MASSIVE LANDSLIDES IN SOFT COMPACTION SHALES

by

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INTRODUCTION

Massive landslides on transportation routes are disruptive and costly. They usually occur in terrain identified as historically unstable but must, nevertheless, be crossed due to location constraint. The risk of failure in this circumstance is inherently high. The result of failure is to impede commercial transport and incur high costs.

Four case histories of massive slides in soft compaction shales of Northern British Columbia and Alberta provide a basis for establishing common features. The mechanics of massive landslides is proposed as a function of geologic stress history and residual strength. The time movement relationship associated with such cases is illustrated. Evaluation and design measures for incorporation in analysis of rugged landslide terrain are recommended.

The case histories which follow are based on records from 1949. Since then to the present there have been numerous improvements in all aspects of geotechnical engineering ranging from techniques of sampling and testing accompanied by the development of improved strength testing equipment to sophistication in stability analyses. However, one of the most important improvements has been the recognition and incorporation of geology and geologic observations in civil engineering practice (Peck 1962). All of these improvements have produced a better insight into investigation, analyses and design of massive landslides on transportation routes in rugged terrain. A coherent picture of the behaviour of soft compaction shales and dense tills now exists and justifies a review of case records of historic landslides in compaction shales.

The compaction shales of Northern British Columbia and Alberta are poorly indurated marine deposits. They display many of the physical characteristics associated with shales in their undisturbed and unweathered state. However, they revert to materials displaying the characteristic of dense overconsolidated clays. When unloaded they are subject to rebound and to the forces of weathering (Hardy, 1957).

These materials have been preconsolidated in their geologic history and are known to display high values of earth pressure at rest (Skempton 1961). Where valleys are formed the processes of rebound due to preconsolidation are accentuated and shear zones due to horizontal strains develop (Matheson 1972). These shear zones are often very narrow bands that often escape identification during exploration.

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In all cases the parent materials are high plastic clays with liquid limits ranging between 50 and 100 percent plastic limits in the range of 20 to 40 percent. The natural moisture content of these materials is usually close to or below the plastic limit except in shear zones or near fissures and joints. The clay minerals usually contain a significant percentage of sodium or potasium montmorillonite which is known to have exceptional water absorbitive capacity.

THE PEACE RIVER BRIDGE

In 1942, during war emergency circumstances the United States public roads administration on behalf of the U.S. Corps of Engineers constructed a suspension bridge across the Peace River at Taylor Flats, B.C. near Fort St. John. During 1956 remedial measures were carried out to correct scouring on foundations of the north suspension tower. During October 1957 after an exceptionally rainy season movements of the north anchor were noted and on October 16th, 1957 the movements accelerated with the north anchor moving riverward the north cable bent tilting forward together with collapse of the stiffening trusses (Thompson 1958, Hardy 1965). A plan of the Peace River slide area is shown in Figure 1 and a cross-section along the centre line of the bridge on the north side shown in Figure 2. Sub-surface exploration identified a zone of remolded material in the compaction shale. This evidence together with cracking noted in the ground adjacent to the north anchor and mud flows in the river provided an insight into the character of movement (Brooker 1958).

A significant factor in the development of failure was erosion in the river channel and particularly at the toe of the slope. This had been a continuous process since the construction of the bridge. Toe erosion and exceptional precipitation along with the failure of a water supply pipeline near where the cable bent are believed to be the factors responsible for the final failure.

Emergency measures to restore traffic first involving a ferry service followed by the construction of a Bailey bridge by the Canadian Corps. of Engineers were undertaken. A new bridge was finally built. At this time in Canada, exploration, testing, and analytic capabilities were relatively primitive as compared with today's methodology. Nevertheless, a reasonable picture of the failure surface was established, a record of movements was developed.

DUNVEGAN SLIDE

In 1957 a highway link between the towns of Grande Prairie and Peace River, Alberta was undertaken requiring the crossing of the Peace River at Dunvegan. The approach to the bridge involved a sidehill cut and fill rising from the Peace River to the south uplands along slopes of the Dunvegan Creek valley. The terrain comprised numerous historic landslides. During the construction of one portion of the fill a massive landslide occurred in August of 1958. The slide area is outlined in Figure 3 and a cross-section through it shown in Figure 4. Because the approach crossed historic slide topography, observations were maintained during construction. Cracks were noted at one location in the fill and in the undisturbed material of the site. For reasons which remain unknown, a deep trench was excavated near the toe beneath the threatened area. Cracking of the ground accelerated with construction of the trench and precipitated a major landslide. (Brooker, 1958, Hardy, R.M., 1962).

Exploration in the area involved the aquisition of thin walled tube samples. However, the sample quality was poor. Open standpipes had been placed in a number of holes and produced inconsistent evidence in respect to peizometric conditions. However the standpipe casings served in determining the seat of movement acting as very crude slope indicators. Observations over the mass demonstrated movement commencing near the toe and retrogressing towards the top. Geologic observations demonstrated the Dunvegan Creek had been eroded as a tributary to the Peace River entirely in the Dunvegan formation a high plasticity compaction shale.

FAILURE MOVEMENTS IN COMPACTION SHALES

Compaction shales in their undisturbed state are brittle. As a result increase of stress produces only small deformations until the peak strength of the material is approached. As strain softening occurs the rate of strain increases. The character of movements associated with failures in these shales is demonstrated by the record of crack widening noted at Dunvegan and as developed for the Peace River Bridge. Cracks are first noticed when they are approximately one-half inch wide. Over several days they increase in width to 1 or 2 inches and then commence to accelerate rapidly over the following 24 hours. Once a crack has reached a width of approximately 6 inches the rate of crack widening is immediately observable and failure is underway. The final rate of movement in a landslide area has been noted as approximately 1 foot per hour lasting for several hours (Thompson 1986). Major portions of the movements take place in approximately 12 hours. Figure 5 demonstrates the general character of movements where the seat of movement is within compaction shale. The character of these movements is consistent with the development of peak strength followed by strain softening resulting in the development of a lower residual strength (Wroth et al 1986).

SMITH BRIDGE SLIDE, ATHABASCA RIVER

The bridge crossing the Athabasca River at Smith was constructed in 1944. In 1949 movements of the north abutment were noted. Records of these movements were maintained from 1949 to 1957. A plan showing the boundary of the landslide area is given in Figure 6 and a cross-section of the slide area is given in Figure 7. Record of movements of the north abutment are shown in Figure 10. The generalized stratographic sequence at the site comprises till over compaction shale. It will be noted on Figures 6 and 7 that the north bank of the Athabasca is subject to erosion which has resulted in an oversteepened toe. The toe area is loaded with the abutment of the bridge. The significant information from this site is a record of movements and a significant amount of movement which has occurred over the years. This circumstance is characteristic of several river crossings in the area.

Adjustments are made in the abutments and approaches to accommodate the movements.

LITTLE SMOKY RIVER SLIDE

The bridge across the Little Smoky River was completed in 1957. It was soon observed that the abutment of the bridge was persistently moving. (Hayley 1968, Thompson & Hayley 1975). Remedial measures were taken to accommodate these movements.

Investigation of the area demonstrated that the pier was in a massive slide area. A plan of the site is given in Figure 8 and a cross-section of the slide area given in Figure 9. This case is important because it is the first in the history of landslide studies in Northwest Canada that adequate instrumentation was installed. Instrumentation consisted of surficial movement gauges, slope indicators and piezometers. The slope indicator data together with surface movement observations demonstrated that the seat of the failure of the slide was within compaction shale and that the failure mass comprised successive blocks retrogressing from the river towards the crown. (Hayley, 1968, Thompson & Hayley, 1975, Morgenstern, 1986). It was further established that the erosion at the toe was the triggering mechanism in the slide area. The rate of movement in the slide area is approximately 150mm per year. Stability studies have demonstrated that an effective stress analyses with residual strength produces a factor of safety close to one when applied to a toe block. The toe block is the first block in the uphill chain of retrogressive movements. This case record is one of the most important because of it's thorough instrumentation, laboratory testing and observations. It marks substantial advance in the technique and knowledge available in a case of the Peace River Slide.

