge on October 16, 1957. The history of the performance of this bridge is given in considerable detail because it is significant to an understanding of the causes of the slide and has not previously been comprehensively reported.

Figure 1 shows the bridge as originally built, looking towards the north bank on which the slide subsequently developed. It was built in 1942 incidental to the construction of the Alaska Highway, now called the Northwest Highway System in Canada, and was designed by the United States Public Roads Administration. It was a suspension bridge with a total length of 2,130 ft, made up of a main span of 930 ft, suspended side spans of 465 ft, and 135-ft simple trusses spanning from the cable bents to the anchor blocks at each end. At the north end the tower and cable bent were carried on spread footings on shale, and the anchor block was also based on shale.

The topography at the north end of the bridge was characterized by a gravel terrace flat about two miles wide at an elevation about 200 ft above the river level. The original slope of the bank at the edge of this flat was close to 1.5:1 down about 170 ft to a second flat about 200 ft wide and about 30 ft above the river level. At the time of construction the main river channel was close to the south shoreline, but by 1949 it had shifted so it was encroaching along the north bank.

The geology in the area of the north bank is characterized by a gravel terrace on the upper flat overlying shale at a depth of about 100 ft, with the contact surface being irregular. The shale has been described geologically as bedrock, and is a black to gray marine shale of Lower Cretaceous origin. The formation is several thousand feet thick in the area. The shale is soft and thinly bedded and frequently includes interbeds of silt. The overlying gravel on the terrace is coarse cohesionless sand and gravel with a slight silt content and is in a medium dense state.

The natural moisture contents ranged from 3 to 35 percent. The lower moisture contents occurred at depths below the surface generally exceeding 50 ft, and for intact shale, which had not been subjected to disturbance or weathering, the moisture contents were generally less than 10 percent. The plasticity values for the shale were all within the medium-to-low plasticity range. They plotted parallel to and slightly above the "A line" on the Casagrande plasticity chart. Laboratory unconfined compressive strengths ranged from about 0.5 to 20 tons per square foot and averaged about 10 tons per square foot. One test gave a strength of 32 and another 56 tons per square foot. Consolidation test results were all characteristic of heavily over-consolidated soils. By extrapolation beyond the preconsolidation load, a compressive index value of about 0.26 was indicated. In general the samples did not indicate high swelling characteristics in the consolidation test, but a maximum swelling pressure of 5.0 tons per square foot was recorded in a "free swell" test with the sample immersed in distilled water.

At the time the bridge was located the site gave the general appearance of being one of the most stable possible locations in the area, except possibly for the fact that a series of springs existed at the contact of the gravel and the shale. However, two things incidental to the bridge construction are particularly pertinent to its subsequent behavior. One of these was that a cut to a depth of about 50 ft was made in the gravel terrace to secure a satisfactory grade for the approach road to the bridge. This removed an appreciable overburden weight from the shale in the area of the north anchor block. The second unfavorable factor was that surface runoff for about a mile north of the bridge was carried down the road ditches and discharged on either side of the bridge anchor block. This drainage area was further increased in 1955. Thus, surface runoff was made available to penetrate through fissures in the shale in the area of the anchor block, a condition that did not exist prior to the bridge construction.

In 1947 it was found that scour had occurred under the foundation pad for the north tower, despite the fact that at the time of construction the foundation excavation had to be taken out with jackhammers and it was considered to be rock excavation. The tower foundation was underpinned in 1948. Some additional scour occurred around the north tower foundation in 1952 and this was corrected by riprapping.

In 1952 there was visual evidence that the bridge deck had lost some of its camber and there appeared to be the possibility that the north anchor block had moved horizontally about 3 in. At that time the Canadian Army, which operates the Northwest Highway System, established a series of check points on the bridge foundations and deck. During the summer of 1957, a year of higher than average precipitation in the area, severe scour occurred on each side of the north anchor block due to high intensity of surface runoff. In September of that year there was visual evidence of possible movement of the north anchor block.

In view of this, officials of the Northwest Highway System made a complete check of their 1952 survey during the period October 3-8, 1957. This showed a possible 1.08-ft shift southward and a definite 3-in. shift westward of the top of the north cable bent. Observations on the north anchor block showed a tilting, as well as shifting toward the south, with a possible movement of 1.6 ft horizontally in the interval 1954 to early October 1957. It is likely that most of this movement occurred during the late summer of 1957, because with this magnitude of total creep noticeable defects in the approach road immediately north of the anchor block would be inevitable. Such in fact were evident between early September and early October 1957. A total settlement estimated at 18 in. in the approach road was observed for a distance of about 18 ft north of the anchor block during this period, cracks were observed in the soil on either side of the anchor block, and surface runoff disappeared behind the anchor.

