University of Nevada
Reno

An Engineering Analysis of the
May 1983 Rock Slope Failure on
Slide Mountain, Nevada

A thesis submitted in partial fulfillment of the
requirements for the degree of
Master of Science in Geological Engineering

by

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February 1986
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Dedication

To my Mom and Dad,

You stood beside me in everything I did, your love and understanding have been with me for as long as I can remember.
Acknowledgements

I would like to thank Dr. Bob Watters, Dr. Gary Norris, and Dr. Burt Slemmons without whose advice in both the field and office phases, none of this thesis would have been possible. The data gathered by the students attending the 1983 Geological Engineering field camp was of incalculable value, and I thank them for their help and companionship.

I would also like to thank the Mackay School of Mines for providing me with financial aid during my graduate studies, and for the laboratory and other equipment used in this investigation.

Thanks is due my present employer—Sergent, Hauskins and Beckwith—for allowing me to use their telephones and computer equipment during the crucial computer analyses phases of this work.

A very special thanks is offered to my wife Kate for her encouragement during the down times and the many hours she spent editing the rough drafts and drafting figures.

I would also like to thank Teresa for her amazing ability to turn my unintelligible scribble into neatly typed pages.

Thanks are due to so many of my friends and family for their help and support. I will not attempt to thank them all by name but hope they will know how I feel.
Abstract

On May 30, 1983 a rockslide occurred on the east side of Slide Mountain, Nevada. Toe heave and slide debris falling in Upper Price Lake caused it to overtop the earth dam at the east end of the lake and flow down Ophir Creek, destroying or damaging another small dam, roads, houses, and other property in its path.

Field and laboratory studies were performed shortly after the slide in order to determine what may have caused it. Slope geometry, joint patterns, and the mechanical properties of the rockmass were determined through direct measurement and verified, where appropriate, by empirical relations from the literature.

Several methods of analysis were considered, and it was decided that the method of slices for both planar and circular failure surfaces, finite element analysis, and toppling failure analysis would be used to analyze the slope. In order to determine what effects various shear strength values would have on the slope as well as attempt to determine the most critical water table elevation, a number of runs of each type of analyses were done. The slope was modeled using shear strengths that included the highest realistic values, the lowest values, and the estimated average or most likely values. For each of the
three shear strengths several analyses were performed. The piezometric surface was successively raised to higher elevations until the factor of safety equaled about 1.0, failure was presumed to occur at this water level.

It was found that the slope most likely failed along a shallow planar surface with a tension crack at its head and, due to the fact that the failure surface did not daylight anywhere in the slope, a substantial amount of buckling (or toe heave) occurred near the toe of the slope. Water was probably high in the slope, but was not so high as to be at the ground surface in the upper two-thirds of the slope.

The entire area comprising the upper elevations of Slide Mountain were investigated and several areas were identified where slide movements had taken place. The failure of a major portion of the east face of the mountain, of which the current slide occupied only a small part, was evident from the obvious massive slide scar and had been identified by previous investigators. The current investigation identified a smaller but significant area on the north face of the mountain that had been previously identified as a glacial cirque. This area was probably oversteepened by glacial activity, but may have had some relatively large wedge failures occur since the recession of the glacier sometime in the Pleistocene.
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CHAPTER 1
INTRODUCTION

1.1 Introduction

At about 11:53 A.M. on Monday, May 30, 1983, a landslide occurred on the east side of Slide Mountain above Upper Price Lake in the Carson Range on the Nevada side of Lake Tahoe (see Figure 1.1).

A portion (about 4%) of the 2.3 million cubic yards of slide debris came to rest in Upper Price Lake (see Figures 1.2 and 1.3 for before and after aerial photos). This slide debris, in conjunction with as much as 15 feet of toe heave in the lake bed, forced the approximately 24 acre feet of water in the lake to overtop the small dam at its east end in what may have been a single pulse of water up to 35 feet high. After destroying the earthfill dam, the wave continued down the main Ophir Creek drainage where it overtopped another small earthfill dam at Lower Price Lake, thus adding another acre foot of water to its volume. It then rushed down the very steep Ophir Creek Canyon picking up rocks and other debris, possibly changing from its original waterflood form into what might be termed a debris flow, and emptied onto a relatively flat alluvial fan in Washoe Valley. As a result of the sharply reduced gradient the debris flow dropped its load of rocks and sand, and thus buried several houses, cars and trucks, and an abandoned school bus. It knocked down power poles causing local power
FIGURE 1.1
Location of Slide Mountain
FIGURE 1.2 Aerial photo of the east side of Slide Mountain before slide occurred
FIGURE 1.3 Aerial photo of the east side of Slide Mountain after slide occurred
outages, completely blocked old U.S. Highway 395, and killed one person while injuring several others.

It is estimated that total property damage easily exceeded $2 million, including an estimated $815,000 in highway repairs and related costs (Nevada Dept. of Transportation, 1983), as well as the replacement or repair of a number of homes.

This is ample evidence that the time and money spent studying the underlying causes and effects of landslide activity is easily justified and that close attention must be paid to the consequences of landslide activity on any water retaining facilities, no matter how small they may be.

1.2 Scope of this Investigation

The main purpose of this study was to investigate the Ophir Creek Landslide in order to determine what factors were primarily responsible for the failure. It was decided that in order to better understand this landslide it would be necessary to look at Slide Mountain as a whole and thus relate this relatively small landslide to a more general understanding of the degredation of this rather unique mountain.

The most important aspects of any slope stability investigation are the determination of:

1. the insitu stresses,
2. the geometry of the slope,
3. the mean orientations of the joint sets within the
rock mass, and
(4) the mechanical properties of the rock mass.
(Duncan and Goodman, 1968; Morgenstern, 1976.)

The insitu stress is only needed if an analysis such as
the finite element method is contemplated. Field tests can
be performed to determine these stresses, but are expensive
and difficult to accomplish. Analytical methods have been
developed to aid in the estimation of insitu stresses
(Naylor and Pande, 1981) and these will be discussed later.
Field observation and lab tests can be employed in the
determination of the other factors and these are discussed
below.

1.2.1 Field Studies

A few weeks after the slide occurred an extensive
topographic survey was performed using standard surveying
equipment as well as a plane table and alidade. The area
from the head of the release zone to the failed dam at Lower
Price Lake, including all areas between these two points
which were effected by either the landslide activity or
resultant flooding, were mapped using the approximate
elevation of the original lake level (as taken from the USGS
7 1/2 minute series, Mt. Rose Quadrangle, 1968) as the
assumed control elevation. In conjunction with the mapping,
reference points were located on the slide mass and
referenced to stationary points in order to monitor possible
future movements.
During this same period many joint orientations were recorded from the available outcrops in and near the slide area. Samples of soil and rock were taken for later lab testing, pits were dug in the sandslide to determine its depth for later volume calculations, and observations were made concerning the general physical appearance of the failed mass and surrounding area. 

During the middle and late summer and early fall of 1983, periodic trips were made to Slide Mountain to gather data and make observations. Joint orientation data and other structural and geological information for most of the remaining areas of the mountain above the elevation of about 7200 feet was gathered. Several theodolite and electronic distance meter traverses were made through the reference points on the slide mass in order to determine if any movement was occurring.

1.2.2 Laboratory Work

Several rock samples were tested (with and without joint fill material), using the portable shear box, in order to determine their mechanical properties.

Samples of soil from the Tree Slide and Sandslide as well as samples of joint fill and fault gouge (see Figures 4.2 and 7.1) were classified using grain size analyses, hydrometer tests, and Atterberg limits determinations. The soil samples from the sand slide were tested to determine their maximum and minimum dry densities. These were then
used in conjunction with field determined values of their pre-failure in situ density in order to calculate the pre-failure in situ relative density of these sands.

Where possible, ASTM standards (Annual Book of ASTM Standards, 1983) were followed. The results of all laboratory tests are found in Appendix A. The results of these tests were used to characterize the materials in the slide mass and other areas of interest. However, it should be noted that much of the test data was used as general background information and, although all test results are listed in the appendix of this report, may not be directly referred to in this text.

1.2.3 Data Analysis

Structural and topographic data were used to construct cross-sections of the pre-failure slope. These cross-sections were appropriately digitized and this information, in conjunction with the material properties, was used as input for various slope stability calculations as well as in a finite element model of the slope. The results of the various calculations were then compared to field observations regarding the nature and extent of failure to determine which failure mechanism prevailed.

Data gathered in other areas of the mountain were used to facilitate investigation of the landslide as well as identify other possible past or future failure areas and their most likely modes of failure.
CHAPTER 2

CULTURAL HISTORY

2.1 A Brief History of the Slide Mountain Area

There is scant evidence of human activity in Washoe Valley (east of the Carson Range at the foot of Slide Mountain) prior to the mid 1800's. Though it can be assumed that various indian cultures were active in the area long before this time, they left no permanent records of their activities. Fur trappers probably penetrated the region in the 1820's or 30's and in the 1840's emigrant parties came through the valley, but none stopped to settle.

In 1852 a man named Clark built a small cabin near what was later to become Franktown and thus became the first white settler in all of what is now Washoe County (Angel, 1881). Over the next few years he was joined by two or three others when, in 1854, a number of Mormon settlers came to the valley and founded Franktown. In that same year, Elder Orson Hyde began construction on the first of many sawmills in the area (Angel, 1881).

With the discovery of the silver and gold deposits of the Comstock Lode in the Virginia range east of Washoe Valley, the population in the valley increased greatly. In 1861 an ore stamp mill was established on Ophir Creek by J. H. Dall and by 1868 there were a total of 15 saw mills and 10 stamp mills in or near the newly established camps of Ophir City and Franktown (Angel, 1881). But, with the
coming of the Virginia and Truckee (V & T) Railroad in 1869, it became cheaper to ship the ore to the Carson River stamp mills and Ophir City and Franktown became nearly abandoned ghost towns.

The key to all this mill activity was the great abundance of water and timber found on the flanks of Slide Mountain. As early as 1860 water was directed from the Tahoe Meadows on the west flank of the mountain, into Lake Sophia (later known as Price's Lake, then Upper Price Lake) then down the Ophir Creek drainage, via ditches and flumes, for use by the steam driven saws in the sawmill located at the outlet of Ophir Creek in Washoe Valley. The boilers at the sawmill were fired by cordwood and required the services of hundreds of woodcutters working on or near Slide Mountain to keep the fires burning (Ratay, 1973). Even after the completion of the V&TRR and the death of the ore mills in the valley, the flanks of Slide Mountain were still a very important source of timber for the mines in the Comstock. Thus the sawmills operated until well into the late 1800's (Ratay, 1973).

2.2 Repeated Flooding on Ophir Creek

In late January of 1874, warm winds and rain on a heavy snowpack caused Lake Sophia to overtop the natural dam that created the lake, wash it out and flood the valley below, destroying the flumes down Ophir Creek but doing little other damage (Goodwin, 1977). William E. Price, who had
recently acquired the sawmill at Ophir Creek, rebuilt the
dam at Price's Lake (Lake Sophia), raising the water level
slightly. He built what was described as a "flimsy, hastily
constructed sand and rock dam" (Goodwin, 1977) and
reconstructed the "V" flumes which he used to transport
timber, cordwood and ice cut from the lake down to his
lumber and wood dump on the V&TRR line on Ophir Creek.

In the following year rapid snowmelt due to Chinook
winds over the snowpack caused the newly constructed dam at
Price's Lake to again be overtopped and destroyed. The
waters rushing down Ophir Creek once again washed out
Price's flumes and once in Washoe Valley, destroyed the V&T
trestle over Ophir Creek, closing the line for 24 hours
(Goodwin, 1977).

The dam was again rebuilt while another small dam was
constructed just below Price's Lake in a small depression in
the Ophir Creek drainage. These dams and the lakes that
would later be called Upper and Lower Price Lakes, working
in conjunction with the newly reconstructed flumes, were
utilized to supply water, timber and ice without further
incident until the summer of 1890. On July 6th of that year
at 4:45 P.M. melt water from the late thaw of the "Winter of
the White Death" caused the dam at Upper Price Lake to again
be overtopped and breached. Price's nephews, working on the
V-flume below the small lower lake then called Little Price
Lake (now called Lower Price Lake), were barely able to
escape the on-rushing waters with their lives. They later described the floodwave as being some 30 feet high and 100 feet wide as it churned down the canyon (Goodwin, 1977).

Apparently the dams were rebuilt again and served a similar function for a number of years. In 1972 the dam at Upper Price Lake was also being used, as it had been since the early 1860's, to divert water from Ophir Creek into the drainage of Franktown Creek. But at this time a modern concrete diversion ditch and a large diameter steel pipeline replaced the wooden flumes and earthen ditches. This water was used solely for agricultural purposes. The dam was again destroyed in the 1983 landslide/flood and at the time of this writing it is again being rebuilt using rock gabions in place of the original earthfill dam. This dam will raise the water level in Upper Price Lake enough so that the concrete ditch and pipe can again be utilized to divert water into the Franktown drainage for irrigation purposes.

2.3 Historic Landslide Activity on Slide Mountain

The early Washoe and Paiute Indians familiar with Slide Mountain gave it a name which translated means "Mountain Which Fell in Upon Itself." In the late 19th century Indians told the story of the ground rumbling and heaving and the entire side of Slide Mountain falling into the valley below (Earl, 1978).

An article in the Washoe Weekly newspaper dated June 10, 1865 (dateline, "Mount Avalanche") belatedly tells of a
huge slide occurring in late November or early December of 1852. The article reports that two men living in Genoa, located in Carson Valley some 30 miles south of Washoe Valley, heard and felt the slide and saw a dust cloud rise thousands of feet into the air. The article goes on to say that five out of the eight emigrants that were crossing the "ancient pass" at Ophir Creek when the slide occurred were buried, as were several cabins near the pass.

This story has never been verified by any other independent sources. It would seem strange that the stories of the three emigrants who escaped the slide would not be recorded elsewhere. One would further have to question why emigrants would be crossing into the Lake Tahoe Basin in the winter months. Also, since the first settler in Washoe Valley reportedly arrived in 1852, it would seem very improbable that any cabins existed near the "ancient pass" at Slide Mountain. However the testimony of the two men in Genoa and the Indian stories would lead one to believe that a slide did occur, possibly in the early 1850's, that was of fairly large proportions.

A story is also told by a man in a woodcutting camp near Slide Mountain of a large slide occurring in the early 1860's (Ratay, 1973). This slide is said to have buried the buildings in the camp early one morning, the men escaping with their lives due to a Chinese cook hearing the slide and warning them by ringing the dinner bell.
A few years later Mark Twain, at the time a writer for the Virginia City Territorial Enterprise, wrote a humorous short story based on a landslide that purportedly occurred on Slide Mountain (Ratay, 1973).

Thus there was a great deal of talk about landsliding on Slide Mountain in the 1850's and 60's, but little of it has been directly substantiated. However, it should be noted that other sources reported a large earthquake (Slemmons, 1965) with an estimated Richter magnitude exceeding 7.0 occurring in the year 1845 or 1852. If this large quake did occur in 1852 it is entirely possible that it could have initiated a large landslide on Slide Mountain.

Also, an earthquake that was reported as being very violent (Richter magnitude estimated to be 7.0-7.5) in Carson City (about 15 miles south of Slide Mountain) occurred in March of 1860 (Slemmons, 1965). It is possible that this event may have precipitated the slide reported by the woodcutter.

In the decades since these earthquakes occurred many other tremors have shaken Washoe Valley and Slide Mountain but all have been of lesser magnitudes (the greatest being 6.7 in 1869) (Slemmons, 1965) and no slide activity has been reported on Slide Mountain until the most recent slide in May of 1983.**

** In connection with this, it should be mentioned that a check of the records of the strong motion seismograph located on the east side of Washoe Valley in the Virginia Range revealed that absolutely no earthquake activity was detected on the day of the latest slide (Ryall, 1983).
Thus it is not possible to state with any degree of confidence when previous historic slide activity occurred on Slide Mountain. However it does seem reasonable to conclude that a very large slide occurred sometime in the 19th century prior to 1860 and that possibly another, smaller slide may have occurred in that year.

The following sections on the geology of the area will discuss evidence for numerous slides in the recent geologic past. But no geologic evidence has yet been uncovered that corroborates the rather limited historic evidence for sliding.
CHAPTER 3
GEOLOGY

3.1 General Location and Geographic Description

Slide Mountain is located near the western border of the state of Nevada, about 6 miles northeast of Lake Tahoe and 20 miles southwest of Reno (see Figure 1.1). It is on the northern end of the crest of the Carson Range, an offshoot of the main Sierra Nevada near the California-Nevada border. The Carson Range splits off from the main Sierra crest at the southern end of Lake Tahoe and is bounded on the west by Lake Tahoe, on the west and north by the Truckee River Canyon and on the east by Washoe Valley. This range is in the transition zone between the Sierra Nevada and the Basin and Range Geomorphic Provinces, but is considered to be within the latter.

The range also is the dividing line between two radically different climatic zones. To the west is the main crest of the Sierra Nevada with greater than 30 inches of precipitation per year being common (Moore, 1952), and farther to the west are the lush Sierra foothills and the highly productive agricultural areas of the Sacramento Valley of California. East of the Carson Range rainfall is scarce due to the rain shadow effect and seldom exceeds 10 inches per year (Moore, 1952). Vegetation is generally sparse and is characterized by Pinon-Juniper forests and sagebrush.
At 9,698 feet in elevation, Slide Mountain is one of the higher peaks in the range. Its flanks drop roughly 5,000 feet to Washoe Valley on the east and to the depths of Lake Tahoe on the west. The eastern flanks of Slide Mountain (as well as the rest of the Carson Range) are drained by a series of small creeks whose waters eventually reach Pyramid Lake (about 40 miles to the northeast) via the Truckee River.

3.2 Geologic History

Sediments deposited in the Cordilleran Eugeosyncline were folded and metamorphosed in the Jurassic-Triassic periods (Moore, 1952). These were then intruded and contact metamorphosed by granodiorite in the late Cretaceous period (Tabor, et al., 1983). Both the metamorphosed sediments and the granodiorite were cut by aplite and pegmatite dikes shortly after the intrusion of the granodiorite (Moore, 1952).

No discernable events occurred until the eruption of the Kate Peak andesite in the late Tertiary period (Moore, 1952). The Kate Peak andesites reached a total thickness of as much as 1200 feet, having erupted from a series of vents along the present crest of the Carson Range. During this same period the Truckee sediments were laid down, having been derived from the erosion of the pre-Jurassic metamorphased sediments (Moore, 1952).

After a great deal of faulting and eroding of the Kate
Peak andesites, the Lousetown Basalts were extruded during the late Pliocene–Pleistocene. Contemporaneous with the latter period of volcanic activity (early Pleistocene) a period of major upwarping and faulting occurred (Thompson and White, 1964). Thus, after much erosion aided by at least four distinct periods of glaciation (Tabor, et al., 1983), the granodiorite core of the range was extensively exposed in the central and southerly portions of the Carson Range as well as in the main Sierran crest.