FEATURES COMMON TO ALL SLIDES

The four cases described above have the following features in common.

1. All slides have their seat of movement in compaction shales which are in a state of rebound.
2. Toe erosion is a factor in all instances.
   In two cases till overlays bedrock and the slide mass continually creeps with the seat of movement within compaction shale.
3. All cases involve retrogressing block type slides proceeding from the toe to the crown.
4. Effective stress analysis provide the greatest consistency of observed failure conditions.
5. All slides are in historic landslide areas.

MECHANICS OF STRENGTH LOSS AND FAILURE

Both the shales and the tills involved in the massive landslides of the foregoing examples involve preconsolidated materials. The numerical means of expressing preconsolidation is the Overconsolidation Ratio, OCR. The definition of OCR in a valley is given in Figure 11. Geologic histories involving glaciation or vast areal erosion result in indeterminate preconsolidation loads. The OCR is the ratio of the maximum effective stress at a point which has existed at some time in the geologic history, i.e. the preconsolidation load, of a deposit to the existing vertical effective stress at the same point. Considering the dimensions of the valleys involved and the depths of movements it is reasonable to conclude that the OCR values range between 10 and 30. (Brooker, 1965).

A history of preloading or overconsolidation produces horizontal earth pressures, K_0, greater than those in normally loaded material. The coefficient of earth pressure at rest is defined as the ratio of the horizontal effect of stress to the vertical effect of stress at a point under a condition of zero lateral strain. K_0 is related to the stress history of the material and the physical properties of the material as graphically defined in Figure 12 (Brooker 1965). It may be noted that the K_0 values range between 1 and 2.5 depending on the value of OCR and the plasticity index of the material. The significance of this in valley formation is that as erosion and valley deepening take place horizontal stresses develop and depending on the plasticity of the material influence valley rebound and therefore strain softening of the materials involved. Evidence has demonstrated that preconsolidated compaction shales are brittle but upon weathering and/or strain softening become plastic.

Research by Skempton (1961) and Bjerrum (1968,1969) throughout has demonstrated that preconsolidated materials display distinct peak and residual strengths as indicated in Figure 13. Evidence accumulated since has shown that the effective angle of shearing resistance is related to the plasticity index of the material and it's state of disturbance. The relationship between the angle of shearing resistance and the plasticity index in high plastic preconsolidated deposits is shown in Figure 14. Experience has demonstrated that these relationships are reliable for practical purposes.

The mechanism of failure in preconsolidated materials is defined by the evidence displayed in Figures 12 and 14. The unloading of a preconsolidated material in a valley results in the development of high values of K_0 and therefore high horizontal stresses (Fig.12) which seek to be relieved. Relief of these stresses results from horizontal strains over extended periods of time (Bjerrum 1968, Bozozuk 1986). Where there is variation in material stiffness and therefore, strain incompatibility of adjacent geologic units, these stresses are relieved by movement within selected weaker zones and result in sub-horizontal or horizontal shear planes (Matheson 1972). The evidence from Dunvegan Slide together with more recent data has clearly demonstrated the existence of shear planes within a rebounding mass (Matheson 1972, Morgenstern 1977). The influence of strain softening on the shear strength of material as defined by its angle of shearing resistance as shown in Figure 14 demonstrates the importance of strain softening. Geologic materials in valleys will exist somewhere between the peak and residual values. The challenge in engineering is to select, based on geologic evidence and the results of sub-surface exploration, an appropriate value of the angle of shearing resistance.

The evaluation must include an estimate of overconsolidation, strain history, regional strike and dip, detailed planar roughness as well as surficial geologic evidence of movements. With these factors included, the relationship in Fig.14 is adequate for reasonable estimates of the angle of shearing resistance for evaluation or design purposes.

The mechanism and information described above provide an insight into the failure mechanics and a means by which a reasonable selection of shearing resistance can be made. The mechanics has been developed from observational evidence and analyses of failures. Although a consistent pattern has developed the application of this to the numerical analyses of slopes for design purposes should still be used together with importance of geologic observations of performance in existing slopes. Application of geologic knowledge within a design is highly important and fundamental to the application of this in design.

APPLICATION AND DESIGN

The significance of the mechanics failure in preconsolidated deposits can be illustrated by the following two examples typifying active and passive approaches to design.

During construction, the freeway from Seattle to the Seattle-Tacoma Airport crossed slopes of preconsolidated materials (Shannon & Wilson, Peck). It was acknowledged that load relief or addition by fills or cuts on the slope may initiate movements resulting in massive failure and enormous costs due to adjacent highrise developments. A unique solution was devised by "wishing a retaining wall into place". A tangent caisson wall was first constructed consisting of large sized caissons excavated tangent to each other forming an inplace wall as illustrated in Figure 15. A wedge of material was removed from in front of the wall to form the highway. The strength of the wall was designed so as to resist uphill earth pressures and prevent movements. Prevention of movements maintain the integrity and strength of the material. The wall was constructed in 1965 and is in service today showing no visible movements in either the wall or the adjacent ground. This is an example of an active design which prevented strains which could cause weakening on selected planes and potential failure.

In the Swan Hill area in northern Alberta oilfield development has resulted in wells on the slopes of preconsolidated materials overlaid by tills. This combination is subject to creep as described in the Little Smoky and the Athabasca River cases. Creep of the materials bent the oilwell caissons to a point where continued production was threatened. The depth and rate of movement of soils was established by slope indicator installations. A slurry trench slot was constructed uphill of the well caisson that the surrounding ground could creep by the caisson. Oilwells have a definite life span and the slot has been constructed so as to accommodate movements over the economic production life of the well. This is an example of a passive solution to the landslide problem. In most cases, once movement has begun, the forces involved in restraining movement are so massive as to preclude economic solution.

In these circumstances the movements are accepted and a design produced to accommodate the movements. Many instances of transportation development in valleys involve historic landslide terrain. This is inherently dangerous and difficult to deal with. Engineering technique now provide the bases for relating geologic observations with sub-surface exploration and subsequent analyses on to define critical features of the mass. Engineering solutions may reside in an active design, passive design or rerouting completely. The options are usually limited by practical considerations.

CONCLUSIONS AND RECOMMENDATIONS

Information provided from four case records provide the basis for the following:

1. Massive movements in soft compaction shales are retrogressive comprised of sequential block sliding commencing at the toe of a slope due to erosion or excavation.

2. The vital movement and hydrologic features of these slopes may be established from instrumentation consisting of
   - o - Slope indicators
   - o - Piezometers
   - o - Surface movements gauges

3. There are, usually, horizontal to sub-horizontal shear planes within the stratigraphy of the slopes. These shear planes have angles of shearing substantially reduced from peak values.

4. The value of the angle of shearing resistance may be established, for all practical purposes, from Fig.14 considering:
   - o Geologic history and geomorphology
   - o Physiography-surficial manifestation of historic movements
   - o Dip and strike of stratigraphy
   - o Character (roughness) of the shear planes.

5. Preconsolidated compaction shale slopes are either in a state of rebound or contain substantial K_0 values. This may be assessed using Fig.12.

6. Subsurface exploration should include several borings using large size, continuous undisturbed core samples (i.e. approximately 150mm). These are aimed at identifying sheared zones.