During 1956 and 1957 a natural gas scrubbing plant was erected on the gravel terrace a few hundred feet to the north and east of the north end of the highway bridge. This involved the erection of a water intake plant at the river with supply and return water lines to and from the plant to handle up to 18,000 gpm. The water intake structure was located about 650 ft west (upstream) of the highway bridge, but the pipelines to and from the plant were carried along the lower bench, below the highway bridge and up the bank about 200 ft east of the centerline of the bridge. The pipelines were constructed with a minimum of disturbance to the natural soil conditions along the bench and up the bank. Their construction involved leveling the surface along the bench, but fill was placed over them rather than placing them in an excavated trench. However, they were placed in an excavated trench up the face of the bank to the east of the bridge.

Construction of the intake structure was commenced in the early winter of 1956-7 at a site about 150 ft east of its final location. It required about a 50-ft deep excavation on

Figure 2. Gas processing plant and pipeline suspension bridge in relation to highway bridge.

the lower bench. Freezing of the walls of the excavation was intended to provide stability during construction. However, heavy freezing temperatures did not occur until early January, and a few days previous to this the north wall of the excavation caved. The structure was then moved 150 ft to the west and founded on steel H-piles driven to refusal in the shale. Some of the piles were battered. This proved to be a fortuitous occurrence in view of the subsequent bank movement.

A natural gas pipeline suspension bridge across the river was also erected during 1956 and 1957 at a location about 700 ft east (downstream) from the highway bridge. This was built with its tower foundations and wind cable anchors on the lower bench and its main cable anchor block in the gravel on the upper bench. These foundations were all built with as little disturbance as possible to the natural soil conditions.

The locations of the natural gas scrubbing plant pumphouse and pipeline suspension bridge in relation to the highway bridge are shown in Figure 2.

The gas processing plant was put into operation during the summer of 1957. No difficulties were encountered with the water supply until October 15th.

Precipitation records in the area have only been available since 1928. An analysis showed that the period June 1 to October 15, 1957, was the wettest on record, and that statistically its occurrence would be at a frequency of once in 30 years. It is also significant that the average precipitation during the annual periods May 1 to October 15 from the time the bridge was built in 1943 until 1956 was 10.2 in., whereas the precipitation for this period in 1957 was 18.0 in. Moreover, in September and early October 1957, there were two heavy falls of wet snow, which would result in a greater percentage of the water going underground into the gravel than would usually occur with rainfall.

At 1:00 p.m. on the afternoon of October 15, 1957, and before officials of the Northwest Highway System were able to arrange for specialist advice following their survey of the conditions in the period October 3-8, the gas processing plant operators detected that the water supply system was not functioning properly. By 3:00 p.m. rocks and soil were being discharged in the return line, and water was flowing from the bank part way up the pipe trench. By midnight that day it was evident that all the water lines were broken. However, attempts were made during the afternoon to keep the plant in operation so that water under pressure was in the lines until about 3:00 a.m. the following morning.