3.2.1 Regional Structure

Regional upwarping in conjunction with predominantly north-south normal faulting are thought to be the agents which formed the Carson Range.

There is a great deal of evidence pointing to the occurrence of some large offsets due to faulting. Along one fault in the vicinity of Slide Mountain, the tertiary volcanics have been displaced as much as 500 feet (Moore, 1952) downward to the west while, across another fault, an offset variously estimated at from 1000 (Thompson and White, 1964) to 3800 feet (Moore, 1952) down to the east has been reported. Thus, it cannot be denied that faulting has played an important part in the formation of the range. However, since a large number of the faults are antithetic (Thompson and White, 1964), a net uplift of the range could only be possible if a general regional uplift occurred. There is evidence of a regional "doming" of the granitic
core of the range (Thompson and White, 1964), and it is postulated that horizontal compressional forces caused this doming.

3.3 Landslide Activity

The landslide activity on Slide Mountain is thought to have begun in the early Pleistocene glacial period and has extended until the present. The old landslide deposits, which can be recognized by their morphology of flats and escarpments and/or by their hummocky topography and the concave scar at their head, have been divided into nine distinct sliding episodes by comparing the differences in weathering of the slide debris as well as carbon 14 dates (Tabor, et al., 1983).

The debris was originally thought to be of glacial origin (Jones and Gianella, 1933), but closer examination revealed their true nature.

It was observed that no evidence exists to indicate that any glaciers in the area went below 6400 feet (in fact most did not get below 8,000 feet) (Moore, 1952), while the debris from Slide Mountain extends to an elevation of about 5,000 feet. Even if this postulated glacier had a large source area (say, Tahoe meadows at the head of Ophir Creek) and thus could have reached the lower elevation, it would have deposited some fragments of the Kate Peak andesites found in the upper Ophir Creek drainage (Moore, 1952).

However, the debris is composed entirely of granodiorite
rocks, and sands and gravels of granitic composition (Tabor, et al., 1983).

Also, the total volume of the debris is roughly equal to the volume of material required to reconstruct the postulated original contours of Slide Mountain (Moore, 1952). Thus all evidence leads to the conclusion that the debris tongue is in fact due to landslide activity originating on the east face of Slide Mountain. Incidentally, had the total volume of the debris, about 125,000,000 cubic yards (Moore, 1952), been deposited in a single episode, it would be one of the 20 largest slides ever identified (see Table 3.1).

Most investigators agree that the eastern side of Slide Mountain has been rendered relatively weak by tectonic movements which have crushed the granodiorite and caused unfavorably oriented joint sets with relatively small joint spacings. This, in conjunction with the oversteepening caused by tectonic activity or, possibly the downcutting action of Ophir Creek, has created conditions favorable to the initiation of large rock slides (Tabor, et al., 1983).

3.4 Structural Geology of Slide Mountain

A large scale geologic map of the area near Slide Mountain is presented in Figure 3.1. A more detailed geologic map of the upper area of Slide Mountain (outlined in Figure 3.1) is presented in Figure 3.2, while a composite aerial photo of approximately the same area before the most recent sliding occurred is shown in Figure 3.3.
## Table 3.1: Landslide Volumes*

<table>
<thead>
<tr>
<th>Locality of Event</th>
<th>Volume ($10^6$ yds.$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tsiolkosky (moon)</td>
<td>1,500,000</td>
</tr>
<tr>
<td>Saidmarreh (Iran)</td>
<td>25,500</td>
</tr>
<tr>
<td>Flims</td>
<td>15,000</td>
</tr>
<tr>
<td>Engleberg</td>
<td>3,800</td>
</tr>
<tr>
<td>Pamir</td>
<td>2,500</td>
</tr>
<tr>
<td>Tamins</td>
<td>1,600</td>
</tr>
<tr>
<td>Siders</td>
<td>1,200 to 2,500</td>
</tr>
<tr>
<td>Fernpass (Austria)</td>
<td>1,300</td>
</tr>
<tr>
<td>Glarnisch</td>
<td>1,000</td>
</tr>
<tr>
<td>Deyen, Glarus</td>
<td>800</td>
</tr>
<tr>
<td>Blackhawk (U.S.A.)</td>
<td>360</td>
</tr>
<tr>
<td>Vaiont (Italy)</td>
<td>320</td>
</tr>
<tr>
<td>Silver Reef (U.S.A.)</td>
<td>280</td>
</tr>
<tr>
<td>Apollo 17 (moon)</td>
<td>250</td>
</tr>
<tr>
<td>Poschivo</td>
<td>190</td>
</tr>
<tr>
<td>Kandertal</td>
<td>180</td>
</tr>
<tr>
<td>Obersee GL</td>
<td>150</td>
</tr>
<tr>
<td>Slide Mountain</td>
<td>125**</td>
</tr>
</tbody>
</table>

**Notes:**

- * Table adapted from Hsu, 1975
- ** from Moore, 1952. This assumes a single sliding episode. However, it is known that there have been at least nine (the first of which was much larger and traveled much further than any that followed) (Tabor, et al, 1983).
**FIGURE 3.1** Geologic map of the area surrounding Slide Mt.

- Qylz: Younger Quaternary landslide deposits.
- Qd: Older Quaternary debris flow and landslide deposits.
- Qt: Quaternary talus deposits.
- Qgr: Quaternary Glacial (moraine and outwash) deposits.
- Qa: Quaternary alluvium.
- Tkt: Tertiary Kate Peak Formation (miocene). Andesite breccias and flows.
- Kg: Cretaceous hornblend-biotite granodiorite.
- pKm: Pre-cretaceous metamorphic rocks.
- nm: (Area not mapped.)
FIGURE 3.2 Small scale geologic map of the upper part of Slide Mt (area within rectangle on FIGURE 3.1).
(Explanations continued on following page)
Figure 3.2 (continued)

- **Oyl**: Younger Quaternary Landslide Deposits
- **Qd**: Older Quaternary Debris Flow and Landslide Deposits
- **Qt**: Quaternary Talus Deposits
- **Qg**: Quaternary Glacial (Morraine and Outwash) Deposits
- **Qa**: Quaternary Alluvium
- **Kg**: Cretaceous Hornblende - Biotite Granodiorite
  (shaded areas are more weathered due to hydration of micas causing extensive damage to rock)
- **pKm**: Pre-Cretaceous Metamorphic Rocks

- Fault
- Lineation
- Slide Area Boundaries
- Vertical Joint
- Strike and Dip of Joint
- Strike and Dip of Surface with Parallel Shears
  (azimuth and plunge of shears indicated)
- Soil Slump/Tension Cracks with Approximate Direction of Movement Indicated
- Striation in Joint Fill, Highly Weathered Rock or Soil with Approximate Direction of Movement Indicated
- Direction of Lean of Trees in Failed Slidemass
FIGURE 3.3 Aerial photo of the upper part of Slide Mountain before recent slide occurred.
Figure 3.1 indicates that the faults that created Little Valley (a graben) to the south of the slide area appear to line up with the faults north of the slide area. It has also been noted that a bend in the fault pattern seems to occur in the immediate area of the slide (Tabor, et al., 1983). This would indicate the possibility of a stress concentration in this area which could account for the relatively large number of faults and topographic linaments mapped in the vicinity of the slide. These features would be a resultant of the higher degree of crushing or jointing produced by the higher stresses. It should also be noted that there is another possible slide area indicated on the map on the north side of Slide Mountain.

3.4.1 Determination of Joint Set Orientations

In order to determine the joint set orientations on Slide Mountain, joint orientation data on more than 1700 joints at over 70 locations on or near Slide Mountain were collected. This data was then entered into a computer data file grouped by location. A program was written that would allow the data for the various locations to be grouped as desired and automatically stored as a data file suitable for use in the program PATCH (Matab, et al, 1972). This program can then calculate the mean joint set orientations and other related information.

Time can be saved by using a computer routine to identify joint clusters and calculate "average" joint
orientations for use in models of jointed rock. PATCH is just such a program.

PATCH takes the field data on dip and dip direction and translates this into the direction cosines of a unit vector passing through the upper hemisphere of a unit sphere. This hemisphere has been divided into "patches" by constructing, mathematically, nine bands between the equator and the pole. These bands are, in turn, divided into a total of 100 quadralateral patches. Thus, by counting the number (or density) of joint normals which pass through each patch and comparing this density to the expected theoretical density calculated using a Poisson random distribution, patches with non-random clusters of joint normals can be identified. Adjacent patches with greater than the expected random density are combined in a suitable manner and thus the members of a joint set or cluster can be identified.

Next the program calculates the mean joint orientation using vector addition and assumes the standard normal distribution in order to construct a 95% probability cone of confidence about the calculated mean joint orientation. The program then checks the members of the set to determine if the set fits a hemispherical normal distribution (as developed by K. J. Arnold, 1941). To do this it calculates the chi-square statistic for the set and compares it to the expected chi-square value for a hemispherical normal distribution at the 95% significance level. If the
chi-square test is passed then a confidence interval is calculated for the mean dip and dip direction. The map in Figure 3.2 illustrates the areal groupings used in the joint orientation analysis.
CHAPTER 4

TYPE OF SLIDING

and

DEFINITION OF SLIDEMASS GEOMETRY

4.1 Landslides

Mass wasting or mass movement is one of the many slope forming processes that have been identified by geomorphologists. However, the importance of mass movement may sometimes be overlooked in the study of geomorphology due to the uniformitarian concepts of geology (Sharpe, 1938), despite the fact that landslides measured in cubic miles rather than feet or yards have been identified or witnessed in many places in the world and recently a landslide of unimaginably large proportions (about 280 cubic miles!) has been identified on the surface of the moon (Hsu, 1975) (see Table 3.1).

As early as 1875, geologists in Europe attempted to classify landslides. Many useful classification systems were developed but they were generally only applicable to a small region and invariably were incomplete. It was not until 1925 that Terzaghi developed a widely applicable system which, for the first time, recognized the great importance of water in the initiation and subsequent behavior of landslides. Unfortunately, this system was never really accepted nor was it used on a widespread basis (Sharpe, 1938).
Sharpe, in his book *Landslides and Related Phenomena* (1938, revised in 1960), presented a classification system which was widely accepted and is used today essentially in its original form. However, Sharpe used the term "landslide" as one of his major classification headings and thus suggests that the more general term mass-movement be used in place of landslide.

He described mass-movement as the downslope movement of soil and/or rock under the influence of gravity without the aid of any transporting medium. He goes on to point out that mass-movement grades into mass-transport as water content is increased.

4.1.1 A Mass-Movement Classification System

Sharpe based his classification system on three basic characteristics of the mass-movement:

1. type of movement (slide or flow),
2. water content, and
3. type of material involved.

First, and most important, he divided mass-movements into two categories, slide or flow, depending on whether a definite slip plane exists which separates the moving slide mass from stable ground or no definite slip plane exists and failure is seen more as a continuous viscous or plastic flow. These two types of failure may be present, to one degree or another, in any given case of mass-movement but it should be classified based on the predominant mode.
These two basic categories were then subdivided into other classes based primarily on water (or ice) content and secondarily on the type of material involved. Table 4.1 lists the various types of mass movements and briefly explains the criteria used for categorization.

Many of the types of mass-movements found in the table do not have a significant bearing on the mass-movement being investigated here. However, a few of the types listed are significant and will be commented on in the following paragraphs.

First, the mass-movement that occurred on Slide Mountain should, indeed, be classified in the general category of slide (or landslide) according to Sharpe's system. Field evidence suggests that failure occurred along a distinct plane (or planes), while the size of the blocks in the slide debris (see Figure 4.1) would indicate that water had no effect on the mass once sliding was initiated and was a small percentage of the total volume of the slide. However, as the schematic diagram depicting the slide area after the failure (Figure 4.2) indicates, there were several sliding modes which occurred at essentially the same time in different locations within the failing mass. There is a rather large volume of material on the upper south edge of the slide mass (next to Ophir Creek) which underwent relatively little movement. This mass moved as three or four large, relatively intact blocks and would thus be best
<table>
<thead>
<tr>
<th>Type of Movement</th>
<th>Rate of Movement</th>
<th>Type and Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow</td>
<td>Slow</td>
<td><strong>Rock-Creen:</strong> slow downslope movement of surficial soil or rock debris, usually imperceptible except to observations of long duration.</td>
</tr>
<tr>
<td></td>
<td>Slow to Rapid</td>
<td><strong>Talus-Creen:</strong> slow downslope movement of surficial soil or rock debris, usually imperceptible except to observations of long duration.</td>
</tr>
<tr>
<td></td>
<td>Slow</td>
<td><strong>Soil-Creen:</strong> water content: dry to saturated (material may include large rock blocks down to fine soil).</td>
</tr>
<tr>
<td></td>
<td>Rapid</td>
<td><strong>Solifluction:</strong> slow movement, generally due to frost action, of saturated rock and soil in alpine or cold regions. Can grade into mudflow.</td>
</tr>
<tr>
<td></td>
<td>Slow to Rapid</td>
<td><strong>Earthflow:</strong> moderate to rapid movement of soil characterized by slumping at its head with flowage of soil to toe causing doming or ridging with crevassed surface. Not fully saturated.</td>
</tr>
<tr>
<td></td>
<td>Rapid</td>
<td><strong>Mudflow:</strong> fully saturated rapid flow of soil and rock mixture. Usually confined to a stream or other channel at steeper gradient than earthflows.</td>
</tr>
<tr>
<td></td>
<td>Slow to Rapid</td>
<td><strong>Debris-Avalanche:</strong> Occurs in humid regions with thick vegetative cover on steep saturated soil slopes. Initiated by a slip at its head. Mowed in long narrow track.</td>
</tr>
<tr>
<td></td>
<td>Rapid</td>
<td><strong>Debris-Slide:</strong> rapid downward movement of unconsolidated earth and debris, no backward rotation, slides or rolls forward forming a hummocky deposit resembling morainal topography. Low water content.</td>
</tr>
<tr>
<td></td>
<td>Slow to Rapid</td>
<td><strong>Debris-Fall:</strong> free falling of earth and debris from a cliff, cave or arch (of minor importance). Low water content.</td>
</tr>
<tr>
<td></td>
<td>Rapid</td>
<td><strong>Rockslide:</strong> rapid movement of newly detached segments of the bedrock sliding on bedding, joint or fault surfaces or any other plane of separation. Low water content.</td>
</tr>
<tr>
<td></td>
<td>Slow to Rapid</td>
<td><strong>Rockfall:</strong> relatively free fall of rock from cliff, steep slope, cave, or arch. Important in talus slope formation. Low water content.</td>
</tr>
</tbody>
</table>

*Adapted from Sharp, 1938
FIGURE 4.1  Photograph of rockslide indicating block sizes
FIGURE 4.2 Schematic diagram of slide area
described as a slump (if rotation occurred), or as Butzer (1976) suggested, a block glide (if no rotation occurred).

The portion of the slide mass in the center of the slide area is composed of large rock fragments which in every case showed numerous freshly broken, unweathered faces. This part of the slide mass may have been a more or less coherent mass when sliding was initiated but very quickly broke up and shattered due to the effects of internal collisions. Thus the predominant character of this part of the mass would necessitate its being classified as a rockslide. However, it might be noted that at the initiation of failure of this portion of the slide (or shortly thereafter), a fair amount of material coming from the upper parts of the source area may well have been in the form of rockfall caused by toppling failures. This point will be clarified in later chapters.

Also, as is noted on the slide area schematic, there is an area which might be classified as a flow. The material found in the extreme north edge of the slide mass, herein referred to as the "sandslide", was predominantly a very fine sand which seemed to have flowed down a low angle slope and over the snow which was on the ground at the time the slide occurred. Thus it is possible that the sandslide should be classified in one of the flow categories (see Section 7.5.1, Liquefaction).

A fourth failure zone, herein called the "tree slide",
is also noted on Figure 4.2. This failure was probably initiated by other slide materials being deposited at its head. This failure could probably be described as a debris slide due to the fact that it involved a brushy, sparsely forested area and, due to the shallowness of the failure surface, a relatively small amount of soil and rock.

As the foregoing discussion would indicate, it is very difficult to classify this landslide using the Sharpe system. This slide would best be called a complex landslide as is suggested in the textbook by Butzer (1976).

The volumes of the various types of mass-movement occurring within the Slide Mountain slide mass were estimated, along with the total volume, and are presented in Table 4.2. These volumes, although very rough estimates, indicate that the slope failure is best described as a complex slump (or block glide) rockslide, since these types of sliding predominated, while the treeslide and sandslide were relatively minor on a volume basis.

4.2 Kinematically Possible Failure Modes

"Kinematics" refers to the motion of a body without specifically knowing the forces which cause this motion (Goodman, 1976). In this case we are interested in determining whether or not it is possible for a slope to fail, given the geometric constraints of the rockmass (structural control) and slope geometry, disregarding the forces required to cause this failure to occur.
Table 4.2: Volumes of Various Components of the Ophir Creek Slide

<table>
<thead>
<tr>
<th>Type of Slide</th>
<th>Volume (yd.(^3) x 10(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump or Block Glide</td>
<td>1470**</td>
</tr>
<tr>
<td>*Rock Slide</td>
<td>650**</td>
</tr>
<tr>
<td>*Tree Slide</td>
<td>120</td>
</tr>
<tr>
<td>Sand Slide</td>
<td>60</td>
</tr>
<tr>
<td>**TOTAL</td>
<td>2300</td>
</tr>
</tbody>
</table>

*Note: Volumes calculated using the average end area method. The total volume of debris coming to rest in Upper Price Lake is 100,000 cubic yards or about 4% of total volume of slide mass.

**It should be noted that, since no drilling was undertaken, it was not possible to determine the thickness of rockslide debris over the underlying slump or block glide and thus the actual volume of the rock slide may be greatly overestimated and that of the slump or block glide underestimated by at least 25%.
A commonly used and extremely practical tool for this type of investigation is stereographic projection. Stereographic projection methods have been used by geologists and engineers to represent three-dimensional planes and lines in space on a two-dimensional surface. It will be assumed here that the reader is familiar with the use of stereographic projection (see Hoek and Bray, 1981) and thus the mechanics of the method will not be discussed.

There are many possible shapes the failure block or slide mass may take, but in most instances four major categories of slope failure geometry are looked at in a slope stability investigation. These are:

1. Planar failure
2. Wedge failure
3. Toppling failure

Planar failure requires a single throughgoing discontinuity that "daylights" or is exposed in the slope face due to its having a lower dip angle than the slope face. This discontinuity must also strike sub-parallel to the slope face. A stereographic projection and a schematic diagram of a slope exhibiting planar failure geometry is shown in Figure 4.3(b).

Wedge failure requires that two (or more) joint sets intersect in such a way that their line of intersection daylights in the slope face. This allows a wedge of
a. Circular failure in overburden soil, waste rock or heavily fractured rock with no identifiable structural pattern.

b. Plane failure in rock with highly ordered structure such as slate.

c. Wedge failure on two intersecting discontinuities.

d. Toppling failure in hard rock which can form columnar structure separated by steeply dipping discontinuities.