8. Precise numerical evaluation of stability is uncertain due to the inevitably indeterminant character of stratigraphic and hydrology. Consequently, stability evaluation may be more of a qualitative nature and thus requires meaningful geologic observations.

9. If rugged terrain in a soft compaction shale stratigraphy is unavoidable satisfactory design measures of an active or passive character are possible.

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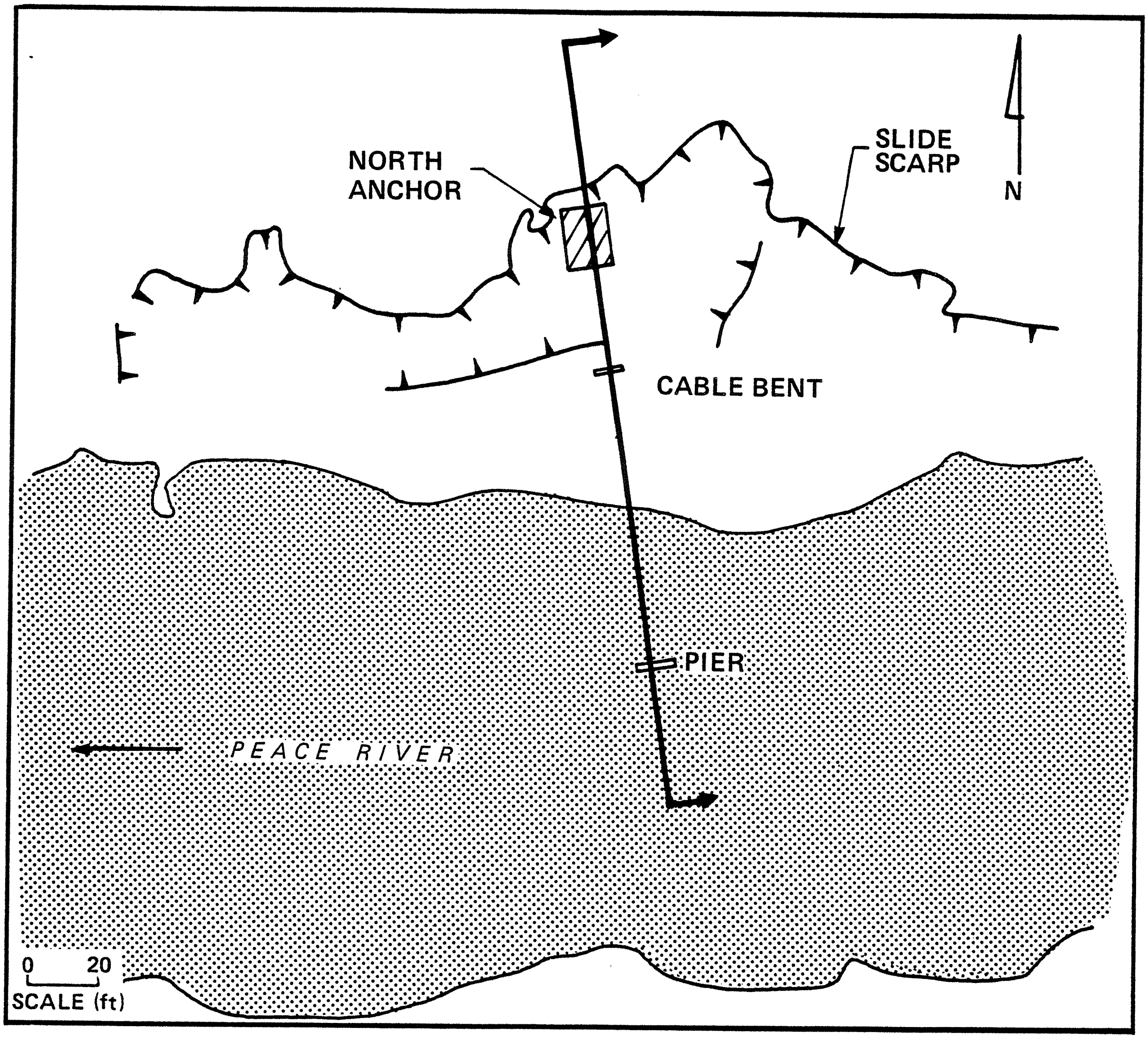

