Delineation of Landslide, Flash Flood, and Debris Flow Hazards in Utah

David S. Bowles

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David S. Bowles

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DELINEATION OF LANDSLIDE, FLASH FLOOD, AND DEBRIS FLOW HAZARDS IN UTAH

Proceedings of a Specialty Conference
Held At
Utah State University, Logan, Utah
June 14-15, 1984

Edited By
David S. Bowles

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Utah State University
Logan, Utah 84322-8200

August 1985
ACKNOWLEDGMENTS

The Specialty Conference on Delineation of Landslide, Flash Flood and Debris Flow Hazards in Utah was organized by Utah State University, Logan, Utah. Financial support for the conference was given by the Utah Water Research Laboratory, the U.S. Geological Survey, the Federal Emergency Management Agency, and the U.S. Bureau of Reclamation. The conference was also co-sponsored by several other organizations including the National Research Council Committee on Natural Disasters, the Utah Section of the American Society of Civil Engineers, the Utah Science and Technology Council, the Utah Geological and Mineral Survey, the National Weather Service, and the U.S. Forest Service. The financial and other contributions of all of these organizations are gratefully acknowledged.

Many individuals played a role in contributing to the success of the conference and ultimately to the production of these proceedings. The conference coordinating committee was responsible for establishing the general structure of the conference and for approving the final conference program. Four program subcommittees were charged with reviewing abstracts, arranging for invited speakers, and organizing technical sessions. The names of individuals who served on these committees are listed on the following page. The efforts of each committee member are gratefully acknowledged. In addition, the vital efforts of the authors of the papers which are included in these proceedings are hereby recognized. Their willingness to document and to share their experiences and knowledge on topics relating to the conference theme made possible a highly successful conference and a very valuable proceedings.

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Conference Coordinator
Conference Coordinator

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Genevieve Atwood, Utah Geological and Mineral Survey
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During 1982, 1983, and 1984, abnormally wet conditions in Utah triggered flash floods, landslides, and debris flows. Por pressure built in hillside soils below melting snows and during prolonged periods of rainfall until the mass suddenly gave way, sometimes as a landslide and other times as a non-Newtonian debris flow that moved rapidly long distances down mountain slopes until finally stiffened by moisture loss or velocity loss because of flatter gradients. Also, runoff from heavy rainfall bursts picked up weathered and other loose material that accumulated on land surfaces over long dry periods.

The sediment laden waters flowed out of mountain canyons onto lowlands where they deposited their loads, filled channels and clogged culverts, and then spread over the land surface to infiltrate, except as intercepted and diverted by streets, storm sewers, and irrigation canals. These were in turn often overtopped to cause flooding in areas with no natural hazard. Snowmelt runoff continued over extended periods, keeping steamflows too high to be contained within the clogged streams, and causing water to flow down streets for weeks disrupting traffic and inundating low-lying property. In closed basins, the waters eventually drain into a terminal lake where rising waters gradually inundate large areas.

This complex of interrelated phenomena created a hazard situation that is greatest at the toe of the mountain slopes and concentrates where mountain canyons drain onto alluvial fans and the water spreads in a pattern that varies substantially from storm to storm. These hillside areas are prime residential sites and command a high price in the market. Development that should not be located in high hazard areas is reasonable a little further downslope where the risk is less. Quantitative methods are needed for mapping flood, debris, and landslide risks in these basin margin areas so that objective decisions can be made on where to locate and how to landscape and design buildings. Monitoring programs and warning systems are needed to track emerging hazards, emergency plans, and get people to respond.

During two spring months of 1983, Utah sustained direct damages from landslides and debris floods in excess of 250 million dollars. Public officials and residents were prepared for water flooding. However, neither the scientific community nor the agencies responsible for dealing with emergency situations were prepared for the widespread landslides and devastating debris flows. At least 92 significant landslides along a 30-mile length of the Wasatch Front Mountains sent torrents of water and debris down on the residential areas below. Along the Wasatch Plateau, more than 1000 landslides occurred. Additional massive landslides in Spanish Fork Canyon, Utah County, created Thistle Lake, and in 12-Mile Canyon, Sanpete County, dammed a river and sent a 30-foot high flash flood surge down the canyon.
These devastating floods, landslides and debris flows were so extensive that 22 of Utah's 28 counties were declared national disaster areas.