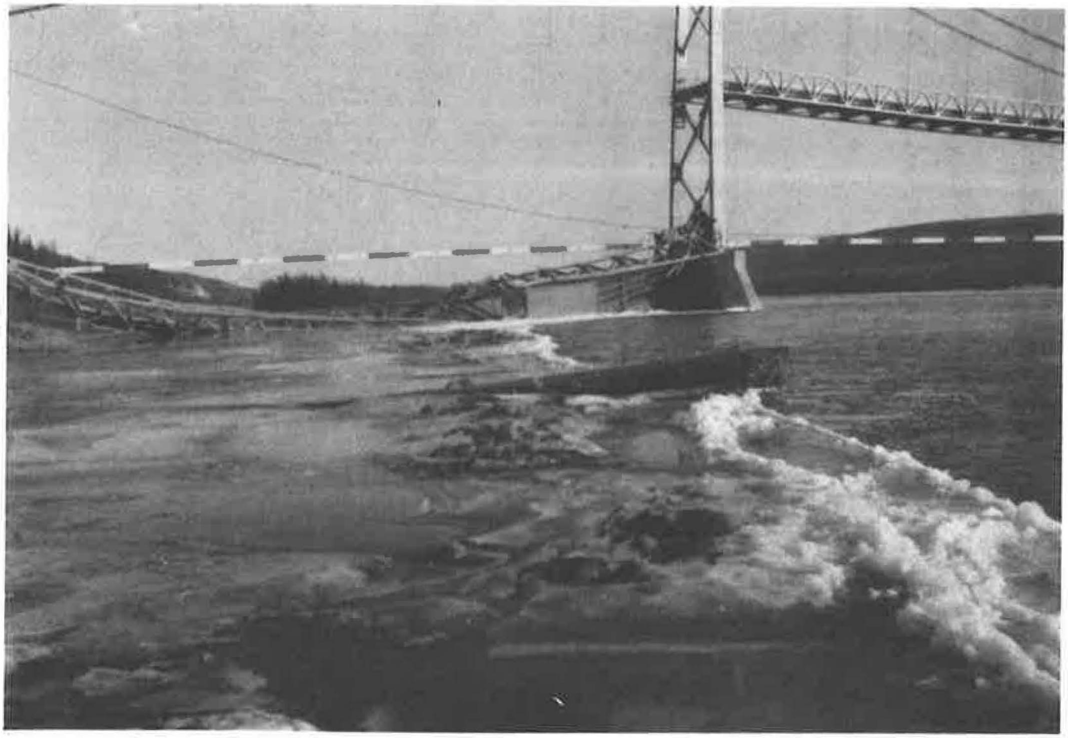

Figure 3. Toe of slide projecting above river surface.

No evidence of additional movement around the north anchor of the bridge was observed until 2:15 a.m. on October 16; at this time a depression of a few inches had occurred in the road surface at the north end of the anchor block. This depression slowly increased, and it was necessary to close the bridge between 4:00 and 5:00 a.m., and by morning the slide was well developed. Collapse of the two north spans of the bridge occurred at 12:40 p.m. on October 16.

It was evident following the collapse that the bank instability was not confined to the area immediately around the north anchor block. It was clear that a major bank movement had developed. The slide extended a distance of about 470 ft east of the highway bridge, and about 625 ft west to within a few feet of the water intake structure. It was thus about 1,100 ft long and was centered about 75 ft west of the bridge centerline. The extent of the slide above the river level was clearly defined by a sheared face at the top and cracks at the ends. The toe of the slide appeared above the water surface at a distance of about 120 ft from the shoreline.

The extent of the bridge collapse is shown in Figures 3, 4, 5 and 6. The toe of the slide is shown in Figure 3 projecting above the water surface in the river. The extent of the slide which developed in the bank is indicated in Figures 6 and 7.

Neither the water intake building nor the pipeline suspension bridge were moved by the slide, except that a wing wall at the east end of the pumphouse was undercut by the movement. However, the water lines were completely wrecked within the slide area, and the distortion of these from the bank movement caused some damage within the pumphouse. These damages, however, were negligible as compared to the damage to the highway bridge. It was subsequently dismantled and replaced by a new bridge.

The cause of the bridge collapse was without question the instability of the bank. The weight of soil within the mass which moved far exceeds the forces acting on the bridge anchor block. It was not a structural failure of the bridge in any sense. The anchor block and cable bent foundation merely rode the slide. However, it appears that deteri-