Schematic diagrams of the various failure modes with stereographic projections of typical slope geometry (From Hoek and Bray, 1981)
material defined by the planes of the slope crest, slope face and the two discontinuities to fail along their line of intersection toward the free face. A diagram of the wedge formed by two joints is shown in Figure 4.3(c) along with a stereographic projection of this same situation.

Toppling failure occurs when a joint set striking subparallel to the slope face dips steeply into the face as shown in Figure 4.3(d). As can be seen from the figure, this type of geometry allows a sheet or column of material to fail by rotating about the lower downslope edge and thus "toppling" down the slope. A stereographic representation of this type of failure mode is shown in the same figure.

4.3 East Face of Slide Mountain

For the purpose of joint data analysis the east face of Slide Mountain was divided into three areas as was shown in Figure 3.2.

The East Slide Area-South was chosen as a separate study area since this is the area where the most recent slide occurred on May 30, 1983.

The East Slide Area-North was taken separately due to the fact that its face is exposed in a more southerly direction than the remainder of the scar and thus it was concluded that a different mode may control the slope failures evident in this area.

The remaining area, designated as the East Slide Area-Middle, is characterized by a rather deep gully
surrounded by rock cliffs. The area has more solid, vertical outcrop, the majority of which is stained a rust brown.

Figures 4.4(a) through (d) present stereographic projections of the major joint sets for the areas on the east side of Slide Mountain as measured in the field and determined with the aid of the computer program PATCH.

If the entire east face slide area is taken as a whole, without the arbitrary divisions discussed above, it is obvious that the most probable failure mode is a planar one on the low angle joint set number three (see Figure 4.4(a)). However, it should be noted that a secondary toppling mode could occur on joint set two. This joint set would also serve as a release surface should any tensile forces develop on the slope. The tension cracks formed would thus become a highly permeable conduit for surface water to enter which would cause a build-up of hydraulic forces in the slope (see Figure 6.1).

As can be seen in Figure 4.4(b), the relative orientations of the joint sets in the East Slide Area—South are not much different than for the entire East Slide Area. Planar failure can occur on joint set number three and toppling can occur on joint set number one, which would also serve as a tensional release surface. It should be further noted that toppling failure is much more likely in this instance. This is due to the fact that in the overall East
FIGURE 4.4 Stereographic Projections (lower hemisphere) of average joint surfaces, their normals (with 95% confidence interval surrounding normal) and slope face (dashed line) Dip/dip direction are indicated.

(a) East Slide Area—All

(b) East Slide Area—South
FIGURE 4.4 Stereographic Projections (lower hemisphere) (continued)

(c) East Slide Area—Middle

(d) East Slide Area—North

Joint Set 1
85/022

Joint Set 2
86/119

Joint Set 3
17/184

Joint Set 1
90/228

Joint Set 2
90/274

Joint Set 3
37/273

Slope Face
29/168

Slope Face
37/122

n1

n2

n3
Slide Area the joint set allowing toppling failure is vertical to subvertical whereas the same joint set in the East Slide Area—South is clearly high angle, but not vertical, and dipping into the slope; thus greatly increasing its tendency to undergo a toppling motion.

The East Slide Area—Middle shows some tendency to allow a planar failure to occur (see Figure 4.4(c)) but the relatively low angle of dip (17 degrees) of joint set number three and the fact that, even allowing for the large 95% confidence band for the strike of this joint set, the orientation of this discontinuity is at least moderately unfavorable for a planar failure. Also, joint set number two would not tend to favor toppling even at its most extreme confidence limits.

Thus it is very difficult to envision any major failure occurring in this middle area. A possible exception might be the case of a major failure on one or both sides of the area "pulling" down rock through the action of side constraint shear forces causing failure via the relatively unfavorably orientated low angle joint set number three.

Examination of the stereographic projection of the joints in the East slide Area—North (Figure 4.4(d)) shows that it is clearly impossible for planar failure to occur on the low angle joint set number three. This joint set is nearly at right angles to the slope and thus does not daylight in a manner which would allow failure. Joint sets
number two and number three do intersect to form a wedge, but the line of intersection is horizontal and thus no sliding could be envisioned here either, except possibly under extreme hydraulic loads.

Joint set number one is oriented in such a way as to make it possible for some toppling failures to occur. These would likely be rather limited since, even at the edge of the extreme confidence band, the joint set is still within a few degrees of vertical. However, with the help of frost wedging and/or hydraulic pressures, toppling failure of columns created by joint sets one and two may be kinematically possible.

Thus in the East Slide Area-North as well as the Middle Area, no failure mode can be envisioned which would result in anything but relatively minor rockfall and talus formation. However, up to this point we have not considered the possibility of a circular failure mode.

4.4 Circular Failure

Circular failure is most often recognized as a type of failure mode that occurs in soil deposits. It has been recognized, however, that if the rockmass in a slope is highly jointed and/or is weak in shear due to weathering or other causes, the slope may fail along a circular surface similar to a soil (see Figure 4.3(a)), irrespective of the joint surfaces present within the rockmass (Hoek and Bray, 1981).
The Sierra Nevada Granodiorites, of which this slope is composed, have been found to be very susceptible to the weathering effects of groundwater. Specifically, the biotite mica found in this type of rock tends to hydrate and expand in the presence of groundwater and thus tends to build up high tensile stresses in the rock. This initiates microfractures and the eventual mechanical breakdown of the portion of the rockmass which is below the water table (Krank, 1980). This results in extensive areas within the rockmass that may be weathered to Grade IV or V (see Table 5.1 for definition of weathering grades) and are thus appreciably weakened (see Table 5.2).

Many areas were found on Slide Mountain where this phenomena was in evidence and these are indicated on the geologic map (Figure 3.2). Field observations further indicated that, for whatever reasons, whenever springs or seeps were found they generally issued forth from the "contact" between the solid, though jointed, rock and the highly weathered (Grades IV-V) rock that is in contact with the groundwater (see Figures 4.5 (a) and (b)). This highly fractured, weathered rock (described as "closely fractured chalky granodiorite" by R. W. Tabor, et al., 1983) could easily be envisioned as allowing a circular failure to occur and this must be viewed as a distinct possibility for any slope on Slide Mountain, especially in the east slide areas. It should be mentioned also that it is possible that joint
FIGURE 4.5 Typical appearance of weathered rock in the East Slide Area

(a) Weathered rock around unweathered core

(b) Close up view of weathered rock (grain size varies between 1/16 and 1/4 inch)
sets, themselves unfavorable with regard to failure, may combine with a circular, planar, or compound planar failure through this weaker rock and thus help initiate failure in what might otherwise appear to be a stable slope.

4.5 Northern Slide Area

A relatively small area on the northern flanks of Slide Mountain (see Figure 3.2) appears to have been formed, at least in part, by slide activity. The deposits below the slope have been mapped as glacial moraines by previous field investigators (Moore, 1952) thus indicating that the steepened area above is a glacial cirque. This investigator does not dispute this but wishes to point out the fact that the joint orientations in this slope (see Figure 4.6(a)) may be conducive to a wedge failure mode along the intersection of joint sets one and two and/or planar failure on joint set one. It should also be noted that a secondary toppling mode on joint set one may be possible on some exposures.

It is entirely possible that glacial action caused the slope to be oversteepened and that slope failure then occurred following the recession of the glacier. It might also be noted that a relatively large number of outcrops showing slickensides (exhibiting alteration zones varying from mere surface alignment of micas to 3/4" thick) were found in this area. These may, in some cases, be glacial striations. The predominant direction of slickensiding is marked on the geologic map (Figure 3.2) as well as on the
stereographic projection (Figure 4.6(a)). The thickness of the alteration zones and the fact that the direction of movement is generally aligned with the "fall line" of the slope would indicate the distinct possibility of these being a result of slope failure. It is, however, difficult to imagine a slope failure in this direction without the aid of non-gravity type forces (e.g., frost wedging and/or hydraulic pressures). However, this will be investigated in more detail in the analysis chapter.

There were no other areas where a relatively major slope failure was evident. Several areas of what appeared to be minor sloughing, grading to small slumps or circular type failures (some with head scarps up to four feet high) were observed just north and just south of the area designated as East Slide Area-North and are marked on the geologic map (Figure 3.2). Much of the area north and east of the main slide scar on the east side of Slide Mountain has been mapped as a "source of rockfall avalanches...and small landslides" on the USGS Geologic Hazards Map for the Washoe Lake area (Tabor, et al, 1983). This investigator would agree with this assessment. However, the shallow failures found in this investigation were above the area mapped as being a source of rockfall avalanches. Therefore it is possible that this source area should be enlarged.

In connection with this, it might also be pointed out that, if all joint measurements east of the summit ridge of
FIGURE 4.6 (a) North Slide Area

Stereographic Projections (lower hemisphere of average joint surfaces, their normals (with 95% confidence interval surrounding normal) and slope face (dashed line). Dip/dip direction are indicated.

FIGURE 4.6 (b) Entire East Side of Slide Mt. (includes both East and North Slide Areas)

Stereographic Projections (lower hemisphere) of average joint surfaces, their normals (with 95% confidence interval surrounding normal) and slope face (dashed line). Dip/dip direction are indicated.
Slide Mountain are analyzed as a group (including both the North and East Slide Areas) it is found that the average joint set orientations (see Figure 4.6(b)) are the same (relative to each other and the respective slope geometry) as those found on the East Slide Area-South.

If anything can be inferred from these averaged values, one cannot avoid noticing that both the high angle and low angle joint sets can be combined to allow the possibility of planar failure (see Figure 4.6(b)). The low angle joint set seems to strike parallel to and daylight in the 34 degree slope face. The high angle set also strikes parallel to the slope face thus forming a tensile release surface as well as a water conduit. The side release surfaces, provided by the vertical joint set (number 1) striking up and down slope, would seem to make this area even more prone to sliding.
CHAPTER 5

MECHANICAL PROPERTIES

5.1 Introduction

As was discussed in the introductory chapter, the definition of the mechanical properties of the slide mass and failure surfaces is an extremely important aspect of any slope stability investigation. Once the slide geometry has been established and the most probable failure modes have been identified, the material properties appropriate to the type of failure and the stress and strain magnitudes within the slope must then be determined.

This determination usually involves both lab and field tests on a number of samples of intact joints, intact rock, and joint fill material.

Direct shear tests of intact joints on either a lab or field test device should be performed. There are a few labs capable of direct shear testing of intact rock but in most cases the cost is prohibitive and therefore a field type direct shear box, similar to that described by Hoek and Bray (1981) is utilized. This test allows one to determine the strength parameters (cohesion ($c$) and friction angle ($\phi$)) for various normal stress ranges.

These strength parameters describe the limiting shear stress ($\tau$) that can be developed on a plane under a given normal stress ($\sigma_n$). This relationship is usually expressed by the linear equation:
\[ \tau = C + \sigma_n \tan \phi \]

However, it is recognized by most workers in the rock mechanics field that this linear relationship (called the Mohr–Coloumb failure envelope) only holds for a narrow range of normal stresses and that the actual envelope is curved, especially in the region of low normal stress (see figure 5.1). Thus the C and \( \phi \) determined in a given shear test is a linear approximation, or tangent, to the actual non-linear failure envelope.

It should be noted here that "cohesion" generally implies that a sample has some finite resistance to shearing at zero normal stress or some finite tensile strength. But for most rock joints this is not the case. Thus it would be best to refer to the parameter C as an "apparent cohesion". The actual curved failure envelope will pass through the origin and the material will, in fact, exhibit no resistance to shearing at zero normal stress.

Most slopes fail along relatively shallow surfaces under low normal stress. This means that the shear stresses mobilized along the failure surface can only be determined with reference to the highly non-linear low normal stress region of the failure envelope. This means that C and \( \phi \) will change drastically with relatively small changes in normal stress (see Figure 5.1) (Hoek & Bray, 1981). Therefore it becomes obvious that any shear testing should be performed at normal stresses appropriate to those likely
FIGURE 5.1 Typical nonlinear failure envelope.

EXAMPLES OF ROUGHNESS PROFILES

A. Rough undulating - tension joints, rough sheeting, rough bedding.  
JRC = 20

B. Smooth undulating - smooth sheeting, non-planar foliation, undulating bedding.  
JRC = 10

C. Smooth nearly planar - planar shear joints, planar foliation, planar bedding.  
JRC = 5

FIGURE 5.2 Barton's definition of Joint Roughness Coefficient (JRC)  
(From Barton, 1973)
to be found in the actual field problem.

In the case of joint fill materials it is usually possible to determine $C$ and $\phi$ (remembering the cautions regarding non-linearity outlined above for rock joints) by using standard soil mechanics lab testing devices such as the direct shear device and the standard triaxial testing device.

If a finite element type analysis is contemplated then it is also necessary to determine the elastic constants ($E, v, G$) for the intact rock, the joints and the joint fill material. This can be done using well established laboratory testing methods. It should be kept in mind that, although geophysical tests can be used to indirectly obtain the elastic constants, these tests are performed at very small strains and thus some correction must be applied in order to get elastic constants that are more appropriate to the rather large strains expected in slope stability problems.

It is a very simple matter to obtain the unit weight of the material in the slope, but this aspect of the problem must not be overlooked. In order to calculate reasonable values for the forces along the possible failure plane or planes it is essential that a good average value of the unit weight of the rock and/or soil material be used in calculating the desired stresses and/or loads.
5.2 Material Properties Considered for Use in this Investigation

Little laboratory testing was performed in this investigation as detailed information was available from previous research by Krank and Watters (1983). The samples used in their investigation were obtained in the vicinity of the slope failure being analyzed in this paper and the rock type was basically identical to the rock type found in the slope.

Krank and Watters tested samples of Sierra Nevada Granodiorites in various weathered states in order to determine their engineering properties. The various weathering categories are described in Table 5.1. Grades I, II, III and IV were tested using rock mechanics testing devices and Grades V and VI, which are essentially soil-like materials, were tested using soil testing devices. The results of these tests are summarized in Table 5.2.

Field observations of the failure plane and scarp of the Ophir Creek Slide showed a layer of highly weathered (Grade V to VI) material remaining on the exposed parts of the failure surfaces. The rock graded through Grade IV to Grade I near the center of a block (as was observed in the unjointed broken blocks found in the slide debris).

5.2.1 Empirical Relations for the Determination of Mechanical Properties

A number of investigators have attempted to develop
<table>
<thead>
<tr>
<th>Type</th>
<th>Grade Abbreviation</th>
<th>Rock Material</th>
<th>Rock Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual Soil</td>
<td>VI</td>
<td>Rw</td>
<td>(same as rock material)</td>
</tr>
<tr>
<td>Completely Weathered</td>
<td>V</td>
<td>Cw</td>
<td>(same as rock material)</td>
</tr>
<tr>
<td>Highly Weathered</td>
<td>IV</td>
<td>Hw</td>
<td>Move: the rock is discoloured and weakened to such an extent that 5 cm diameter cores can be readily broken up by hand across the rock fabric.</td>
</tr>
<tr>
<td>Moderately Weathered</td>
<td>III</td>
<td>Mw</td>
<td>The rock is discoloured and noticeably weakened, but 5 cm (2 inch) drill cores cannot be broken by hand across the fabric.</td>
</tr>
<tr>
<td>Slightly Weathered</td>
<td>II</td>
<td>Sw</td>
<td>The rock is slightly discoloured and slightly weaker than fresh rock.</td>
</tr>
<tr>
<td>Fresh</td>
<td>I</td>
<td>Fr</td>
<td>The rock shows no discolouration, loss of strength, or any other effect due to weathering.</td>
</tr>
</tbody>
</table>

*adapted from Krank and Watters, 1983*
Table 5.2: Range of Physical Properties of Sierra Nevada Granodiorites* (laboratory results)

<table>
<thead>
<tr>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Static Modulus** (E)</td>
<td>Shear Modulus** (G)</td>
</tr>
<tr>
<td>I</td>
<td>2.62-2.75</td>
<td>—</td>
<td>—</td>
<td>16.0-23.97</td>
<td>0.19-0.28</td>
<td>4.53-4.96</td>
<td>1.9-2.1</td>
</tr>
<tr>
<td>II</td>
<td>2.62-2.55</td>
<td>—</td>
<td>—</td>
<td>3.77-14.04</td>
<td>0.16-.36</td>
<td>2.10-2.21</td>
<td>0.8-1.0</td>
</tr>
<tr>
<td>III</td>
<td>2.56-2.62</td>
<td>—</td>
<td>—</td>
<td>4.79-6.49</td>
<td>0.29-.37</td>
<td>1.36-1.66</td>
<td>0.5-0.6</td>
</tr>
<tr>
<td>IV</td>
<td>2.34-2.56</td>
<td>29-32</td>
<td>34-35</td>
<td>1.24-3.48</td>
<td>0.26-.27</td>
<td>N</td>
<td>.57-.85</td>
</tr>
<tr>
<td>V</td>
<td>2.02</td>
<td>46-46.5</td>
<td>0-2.8</td>
<td>0-1.50</td>
<td>N</td>
<td>N</td>
<td>.0004-.002</td>
</tr>
<tr>
<td>VI</td>
<td>1.50</td>
<td>34.6</td>
<td>0-4.9</td>
<td>0</td>
<td>N</td>
<td>N</td>
<td>N</td>
</tr>
</tbody>
</table>

Notes:
* adapted from Krank and Watters, 1983
** x 10^6 psi
N = not able to calculate
-- = not determined
empirical relations which would allow the engineer to determine the properties of rock joints from easily obtainable lab and/or field data.

Early work in this area was done by Patton (1966). He used a concept borrowed from soil mechanics which recognizes that the roughness of the plane being sheared has a significant effect on the shear strength mobilized. He proposed the use of the term $\tan (\phi + i)$ in place of $\tan \phi$ in the linear equation for shear strength. He defined $\phi$ as the basic or intrinsic friction angle that would be measured on a perfectly smooth laboratory test sample or on a test sample that has been sheared a great distance (otherwise known as the residual friction angle [$\phi_r$]). The $i$ is defined as the average angle of inclination of the surface roughness asperities as found on joint surfaces in the field. This concept is well accepted today and serves to emphasize the fact that lab tests of small sample joints will not reflect the actual shearing conditions experienced by large joint surfaces in the field. Field data and lab tests indicate that $i$ may be as high as 40-50 degrees for low normal stresses (Hoek & Bray, 1981) and thus with an average $\phi_r$ being on the order of 30 degrees this means the actual effective $\phi$ may be as high as 80 degrees. The effective friction angle varies with normal stress due to the fact that at high normal stresses the smaller, "second order" asperities are broken off as shearing proceeds and
thus only the larger, lower angle, "first order" asperities are important at these higher stresses.

The action of shearing thus causes the asperities to ride up thus causing dilation. This was studied in great detail by Landanyi and Archambault (1970). They developed the following empirical strength criterion:

\[ \tau = \frac{\tau_r}{1 - (1 - a_s) \dot{v} \tan \varphi} \]

where

- \( a_s \) = the proportion of the surface which is sheared through intact rock,
- \( \dot{v} \) = the dilatation rate \( \frac{dv}{du} \) at peak shear strength,
- \( \tau_r \) = the shear strength of the intact material.