FIGURE 1 PEACE RIVER BRIDGE FAILURE – PLAN

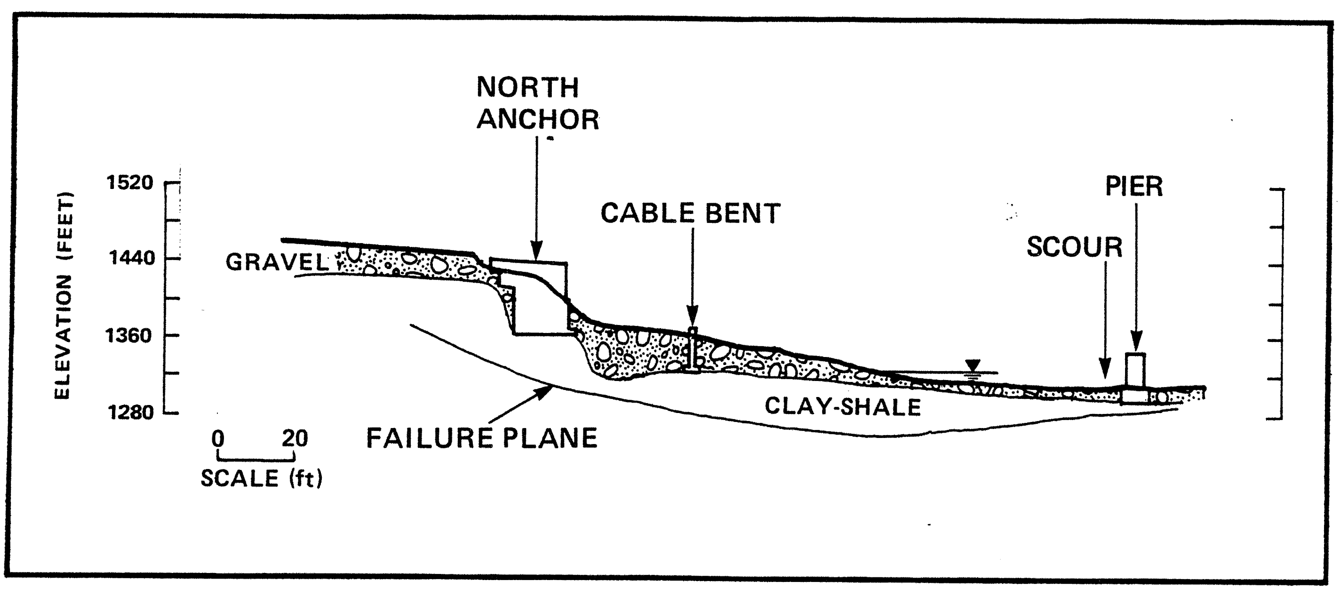

FIGURE 2 PEACE RIVER SLIDE – CROSS SECTION

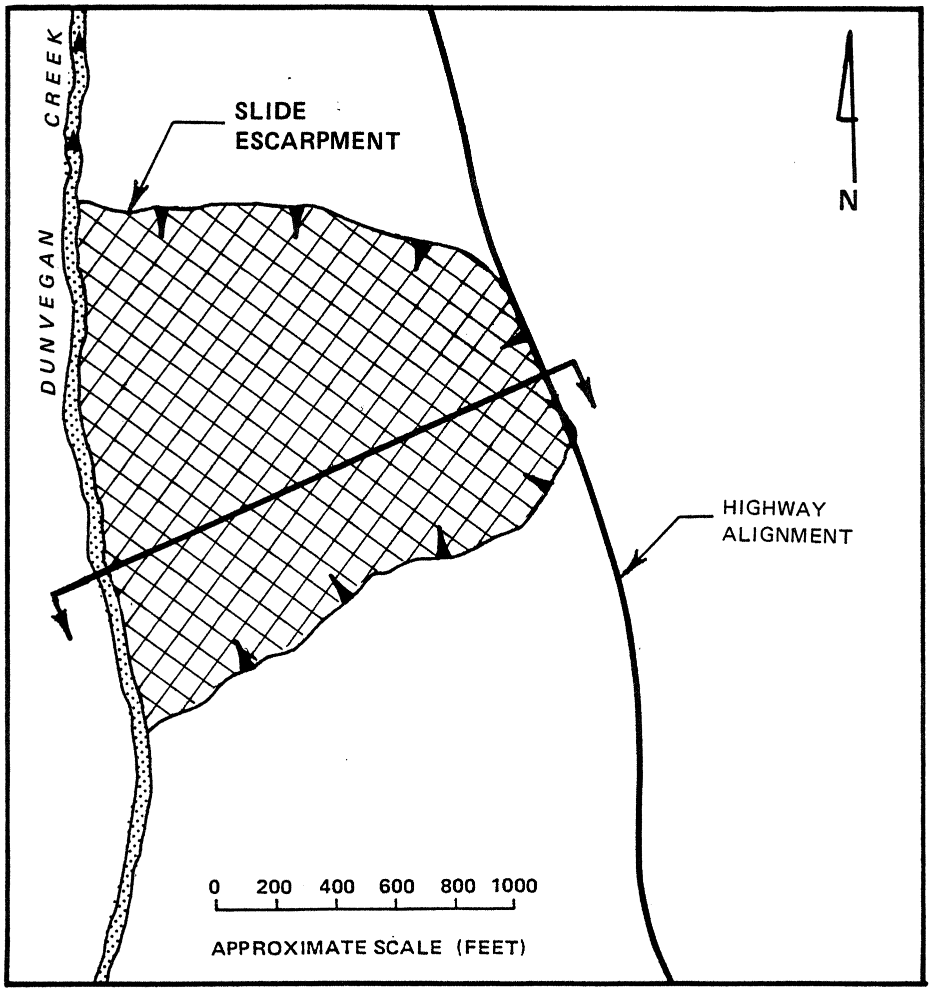

FIGURE 3 DUNVEGAN HILL – SITE PLAN

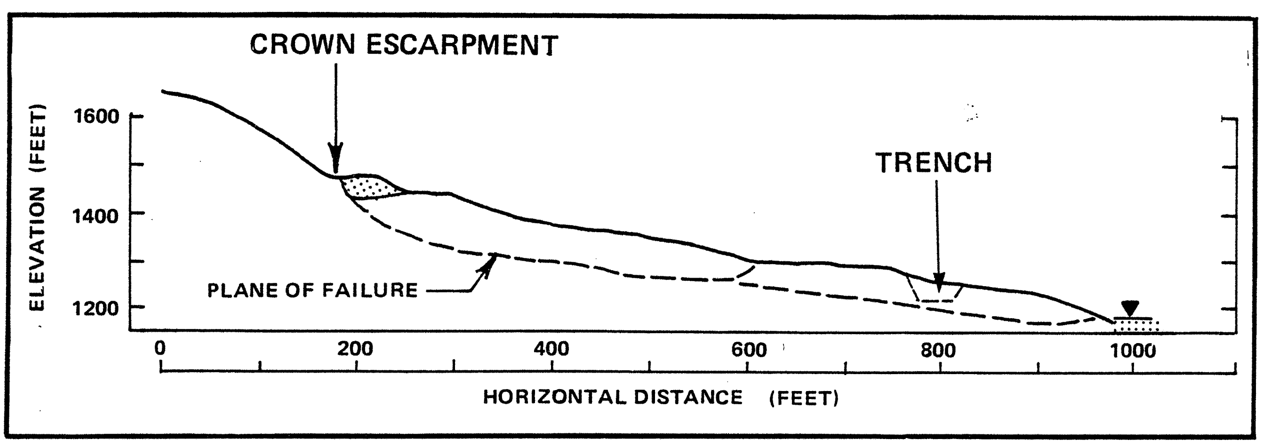

FIGURE 4 DUNVEGAN HILL – CROSS SECTION OF SLIDE AREA

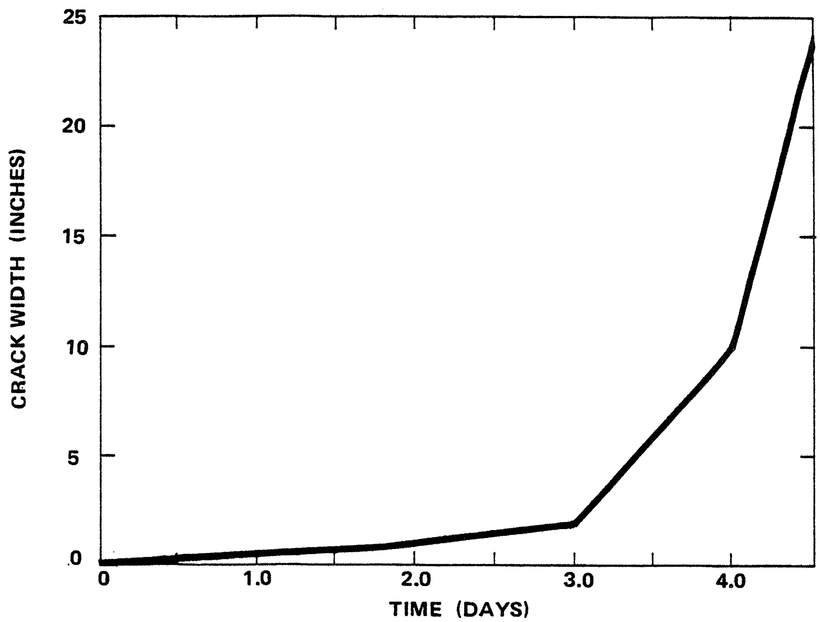

FIGURE 5 MASSIVE LANDSLIDES
CRACK MOVEMENTS AT FAILURE (DUNVEGAN)