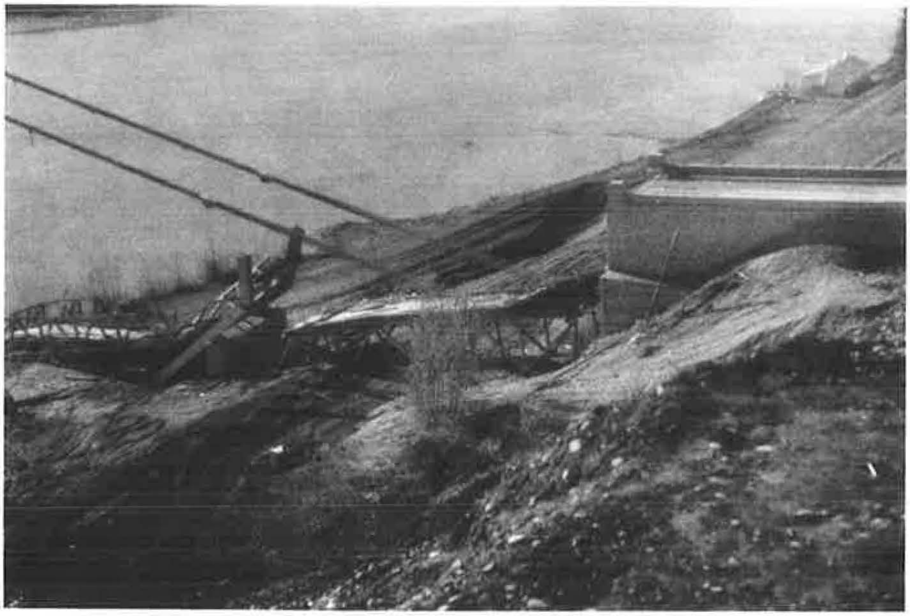

Figure 4. Slide area, showing movement of anchor block.

oration of the stability conditions in the bank commenced soon after completion of the bridge and they continued to deteriorate for a period of years.

The sequence of events and the known characteristics of the shales existing in the bank suggest that the approach road cut into the natural bank at the north end of the bridge, plus the concentration of surface runoff in the highway ditches on either side of the anchor block, were factors that probably initiated the deterioration of the shale. The excessive rainfall during the previous summer undoubtedly accelerated the deterioration of the subsoil conditions. However, the intensity of rainfall in that period was not greater than should have been normally expected throughout the life of the bridge.

Of more problematical significance is the effect of the construction activities in the area of the slide during 1957, particularly the construction of the water lines. There is no evidence in the sequence of events leading up to the collapse that the construction activities had any appreciable effect on the stability conditions. However, it does appear that the broken water lines evident for the 24-hr period previous to the collapse greatly aggravated the final deterioration of the stability conditions. If the broken water lines had not been in existence, the rate at which the conditions progressed towards collapse would undoubtedly have been much slower, and possibly would have been such that remedial measures could have been taken before the bridge was finally destroyed.

It is, of course, pertinent to inquire as to whether the water lines broke due to defects inherent in one or another of them, or whether the creep of the bank broke a water line and then the loss of water into the bank created seepage pressures and accelerated the stability deterioration. This question can be answered only by inference. The lines were all pressure-tested before the plant was put into service, and the failure of pressure-tested pipes due to inherent defects is rare in pipeline experience. In view of the history of creep of the bridge anchor over the years, it is at least logical to conclude that it was movements in the bank which ruptured the water lines.

Extensive investigations were made by several authorities following the collapse to establish the reasons for the failure and to assess the stability of the water intake and pipeline bridge structures located adjacent to each end of the slide. Efforts were, of course, made to identify the position of the failure surface. This proved difficult to positively identify. However, in two test holes significant increases in natural mois-

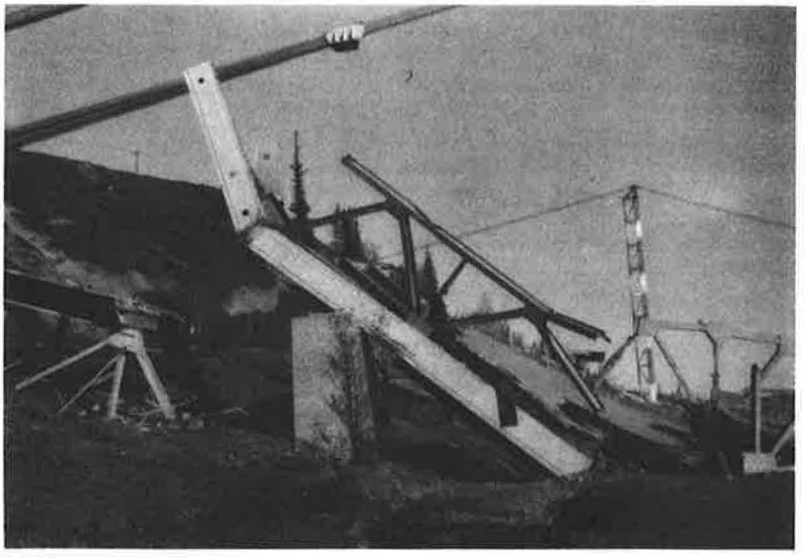

Figure 5. Collapsed cable bent.

ture contents occurred and some evidence of soil disturbance was detected at elevations 1285 and 1290. These were 30 and 35 ft, respectively, below the river level at the time of the collapse. Although the natural moisture contents in this zone showed a bulge in the moisture content profile for depth, they were only about 20 percent.