They also found that \( \tau_r \) can be found by using the empirical relation:

\[ \tau_r = \frac{\sigma_j (1-n)^{0.5}}{n^{1.5}} \left[ 1 + n \left( \frac{\sigma_j}{\sigma_j} \right) \right]^{0.5} \]

where

- \( \sigma_j \) = is the uniaxial compressive strength of the joint wall material (which is generally less than the unweathered rock farther from the joint surface).
\[ n = \text{the ratio of uniaxial compressive to uniaxial tensile strength of the rock material (} n = 10 \text{ for most rocks).} \]

Since \( v \) and \( a_s \) are hard to measure, Ladanyi and Archambault developed the following empirical relationships to be used to calculate \( v \) and \( a_s \):

\[
\dot{v} = (a_s - \frac{\sigma^n_j}{\sigma^n_j})^k \tan i
\]

\[
a_s = 1 - (1 - \frac{\sigma^n_j}{\sigma^n_j})^L
\]

where, for rough rock surfaces, \( K = 4 \) and \( L = 1.5 \).

However, the difficulty of measuring the parameters needed and the many empirical relationships required to make use of the Ladanyi and Archambault equations makes them somewhat unwieldy and reduces their practical usefulness.

Barton (1973) has done a great deal of testing on model tensile failure joints and has developed an equation based on easily measured quantities. His equation is:

\[
= \sigma'_n \tan (\phi + \text{JRC} \log_{10} \frac{\sigma'_j}{\sigma'_n})
\]

where

\( \text{JRC} = \text{the joint roughness coefficient,} \)

\( \sigma'_j = \text{joint wall compressive strength.} \)

\( \sigma'_n = \text{effective normal stress.} \)

The JRC is determined by field observations of the joint surface undulations and can be estimated with the aid
of the diagrams found in Figure 5.2.

This empirical relationship can only be used when rock-rock contact is still maintained, although joint surfaces may be weathered, and filled joints are excluded.

Barton further notes that tension joints in multi-mineralic rocks have been shown to be very rough, whereas those joints formed by shearing due to tectonic processes are generally more smooth and planar. He also points out that the rougher the joint surface the more pronounced the curvature of the failure envelope.

Barton recommends that \((\phi + JRC \log_{10} \frac{\sigma^j}{\sigma^c})\) not exceed 70 to 76 degrees since reported values of \(\tan^{-1} \frac{\gamma}{\sigma^c}\) in the literature are rarely beyond this range.

In back analyzing failed slopes Barton found that, when no direct tests of joint wall rock compressive strength are possible, the ratio \(\frac{\sigma^j}{\sigma^c}\) (where \(\sigma^c = \) compressive strength of weathered rock) can be taken as equal to 1/4 to obtain a lower bound estimate of joint shear strength, while \(\sigma^j = \sigma^c\) gives the upper bound strength.

It has been found that water reduces the compressive strength of laboratory specimens from 19-33% (Barton, 1973). Thus, for cases where the failure surface is below the water table it is suggested that \(\sigma^j\) be determined on saturated specimens or \(\sigma^j\) be replaced by \(0.75 \sigma^j\) if no saturated test data is available. Thus Barton's failure criteria becomes:

\[
\psi = \sigma^j \tan (\phi + JRC \log_{10} \frac{0.75 \sigma^j}{\sigma^c})
\]
Using this equation and various combinations of joint wall rock strength values from Table 5.2, a range of shear strength envelopes can be determined (see Table 5.4 and Figure 5.3) and used to analyze the slope.

Research has shown that when a rock slope is cut by several closely spaced joint sets it will behave in a manner that is similar to a soil slope. This means that the most likely mode of failure will be circular (Hoek & Bray, 1981). In this case the strength parameters must be calculated differently since part of the failure surface will pass through intact rock and part will follow joint planes.

An empirical equation has been proposed by Hoek and Brown (1980) (Hoek and Bray, 1981) for determining the failure envelope based on large scale laboratory triaxial tests of coarse grained material. The relation they suggest is:

\[ \tau = A \sigma_c \left( \frac{\sigma_n}{\sigma_c} - T \right)^3 \]

where

\[ T = \frac{1}{2} \left( m - (m^2 + 45) \cdot 5 \right) \]

m and s are constants relating to the shape and degree of interlocking of the individual pieces of rock in the mass, and A and B are constants defining the shape of the Mohr envelope.
Effective Stress $\sigma^*$ (psf - Assuming water table at ground surface)
(depth below surface, in feet, in parentheses.)

FIGURE 5.3 Comparison between various empirical shear strength criteria and laboratory test results.
It was assumed that the effective stress law applies, thus in place of $\sigma_n$ the effective stress $\sigma'_n (= \sigma_n - u)$ should be used in cases where the failure surface is below the water table.

Hoek and Brown ran regression analyses on the data from many tests for various types and shapes of rocks and values for the constants $m$, $s$, $A$ and $B$ were obtained. These were categorized and published in tabular form. Table 5.3 lists the values of the constants that apply to the material in the Ophir Creek Slide. The resulting range of failure envelopes can also be found in Table 5.4 and are plotted in Figure 5.3. Thus a range of possible empirical failure criteria can be derived for use in a circular failure analyses.

5.2.2 Determination of the Material Properties for a Finite Element Analysis

If a Finite Element analysis is performed assuming homogeneous, isotropic, linear elastic materials then some value or values for the material property matrix must be chosen, recognizing that most geologic materials are at least slightly non-linear and can, in many cases, be highly non-linear. This added to the discontinuous nature of the rock mass makes the choice of elastic constants very difficult.

Goodman and Duncan (1968), in describing their orthotropic continuum finite element analysis (see Chapter
Table 5.3: Approximate Relationship Between Rock Mass Quality and Empirical Constants for Hoek and Brown Shear Strength Criteria

<table>
<thead>
<tr>
<th>QUALITY ROCK MASS</th>
<th>m</th>
<th>s</th>
<th>A</th>
<th>B</th>
<th>T</th>
</tr>
</thead>
<tbody>
<tr>
<td>GOOD</td>
<td>2.5</td>
<td>0.004</td>
<td>0.346</td>
<td>0.700</td>
<td>-0.002</td>
</tr>
<tr>
<td>fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CSIR rating 55</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NGI rating 10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FAIR</td>
<td>0.50</td>
<td>0.0001</td>
<td>0.346</td>
<td>0.700</td>
<td>-0.0002</td>
</tr>
<tr>
<td>several sets of moderately weathered joints spaced at 0.3 to 1m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CSIR rating 44</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NGI rating 1.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>POOR</td>
<td>0.13</td>
<td>0.00001</td>
<td>0.203</td>
<td>0.686</td>
<td>-0.0001</td>
</tr>
<tr>
<td>numerous weathered joints at 30 to 500mm with some gouge - clean waste rock</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CSIR rating 23</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NGI rating 0.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Coarse Grained Polymerallic Igneous & Metamorphic Crystalline Rocks (amphibolite, gabbro gneiss, granite, norite, and quartz-diorite)
suggest that the elastic properties of a rock mass containing three orthogonal joint sets can be calculated knowing the joint spacing of each joint set \((S_1, S_2, S_3)\) and the shear and normal stiffnesses \((K_s \text{ and } K_n, \text{ respectively})\) of each joint set. The following equations can then be used to model a jointed medium as an equivalent continuum when the intact rock blocks are much stiffer than the joints:

\[
\begin{align*}
E_1 &= S_1 K_n \\
E_2 &= S_2 K_n \\
E_3 &= S_3 K_n \\
G_{12} &= \frac{S_1 S_2 K_s}{S_1 K_s + S_2 K_s} \\
G_{13} &= \frac{S_1 S_3 K_s}{S_1 K_s + S_3 K_s} \\
G_{23} &= \frac{S_2 S_3 K_s}{S_2 K_s + S_3 K_s} \\
v_{12} &= v_{13} = 0 \\
v_{23} &= v_{21} = 0 \\
v_{31} &= v_{32} = 0
\end{align*}
\]

where,

\(E\) = Deformation Modulus
\(G\) = Shear Modulus
\(v\) = Poisson's Ratio
In this analysis the joint shear stiffness can be estimated from portable shear box tests performed on joint samples. However, since no tests were performed by the author in order to determine the normal stiffness of the joints, an equation similar to that presented by Duncan and Goodman (1968) was utilized to calculate the equivalent deformation modulus \( E_n \) of the rockmass.

The total deformation \( \delta \) of a given block of rock is the sum of the joint deformation \( \delta_j \), plus the deformation of the rock core \( \delta_r \):

\[
\delta = \delta_r + \delta_j
\]

or, noting that the deformation can be calculated by multiplying the rock joint spacing \( S \) and joint thickness \( T_j \) by the respective strains \( \varepsilon \),

\[
\delta = \varepsilon_r S + \varepsilon_j T_j
\]

The deformation moduli can be used to calculate the strains, given the stresses and this results in

\[
\delta = \frac{\sigma_r S}{E_r} + \frac{\sigma_j T_j}{E_j}
\]

Assuming that the normal stress in the rock and the joint are equal, then the following equation can be written:

\[
\delta = \frac{\sigma_r (S + T_j)}{E_{equiv.}} = \frac{\sigma S}{E_r} + \frac{\sigma_j T_j}{E_j}
\]

or solving for \( E_{equiv.} \):
Table 5.4 shows various calculated values of $E_{equiv}$ for a range in values of $S$ and $T_j$, assuming $E_r = 1.51 \times 10^6$ psi (as determined by Krank and Watters for grade III rock) and $E_j = 3040$ psi (as given by Lambe and Whitman [1969] for dense sand at a confining pressure of 21 psi).

5.3 Conclusions Regarding Material Properties

When it is possible to conduct a very thorough site investigation and lab testing program, then the material properties obtained can be used with a high level of confidence. However, when only a minimum of field and lab work can be performed, the strength parameters obtained must be used with a great deal of caution.

In the case of the Ophir Creek Slide only six sets of joint shear tests were performed and thus the data obtained (see Appendix A) cannot be used with a high level of confidence. However, it is possible to compare these test results with the various empirical equations outlined in this chapter in order to bracket a reasonable range of strength parameters.

Table 5.4 is a summary of the various strength parameters that could be used in the analysis of the Ophir Creek Slide. Also included are values obtained from the literature (for comparison) as well as the values used for the constants employed in the calculation of the various
strength parameters. The graph in Figure 5.3 allows a comparison of some of the various shear strength criteria from Table 5.4 in the stress range relevant for this slope. It will be noticed that the Hoek and Bray empirical criterion for fair and poor rock masses (for circular failures) brackets the values obtained in tests done by this author as well as the average properties for Grade V rock determined from Krank and Watters' studies. The Krank and Watters' data are bracketed even more closely by the curves plotted from Barton's shear strength criteria assuming the joint wall strength ($C'_{j}$) to be that of Grade III through Grade V rock as determined by Krank and Watters. Thus, since field observation indicated that the joint walls and fill were generally Grade V to VI rock, it is thought that the values of $C = 2.8$, $\phi = 45.3$ (Krank and Watters) would be a good value for the average joint shear strength, while the values of $C = 3$ psi and $\phi = 37$ degrees are good estimates of the lower bound joint shear strength (see shear test 4), and that the upper bound strength would be best described by the shear strength as determined from shear test 6 using the Grade IV sample, or $C = 48.8$ psi, $\phi = 42$ degrees. These will be used in both planar and circular failure analyses. The results will then be compared with Barton's non-linear envelope for $\sigma'_{j}$ from Grade IV rock to see if the factor of safety will vary significantly.

Table 5.4 also lists various values for the elastic
constants which could be used in a finite element analysis. It should be noted that the value for $E$ from Krank and Watters (1200 psi) is in close agreement with values based on data for loose, medium to fine quartz sands from Lamb and Whitman (1969). It should further be noted that the other values quoted from Lambe and Whitman demonstrate the non-linearity of the material properties of geologic materials. $G$ for sands also changes with strain (or stress) as shown by the values taken from Norris (1983) (based on Harden and Drnevich, 1972). It must also be realized that commonly used values of Poisson's ratio represent the case where strains are small and no failure has occurred. Once shear failure has occurred Poisson's ratio may, in some cases, exceed .5 (Sowers, 1979; Lambe and Whitman, 1969). This, again, demonstrates non-linear behavior and serves to underline the difficulties inherent in the use of linear approximations with geologic materials.

Also found in Table 5.4 is a list of the material properties that will be used for an orthotropic continuum analysis. Since field observation indicates that the high angle joint set had the smaller joint spacing ($S_1$), only combinations of joint spacing where $S_1 < S_2$ will be considered for use here. Also, field observation of the rock slide debris would indicate that the smallest dimension of a single rock fragment generally exceeded one foot and, in most cases, did not exceed 10 feet. The largest
dimension of the individual blocks in the rock slide varied from as small as 2 or 3 feet to larger than 30 feet with the average being from 5 to 10 feet. Thus the combinations \( S_1 = 5 \text{ feet}, S_2 = 1 \text{ foot}; S_1 = 10 \text{ feet}, S_2 = 1 \text{ foot}; \) and \( S_1 = 10 \text{ feet}, S_2 = 5 \text{ feet} \) will probably give the most reasonable values of \( E \) for this slope. The shear tests which were used to obtain the value of joint shear stiffness \( (K_s) \) yielded varying values for \( K_s \) according to the applied normal force. Two representative values were chosen (as noted in Table 5.4) and \( G_{12} \) was calculated for both values of \( K_s \) for each combination of joint spacings. These two values were then averaged in order to obtain the value to be used in the analysis.
Table 5.4: A Summary of Empirically and Laboratory Determined Mechanical Properties for The Materials in the Ophir Creek Slide

Properties (units are pounds and inches)

<table>
<thead>
<tr>
<th>Source or Method of Determination</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIMIT EQUILIBRIUM (including planar, wedge, toppling, and circular failure modes):</td>
</tr>
<tr>
<td><strong>Shear Strength</strong> ($\mathcal{T}$)</td>
</tr>
<tr>
<td><strong>Grade III Rock:</strong> (joints)</td>
</tr>
<tr>
<td>$\mathcal{T} = \sigma'<em>n \tan [10 \log</em>{10} \left( \frac{4230}{\sigma'_n} \right) + 32.5^\circ]$</td>
</tr>
<tr>
<td>$10 \log_{10} \left( \frac{4230}{\sigma'_n} \right) + 32.5^\circ \leq 70^\circ$</td>
</tr>
<tr>
<td>$\mathcal{T} = 14.5 + \sigma'_n \tan 29^\circ$ (no joint fill)</td>
</tr>
<tr>
<td>$\mathcal{T} = 8 + \sigma'_n \tan 32^\circ$ (fine joint fill)</td>
</tr>
<tr>
<td>$\mathcal{T} = 16 + \sigma'_n \tan 40^\circ$ (no joint fill)</td>
</tr>
<tr>
<td>$\mathcal{T} = 3 + \sigma'_n \tan 37^\circ$ (coarse joint fill)</td>
</tr>
<tr>
<td>$\mathcal{T} = 8 + \sigma'_n \tan 35^\circ$ (fine joint fill)</td>
</tr>
<tr>
<td>$\mathcal{T} = 48.8 + \sigma'_n \tan 42^\circ$ (no joint—sheared on obvious plane of weakness)</td>
</tr>
</tbody>
</table>

Barton\textsuperscript{a} lab tests on matched rock surfaces (see Appendix)

Lab test on intact rock specimen

\textsuperscript{a} Barton
### Table 5.4 (continued)

<table>
<thead>
<tr>
<th>Properties</th>
<th>Source or Method of Determination</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Grade IV Rock:</strong></td>
<td></td>
</tr>
<tr>
<td>(joints)</td>
<td></td>
</tr>
<tr>
<td>( \tau = \sigma'<em>n \tan \left[ 10 \log</em>{10} \frac{1770}{\sigma'_n} + 32.5^\circ \right] )</td>
<td>Barton\textsuperscript{b}</td>
</tr>
<tr>
<td>(rock material)</td>
<td></td>
</tr>
<tr>
<td>( \tau = 35 + \sigma'_n \tan 30^\circ )</td>
<td>Krank &amp; Watters</td>
</tr>
<tr>
<td><strong>Grade V Rock:</strong></td>
<td></td>
</tr>
<tr>
<td>(joints)</td>
<td></td>
</tr>
<tr>
<td>( \tau = \sigma'<em>n \tan \left[ 10 \log</em>{10} \frac{560}{\sigma'_n} + 32.5^\circ \right] )</td>
<td>Barton\textsuperscript{c}</td>
</tr>
<tr>
<td>(rock material)</td>
<td></td>
</tr>
<tr>
<td>( \tau = 2.8 + \sigma'_n \tan 46.3 )</td>
<td>Krank &amp; Watters</td>
</tr>
</tbody>
</table>
### Table 5.4 (continued)

<table>
<thead>
<tr>
<th>Soil (joint fill) (c=0)</th>
<th>Source or Method of Determination</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi ) peak = 39° (loose, regular, well-graded sand)</td>
<td>Lambe &amp; Whitman</td>
</tr>
<tr>
<td>( \phi ) peak = 45° (dense, regular, well-graded sand)</td>
<td>Lambe &amp; Whitman</td>
</tr>
<tr>
<td>( \phi ) peak = 33° - 41° (sandy gravel)</td>
<td>Lambe &amp; Whitman</td>
</tr>
<tr>
<td>( \phi_r ) = 26° (medium, fine sand)</td>
<td>Lambe &amp; Whitman</td>
</tr>
<tr>
<td>( \phi ) peak = 40° (granite w/sandy loam joint fill)</td>
<td>Lambe &amp; Whitman</td>
</tr>
</tbody>
</table>

**CIRCULAR FAILURE (failure partially through intact rock):**

- **Good quality rockmass:**
  \[ \gamma = 7.576 \left( \sigma'_n + 11.28 \right) \cdot 707 \text{ psi} \]
  (fresh to slightly weathered w/joints 1-3 m.)

- **Fair quality rockmass:**
  \[ \gamma = 3.56 \left( \sigma'_n + .472 \right) \cdot 700 \text{ psi} \]
  (several sets of moderately weathered joints spaced .3-1 m.)