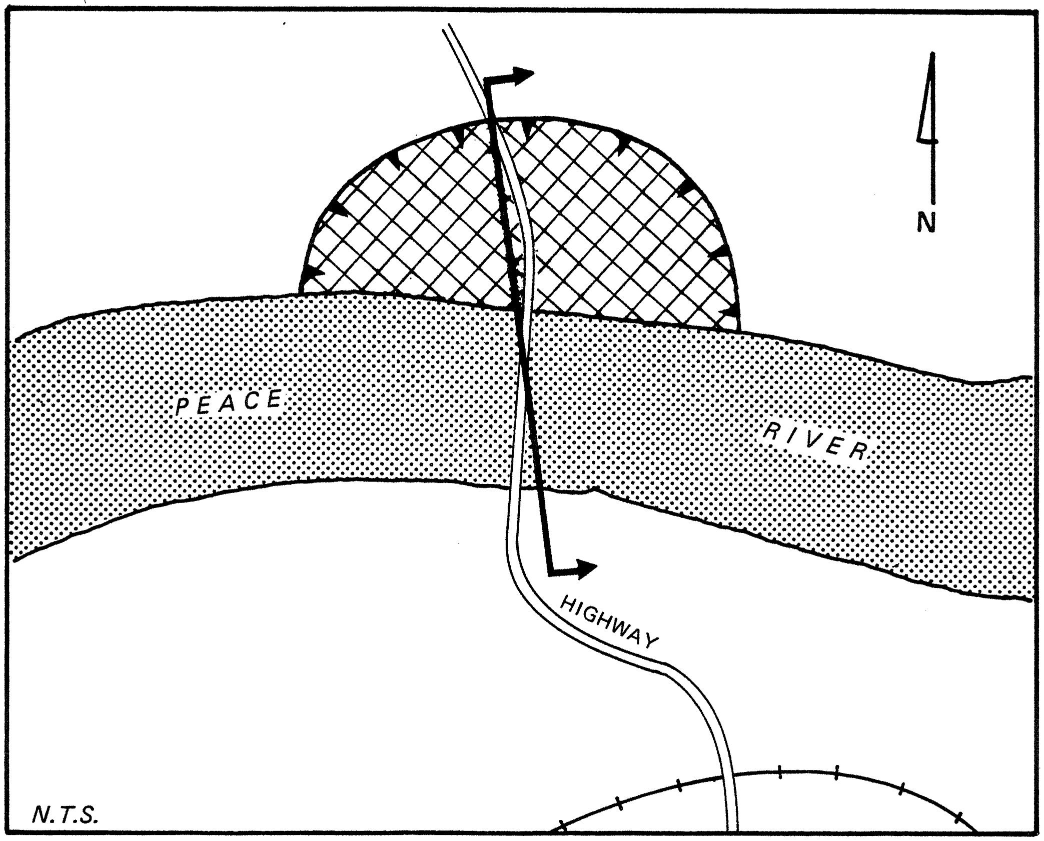

FIGURE 6 PLAN OF AREA – SMITH

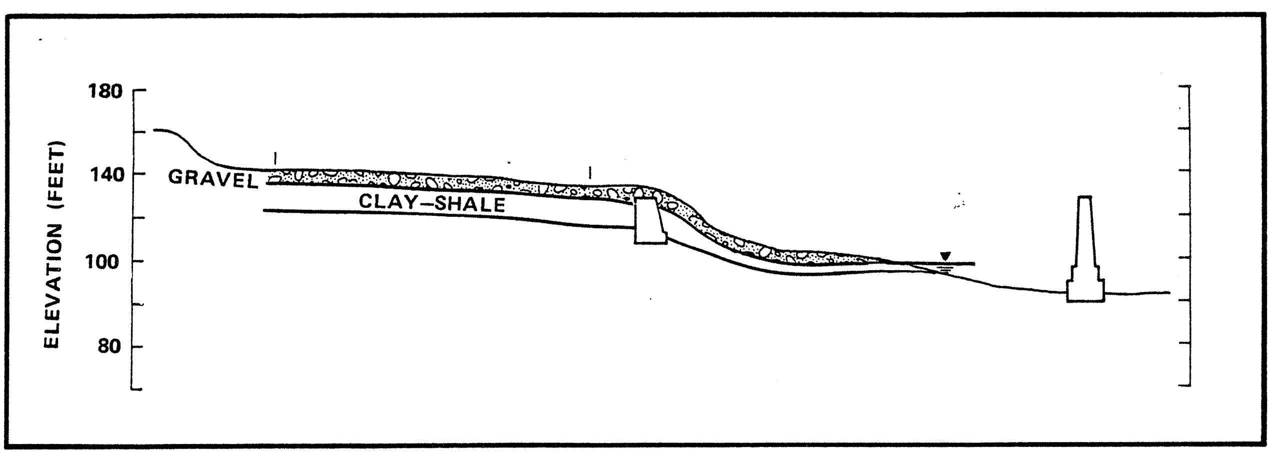

FIGURE 7 SMITH BRIDGE – CROSS SECTION

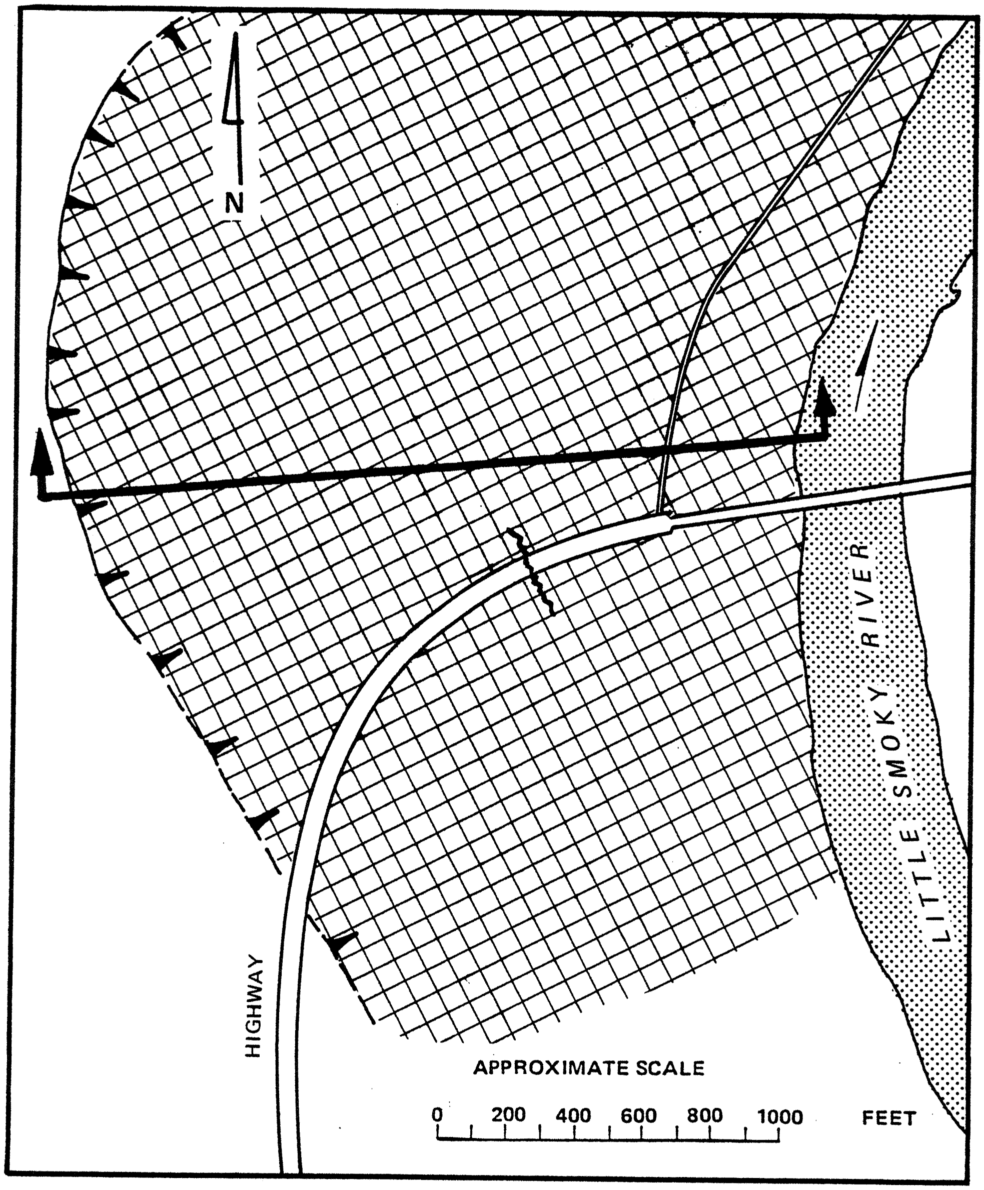

FIGURE 8 LITTLE SMOKY SLIDE AREA – SITE PLAN