Much information on the scientific, engineering, and geologic aspects of these hazards, as well as the social aspects of human response has been gathered by a number of agencies and individuals. A specialty conference was organized in June 1984 as a forum for documentation and exchange of this vital information and its interpretation to improve the state-of-the-art of hazard assessment and effectiveness of responses. This volume is the proceedings of that conference.

The 2-day specialty conference was structured to assemble descriptive information on these hazards in Utah and similar events from elsewhere for use in advancing the state-of-the-art of hazard assessment, damage reduction, and control measure design. The relevant hydrologic, hydraulic, geologic, economic, and social theory, which has been formulated over the years, has been blessed with a wealth of empirical data for validation and calibration. The purpose of the conference was to stimulate use of the data in improving the state-of-the-art of hazard assessment and the effectiveness of remedial activity.

Papers were solicited through a widely distributed call for papers. Contributed abstracts were reviewed by four program subcommittees in the program areas of Geologic Hazards, Flood Hazards, Hazard Mitigation Measures, and Emergency Preparedness/Response. Each subcommittee arranged the accepted abstracts into technical sessions and added invited papers to make well-rounded sessions. In addition, four keynote addresses were invited from authorities in each of the four program areas. The conference was opened with an address by Professor John F. Kennedy, University of Iowa and Chairman, National Research Council, Committee on Natural Disasters.

A total of 38 papers were presented in 14 technical sessions. An additional 14 poster presentations were made during breaks. After the luncheons, current events updates were given by Genevieve Atwood, Director, Utah Geological and Mineral Survey, and Lorayne Tempest, Director, Utah Comprehensive Emergency Management Office. In addition, a special evening session was organized to provide a variety of perspectives on the Thistle Landslide. These perspectives included those of the Utah Geological and Mineral Survey, the State Engineer, consultants to the Denver and Rio Grande Western Railroad, the Utah Department of Transportation, and the consultants responsible for the emergency draining of Thistle Lake.

Following the technical sessions on the last day of the conference, a field trip group traveled from Logan to Salt Lake City, making a brief stop to view effects of a debris flow at Willard. After spending Friday night in Salt Lake City, the first stop on Saturday morning was in the Farmington Area of Davis County to view the location of debris flow damage. The second stop was in downtown Salt Lake City where City Creek flowed down State Street in 1983. The Thistle Landslide and reservoir site was the third stop. After touring the Thistle area, the group traveled to Utah Lake where Interstate 15 has been diked to prevent the rising water from submerging the road. The
final stop was at the flooded Salt Air resort on the southern shore of the Great Salt Lake.

A total of 226 people attended the specialty conference on June 14 and 15, 1984. Participants came from throughout the southwestern U.S. and from other locations around the nation. They represented a wide range of professions and employment affiliations. Many favorable comments were received on the uniformly high quality of technical presentations and the well-rounded coverage of the program areas.

These proceedings contain most of the papers presented at the conference. Where a written text was not prepared for a paper, the abstract is included instead. The proceedings is organized into six sessions beginning with an Overview of the 1983 Utah Diasters. The following four sections correspond to the program areas into which the conference was organized: Geologic Hazards, Hydrologic Hazards, Hazard Mitigation Measures, and Emergency Preparedness/Response. Papers in these sections are grouped into subsections. Section VI contains papers from the special session on the Thistle Landslide.

It is hoped that the printed version of the conference proceedings will make the information which was presented at the conference accessible to an even wider audience.

David S. Bowles
Conference Coordinator
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SECTION I - OVERVIEW OF 1983 UTAH NATURAL DISASTERS
THE UTAH LANDSLIDES, DEBRIS FLOWS, AND FLOODS
OF MAY AND JUNE 1983

by Loren R. Anderson, Jeffrey R. Keaton, Thomas F. Saarinen, and Wade G. Wells, II

The authors were appointed by the Committee on Natural Disasters of the National Research Council to serve as a team to investigate the Utah events of May and June 1983. This paper, which gives a general description of the Utah event comprises Chapter I of the team report and is being printed here with the permission of the National Research Council.