- **Poor quality rockmass:**
  \[ \gamma = 1.62 \left( \sigma'_n + .075 \right) \cdot 686 \text{ psi} \]
  (numerous weathered joints at 50-500 mm spacing with some gouge)
Table 5.4 (continued)

<table>
<thead>
<tr>
<th>Source or Method of Determination</th>
<th>Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>FINITE ELEMENT METHOD</td>
<td></td>
</tr>
<tr>
<td>Elastic Constants</td>
<td></td>
</tr>
<tr>
<td>(Rockmass)</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Grade</th>
<th>$E$ (GPa)</th>
<th>$G$ (GPa)</th>
<th>$v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>III</td>
<td>1.51 x 10^6</td>
<td>0.55 x 10^6</td>
<td>0.33</td>
</tr>
<tr>
<td>IV</td>
<td>0.71 x 10^6</td>
<td>0.25 x 10^6</td>
<td>0.27</td>
</tr>
<tr>
<td>V</td>
<td>0.0012 x 10^6</td>
<td>(not available)</td>
<td>(not available)</td>
</tr>
</tbody>
</table>

$E_d = 1.02 \times 10^6$ (for RQD = 80%, rock strength = 39 MPa (Gr. III), joint spacing = 1-3 m., soft wall rock, unfavorable joint orientation, moderate H₂O pres.)

Krank & Watters

RMR₉ = 47
Table 5.4 (continued)

Properties

\( E_d = 1.7 \times 10^6 \) (same as above except

- RQD = 50, joint spacing = 1 cm. –
- 3 m., very favorable joint orientation)

Soil (Joint Fill) (sand)

Confining Pressure:

7 psi

\[ \begin{align*}
E & = 1180 \text{ (loose)} \\
G & = 455 \\
E & = 1690 \text{ (dense)} \\
G & = 650
\end{align*} \]

21 psi

\[ \begin{align*}
E & = 1860 \text{ (loose)} \\
G & = 715 \\
E & = 3040 \text{ (dense)} \\
G & = 1170
\end{align*} \]

for large shear strains (\( \gamma \)):

\[ \begin{align*}
\gamma = 1\%, & \quad G = 318 \text{ psi} \\
\gamma = 1\%, & \quad G = 2070 \text{ psi}
\end{align*} \]

Source or Method of Determination

- RMR = 53
- Lambe & Whitman
  - from dynamics class notes/Hardin & Drnevich
## Table 5.4 (continued)

**ORTHOTROPIC CONTINUUM ANALYSIS**

**Poisson's Ratio:**

\[ \nu_{12} = 0 \]

**Deformation Modulus:**

<table>
<thead>
<tr>
<th>( S ) (feet)</th>
<th>( T_j ) (inches)</th>
<th>( E_n ) (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>18</td>
<td>( 4.2 \times 10^4 )</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>( 6.1 \times 10^4 )</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>( 1.2 \times 10^5 )</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>( 2.1 \times 10^5 )</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>( 4.9 \times 10^5 )</td>
</tr>
<tr>
<td>10</td>
<td>18</td>
<td>( 2.3 \times 10^4 )</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>( 3.3 \times 10^4 )</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>( 6.1 \times 10^4 )</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>( 1.2 \times 10^5 )</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>( 3.0 \times 10^5 )</td>
</tr>
<tr>
<td>5</td>
<td>18</td>
<td>( 1.3 \times 10^4 )</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>( 1.8 \times 10^4 )</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>( 3.3 \times 10^4 )</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>( 6.1 \times 10^4 )</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>( 1.7 \times 10^5 )</td>
</tr>
</tbody>
</table>

Source or Method of Determination: Goodman and Duncan
Table 5.4 (continued)

<table>
<thead>
<tr>
<th>Source or Method of Determination</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear test</td>
</tr>
<tr>
<td>Grade IV-V Rock</td>
</tr>
<tr>
<td>(see appendix)</td>
</tr>
<tr>
<td>(tests 6A &amp; B)</td>
</tr>
<tr>
<td>Goodman &amp; Duncan</td>
</tr>
<tr>
<td>(see this text for equations used)</td>
</tr>
</tbody>
</table>

**Shear Modulus ($G_{12}$) given various joint spacings:**

(psi x 10³)

<table>
<thead>
<tr>
<th>$s_2$(feet):</th>
<th>.25</th>
<th>1</th>
<th>5</th>
<th>10</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_1$(feet)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$K_{q_{\text{min}}}$</td>
<td>2.5</td>
<td>8.8</td>
<td>26.4</td>
<td>35.1</td>
<td>42.2</td>
</tr>
<tr>
<td>5 avq</td>
<td>7.0</td>
<td>24.4</td>
<td>73.2</td>
<td>97.6</td>
<td>117.1</td>
</tr>
<tr>
<td>$K_{q_{\text{max}}}$</td>
<td>11.4</td>
<td>40.0</td>
<td>120.0</td>
<td>160.0</td>
<td>192.0</td>
</tr>
<tr>
<td>$K_{q_{\text{min}}}$</td>
<td>2.6</td>
<td>9.6</td>
<td>35.1</td>
<td>52.7</td>
<td>70.3</td>
</tr>
<tr>
<td>10 avq</td>
<td>7.2</td>
<td>26.6</td>
<td>97.6</td>
<td>146.4</td>
<td>195.2</td>
</tr>
<tr>
<td>$K_{q_{\text{max}}}$</td>
<td>11.7</td>
<td>43.6</td>
<td>160.0</td>
<td>240.0</td>
<td>320.0</td>
</tr>
<tr>
<td>$K_{q_{\text{min}}}$</td>
<td>2.6</td>
<td>10.0</td>
<td>42.2</td>
<td>70.3</td>
<td>105.4</td>
</tr>
<tr>
<td>20 avq</td>
<td>7.3</td>
<td>27.9</td>
<td>117.1</td>
<td>195.2</td>
<td>292.7</td>
</tr>
<tr>
<td>$K_{q_{\text{max}}}$</td>
<td>11.9</td>
<td>45.7</td>
<td>192.0</td>
<td>320.0</td>
<td>480.0</td>
</tr>
</tbody>
</table>

**NOTES:**

- $s_1 = $ Spacing perpendicular to low angle joint set
- $s_2 = $ Spacing parallel to low angle joint set

$K_{q_{\text{min}}} = K_{s_1} = K_{s_2} = 878.6 \text{ psi/in}$

$K_{q_{\text{max}}} = K_{s_1} = K_{s_2} = 4000 \text{ psi/in}$
NOTES:

a JRC assumed to be 10, $\phi_r = 32.5$ (Barton, 1973), $\sigma_{j} = 4230$ psi (Krank & Watters)

b JRC = 10, $r = 32.5^\circ, .75 \sigma_{j} = 1770$ (Krank & Watters)

c JRC = 10, $r = 32.5^\circ, .75 \sigma_{j} = 550$ (Krank & Watters)

d $m = 2.5, s = .004, A = .603, B = .707, T = -.002$ (see text for equation)

e $m = .5, \beta = .0001, A = .346, B = .700, T = -.0002$ (see text for equation)

f $m = .13, s = .0001, A = .203, B = .686, T = -.0001$ (see text for equation)

g RMR = Rock Mass Rating as described by Goodman (1976)

h $E$ calculated from given $V_L$ for medium to fine quartz sands, $G$ calculated using

$$G = \frac{E}{2(1+v)} \quad \text{where} \quad v = .30$$

i $G = 100K_2 (\sigma'_m)$ psf, for $K_0 = .4$

$$\phi = 36^\circ$$

$$\sigma'_m = 3000 \text{ psf} \quad \text{(assuming 10 feet below surface with rock unit wt = 150 pcf, and water at surface then}$$

$$= \frac{(160-62.4) (10') (1+2K_0)}{3}$$
CHAPTER 6

METHODS OF ANALYSIS

6.1 Introduction

There are many methods of analysis currently being used to analyze slopes, given the slope geometry and material properties. However, these can be subdivided into two basic categories:

(1) Limit equilibrium analysis,
(2) Stress and deformation type of analysis.

In limit equilibrium analysis a static (or pseudo static, if dynamic forces are considered) freebody analysis is performed on the assumed failure mass and a Factor of Safety (FS) against sliding is calculated by dividing the magnitude of the resultant of all stabilizing forces (acting along the failure surface in the anticipated direction of failure) by the destabilizing forces (acting on the same surface but opposite in direction). When the stabilizing and destabilizing forces are equal the \( FS = 1 \) and failure is presumed to occur below this limit, thus the name limit equilibrium.

The most common example of the stress and deformation type of analysis is Finite Element (FE) Analysis. In FE analysis the rock mass is divided into numerous compartments or elements of finite but relatively small dimensions and the differential equations of equilibrium for every element are used in conjunction with the strain relation developed
in the theory of elasticity to obtain the unknown displacements of specified points within each element. These displacements are then used along with the material stress-strain properties to compute the stresses in the elements.

It should be noted that the main difference between the limit equilibrium methods and the stress and deformation methods is that in the limit equilibrium methods there are no calculations of the strains or deformations within the slope.

There are many refinements which have helped in increasing the accuracy and usefulness of the two categories of analyses and these will be discussed in subsequent sections of this paper.

6.2 Limit Equilibrium Analyses (Hoek and Bray, 1981)

6.2.1 Planar Failure

Planar failure is the simplest failure mode. Like most things in geotechnical engineering, the fact that it is simple almost precludes its occurrence in nature. However, it can and does occur in natural as well as manmade slopes and, more importantly, is an excellent tool for understanding the basic concepts of limit equilibrium as applied to slope stability.

Planar failure, as the name implies, is generally assumed to occur on a single continuous surface that strikes subparallel ($\pm 20^\circ$) to the slope face at a dip angle less
than that of the slope face (see Figure 6.1). The geometry of this type of failure allows it to be analyzed in two dimensions, assuming that the lateral extent of the slope face is large compared to its height and that there are side release surfaces which provide a negligible resistance to sliding.

Since gravity forces will generally act to create a zone of tension near the crest of the slope and since the tensile strength of the rock is generally assumed to be zero (especially if another, nearly vertical, joint set is present), a tension crack is generally incorporated in the analysis as shown in Figure 6.1.

In natural slopes the only forces acting on the slope are the weight force and, if there is water present, the resultant of the distributed water pressure. Generally, the water pressure distribution along the failure plane is unknown due to uncertainties in the permeability of the failure surface. Therefore it is usually necessary to assume one of two pressure distributions: full hydrostatic pressure on all surfaces or a linear distribution from zero pressure at the top surface and toe of the slope to a maximum at a point below the crest. The former will occur if ice or clay block drainage at the toe whereas the latter probably represents the conditions present in most slopes and is more commonly used in slope stability analysis (see Figure 6.1). A third option is to use the piezometric
FIGURE 6.1 Planar failure showing tension crack with typical water pressure distribution and resultant water forces (U and V). Definition of variables in FS calculation are also shown. (From Hoek and Bray, 1981)

FIGURE 6.2 Cross-section of circular failure showing a typical slice and the forces acting on it. (From Hoek and Bray, 1981)
surface as an indicator of pressures in the slope and derive a distribution based on this.

Other types of forces can be considered (i.e., rock bolts, earthquakes, etc.) but these are not relevant to the natural slope studied in this paper and therefore will not be discussed.

The forces acting on the failure mass are then resolved into components perpendicular and parallel to the assumed failure plane. If a Mohr-Coloumb type failure criterion is used then the shear force mobilized can be calculated knowing the normal force acting on the failure plane (see Chapter 5).

It should be noted that, since the point(s) of application of these forces are unknown, limit analysis assumes that all forces act through the center of gravity of the failure mass and thus no moments need be considered. This assumption results in some error but its magnitude has been found to be negligible (Hoek and Bray, 1981).

Once all the forces acting along the failure plane have been determined, the factor of safety can be calculated. For the case shown in Figure 5.1 (assuming a Mohr-Coloumb type linear failure envelope) the factor of safety can be calculated using the equation:

$$FS = \frac{CL - (W\cos\psi_p - U - V\sin\psi_p) \tan\phi}{W\sin\psi_p - V\cos\psi_p}$$
where \( W, V \) and \( \psi_p \) are as defined in Figure 6.2.

### 6.2.2 Wedge Failure

Once an understanding of the basic approach to limit equilibrium is attained using the planar failure mode, it is a relatively simple matter to extend this reasoning to include the case of the intersection of two planar discontinuities in order to form a three dimensional wedge as illustrated in Figure 4.3(c). This type of failure is much more common than the planar failure and it can be seen that the planar failure is merely a limiting case of the more general wedge failure mode.

Again it is assumed that the failure wedge is a rigid body that fails by translating in the direction of the intersection of the two planes that define the bottom of the wedge. Since the failure wedge is assumed to be a rigid body in translation, resisting shear forces on both planes will be in a direction parallel to the line of intersection of the planes and normal forces will be perpendicular to these same planes.

An important and sometimes difficult aspect of wedge failure analysis is the determination of the wedge geometry. This can be done fairly easily using standard stereonet plotting techniques or, for computer use, vector operations. The development of the equations will not be detailed here (see Chapter 7), but the general idea of analysis is the same. All forces acting on the wedge (including, in the
more general case, forces due to water pressures) are resolved into components parallel and perpendicular to the line of intersection of the two joint planes. The FS is calculated as before by dividing the stabilizing forces by the destabilizing forces and failure is presumed to occur when $\text{FS} < 1$.

### 6.2.3 Circular Failure

Over the years, both field observation as well as theoretical considerations have indicated that soil slopes fail along a path that, in cross section, closely resembles a circle (Figure 6.2) (Lambe and Whitman, 1969; Hoek and Bray, 1975). There are a number of methods of circular failure analysis but almost all of the popular methods involve dividing the assumed failure mass into a series of vertical slices, thus the name "Method of Slices". Each slice is analyzed statically in order to determine the magnitude of the forces acting along the base of the slice. These are then summed to obtain the overall equilibrium equations for the slide mass. As can be seen from Figure 6.2, there are more unknowns than there are equations of equilibrium. Thus, in most methods of slices, some simplifying assumptions are made regarding the magnitude and/or the line of action of the resultant of the side forces in order to solve the equations. These assumptions as well as the accuracy of the resulting FS will vary with the method employed.
Since greater accuracy can be obtained by increasing the number of slices, this type of analysis is generally performed with the aid of a computer. However, for a preliminary analysis, hand calculations can be performed using as few as eight slices. Hoek and Bray (1981) have worked out detailed charts for circular failure analysis for cases of homogeneous slopes with varying piezometric elevations, assuming a failure circle passing through the toe of the slope. In conjunction with this, it should be noted that analyses of many slopes have shown that the circle with the lowest factor of safety will, in most cases, pass through the toe of the slope (Hoek and Bray, 1981).

For slopes with irregular slope profiles and/or material properties that vary with location within the slope, it is necessary to use one of the available computer programs to perform the analysis.

6.2.4 Toppling Failure

Although toppling failure is included here under limit equilibrium methods of analysis, it is not entirely accurate to classify it in this way.

Toppling failure occurs when a joint set dips steeply into the slope thus allowing relatively thin columns or plates of rock to rotate about a point or line at the base of the block on the side nearest the open face and thus "toppling" towards the free slope face.

The simplest form of this can be investigated using the
familiar block on a plane model (Figure 6.3). It can be seen that the weight force acting through the center of gravity of the block will create a moment about the downslope toe of the block (point A). If the weight force passes through a point upslope of point A it will tend to have a stabilizing effect. However, if the size and shape of the block is such that the weight force passes through a point downslope of point A, then a destabilizing moment is created and, unless a force is applied on the downslope side of the block, it will topple. As can be seen from the diagram, the limiting condition can be defined by the case when the weight force passes through point A. This can be defined by the equation:

$$\tan \Theta = \frac{b}{h}$$

In otherwords, if \(\tan \Theta > \frac{b}{h}\) toppling will occur and if it is less the slope is stable. It should be noted that the block can slide as well as topple and this condition must also be checked (see Planar Failure, above).

This simple model serves to illustrate the failure mechanism, but seldom do we find a situation where a single block can be analyzed. In most cases many blocks will interact as shown in Figure 6.4. It can be seen that the key to the stability of a toppling slope is the ability of the blocks near the toe to remain stable under the forces exerted on them by the blocks upslope (Goodman and Bray,
(a) Stable (b) Limiting Condition (c) Unstable

FIGURE 6.3 Simplified analysis of toppling failure.

FIGURE 6.4 Toppling failure geometry on a typical slope showing the large number of variables involved. (From Hoek and Bray, 1981)
Thus the factor of safety can be expressed as the ratio of the coefficient of friction ($\mu$) available to the coefficient of friction required for the slope to remain stable.

The geometry of toppling failures indicates that when enough rotation occurs the slope will again become stable. This is due to the fact that edge to face contacts will become face to face contacts and thus greatly increase the area contributing to shear forces along the sides of the block which will tend to be stabilizing forces.

Toppling has only recently been shown to be an important failure mechanism and much work remains to be done. As Figure 5.4 suggests, a great deal must be known about the geometry of the slope, especially joint spacing, before a meaningful analysis can be performed. Computer methods are generally required due to the large number of variables and the complexity of the calculations. New methods, such as Cundall's discrete element approach, are making great progress in the solution of toppling as well as other types of rigid block failures (Goodman and Bray, 1976).

6.2.5 Modifications to Basic Limit Equilibrium Analyses

Few modifications to the limit equilibrium approach to slope stability have been successfully integrated into practical usage.

Due to the increased use of the computer many
refinements such as the addition of a possible tension crack to the wedge analysis, the possibility of defining a wedge by more than two joint sets, the analysis of wedge toppling, etc., have been added to the analysis. These refinements increase the accuracy of the analysis but stay within the general framework of the original basic limit equilibrium analysis.

As discussed in Chapter 5, there are major problems with accurately describing the shear strength of joints. This relates to the fact that, at low normal stress, the failure envelope is non-linear. Further inaccuracies are built into any limit equilibrium analyses due to the fact that both cohesion and friction angle are dependent upon displacement (Von Thun, 1977) and displacement is not calculated in a limit equilibrium analysis. Also, since displacements will vary throughout the slope, the amount of shear resistance developed on the failure plane or planes will also vary with location on the failure surface (Von Thun, 1977; Manfredini and Martinetti, 1977).

Barton has attempted to address these problems by using what he describes as a "Progressive Failure" method of analysis (Barton, 1971a). The following section will attempt to explain this in detail.

**Barton's Progressive Failure Analysis:** Barton's Progressive Failure concept has to date only been applied to plane failure geometries, but this author sees no reason why
it could not be extended to include the more general wedge failure.

The progressive failure concept assumes that the only stresses developed in the slope are due to overburden pressure and that stress concentrations will not occur at the toe of the slope as is predicted by elasticity based models (e.g., finite element methods). This is due to the fact that stress is redistributed in a discontinuous (jointed) rock mass so that stresses are very nearly lithostatic. This fact has been demonstrated in model tests as well as various discrete element analyses (Barton, 1971a).

Barton's analysis incorporates sliding on one joint set and tensile failures on a second joint set. The 2nd joint set is assumed to be vertical and thus divides the slidemass into slices in a manner similar to circular failure analysis. This will generally be a valid assumption since in most jointed rock slopes there will be a vertical or nearly vertical joint set across which non-zero tensile forces would cause tensile failure.

However, it should be noted that although the slidemass is divided into slices and is thus similar to circular failure analysis, there is no rotational component of motion. The failure is purely translational along a single joint set. This means that there is no shearing between slices and any forces transmitted across slice boundaries.
can only act normal to those boundaries. Barton's analysis also attempts to incorporate non-linear material properties into the analysis by assigning \( \sigma \) and \( \phi \) according to the stress acting at the base of each slice.