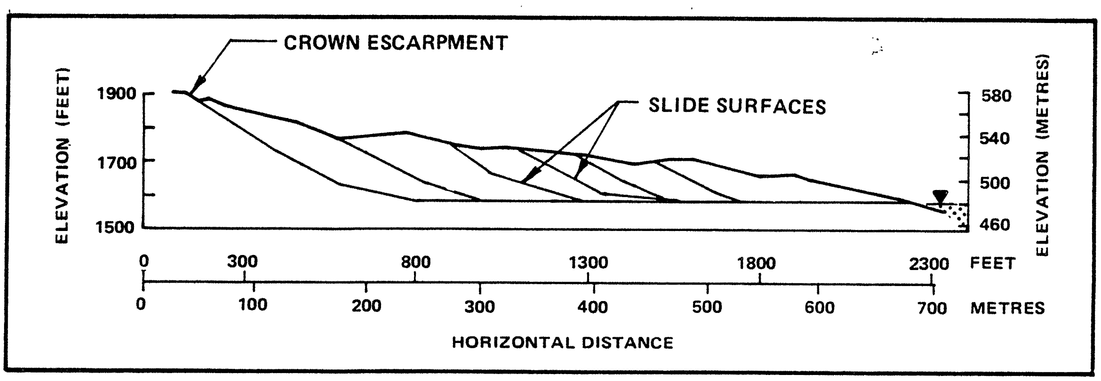

FIGURE 9 LITTLE SMOKY RIVER STRATIGRAPHIC PROFILE (HAYLEY)

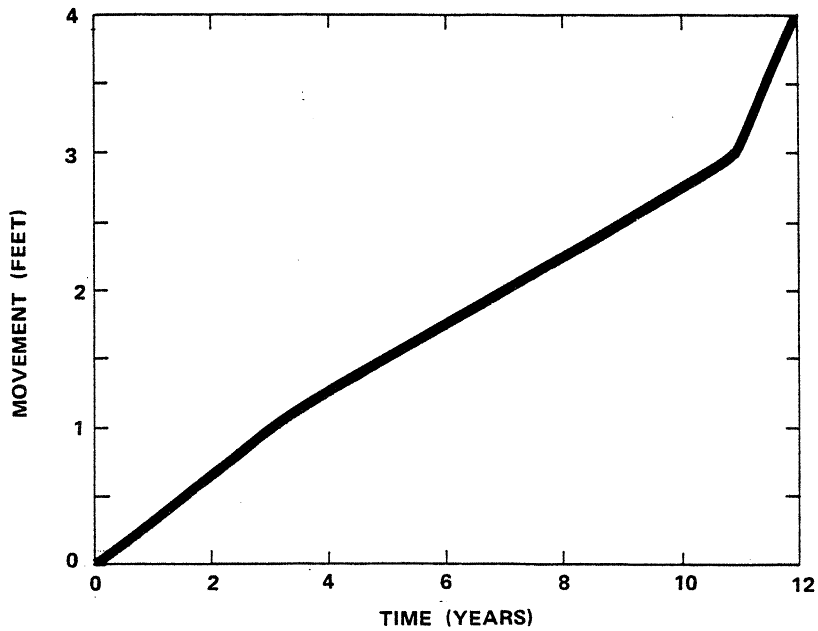

FIGURE 10 CREEP MOVEMENTS
SMITH BRIDGE, ATHABASCA RIVER – (1944-1957)