All samples taken were badly fissured and difficult to handle for test purposes. However, one series of carefully run consolidated undrained triaxial compression tests with pore pressure measurements defined an effective angle of internal friction of 30.6° with a cohesion value of 0.75 tons per square foot. However, the data could have been interpreted equally as well to give an angle of internal friction of 36° with no cohesion.

Stability analyses were made for various cross-sections through the slide area. On the section along the centerline of the bridge with the forces of the anchor block acting, a sliding block analysis with its base at elevation 1285 at the toe and 1290 below the anchor block gave the most realistic results. Using an effective stress analysis with an angle of internal friction of 30.6°, an excess hydrostatic head on the base of the sliding block closely equal to the elevation of the contact between the shale and the gravel of the terrace was required for a factor of safety of one.

This is a realistic result to the extent that the broken water lines could be expected to produce a head in the area of the anchor block that would be close to the contact between the pervious gravel and the underlying clay shale. The seepage pressures would be transmitted through the shale to the failure surface through fissures. However, it is unrealistic in several respects. First, the cohesion component of shearing strength has been neglected in the analysis. Second, if the alternative interpretation of the soil test data is used by which the cohesion is zero and the angle of internal friction is 36°, a considerably greater hydrostatic pressure is required on the failure surface for a factor of safety of unity. Third, the analysis has been made assuming the forces from the anchor block were resisted entirely by a slice of soil through the slide of width only equal to the width of the anchor block; if they had been distributed over the 1,100-ft width of the slide area, the average soil strength required to resist them would be much less. Fourth, failure conditions were imminent before water from the broken mains was available to provide the high seepage pressures on the failure surface necessary to show a factor of safety of unity. The effect of each of these factors is to lower the actual average shearing stress existing on the failure surface as compared to what can be estimated as being available using conventional concepts of the shearing strength of soils.

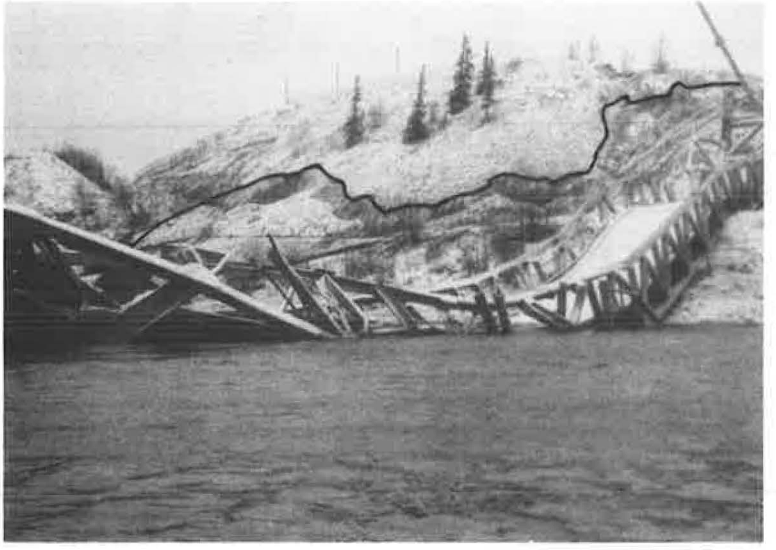

Figure 6. Slide escarpment to west of highway bridge.