It should be noted that for many rock slope analyses the friction angle is used to obtain the magnitude of the shear strength mobilized and that the contribution to shear strength due to cohesion is assumed to be zero. In Barton's method a curved failure envelope is used and thus \( \sigma \) will not be zero except at zero normal stress and \( \phi \) and \( \sigma \) will vary with stress. In other words, a linear approximation tangent to the curved envelope at the desired stress level is used in a piecewise linear fashion. Thus, slope problems that would, with \( \sigma = 0 \), be dimensionless, now become dependent upon slope height.

Barton developed a concept he calls "unstable excess". The unstable excess \( (P) \), for a given slice, is calculated by summing the forces acting on the slice, including the shear strength mobilized, in the direction of the failure surface (with upslope taken as the positive direction). Then any slice having a positive \( P \) (net force acting up the slope) will be stable; while a slice with a negative \( P \) (net force acting down slope) will be unstable.

As with most other limit equilibrium methods, it is assumed that all forces act through the center of gravity of
the slice and thus no moments are created. It is further assumed that the unstable excess (or negative P) of any slice is immediately transferred to the slice downslope from it and, as stated earlier, this transfer of force causes no tendencies for rotation of either slice.

A table can then be constructed indicating the P value for each slice. Since the vertical joint set is assumed to have zero tensile strength a tension crack will form between the slices where the sign of P changes from positive to negative near the crest of the slope. Thus the location of the tension crack can be calculated for a given slope height (H).

Also the table of P values will tell you which slices are overstressed (negative P) and which are not (positive P). Thus failure zones (or zones of negative P) can be identified. As H is increased the location of the tension crack and the length and depth of the overstressed zone changes (Figure 6.5). Thus, as erosion (or excavation) of material at the base of the slope increases, a progressive failure zone is developed and will result in a stepped failure surface as indicated in Figure 6.5.

Barton also notes that as this progressive failure is occurring the slices in the overstressed zone will undergo increasing displacements and thus can be expected to mobilize shear stresses proportional to the residual friction angle (φ_r) rather than peak friction angle (φ_peak).
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Tension crack formation

Residual Block

Joints

Overstress zone changes as slope erodes thus creating a stepped failure surface.

Figure 6.5 Barton's Progressive Failure

Proportional limit

Plastic Zone

Figure 6.6 Stress-strain diagrams

(a) Linear

(b) Nonlinear
It is not obvious that displacements can occur or that they will be large enough to reach a $\phi_r$ condition. But Barton cites field evidence that indicates this is true. He also notes that as joints shear under low normal stress they dilate. This would tend to increase permeability, accelerate weathering, and thus further reduce joint shear strength.

Thus the failure mass can be divided into a residual block and a toe block (figure 6.5). The $P$ values are then summed for all slices (excluding those above the tension crack, since no forces will be transmitted across this boundary) for a number of different slope heights and these sums are plotted against the slope heights. Where this curve crosses the line $P = 0$ the slope height at failure can be determined. In other words, the height where the total unstable excess equals zero is also the height where the $FS = 1$.

Other Refinements: Von Thun has suggested that the assumption of rigid body translation in limit equilibrium analysis of wedge failure tends to cause the normal force on the two failure planes to be overestimated, since a wedge will also tend to change shape in some manner perpendicular to the direction of sliding (Von Thun, 1977). This would produce a shear stress in that direction and thus tend to reduce the net shear force in the direction of sliding. Thus the shear resistance mobilized on the failure planes
will be overestimated as will the factor of safety. He suggests that the slide wedge be divided into slices and that each slice be partitioned along a vertical plane passing through the line of intersection of the two failure planes. The weight of each of the partitioned slices will generate a maximum shear force according to the appropriate failure criterion directed in the dip direction of the plane it rests upon, thus reducing the shear resistance mobilized in the failure direction as well as the calculated factor of safety.

Finite element models of wedge failure using good joint elements indicated that wedge partitioning generally defined the lower bound FS while the upper bound FS is obtained from the normal wedge analysis (Von Thun, 1977).

It is suggested that perhaps this method could be combined with Barton's unstable excess concept to arrive at a good estimate of the FS in wedge failure analysis.

6.3 Stress and Displacement Methods

6.3.1 Introduction

There are three methods currently in use which yield the stress and displacement fields within a region or structure. These are: the theory of elasticity, the Finite Element Method (FEM), and the Finite Difference Method (FDM) (Desai, 1972).

The most basic method of determining the stress and deformation fields in a structure or body is through the use
of the theory of elasticity. This method requires that an essentially closed form solution of a set of partial differential equations be obtained. This equation is usually in the form of an infinite series approximation giving the stress or deformation at a point on or in the structure as a function of the coordinates of the point and the elastic constants for the structure. Solutions for simply shaped bodies with relatively simple loadings and boundary conditions are available, but solutions for odd shaped structures made of several types of material are extremely difficult to obtain.

The FDM is a numerical method that can be used when the governing differential equations for equilibrium and the boundary conditions are known, but an infinite series or other solution cannot be obtained. Stresses or displacements can be calculated at specific points within the structure, but between the points an interpolation function must be used in a separate calculation. As with theory of elasticity solutions, it is often possible to formulate the differential equations for a continuous homogeneous structure. But when complex boundary conditions or loads are included, and/or a non-homogeneous structure must be analyzed, the equations may not be possible to formulate, and when they are it is usually not possible to solve them (Desai, 1972).

The FEM does not have these disadvantages. The FEM
developed out of the idea that the structure as a whole can be characterized by the sum of its parts. Structural engineers have used this form of analysis for years to calculate the internal forces in framed (non-continuous) structures. More recent developments allow this concept to be taken one step farther by allowing the area within a non-homogeneous continuous structure to be divided into subareas that are assumed to be homogeneous. These subareas are known as elements.

Elements are defined by the coordinates of their corners (which are referred to as nodes). In three dimensions, each node can have up to six displacements and rotations also referred to as degrees of freedom (dof).

The essence of the FEM is the concept of a "piecewise approximation of a function \( \Phi \), by means of polynomials, each defined over a small region and expressed in terms of nodal values of the function" (Cook, 1981). \( \Phi \) represents the stress or displacement function in structural mechanics but can be voltage, hydraulic head, etc., depending on the type of field being studied.

The FEM was first suggested in the early 1940's and became a more or less practical reality in the mid-fifties with the development of the digital computer. This is due to the fact that hand calculations for even the simplest structure are prohibitively cumbersome due to the rather large matrix operations that are a part of the method.
6.3.2 Finite Element Method in Structural Mechanics (Cook, 1981)

The FEM can be used with either stresses or displacements as the primary unknowns. In structural mechanics the displacement method is, by far, the most common.

The displacement method makes use of a generalized Hooke's Law to arrive at the equations of equilibrium. Thus for an element,

$$[k] \{d\} = \{f\} \quad (6.1)$$

where

- $[k] = \text{element stiffness matrix}$
- $\{d\} = \text{element nodal displacement vector}$
- $\{f\} = \text{vector of element nodal loads}$.

Careful examination of Equation (6.1) will reveal that the $j$th row of $[k]$ can be interpreted as the nodal forces or moments that must be applied to the element in order to cause a unit displacement or rotation of any one degree of freedom.

Once the stiffness matrix for the elements have been formulated then these can be combined, in an expanded form, to obtain the structure stiffness matrix $[K]$. The structure stiffness matrix thus relates deformations of the structure to the forces applied to the structure by the relation

$$[K] \{D\} = \{R\}.$$
This can then be solved for the displacements (D) or

\[
\{D\} = [K]^{-1} \{R\}
\]

and thus the translations and/or rotations of each dof can be found. From these displacements the stresses can be calculated (knowing the material properties) from the relation

\[
\{\sigma\} = [E]\{\varepsilon\}
\]

where, in two dimensions,

\[
\{\varepsilon\} = \begin{bmatrix}
\frac{\partial}{\partial x} & 0 \\
0 & \frac{\partial}{\partial y}
\end{bmatrix}
\begin{bmatrix}
u \\
v
\end{bmatrix}
\]

with

\[
[E] = \text{material property matrix}
\]

\[
\{\sigma\} = \text{vector of normal and shear stresses}
\]

\[
\{\varepsilon\} = \text{strain matrix}
\]

\[
u, v = \text{displacements in x and y directions, respectively.}
\]

It should be noted that these relations, as formulated, assume small strains and thus linear material properties (constant moduli). Non-linear problems will be discussed later in this paper.

Many situations in geotechnical engineering can be formulated as plane strain problems. Thus \(\varepsilon_z = 0\) and \(\tau_{yx}\)
Therefore, in two dimensions, the material matrix will simplify to

\[
E = \begin{bmatrix}
1-v & v & 0 \\
v & 1-v & 0 \\
0 & 0 & \frac{1-2v}{2}
\end{bmatrix}
\]

In connection with this, it should be noted here that the capabilities of the FEM far exceed the knowledge of material behavior upon which the analysis must be based and, further, that anisotropic and/or nonlinear material properties (common in geotechnical applications) can cause significant error.

Another problem often encountered in geotechnical practice, and found to have a dominant effect on slope stability problems in particular (Duncan and Goodman, 1968; Chowdhury, 1976), are the initial, or insitu, stresses. If they can be determined, insitu stresses are easily handled by FEM.

Solid Elements: The previously outlined formulation of the stiffness matrix and thus the equations of equilibrium cannot be generalized to a solid element. This is because a solid will have an infinite number of degrees of freedom and thus requires a different formulation method utilizing numerical integration across the element or structure.

The most commonly used concept is referred to as the Raleigh-Ritz method. In this method the total potential
energy \( (\Pi_p) \) for the structure is minimized with respect to displacement to determine the stable equilibrium configuration.

In this method it is assumed that a polynomial of a given degree can be used to approximate the displacements of points on the interior of the element given the displacements of the nodes. Since a solid will have an infinite number of degrees of freedom, this will not yield an exact solution but as the number of nodes is increased it will approach the "exact" solution. The coefficients of these polynomials in \( x \), \( y \), and \( z \) can be obtained by setting \( \frac{\delta \Pi_p}{\delta d} = 0 \), remembering that the partial derivatives of the displacement function \( d \) with respect to \( x \), \( y \), and \( z \) are equal to the strains.

However, since these coefficients have little or no physical meaning, the displacement equations are better expressed in terms of the local element coordinates. These are also used to obtain the displacements within an element by interpolation given the displacements at the nodes and are based on the original displacement functions. These relations are generally referred to as shape functions.

Thus, using the shape functions and the minimum potential energy concept, an expression can be derived which can be utilized to solve for the nodal displacements within a solid structure given the geometry of the structure and the applied forces. These displacements will be "exact" if
the forces are applied at the nodes and approximate for distributed loads (i.e., gravity loads and water pressure loads) or point loads not applied at the nodes.

There are a large number of elements that have been developed and, as the FEM gains popularity, more will be developed to fit the many ways in which a given element may be expected to perform.

One of these, the isoparametric element, has found wide usage in many fields. An element is termed "isoparametric" if the same shape function, expressed in terms of a local curvilinear coordinate system, (1) can be used to calculate the displacements within an element, given the displacements of the nodes; and (2) can be used to determine the global coordinates of a point within the element given the local coordinates of that point.

These elements can have curved boundaries and have been found to be more accurate than other types of elements. They can thus perform the same task with the same level of accuracy as other elements while requiring fewer elements and thus less computer time. The main drawback to isoparametric elements is that they cannot have rotational degrees of freedom. However, this does not generally pose a significant problem in most applications.

6.3.3 The Finite Element Method in Nonlinear Structures

It is well known that most problems in geotechnical engineering, especially those involving large strains, are
nonlinear in nature. The nonlinearity can take on one of two forms: (1) geometric nonlinearity and (2) material nonlinearity (Cook, 1981).

Geometric Nonlinearity: Geometric nonlinearity may result when very large strains are encountered in a structure. These excessive strains can cause the geometry of the structure to be changed so radically that the stiffness of the structure is altered. This has not been found to be an important consideration in geotechnical applications (Desai, 1972) and will not be discussed further.

Material Nonlinearity: When strains go beyond the proportional limit, if the material can, indeed, be said to have a proportional limit, and into the plastic region (Figure 6.6(a)) the problem becomes nonlinear. Also, as shown in Figure 6.6(b), geologic materials are generally extremely nonlinear throughout the strain range. Thus it is obvious that, in order to arrive at realistic values for stresses and deformations using FEM, it is very important to attempt to incorporate material nonlinearities into the analysis.

The problem is that the equations of equilibrium must be formulated according to the displacements since \([K]\) now becomes a function of displacement. But the displacements are unknown prior to the analysis. Also, if plastic strains
take place, the problem is path dependant. Since many of the available FE programs were written for use in structural engineering and thus are designed to consider only elastic materials, a number of schemes utilizing these programs have been developed. Most of the methods employed fall in one of two categories: (1) incremental method and, (2) the iterative method (Desai, 1972).

In the incremental method the load is applied in increments and a stiffness matrix based on the displacements from the previous increment is used to calculate the displacements for the current increment. The displacements are summed, the stiffness matrix is updated, and the process is repeated until the full load is reached.

In the iterative method the full load is applied with the stiffness determined assuming zero strains. The displacements are calculated and the stiffness matrix is changed to fit these displacements and the full load is again applied in a new analysis. This process is repeated until the desired level of accuracy is reached.

The reader should note that the above two methods can be combined such that the iterative method can be used within each increment of loading in the incremental method.

5.3.4 The Finite Element Method as it Applies to Rock Slope Stability

Studies have shown that, if reasonable stress-strain
characteristics are employed, the agreement between the
displacements calculated by FEM and measured displacements
in slopes is very good. The method has been found to be
useful in the prediction of local failure zones as well as
for general slope stability (Duncan, 1972). However, in
connection with slope failure problems, it should be noted
that when the behavior of the entire slope is governed by
failed zones the results from FE analysis have been less
than satisfactory (Duncan, 1972). Thus in some slope
problems FE analysis is not much better than limit
equilibrium methods and is far more expensive.

However, when the expense is justified there are three
factors that are essential to the building of a reasonable
FE model of the slope. These are:

1) Geometry of the slope,
2) Material properties, and
3) In situ Stresses.

The geometry of the slope is defined by the ground
surface and by the discontinuities in the rock mass.
Although there are many uncertainties associated with the
determination of the orientations of the discontinuities
within the rock mass, it is probably the best defined aspect
of the slope stability analysis. Since this subject was
discussed thoroughly elsewhere in this paper no further
discussion will be presented here.

Material properties are probably the most important
aspect of a slope stability analysis. This subject is also covered elsewhere in this paper and therefore will not be discussed further.

Insitu stresses have been found to be of importance according to FE studies done by Goodman and Duncan (1963). However, the methods developed to date for the determination of insitu stresses are difficult and expensive to perform. Also, a complete analysis would require a knowledge of insitu normal and shear stresses acting on the various discontinuity surfaces within the rock mass and no practical method for obtaining this information has been developed.

Many methods have been suggested for estimating insitu stresses. The simplest being to assume that self weight or gravity force is the only important force acting. Thus the vertical stress becomes \( \gamma \cdot z \) (\( \gamma \) = total unit weight of the in place material, \( z \) = depth below the surface) and the horizontal stress is assumed to be \( K \cdot \gamma \cdot z \) where \( K \) is a constant chosen by the engineer in some rational manner. If so-called "\( K_0 \) conditions" are expected then \( K = K_0 = \frac{v}{1-v} \) can be used.

However, for any case other than a horizontal ground surface, the depth below the surface will give a poor estimate of actual vertical stresses. Duncan (1972) has suggested that a FE analysis be run with gravity force only, then the calculated vertical stress can be used in conjunction with \( K_0 \) to estimate the insitu horizontal
stresses. Goodman and Duncan (1968), when investigating excavations in a horizontal surface, varied $K$ over a reasonable range to determine the sensitivity of the analyses to changes in horizontal stress. This same method could be employed in other slope studies.

**Linear Finite Element Analysis of Slopes:** Goodman and Duncan (1968) have suggested five different types of linear models for FE analysis of slopes. They are:

1. unjointed linear elastic analysis,
2. ubiquitous joint analysis,
3. orthotropic continuum analysis,
4. two-dimensional joint element analysis, and
5. one-dimensional joint element analysis.

The unjointed linear elastic analysis employs the standard FE analysis with the force of gravity applied and the stresses and displacements calculated in the normal manner.

The ubiquitous joint analysis takes the stresses from the unjointed linear elastic analysis and uses them to determine the resultant normal and shear stresses on variously oriented planes at the points within the slope where the stresses were calculated. It then compares those values of stresses to the appropriate strength criterion to determine if failure is probable at that joint orientation. This method assumes that a joint of any given orientation
will pass through all points in the slope, thus its name: ubiquitous joint analysis.

The orthotropic continuum analysis requires three sets of orthogonal or nearly orthogonal joints. It further assumes that the jointed rock mass can then be replaced by an elastic orthotropic continuum that incorporates the properties of the joints plus the intact rock in a set of three orthogonal material property vectors (see Chapter 5).

The two-dimensional joint element analysis attempts to model the joints and the intact rock blocks with elements having different material properties. This is a useful method for "thick" joints containing much weathered rock and/or infilling.

The one-dimensional joint element analysis utilizes an element developed by Goodman, Taylor, and Brekke (1968). This element was designed to model the type of joint that is very "thin" or where there is good rock to rock contact. It was found that the results obtained using this one-dimensional joint element did not vary significantly from the two-dimensional joint element if equivalent properties were used for both types of elements (Goodman and Duncan, 1968).

A problem that was mentioned by Goodman and Duncan (1968), but not addressed, was that of tension across joints. Most workers in the rock mechanics field recognize that rock is relatively weak in tension. Thus, except for
the case of healed mineralized joints, it is a good assumption that the tensile strength of a joint is very nearly zero. However, the FE studies done by Goodman and Duncan as well as field observations would suggest that somewhere at or behind the crest of a slope an area of tensile stresses will build up. But if the material of the slope is jointed these tensile stresses would not build up, they would instead be transferred to another part of the slope. A method of analysis utilizing the FEM with a provision for the transfer of tensile stresses to neighboring elements has been suggested (Zienkiewicz et al., 1971). In the "no tension" method an unjointed linear elastic FE analysis is performed with all forces (including weight, water pressures, etc.) acting all at once. The output is analyzed and any elements with tensile stresses are identified. The stresses within these elements are then negated by applying self equilibrating nodal forces to the nodes of the element in question. The analysis is then repeated using the original geometry and full forces, each time applying nodal forces in order to eliminate any tensile stresses, until all tensile stresses in the slope become zero or nearly zero. In this way it is possible to approximate a jointed material in an analysis where the material is treated as an elastic continuum.