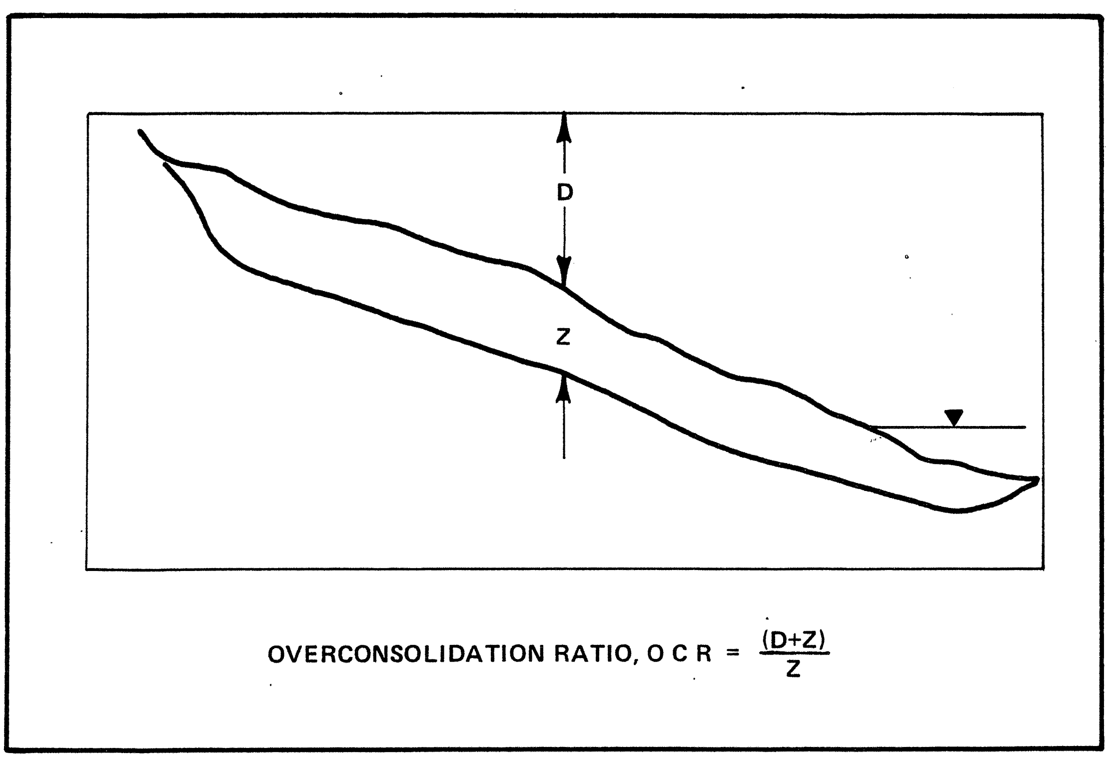

FIGURE 11 DEFINITION OF OVERCONSOLIDATION RATIO

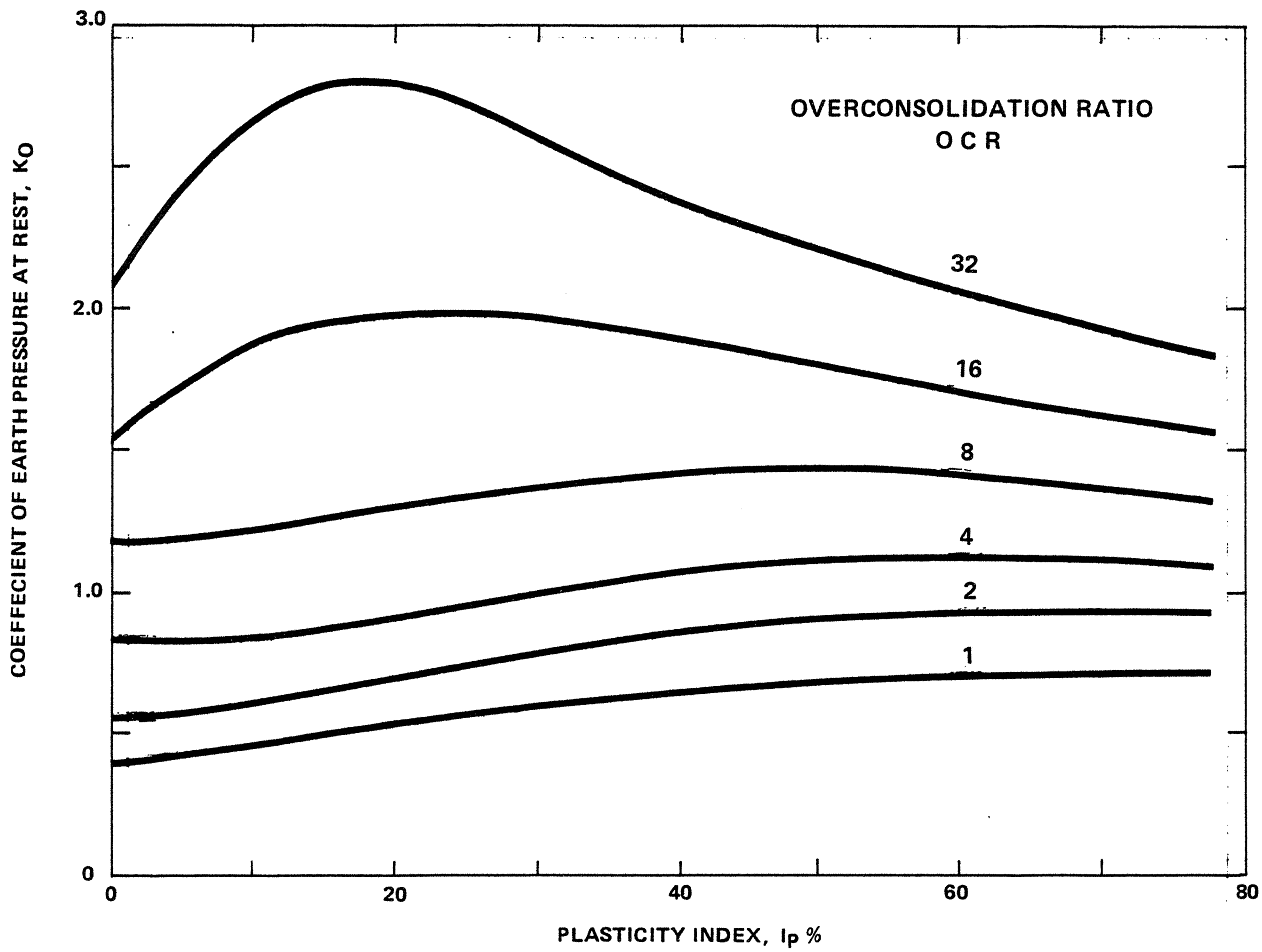

FIGURE 12 RELATIONSHIP BETWEEN K_O, I_p AND OCR

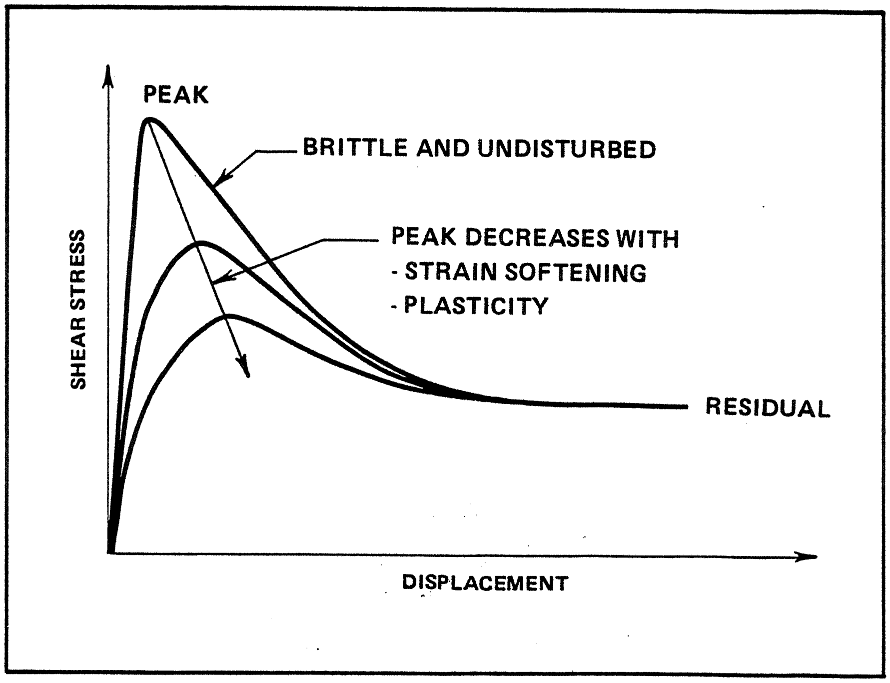

FIGURE 13 PEAK AND RESIDUAL RELATIONSHIP

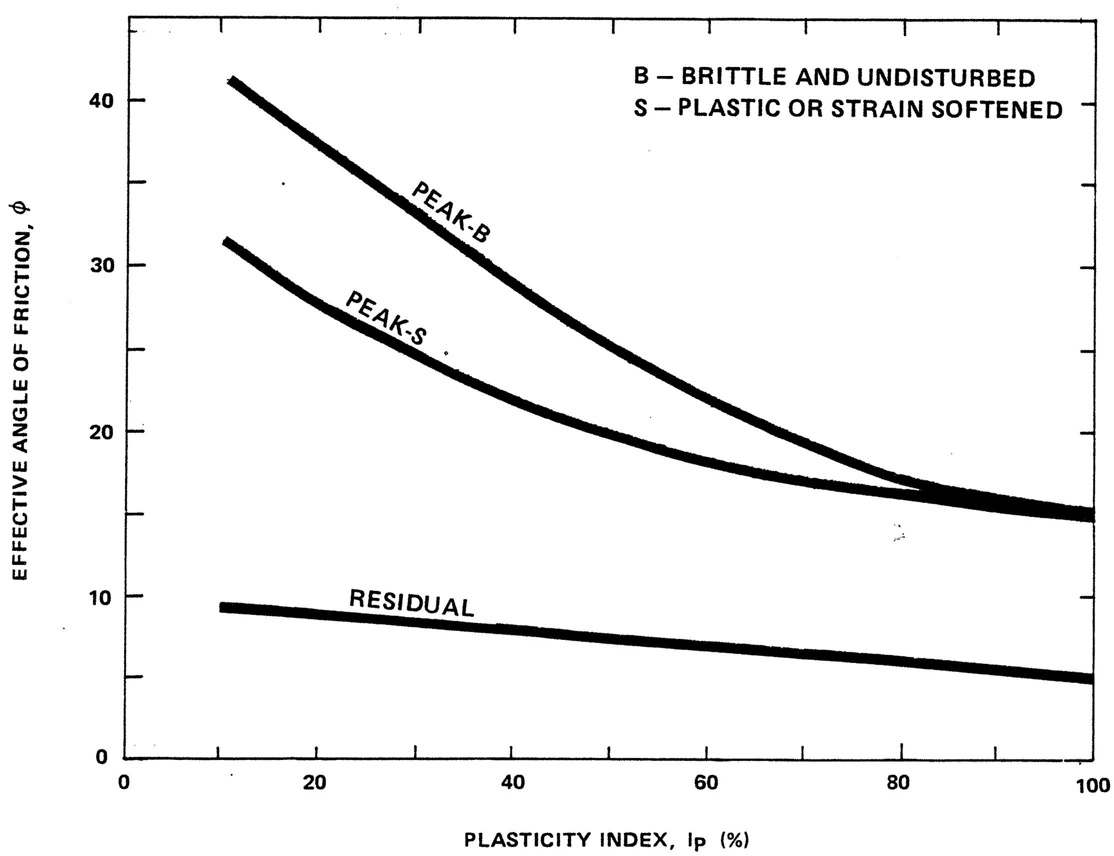

FIGURE 14 FRICTION ANGLE vs. I_p – (PEAK AND RESIDUAL)