DESCRIPTION OF THE UTAH DISASTER

During approximately three months in the spring of 1983, the State of Utah, with a population of about 2 million people, sustained direct damages from landslides, debris flows, mud floods, and flooding in excess of $400 million. Based on hydrologic records of precipitation and on snow surveys, which indicated that snowpacks were from 150 to 400 percent of normal, Utah’s public officials and citizens were prepared for above-normal stream flows and some water flooding. However, neither the scientific community nor state emergency agencies foresaw or prepared for the widespread landsliding, devastating debris (and mud) flows, and mud floods that were initiated from mountain landslides. These disastrous events were so widespread and extensive that 22 of the 29 counties in the state were declared national disaster areas (Figure 1).

Ninety-two landslides that occurred during the spring of 1983 have been identified along a 15-mile length of the highly populated Wasatch Front (R. T. Pack, Utah State University, Logan, personal communication, 1983). Sixty-eight of these landslides are mapped in Figure 2, which shows a portion of this area. Along the Wasatch Plateau, in the central portion of Utah, nearly a thousand landslides occurred. Many of these were reactivations of large preexisting landslides.

The nationally publicized Thistle landslide, which began on April 9, 1983, was also the reactivation of an old slide. It resulted in a massive
Areas of Utah that sustained damage during the 1981 disaster events.
FIGURE 2 Distribution of the spring 1983 landslides from Bountiful to Farmington, Utah.
220-ft high dam across Spanish Fork Canyon, creating Thistle Lake (see Figure 3). The lake completely inundated the unincorporated community of Thistle and three major transportation arteries: the Denver and Rio Grande Western Railway, U.S. Highway 89, and U.S. Highway 6/50. There was also a significant possibility that the dam would fail and cause flooding of Spanish Fork and other areas downstream. This massive slide was responsible for Utah's first Presidential disaster declaration, which was issued on April 30, 1983. Preliminary estimates indicate that the Thistle landslide may be the single most expensive landslide in U.S. history.

Another massive slide occurred in Twelve Mile Canyon, east of Gunnison, Utah, in the Manti-LaSal National Forest, damming the Twelve Mile River. On May 25 this small dam failed, sending a 30-ft-high flash-flood surge down the canyon. Another landslide in the same canyon threatened Twin Lakes Reservoir and required the lake to be drained.

On May 30, without warning, a major debris flow from the Rudd Creek watershed, which has a drainage area of 0.6 square miles, inundated a three block by three block residential area of Farmington, Utah. Eventually this and subsequent debris flows deposited approximately 90,000 yd$^3$ of material. Photographs of this event are shown in Figures 4, 5, and 6.
FIGURE 4 Overview of the Farmington debris fan.

FIGURE 5 Damage from the Farmington debris flow. Source: Courtesy the Salt Lake Tribune.
FIGURE 6  Two views of damage from the Farmington debris flow.
The Farmington event was followed on Tuesday night, May 31, by a mud flood in Bountiful, Utah. A landslide in Ward Canyon (see Figure 7) initiated a debris flow that entered Stone Creek. This debris flow in turn caused a mud flood along Stone Creek west of about 900 East Street in Bountiful. The mud flood caused extensive damage, as illustrated in Figures 8 and 9.

The high snowpack and above-normal temperatures of late May and early June, which contributed to the initiation of landslides, also caused severe flooding throughout much of the state. The most publicized flooding occurred in Salt Lake City, where flood waters were diverted down two major streets, 1300 South Street and State Street, as shown in Figures 10 and 11. Flooding also disrupted highway traffic in a number of locations and damaged agricultural land and operations.

On June 16 crews began shoring up the DMAD dam near Delta, Utah, after the spillway developed problems from the high runoff. This dam failed about one week later, flooding and causing extensive property damage in the small community of Deseret, Utah.

The high runoff of the past several years has significantly increased the levels and surface area of both Utah Lake and the Great Salt Lake. On December 13, 1983, the surface of the Great Salt Lake was 1.3 million acres, compared with a surface area of 600,000 acres at its low point in 1963. This has caused extensive flooding around the perimeters of the lakes. Agricultural land, state parks, and Interstate 15 near Provo, Utah, were all affected by the high level of Utah Lake. The rise in the level of the Great Salt Lake has caused significant financial losses. For instance, recreational areas (see Figure 12), the mineral industry, and the Southern Pacific Railroad have been adversely affected by the rising waters. The impact will be even greater in the spring of 1984, when the water level is projected to rise even higher. The State of Utah is exploring several alternatives to control the level of the lake. These alternatives include breaching a railroad causeway that divides the lake into two sections and pumping water into a large basin in the desert area west of the lake.