Nonlinear analysis was covered earlier in this chapter but it should be noted here that, although geologic
materials are nonlinear, it is possible to back analyze a failed slope to determine its equivalent linear properties (Duncan, 1972). In order to do this it is necessary to assume some appropriate material properties and use these in a linear FE analysis. The calculated displacements can then be compared with those actually measured in the field and the material properties of each element or element type changed appropriately and another analysis run using these new properties. This sequence is repeated until the calculated and actual displacements match within the desired range of accuracy.

The results of such a study would only give the equivalent linear properties of the particular slope investigated. These properties could not be generalized to other slopes or materials unless the stresses anticipated and the materials found in the slope were very similar or identical to those of the slope originally studied.

6.4 Conclusions

There are a large variety of possible methods of analysis but it must be remembered that the geometry of the slope and its discontinuities will generally reduce the choice of methods greatly. However, there is always the choice between a limit equilibrium type of analysis and a stress or displacement type. Economics and/or lack of access to a large computer will generally restrict the use of the stress or displacement methods. But, whatever type
of analysis is chosen, it must be remembered that any analysis is only as good as the data used in that analysis. Thus it is of utmost importance to choose the material properties, geometry, water pressure distribution, and insitu stresses based on the best available field or lab evidence.
CHAPTER 7

ANALYSES and CONCLUSIONS

In Chapter 1 it was stated that there are four basic aspects to any slope stability investigation. These are the determination of:

1. Insitu stresses,
2. The geometry of the slope,
3. The mean orientations of the joints within the rockmass, and
4. The mechanical properties of the rockmass.

As was discussed earlier, there are many methods for the determination of the insitu stresses. However, all the available methods are expensive and the accuracy and usefulness of the results are open to question. Therefore, due to a lack of the proper equipment and the necessary funds, no initial insitu stress determination was undertaken for this analysis.

7.1 Geometry of the Slopes Studied

In order to determine the geometry of the various slopes, 7.5 minute USGS topographic quadrangles (1968) were obtained and profiles were constructed based on the map topography. For the East Slide Area - South, the cross section AA' (Figure 7.1) was used to create the profile shown in Figure 7.2.

No cross sections or profiles were developed for either
**FIGURE 7.1** Cross section, soil sample, test pit and geophysical testing location.
FIGURE 7.2 Profile of East Slide Area-South showing water levels
the East Slide Area-Middle or East Slide Area-North since no significant, kinematically possible failure modes were identified in these areas.

Since it was determined that there existed some possibility of a wedge type failure in the North slide area, the slope was modeled by the simple three plane profile shown later in Figure 7.6.

7.2 The Mean Orientations of the Joints within the Rockmass

In Chapter 4, the mean joint orientations were presented in the form of stereographic projections for the various areas of Slide Mountain. These mean joint orientations were determined by statistical analysis of the joint orientation data collected in the field. For easy reference, the mean joint orientations used in the various analyses that follow are reproduced in tabular form in Table 7.1.

There are two joint sets which were found to be of importance in the analysis of the East Slide Area-South. These are the nearly vertical joint set number two and the lower angle joint set number three. Joint set number three was found to allow the possibility of planar failure with the formation of a tension crack and water conduit along joint set two. Joint set two also offers the possibility of a toppling failure mode.
Table 7.1: Mean Joint Orientations Used in Analysis

<table>
<thead>
<tr>
<th>Area</th>
<th>Joint Set Designation</th>
<th>Joint Set Number*</th>
<th>Dip (Degrees)</th>
<th>Dip Direction (Azimuth in Degrees)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>East Slide Area</td>
<td>South</td>
<td>2</td>
<td>82</td>
<td>313</td>
<td>Allows toppling, may form tension cracks</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>29</td>
<td>140</td>
<td></td>
</tr>
<tr>
<td></td>
<td>North</td>
<td>1</td>
<td>90</td>
<td>188</td>
<td>Joint sets 1 and 2 may form a column of rock which may topple about its base on joint set 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>90</td>
<td>274</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>37</td>
<td>273</td>
<td></td>
</tr>
<tr>
<td>North Slide Area</td>
<td>1</td>
<td>87</td>
<td>347</td>
<td></td>
<td>Joint sets 1 and 2 form wedge</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>28</td>
<td>098</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* See Stereographic Projections - Chapter 3
No kinematically possible failure modes were found given the joint orientations in the East Slide Area-Middle.

In the East Slide Area-North, it was found that there existed the possibility of minor toppling failures occurring through the combined effects of joint sets one and two forming rock columns which could topple at the base on joint set three.

It was determined that in the North Slide Area there was a kinematically possible wedge failure mode. This mode involved the possible failure of a wedge or wedges of rock formed by the intersection of joint sets one and two.

Nearly all areas of Slide Mountain could be subject to circular failure in the rock zones that show weakening or accelerated weathering due to being (or having been) below the water table, or having been crushed by past tectonic movements. However, of the areas studied in detail in this investigation, only the East Slide Area-South was found to be appreciably affected by excess rock weathering processes or tectonic movements, and thus only this area is analyzed with regard to circular failure modes.

7.3 Mechanical Properties of the Rockmass

The material properties chosen for use in the various methods of analysis are summarized in Table 7.2.

In general, when test data was available these strengths were used in the analyses if the empirical strength criteria supported the test results. Thus for the
### Table 7.2: Summary of Material Properties Used for the Various Analyses

<table>
<thead>
<tr>
<th>Type of Analysis</th>
<th>Material Properties (( = 152.9 ) pcf in all cases)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>East Slide Area—South:</strong></td>
<td></td>
</tr>
<tr>
<td>Circular Failure</td>
<td></td>
</tr>
<tr>
<td>Planar Failure</td>
<td></td>
</tr>
<tr>
<td>Lower Bound Shear Strength:</td>
<td>( C = 3 ) psi ( \phi = 37^\circ )</td>
</tr>
<tr>
<td>Average Shear Strength:</td>
<td>( C = 2.8 ) psi ( \phi = 48^\circ )</td>
</tr>
<tr>
<td>Upper Bound Shear Strength:</td>
<td>( C = 48.8 ) psi ( \phi = 42^\circ )</td>
</tr>
<tr>
<td>Barton's Non-Linear Strength Envelope:</td>
<td>( 560 ) psi* = ( n \tan (10 \log_{10} n + 32.5^\circ) )</td>
</tr>
<tr>
<td><strong>Planar Failure</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Barton Progressive Failure</strong></td>
<td></td>
</tr>
<tr>
<td><strong>FEM Analysis</strong></td>
<td></td>
</tr>
<tr>
<td>( E_1 = 3.0 \times 10^7 ) psf (normal to joint set #3)</td>
<td></td>
</tr>
<tr>
<td>( E_2 = 8.8 \times 10^6 ) psf (parallel to joint set #3)</td>
<td></td>
</tr>
<tr>
<td>( G_{12} = 1.0 \times 10^7 ) psf (10' x 20' block size using low shear stiffness ( [K_{sm,n}] ))</td>
<td></td>
</tr>
<tr>
<td>( v = .001 )</td>
<td></td>
</tr>
<tr>
<td><strong>North Slide Area:</strong></td>
<td></td>
</tr>
<tr>
<td>Wedge Failure</td>
<td>( C = 16 ) psi ( \phi = 40^\circ )</td>
</tr>
</tbody>
</table>

* 560 psi was obtained by taking the unconfined compressive strength of the joint wall to be that of average Grade V Rock (Krank and Watters, 1982) and multiplying it by .75 in order to account for strength loss due to saturation, as suggested by Barton (1973)
planar, circular, and wedge-failure modes the upper bound shear strength was taken as $\phi = 42$ degrees, $C = 48.3$ psi (shear test 6), the lower bound as $\phi = 37$ degrees, $C = 3.0$ psi (shear test 4), with the most probable, or average, strength represented by $\phi = 46$ degrees, $C = 2.8$ psi (Krank and Watters; Grade V). The Barton non-linear envelope (using the average unconfined compressive strength from Krank and Watters-Grade V as $\sigma_j$) was also used in circular and planar failure analyses.

The elastic constants used in the FE analysis of the failed slope in the East Slide Area-South are also presented in Table 7.2. The deformation modulus was calculated for the orthotropic continuum analysis by using what was thought to be the most representative joint spacing ($S$) and joint thickness ($T_j$). In the direction perpendicular to the low angle joint set three the joint spacing was estimated to be about 20 feet with a joint thickness of about three inches, resulting in an equivalent deformation modulus ($E_1$) of $3.0 \times 10^7$ psf ($2.1 \times 10^5$ psi). In the direction parallel to joint set three, the joint spacing was estimated to average about 10 feet while the joint thickness, due to a greater amount of weathering, was estimated to be about six inches resulting in $E_2 = 8.8 \times 10^6$ psf ($6.1 \times 10^4$ psi). The shear modulus ($G$) for 10 foot x 20 foot blocks was calculated as $1.0 \times 10^7$ psf ($7.0 \times 10^4$ psi) using the minimum shear stiffness ($K_{s_{\text{min}}}$). Poisson’s Ratio, although assumed to be
0.0 for the Duncan and Goodman model to be valid, was taken as 0.001 in order to avoid computational difficulties.

7.4 Methods used to Perform the Analyses

7.4.1 East Slide Area-South

In order to analyze the failures in the East Slide Area-South, a number of types of analyses were used. In all but the toppling failure and liquefaction analyses a similar approach was taken. This approach was to set up the geometry of the slope, assign the appropriate material properties, then run the analysis a number of times varying the water level in the slope in an attempt to determine the effects of water on the stability of the slope.

As shown in Figure 7.2, the base level or lowest possible water table elevation in the cross section was arbitrarily assumed to be the approximate current stream bed elevation of Ophir Creek. The water table elevation was then raised from this base elevation until reaching the ground surface.

It should be noted that the water table was assumed to be at a static level and no seepage forces were considered in any of the analyses. This may be an unrealistic assumption given the fact that prior to the slide several very warm days caused rapid snowmelt and this may have caused significant seepage forces to be developed. However, since no reasonable estimate could be made of the magnitude or direction of seepage forces as well as the fact that none
of the slope stability models provided for the inclusion of these forces, no seepage forces were considered in the analyses.

The failure modes investigated for the East Slide Area-South were:

1. Planar failure,
2. Circular failure, and
3. Toppling failure.

**Planar failure:** The dip and dip directions of the joints near the failed slope (see Figures 3.2 and 4.4) would indicate that a planar failure is a very good possibility. Also, seismic refraction studies done on the block slide area (see Figure 7.1) indicated that a denser, less weathered rock was located about 40 feet below the surface. Thus it might be concluded that a planar failure may have occurred as illustrated in Figure 7.3 with the failure plan passing through the point determined by the geophysical data. In order to investigate this possibility, a computer program that employs the method of slices and allows the analyses of planar failure modes was used in conjunction with the slope and failure plane geometry shown in Figure 7.3. (See Circular failure, this section, for a description of the program.)

The high, average, and low shear strength criteria, as well as the nonlinear strength envelope of Barton (approximated by changing the $\phi$ and $C$ parameters as a
FIGURE 7.3 Profile of East Slide Area - South showing geometries for planar and circular failures and location of origin of coordinates (in order to locate centers of circular failure surfaces).
function of depth below the surface in a layered model) were each used individually as the water table was raised in steps (see Figure 7.2) until reaching the surface.

**Circular failure:** In order to investigate the possibility of circular failure as well as planar failure, the program GENSAM (General Slip Surface Analysis including Monte Carlo Simulation) developed by S. G. Wright (1976), was used with the same slope surface geometry as was used in the planar failure analyses (see Figure 7.2). As with the planar failure analyses, the full range of shear strengths were used in conjunction with varying the piezometric surface in a series of runs.

The program GENSAM is based on the method of slices as developed by Spencer (1967). However Spencer's method was generalized by Wright to include the analysis of general planar failure surfaces in addition to circular surfaces and to allow for the inclusion of arbitrary surface loads.

Spencer's method completely satisfies static equilibrium, including both forces and moments, by making the simplifying assumption that all interslice forces are parallel to each other. Because it satisfies complete equilibrium this method can be considered to be one of the most rigorous and, therefore, most accurate limit equilibrium methods available.

**Toppling failure:** As discussed in Chapter 4, the high angle joint set number one (Figure 4.4(b)) is oriented in
such a way as to make toppling failure kinematically possible. This analysis was performed using the simple block on an inclined plane model as described in Chapter 4.

Finite element analysis: In addition to the limit equilibrium analyses performed as described above, a FE slope stability analysis was also attempted. This modeled the slope as having the same ground surface geometry as was used for both the circular and planar failure analyses, and utilized the Orthotropic Continuum (Duncan and Goodman, 1968) concept in order to assign equivalent material properties to the jointed rock slope.

The finite element model employed the program SAP IV (Bathe, et al., 1973), and consisted of 293 plane strain linear elastic elements and, initially, 322 nodes (see Figure 7.4).

In order to model the formation of tension cracks, a series of runs were done with gravity increased from 0.1 g (1 g = the acceleration of gravity = 32.2 ft/sec$^2$) by 0.2 g steps until the full weight, or 1.0 g, was being applied to the slope. After each step the elements which had developed tensile stresses were noted and extra nodes were added in order to create "tension cracks" in the slope (see Figure 7.5). This method of introducing tension cracks caused the number of nodes to be increased from the original 322 to 352.

During this process, the piezometric surface was kept
FIGURE 7.4 Finite element model used in the analysis of the recent slope failure in the East Slide Area South.
Water Filled Tension Crack

EQUIVALENT NODAL FORCES:

\[ NF_1 = -NF_3 = \frac{1}{3}F_{\text{water}} \]

\[ NF_4 = -NF_2 = \frac{2}{3}F_{\text{water}} + \frac{1}{2}F_{\text{water}} + \frac{1}{3}F_{\text{water}} \]

\[ F_{\text{water}} = \text{WATER FORCE ON SIDE OF ELEMENT } n \]

\[ NF_n = \text{EQUIVALENT WATER FORCE TO BE APPLIED TO NODE } n \]

FIGURE 7.5 Schematic diagram illustrating how tension cracks were formed and equivalent water forces applied to nodes in finite element model of the slope.
at the base level. Water forces were modeled by giving the rock mass below the water table a unit weight equal to the buoyant unit weight of the rock (90.5 pcf), while placing equivalent horizontal nodal forces on elements with tension cracks which terminated below the water table (see Figure 7.5).

Once the locations of the tension cracks were established, the piezometric surface was successively raised by giving the material below the new water table the lower, buoyant unit weight. Appropriate equivalent nodal forces were then applied to those nodes which were in submerged tension cracks.

7.4.2 East Slide Area-Middle

Since no failure modes were found to be kinematically possible, no analysis was performed for this area.

7.4.3 East Slide Area-North

Since it was previously determined that only a toppling mode of failure was possible in this area, the simple block on a plan analysis was used.

7.4.4 North Slide Area

It was found that, kinematically, the most probable mode of slope failure in the North Slide Area is a wedge failure.

Thus the analysis of this slope was performed using the wedge failure analysis as described by Hoek and Bray (1981) and outlined in Chapter 4. This analysis was performed
using the equation:

\[
F = \frac{3}{YH} (C_A \cdot X + C_B \cdot X) + (A - \frac{g}{2} \cdot X) \tan \phi_A \\
+ (B - \frac{g}{2} \cdot Y) \tan \phi_B
\]

(refer to Figure 7.6 for the geometry of the problem)

- \( F \) = Factor of safety against wedge failure
- \( C_A \) and \( C_B \) = cohesion of Planes A and B, respectively
- \( \phi_A \) and \( \phi_B \) = friction angles of planes A and B, respectively
- \( g \) = unit weight of the rock
- \( g_w \) = unit weight of water (62.4 pcf)
- \( H \) = total height of the wedge

\[
X = \frac{\sin \Theta_{24}}{\sin \Theta_{13}} \\
Y = \frac{\sin \Theta_{34} \cdot \cos \Theta_{11b} \cdot \cos \Theta_{na nb}}{\sin \Theta_{5} \cdot \sin^2 \Theta_{na nb}} \\
A = \frac{\cos \Theta_a - \cos \Theta_b \cdot \cos \Theta_{na nb}}{\sin \Theta_5 \cdot \sin^2 \Theta_{na nb}} \\
B = \frac{\cos \Theta_b - \cos \Theta_a \cdot \cos \Theta_{na nb}}{\sin \Theta_5 \cdot \sin^2 \Theta_{na nb}}
\]

- \( \Theta_a \) and \( \Theta_b \) = dips of planes A and B, respectively
- \( \Theta_5 \) = the dip of the line 5 (the intersection of Planes A and B)
- \( \Theta_{24} \) = the angle of intersection of lines 2 and 4
- \( \Theta_{45} \) = the angle of intersection of lines 4 and 5
FIGURE 7.6 Diagram describing the geometry of the wedge failure analysis used to analyse the North Slide Area

a. Pictorial view of wedge showing the numbering of intersection lines and planes.

b. View normal to the line of intersection 5 showing the total wedge height and the water pressure distribution.
\( \theta_{2na} \) = the angle of intersection of line 2 with the normal to plane A

\( \theta_{13} \) = the angle of intersection of line 1 and line 3

\( \theta_{35} \) = the angle of intersection of line 3 and line 5

\( \theta_{1nb} \) = the angle of intersection of line 1 and the normal to plane B

\( \theta_{na \ nb} \) = angle of intersection of the normals to planes A and B.

All of the angles for this equation can be obtained using either stereonet or vector analysis.

This model assumes the worst case water condition, that is, water at the surface. Water pressure forces are determined by assuming the maximum pressure will occur at a location on line 5 one-half \( H \) above the wedge tip and that water pressure will be reduced to zero along the lines 1, 2, 3, and 4 where they daylight.

7.5 Results of the Analyses

7.5.1 East Slide Area-South

**Circular and Planar Modes:** The resulting factors of safety for the block slide area, given the various strength criteria and water table levels for both planar and circular failure modes, are summarized in Table 7.3.