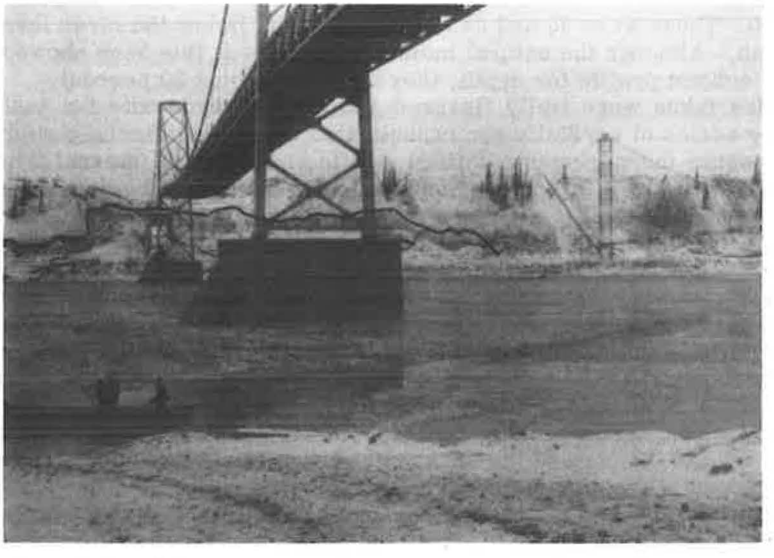

Figure 7. Slide escarpment to east of highway bridge.

**Within the scope of this symposium this case history illustrates a situation in which, at the time of the original construction, the soils involved at the site were considered to be bedrock and they had the properties of soft rocks. However, normal construction practices appear to have had the result of greatly accelerating the disintegration of these rocks. The result has been that within a period of only a fraction of the normal potential life of the structure, deterioration of the strength characteristics proceeded to the point where disastrous failure of the engineering works occurred. Moreover, analyses of the failure conditions indicate that the shearing stresses mobilized on the failure surface were considerably less than can be predicted by conventional determination of the shearing strength of these soils.**

Several other cases of slides in similar types of clay shale have come to the attention of the author in which the same difficulties have arisen in attempts to analyze the conditions. At two of these comprehensive soil investigations were undertaken. These two, known as the Dunvegan and Little Smoky River slides, have been reported in detail elsewhere (1).

The Dunvegan slide involved the failure of a highway embankment built on a slope of 6.5:1 with an over-all height of 250 ft. The embankment was to be built to a height of 100 ft, but when a height of 70 ft had been reached instability developed as the frost was leaving the ground in the spring of 1959. The surface area within the slice was about 50 acres and 4 to 6 million cubic yards of soil moved.

The Little Smoky River slide involved movement of more than 2 ft in a bridge pier foundation founded on timber piles driven to refusal in the clay shale.

In both these cases conventional stability analyses show that they should have had a substantial factor of safety against movement. Total stress analyses are completely unrealistic using shear strengths from tests that appear to be at all reasonable. Effective stress analyses using shear strength parameters determined from consolidated undrained triaxial tests with pore pressure measurements give somewhat more realistic values for factor of safety. However, to secure an effective stress low enough to give a factor of safety of unity, a magnitude of pore pressure must be assumed that greatly exceeds any realistic value indicated by the subsoil investigations.

## THEORETICAL CONSIDERATIONS

It is pertinent to consider possible modifications to conventional shearing strength theories that will permit more realistic stability analyses to be made where these types of clay shales are involved. The concepts of Lambe (2) for the shearing strength of clay soils, in which forces of attraction and repulsion between the soil particles are postulated, appear to offer possibilities in providing a modification to the conventional effective stress stability analysis that will more accurately check the observed field performance of these soil types.

Release of stress in recent geological time on these heavily over-consolidated soils results in elastic readjustments within the soil mass which may result in fissuring and fracturing of the brittle soil mass, particularly at comparatively shallow depths and along river valleys. One effect of this is that water can more readily penetrate the soil, which if it has access to free water undergoes a marked reduction in shearing strength. Lambe's theory postulates that this occurs due to changes in the forces of attraction and repulsion between the soil particles.

The modifications of the inter-particle forces appear to be due to several factors, and are influenced by the types of clay minerals making up the soil particles, the types of exchangeable ions in the adsorbed water films, and the salt content of the free water as compared to that of the adsorbed films. A difference in the salt content between the two results in the development of an osmotic pressure within the adsorbed films, which increases the forces of repulsion between the soil particles. The existence of such osmotic pressures in cohesive soils is taken from physico-chemical concepts, applied to fine-grained soils in an aqueous environment, and the application of these principles to soil masses has been discussed in recent years by a number of writers including Bolt (3), Ladd (4), Moum and Rosenquist (5), Seed, Mitchell and Chan (6), Olson (7), and Scott (8).