LONG DURATION—A UNIQUE FEATURE

The duration of the Utah disaster was long in comparison with such natural disasters as earthquakes and hurricanes, and the disaster may extend into another spring if current weather patterns continue as expected. The long duration of the disaster provided unique opportunities for documenting the various phases of the event. Although flooding and rising lake levels were already occurring in the fall of 1982, the events in Utah did not become a disaster until about April 9, 1983, when movement was first detected in the Thistle landslide. After President Reagan declared the Thistle landslide a major disaster on April 30, headquarters for the Federal Emergency Management Agency (FEMA) were set up in Provo, Utah, and a disaster team was established. Because of this action, the FEMA disaster team was already assembled and working in Utah when the major flooding, landslides, and debris flows of late May and early June started.
FIGURE 7 Landslide in Ward Canyon that initiated the Bountiful mud flood.

FIGURE 8 Damage in Bountiful, Utah, from the mud flood. Source: Courtesy the Salt Lake Tribune.
FIGURE 9 Damage in Bountiful, Utah, from the mud flood. Source: Courtesy the Salt Lake Tribune.

FIGURE 10 1300 South Street in Salt Lake City, Utah, during the flooding. Source: Courtesy the Salt Lake Tribune.
FIGURE 11 Two views of State Street in Salt Lake City, Utah, during the flooding.
FIGURE 12 Flooded recreational areas along the Great Salt Lake. Source: Courtesy the Salt Lake Tribune.
RESEARCH OPPORTUNITIES

The recent events in Utah have provided an opportunity for scientific research on major landslides, debris flows, urban flood routing, and urban planning for disaster mitigation. They have also offered an opportunity for social science research to evaluate the response of communities to the disaster. Social science research might examine the diversion of streams into streets to minimize flood damage, mitigation measures based on worst-case scenarios, the high rate of volunteer activity, and the generally high quality of the disaster response by local and state governments.

PURPOSE AND SCOPE

The purpose of this reconnaissance study was (1) to provide a conveniently available account of the disaster for historical purposes and (2) to identify cases where an in-depth study would add to the ability to analyze and forecast such disasters.

Because the Utah disaster of 1983 was geographically widespread and geologically complex, the postdisaster reconnaissance team had to limit the scope of its study. The types of events that occurred are described in general terms, but only selected examples of each type of event are discussed in detail. The type of events examined in this report include flooding (both by water and mud), major landslides, and debris and mud flows. The local, state, and federal responses to the disaster and hazard mitigation policies are also analyzed.

Chapters 2 through 8 of the report describe the disaster and the responses to the disaster. After a chapter describing research that has been initiated, Chapter 10 presents conclusions and recommendations.

Copies of the full report can be obtained from the Committee on Natural Disasters of the National Research Council.
Utah had never before experienced the combination of emergencies which occurred during the spring of 1983. The landslide in Spanish Fork Canyon, widespread flooding from high runoffs, debris flows, and the failure of the DMAD Dam combined to create the worst statewide disaster in Utah's history.

By the end of the disaster incident period on July 1, 1983, 22 counties were included in the Presidential Disaster Declaration. The 1983 Utah flood damage costs totalled $478,098,555 which included Public Assistance, Private Federal, Thistle, Private Relief Organizations, and Agricultural Damages.

During this past year, the State of Utah and its local governments faced the threat of a 1984 spring flooding and mudslide disaster of major proportions. Major snowstorms, combined with cool weather during the winter and spring months of 1984 dramatically increased the flooding and mudslide potential, especially in the northern and central portions of the state. Even as late as the first of May of this spring, the National Weather Service's hydro meteorological flood threat index was set at 9.4 on a scale of 10.

However, Utah did not experience a sudden rise in temperature during May or early June as was the case last year. Favorable temperatures and precipitation patterns during May and the first half of June, substantially, reduced the threat of extensive widespread snowmelt runoff flooding. During this period, about 60-80 percent of the lower elevation snow melted during May, in which is described, as an ordinary fashion. In addition, during this same period, a reduction of approximately 60 percent of the higher elevation snowpack occurred at a rate of 2 to 5 percent per day.