The results for both planar and circular failure modes
### Table 7.3: Results of Circular and Planar Failure Analyses: East Slide Area-South

<table>
<thead>
<tr>
<th>Water Table Number</th>
<th>Shear Strength</th>
<th>F.S.</th>
<th>Radius of Arc (feet)</th>
<th>Center Coordinates&lt;sup&gt;c&lt;/sup&gt; (Elev.-MSL)</th>
<th>Planar Failure&lt;sup&gt;d&lt;/sup&gt; F.S.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water #2</td>
<td>L</td>
<td>0.87</td>
<td>1461</td>
<td>X 2260   Y 8690</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>M</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>0.83</td>
<td>1521</td>
<td>X 2240   Y 8770</td>
<td>1.23</td>
</tr>
<tr>
<td>Water #3</td>
<td>L</td>
<td>0.82</td>
<td>1540</td>
<td>X 2160   Y 8790</td>
<td>1.08</td>
</tr>
<tr>
<td></td>
<td>M</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>0.88</td>
<td>1384</td>
<td>X 1940   Y 8670</td>
<td>1.17</td>
</tr>
<tr>
<td>Water #4</td>
<td>L</td>
<td>0.76</td>
<td>1881</td>
<td>X 2320   Y 9110</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>M</td>
<td>1.73</td>
<td>1891</td>
<td>X 2320   Y 9110</td>
<td>1.18</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>0.81</td>
<td>1740</td>
<td>X 2240   Y 8990</td>
<td>1.11</td>
</tr>
<tr>
<td>Water #5</td>
<td>L</td>
<td>0.64</td>
<td>2789</td>
<td>X 2680   Y 9990</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td>M</td>
<td>0.88</td>
<td>2403</td>
<td>X 2460   Y 9670</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>0.72</td>
<td>1980</td>
<td>X 2260   Y 9230</td>
<td>0.84</td>
</tr>
<tr>
<td>Water #6</td>
<td>L</td>
<td>0.56</td>
<td>3274</td>
<td>X 2680   Y 10610</td>
<td>0.63</td>
</tr>
<tr>
<td></td>
<td>M</td>
<td>0.78</td>
<td>3640</td>
<td>X 3000   Y 10790</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>1.27</td>
<td>2000</td>
<td>X 2040   Y 9250</td>
<td>1.53</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>0.72</td>
<td>2284</td>
<td>X 2140   Y 9570</td>
<td>--</td>
</tr>
</tbody>
</table>

**NOTES:**

a. Refer to Figure 7.2 for water tables. Water table #1 was not used since all circles for water #2 were above this level.

b. L = low strength; M = average strength, H = high strength, and B = Barton's nonlinear strength criteria.

c. See Figure 7.2 for location of origin of x-coordinates.

d. Refer to Figure 7.3.
indicate that, using the low strength criteria, the slope will fail for all water levels above water #2. This is clearly unreasonable since water levels on the slope would most probably exceed this every year. Also, if the high strength criteria is used, the slope does not fail in either mode, even with water at the surface. This, again, is obviously unreasonable given the fact that the slope has failed. This leads to the conclusion that the "average" shear strength parameters seem to yield the most reasonable results.

The Barton strength criteria seems to yield reasonable factors of safety for the planar failure mode, but in the case of the circular failure mode, it yields similar results to that of the low strength parameters.

Thus it would seem that the "average" shear strength parameters generally gave the most reliable and plausible factors of safety for both circular and planar failure modes. These factors of safety indicate that water levels #4 (in the case of circular failure) and #5 (in the case of planar failure) are the critical water table locations for this slope.

Inspection of the critical failure circle (Figure 7.3) determined using the average shear strength parameters reveals that this failure surface location cannot be supported by the field evidence. This circle would exit from a point very near the failed dam on Upper Price Lake
and no indication of slope failure damage was found in this area.

A progressive failure analysis (as described by Barton and outlined in Chapter 4) was performed using the same Barton strength criteria as was used for circular failure. This analysis indicated that no tension cracks or failure zones would develop in the slope even with the water table at the surface. This leads to the conclusion that even though the progressive failure mechanism as described by Barton may well be appropriate for this slope, the Barton nonlinear failure criteria is not valid for joints where weathering has proceeded to the Grade V or VI stage. Barton suggests that this criteria is valid for weathered joints but not for filled joints. However, it would be difficult to establish that there is any appreciable difference in the mechanical behavior of joint fill versus a Grade V or VI weathered rock material. Thus it is suggested here that Barton's strength criteria only be used on rock joints where joint weathering has not gone beyond Grade IV.

Toppling failure mode: Using the simple block on a plane analysis of toppling involving slabs created by joint sets one and three (Figure 4.4(b)) results in the following relationship:

\[ \theta = 0.3 \]

where
The above relationship indicates that a 20 foot high block would have to have a base length of at least 5.6 feet in order to be marginally stable, while a base dimension of anything less than 5.6 feet would fail by toppling. Field observations indicate that blocks of such dimensions could have been created by the observed jointing patterns, especially in the area above the scarp at the head of the recent slide.

**Finite element analysis:** A finite element analysis was performed in order to establish the probable location of tension cracking in the slope, as well as define what areas within the slope were overstressed in shear.

Figure 7.7 indicates the locations of the tension cracks as determined in the FE analysis as well as the location of the actual tension crack/scarp as determined in the field. Note that tensile stresses were not only developed at or near the crest, as might be expected, but also just beyond the toe of the slope. This is the result of toe heave, or bulging, inducing tensile stresses and thus causing tensile cracking.

Once the tension cracks were established the "water table" was incrementally raised in the slope and a finite element analysis run with full gravity forces acting on the slope. The weakness of this type of analysis is that...
FIGURE 7.7 Finite element model showing locations of tension cracks (dashed lines).
once the shear stresses in an element reach the shear strength of the rock mass, then, in order to model the behavior of the actual slope, large deformations would have to occur with the shear stresses remaining essentially constant. Thus it would be necessary to change the shear modulus of the failed elements in such a way as to limit the shear stresses in the "failed" element to the failure stresses. The FE program used made no allowance for this type of behavior and no attempt was made to simulate this effect by changing the elastic properties on successive runs as the water table was raised. This means that a "failed" element would not cause stresses to be redistributed to the neighboring elements, and thus tended to make the slope more stable than it would be if more realistic material behavior could be modeled.

The above discussed complication, plus the fact that a linear elastic FE model was used, greatly limits the usefulness of the results of this type of analysis. Nevertheless, some insight can be gained regarding which areas in the slope are the most susceptible to shear failure, and thus what is the most likely mode of failure.

Through the use of a computer program written by the author, the output from the finite element model was evaluated in order to determine the shearing stresses developed at the center of each element in the direction of the low angle joint set number three. These shear stresses
were then compared to those that could be developed given the normal stress on the low angle plane (as calculated in the FE model for the element being considered) and the appropriate shear strength parameters, C and $\phi$.

Contours of the element factors of safety in the slope are shown in Figure 7.8. These factors of safety were determined by using the low shear strength parameters ($\phi = 37$ degrees, $C = 3$ psi) for the case of water at the surface of the slope (water #6).

As Figure 7.8 illustrates, there are areas near the crest, at midslope, and at the toe that are beyond or relatively close to failure. The area near the crest involves a large zone where the factor of safety is at 1.0 or below. Except for a very thin zone beyond the toe of the slope, no other areas show factors of safety below 1.0.

However, if the factor of safety contours are viewed as showing relative susceptibility to shear failure instead of absolute factors of safety, the most likely failure zones can be identified. It is interesting to note that the contour for a factor of safety of 2.0 passes through the point determined by seismic refraction to be on the failure plane, and also passes through a point at or very near the projected base of the tension crack/scarp of the slide (as determined by field mapping). Also, the general shape of the contours as well as the values of the factors of safety would indicate the possibility of a relatively deep failure.
FIGURE 7.8  Factor of safety contours determined by finite element analysis
zone near the crest, and a shallow failure zone from midslope to somewhat beyond the toe. A planar failure would produce this type of geometry while a circular failure with similar geometry would be very uncharacteristic (Lambe and Whitman, 1969).

**Liquefaction:** Liquefaction and/or cyclic mobility generally only occurs in saturated cohesionless soils (sands and silts) due to cyclic loading usually caused by earthquake induced ground motions. Liquefaction occurs when effective stress is reduced to zero by pore pressure buildup induced by repeated shear strain reversals. When this occurs the soil looses shear strength, becomes essentially a viscous fluid, and fails. Cyclic mobility, on the other hand, occurs in denser soils. The pore pressure buildup causes loss of shear strength but results in a limited amount of shear strain (generally not greater than 20 percent [Seed, 1975]) before pore pressures are reduced and shear strength is regained.

There are several factors which are important in assessing the potential for liquefaction and/or cyclic mobility. Of these, the most import are: 1) the ratio of dynamic shear stresses in the soil to the vertical effective stress, and 2) the relative density ($D_r$) of the soil.

There is a practical maximum acceleration and thus a maximum shear stress that can be produced by even the largest earthquakes. As the soil gets deeper, thus
increasing the effective stresses, the ratio of maximum possible shear stress to effective stress tends to become smaller. This factor generally precludes liquefaction and/or cyclic mobility at depths greater than approximately 50 feet below the ground surface. Also, most researchers agree that there is a relative density beyond which liquefaction cannot occur. Generally, beyond a Dr = 70 percent, neither liquefaction nor cyclic mobility can occur (Casagrande, 1975).

Liquefaction can occur in sands below a relative density of about 40%, and cyclic mobility may occur at relative densities greater than this (Casagrande, 1975; De Alba, et al., 1975; Seed, 1976). Field and lab tests of the material in the sand slide indicate that it was at a Dr of about 53% and field observation indicates that it was saturated prior to sliding. Thus it is obvious that this deposit was capable of undergoing large strains under dynamic loading and may have been capable of liquefying. The chart presented by Seed (1975) relating limiting shear strain to Dr indicates that this material could undergo shear strains exceeding 30%, and thus could fail catastrophically under dynamic loading.

Tokimatsu and Yoshimi (1984) present data that indicates that the majority of the instances of liquefaction occur in soils having a mean grain size (D50) of less than .6 mm. They also indicate that a clean sand with an SPT
blowcount (corrected to 1 tsf) of less than 10 is very likely to liquify. The sand in the sand slide had a \( D_{50} = 0.57 \) mm (see Appendix A) and a corrected SPT blowcount of approximately 5 (calculated from \( D_r \) correlation [Gibbs and Holtz, 1957]), and thus gives further support to the susceptibility of this deposit to liquefaction.

It is postulated here that the ground motions induced by the various components of the slide (block slide and rockslide) caused the sand in the sand slide area to liquify, fail, and slide over the unmelted snow below the failure zones. The fact that the sand was deposited in an extremely loose state (\( \gamma_d = 81 \) pcf, less than the lab determined minimum dry density of 90 pcf) gives further evidence as to its liquid form just prior to deposition.

**Tree slide area:** Little is known about this area due to the fact that it was not recognized as a separate failure zone until all field work was complete. Careful study of aerial photographs taken the day after the slide occurred revealed the nature of the failure in this area.

Field observations indicated that this area contained failure debris which belonged in neither the sand slide nor the rock slide. This tree and brush covered debris had a medium to coarse sand matrix (see grain size distribution curves, Appendix A) and a fair amount of reinforcing due to plant roots. This material was on a slope of about 34° prior to sliding. This angle is very close to and possibly
greater than the angle of repose of the unreinforced sand soil. The root reinforcement undoubtedly acted to allow the slope to stand as the sand and pebbles coming downslope built up on the plants and allowed the slope to become progressively oversteepened.

It is presumed that this slope was marginally stable under the high water table conditions prevailing at the time of the slope failures above it. Thus it is conjectured that this slope failed when its head area was surcharged by materials coming from both the rock and sand slides. About half of the failed mass of the tree slide came to rest in the bed of Upper Price Lake.

7.5.2 Other Areas

East Slide Area-Middle: The East Slide Area-Middle was found to have no kinematically possible failure modes.

East Slide Area-North: The East Slide Area-North was analyzed for simple toppling failure and it was determined that, with the aid of ice wedging of the rock pillars, minor rockfall and talus slope formation could be expected.

North Slide Area: A series of wedge failure analyses with the height of the wedge varied between 200 and 800 feet was performed to investigate the stability of the North Slide Area. Since many examples of slickensided rock surfaces were found in this area, it is probable that failures occurred on joints having good rock-to-rock contact. This, added to the fact that only a small
percentage of the rock in this area was found to be weathered or otherwise broken down, leads to the conclusions that shear strength properties associated with matched rock surfaces with no fill would best represent field conditions. Thus the values of $C = 15$ psi and $\phi = 40$ degrees were used (direct shear test 3).

The results, as presented in Table 7.4, indicate that failure would occur when the slope height is somewhere near 600 feet. As can be seen on the topographic map (Figure 7.1) the current slope height is between 600 and 800 feet and thus tends to lend some support to the idea that a series of intermediate to large sized wedge failures may have occurred in the recent past. However, it should be noted that the slickensides mentioned earlier were in a direction which had an average azimuth of about 15 degrees, while the direction of the wedge failure(s) would most likely be at an azimuth of about 45 degrees. Variation in joint orientations within the rockmass of the slope could conceivably account for this discrepancy. However, no direct field evidence was found which could provide a sure indication of the postulated wedge failures.

7.6 Conclusions

The various methods of slope stability analyses used, as well as the preponderance of field evidence, seem to indicate that the failure geometry of the most recent slide on Slide Mountain was most probably planar in nature. The
Table 7.4: Results of Wedge Failure Analysis:

<table>
<thead>
<tr>
<th>Height of Wedge (feet)</th>
<th>Factor of Safetya</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>2.7</td>
</tr>
<tr>
<td>400</td>
<td>1.4</td>
</tr>
<tr>
<td>600</td>
<td>0.9</td>
</tr>
<tr>
<td>800</td>
<td>0.7</td>
</tr>
</tbody>
</table>

- **Shear Strength Parameters Used:**
  
  \[ C = 16 \text{ psi} \]
  
  \[ \phi = 40^\circ \]
strength of the joint surfaces were reduced by long term weathering processes while the very high water table elevation caused further shear strength reduction. The reduction in joint shear strength induced by weathering combined with high hydraulic forces in the tension cracks and along the failure plane to initiate the slide.

The slide was found to consist of four component parts. It is thought that the main failure, a block glide, was initiated first. This removed the constraining forces on the lower blocks in the head area, allowing these to fail in a mode that involved both sliding and toppling failure. As these blocks began moving down the slope, they were quickly broken into small pieces, thus forming a rockslide which was funneled down the slope to the left of the block glide, dislodging other blocks as it moved downslope.

It is thought that the ground motions initiated by the block glide and the rock slide initiated liquefaction of the surface layer of the clean, uniform, fine-grained sand deposit found along the northern edge of the rock slide, causing it to flow rapidly downslope.

Portions of the rock slide and sand slide debris were deposited at the top of the sparsely forested tree slide area on the slopes just above the northeast end of the lake. This surcharge at the head of the slope apparently caused this relatively insignificant slope to fail and slide into the lake below. This material, although only about 5% of
the total slide mass, made up about 80% of the material that actually reached Upper Price Lake. This material undoubtedly aided in forcing the water out of the lake and in combination with some material from the rock slide buried about two-thirds of the old lake bed.

This author believes that toe heave associated with the block glide was probably a major factor in the essentially instantaneous emptying of Upper Price Lake. As depicted in Figure 7.3, the planar failure surface probably passed about 20 feet below the surface at the toe of the block glide and thus would be expected to cause some form of buckling at the toe. Pressure ridges observed at the toe of the block glide as well as survey data indicate that toe heave was most likely large (on the order of 15 feet). Also, it was observed that a piece of sandy soil reinforced by plant roots was dislodged from the tree slide on the northeast side of the Lake. This mat of soil skidded across the muddy lake bed and came to rest after about five feet of travel (as evidenced by the marks in the mud indicating the direction and extent of travel). The fact that this mat of sandy soil left striations in the mud and was not dissolved or eroded would seem to indicate that the lake was empty by the time the tree slide reached this point. This observation tends to lend further support to the idea that toe heave caused by the block guide may have emptied the lake before any slide debris could reach its shores.
It is further concluded that the area herein called the North Slide Area may have experienced failures of rock wedges up to 500 feet in height. However, this is based on stability calculations only and no positive field evidence of this was found.

The two areas discussed above would seem to be the only areas on Slide Mountain capable of major failure. However, it must be pointed out that, on a large scale, the entire east side of Slide Mountain has been intensively fractured by tectonic forces and water induced mechanical weathering of the granodiorite bedrock. Thus, when tectonic and erosional forces again combine to cause further oversteepening of these slopes, there is little doubt that major slope failure will occur and Slide Mountain will continue to earn its name.
FIGURE 7.9  "Geologic Hazards"
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PERCENT FINER BY WEIGHT

LEGEND

SAMPLE | DEPTH INTERVAL (FT) | UNIFIED SOIL CLASSIFICATION
--- | --- | ---
1 SAND SLIDE | SURFACE | BP

FIGURE A1
PARTICLE SIZE DISTRIBUTION CURVE
FIGURE A2
PARTICLE SIZE DISTRIBUTION CURVE
FIGURE A3
PARTICLE SIZE DISTRIBUTION CURVE
PARTICLE SIZE DISTRIBUTION CURVE

LEGEND

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>DEPTH INTERVAL (FT)</th>
<th>UNIFIED SOIL CLASSIFICATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1 TREE AREA</td>
<td>SURFACE</td>
<td>SP</td>
</tr>
</tbody>
</table>

FIGURE A4
PARTICLE SIZE DISTRIBUTION CURVE
FIGURE A5
PARTICLE SIZE DISTRIBUTION CURVE
FIGURE A6
PARTICLE SIZE DISTRIBUTION CURVE
FIGURE A7
PARTICLE SIZE DISTRIBUTION CURVE
FIGURE A8
PARTICLE SIZE DISTRIBUTION CURVE
FIGURE A9
DIRECT SHEAR TEST: SAMPLE NUMBER 2
MATCHED ROCK SURFACES WITH AND WITHOUT FINE SAND JOINT FILL
Figure A.10
Direct Shear Test: Sample Number 1

Matched rock surfaces sheared without joint fill and with both fine and coarse joint fill.

Legend:
- △ Test #3 (No Joint Fill)
- □ Test #4 (Coarse Sand Joint Fill)
- ○ Test #5 (Fine Sand Joint Fill)
**FIGURE A11**
**DIRECT SHEAR TEST NUMBER 6**
ROCK SAMPLE SHEARED THROUGH PREVIOUSLY UNSHEARED PLANE OF WEAKNESS
FIGURE A12
DIRECT SHEAR TEST NUMBER 6
ROCK SAMPLE SHEARED THROUGH PREVIOUSLY UNSHEARED PLANE OF WEAKNESS
ARRIVAL TIME (sec)

0.10

DISTANCE FROM SHOT POINT (feet)

0 20 40 60 80 100 120 140 160

V1 = 1070 ft/sec

V2 = 2280 ft/sec

FIGURE A13
SEISMIC REFRACTION
LINe 1, SHOT 5 (UP DIP)
FIGURE A14
SEISMIC REFRACTION  LINE 1, SHOT 7 (DOWN DIP)

ARRIVAL TIME (sec)

DISTANCE FROM SHOT POINT (feet)

V₁ = 1013 ft/sec
V₂ = 1500 ft/sec
FIGURE A15
SEISMIC REFRACTION LINE 2, SHOT 7 (DOWN DIP)
FIGURE A16
SEISMIC REFRACTION  LINE 2, SHOT 10 (DOWN DIP)
ARRIVAL TIME (sec)

DISTANCE FROM SHOT POINT (feet)

V1 = 1150 ft/sec

V2 = 6360 ft/sec

FIGURE A17

SEISMIC REFRACTION LINE 2, SHOT 12 (DOWN DIP)
ARRIVAL TIME (sec)

SEISMIC REFRACTION

FIGURE A18
LINE 2 SHOT 13 (DOWN DIP)

V1 = 1180 ft/sec

V2 = 2950 ft/sec