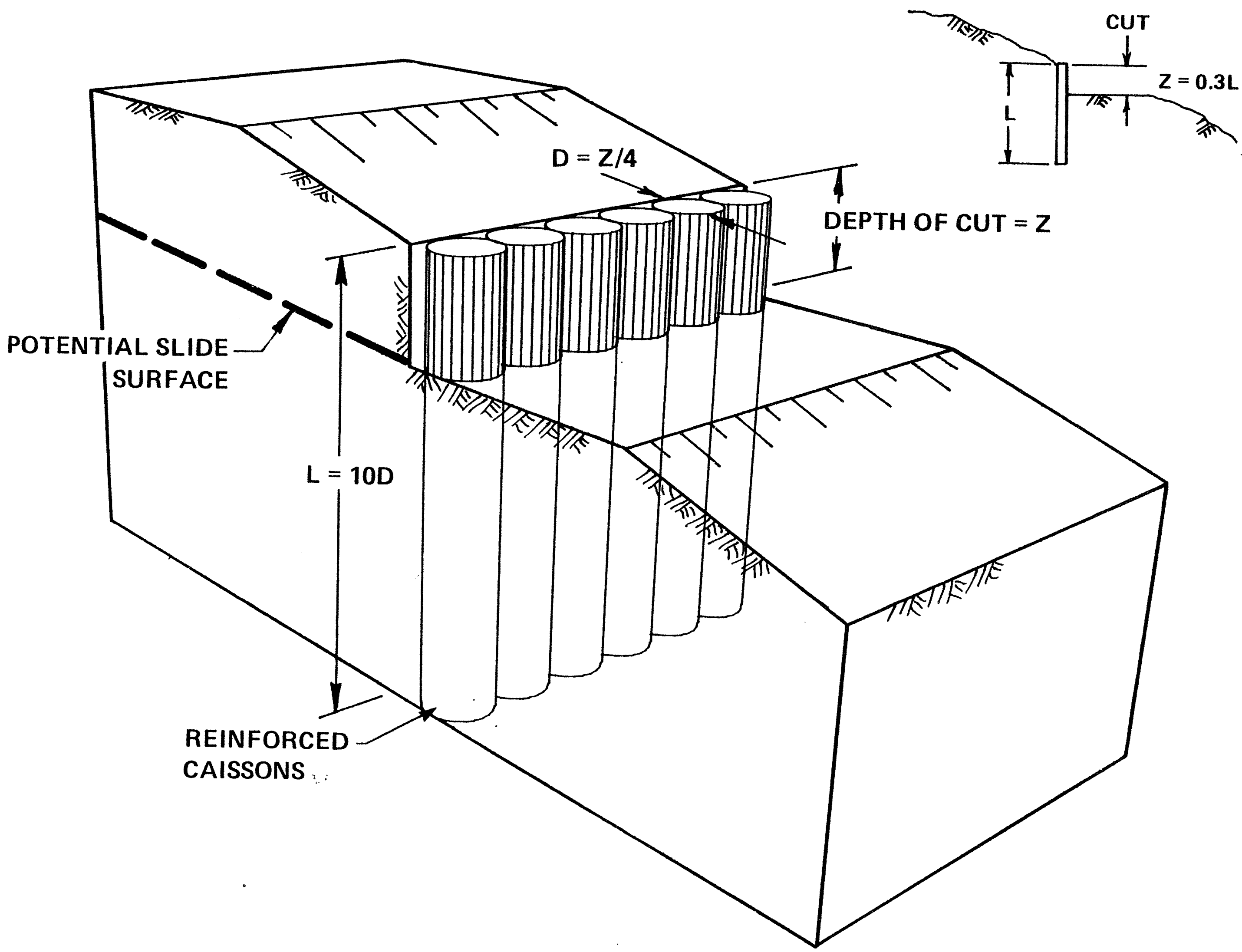

FIGURE 15 STABILITY MAINTENANCE BY TANGENT CAISSON WALL (SEATTLE—TACOMA)

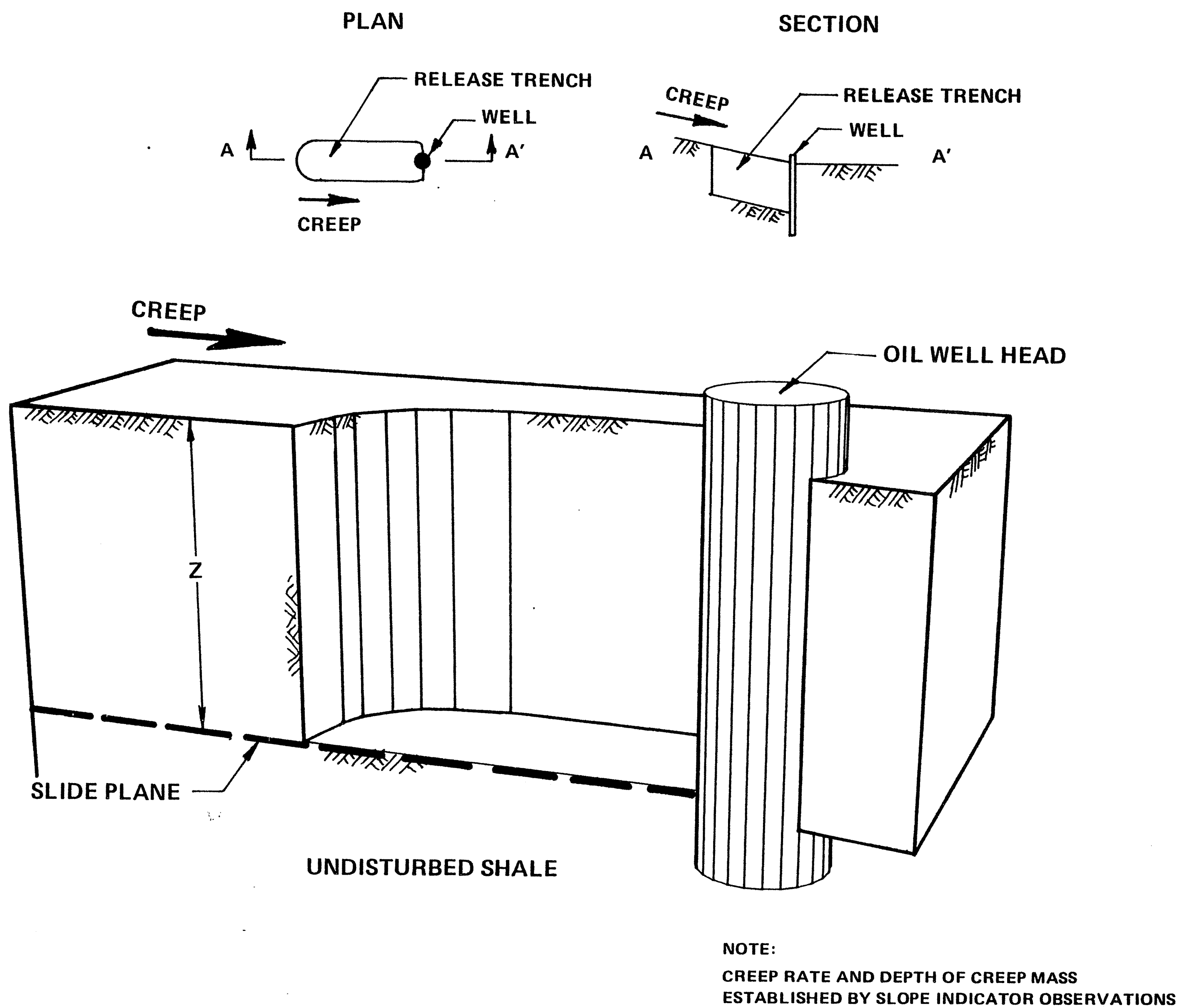

FIGURE 16 LOAD RELIEF BY SLURRY TRENCH (SWAN HILLS)