From the practical point of view there are several important implications of these concepts. First, because the osmotic pressure exists within the adsorbed films and not in the free water, it is not reflected in piezometric pressure measurements. Second, the increase in forces of repulsion manifests itself as a swelling pressure in the soil. Consolidation tests run so that a measure of the swelling pressure developed is determined, therefore give an indication of the extent to which osmotic pressures can develop in the soil, but this appears to be the only routine laboratory test in the field of soil mechanics which is capable of identifying soils susceptible to the development of appreciable osmotic pressures. Third, an increase in repulsive forces between the particles will result in a decrease in effective stress in the soil mass within the meaning of currently accepted concepts of effective stress.

A characteristic of some of the soils at each of the Peace River Bridge, Dunvegan and Little Smoky River sites, as well as at many other sites in the area of the occurence of these soil types, is that they exhibit high swelling pressures as measured in laboratory consolidation tests. The swelling pressure, however, is not a soil constant. It is affected by a number of factors (8). It is greatly affected by the salt content of the immersing water used in the test, and appears to be a maximum for distilled water. It is significant that surface runoff and snow melt are initially distilled water, and their subsequent salt contents must be taken up by contact with soil. Values of swelling pressures measured in consolidation tests do not usually exceed about 4 or 5 tons per square foot for these soil types, but swelling pressures as high as 35 tons per square foot have been measured. A wide range of values is usually obtained in samples from any particular site and many tests will show no swelling pressure. Appreciable swelling pressures do not appear to be necessarily associated only with montmorillonite clay mineral content, or with soils only of high plasticity. A limited number of tests indicate that maximum swelling pressures are developed with sodium-type soils and those with a high content of sodium-exchangeable ions (7) (9).

From the point of view of stability analyses of slopes in these soils, the most important factor emerging from these theoretical concepts is that the osmotic pressures reduce the effective stress within the soil mass in the zones where such pressures develop. It therefore is expedient to take this fact into consideration by a modification of the conventional relationship for the effective shearing strength of a soil:

$$s = c' + p_e \tan \phi' \tag{1}$$

in which

$s$ = effective shearing strength;
$c'$ = effective cohesion;
$\phi'$ = effective angle of internal friction; and
$p_e$ = effective normal stress = $(p-u)$;
$p$ = total normal stress; and
$u$ = pore pressure measurable by a piezometer.

Assuming that the osmotic pressures act to reduce the effective normal stress, and that their effect can be estimated from swelling pressures determined in consolidation tests,

$$p_e = p-u-p_s \tag{2}$$

in which

$p_s$ is the swelling pressure estimated from consolidation tests.

Eq. 1 can then be written

$$s = c' + (p-u-p_s) \tan \phi' \tag{3}$$

For the Dunvegan and Little Smoky River slides, for which comprehensive soil test data were available, stability analyses show that for a factor of safety of unity in the slide areas, the required swelling pressure is within the range of measured swelling pressures in consolidation tests on samples from the slide areas (1). Sufficient swelling pressure test data were not available from the Peace River Bridge slide area to permit an accurate estimate of swelling pressure to be made.

## PRACTICAL CONSIDERATIONS

Further research on the physical-chemical characteristics of these clay shales, the swelling mechanism, and methods for measuring the swelling forces tending to reduce the effective stress is obviously desirable. A wide field for research has been opened up by the concepts. Present knowledge does not indicate a simple solution to the stabilization of slopes in such materials. The concepts, of course, do not condemn the traditional methods for stabilizing slopes, but they do suggest a change in emphasis in some aspects of the standard practice.

Drainage remains of prime importance in improving stability, but the possible effect of surface runoff of low salt content in inducing high osmotic pressures suggests that major attention should be given to the control of surface runoff. Slope flattening by means of toe loadings is still desirable, provided it does not block drainage. However, slope flattening by excavation of the upper portion of the slope may precipitate instability if it results in more ready access of water, particularly surface runoff, to the subsoil. The most objectionable practice with these soil types is the installation of drainage systems to relieve anticipated pore pressures if these are installed so that surface runoff has access to the system, or if it is possible for water to back up into the system from a river at high flow stages.

## ACKNOWLEDGMENTS

The information concerning the Peace River Bridge failure and the data from the subsequent investigations were available to the author in his capacity as consultant to Westcoast Transmission Company, Ltd., and Pacific Petroleums, Ltd., the owners of the natural gas facilities at Taylor. Independent investigations were made on behalf of the Department of National Defense of Canada. Technical information was pooled. Data concerning the properties of clay shales were available from the records of R. M. Hardy & Associates, Ltd., consulting engineers. Special stability studies were made by E. W. Brooker (10) under the general direction of the author.

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