Although Utah was spared the widespread and devastating damages to public and private property and facilities that occurred last spring, substantial reported damage, primarily to public facilities, in 16 of the state's 29 counties did occur. The deep snowpacks caused record breaking snow melt runoffs in a number of rivers and streams in Northern and Central Utah. Also, because of the extremely saturated soil conditions along the

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1Luncheon remarks.
2Lorayne Tempest is Director, Utah State Division of Comprehensive Emergency Management.
foothills and in the mountains of these same areas, numerous mudslides and debris flows occurred. It should be noted, that the primary factors that contributed to a large scale reduction in damage to public as well as private property and facilities were 1) favorable weather and precipitation patterns during May; 2) improvements to flood and debris flood control structures; and 3) effective damage mitigation projects.

The 1984 public assistance flood damages are currently being reported at $31,807,945 by the state and 16 counties. The 1983 public assistance damage was reported at $45 million. Governor Matheson is requesting another Presidential Disaster Declaration for the state and counties impacted by this year's flooding.

According to the National Weather Service we are in the third year of a four year wet cycle. That means, we could have flooding problems in 1985. My office is again ready to meet the challenge and provide support and assistance to any area of the state that will experience flooding problems.
SECTION II - GEOLOGICAL HAZARDS
Keynote Address

IMPLICATIONS OF THE CURRENT WET CYCLE TO LANDSLIDES IN UTAH

by Robert W. Fleming1 and Robert L. Schuster1

ABSTRACT

The damaging landslides in Utah in 1983 and 1984 can be attributed to an abrupt increase in precipitation that is unprecedented in Utah's more than 100-year history. Among the many types of landslides that occurred, reactivations of large relatively slow-moving landslides and soil slip-debris flows were most common. The evaluation of hazards from these types of landslides is inhibited by a lack of basic research information on precipitation-subsurface water response. The disaster has provided an opportunity to greatly improve our understanding of landslide distribution, mechanics, and processes.

INTRODUCTION

Slope-failure problems in Utah in 1983 and 1984 have been the most visible result of a general increase in landslide activity in the western United States during the past 10 years. This increase in slope failures can be ascribed to a reversal of a long-term trend of a drier-than-normal climate. When the current wet cycle again reverses, we can expect a return to lessened hillslope problems in Utah.

The reason for the abnormal amount of landsliding in Utah during 1983 and 1984 can be inferred from the long-term precipitation record for Utah. Figure 1 is a graph showing cumulative departure from mean annual rainfall at the Salt Lake City Airport for the period 1875-1984. Average annual precipitation for the period 1885-1984 is 14.92 inches. Note that between about 1890 and 1966, there was an accumulating deficit of precipitation punctuated by a few short intervals of above-average precipitation. By 1966, this cumulative deficit from mean annual precipitation was about 50 inches. Between 1966 and 1982, about half of the deficit was made up by small annual surpluses of precipitation. A major storm in Utah in September 1982 signaled the beginning of 2 years of near-record precipitation that has completely removed the long-term moisture deficit. The current wet cycle is unprecedented for the period

Jahns (1969) recognized the significance of long-term precipitation to the abundance of landsliding in the Los Angeles, California, area. Combining data from growth rings of trees with measured precipitation, he demonstrated a long-term cyclic pattern to precipitation. Precipitation patterns were compared to historical records of landslides along the California coastal area. He concluded that there is an unmistakable correlation between the incidence of landslides and climatic wet cycles. Further, in an observation that may be significant to landslide problems in Utah, he noted a tendency for landslide activity to be most vigorous and widespread during the early parts of wet cycles, and that it begins to taper off during their later parts.

Jahns (1969) speculated on the consequences of an inevitable return of a wet cycle. His conjecture has proven accurate for much of the western United States and particularly for Utah:

"An upward swing in the cyclic moisture curve inescapably promotes higher mobility of the terrain. In some contrast to events of the past two decades, increased flood erosion and deposition, further visitations by flowing masses of debris, the appearance of many new landslides, and some reactivation of existing ones now can be expected during the next decade or so...."
Existing works for flood control will be severely challenged, and immediate needs for additional installations will become rather apparent. As the levels of groundwater gradually rise, a host of more subtle effects may well make an appearance. The lower parts of some cuts will begin to weep, long-dry springs will become active again, and clear streams will grace many canyons that for a long time have been occupied by no more than occasional muddy floodwaters.

As a consequence of the current wet cycle, hillslopes in Utah are probably experiencing groundwater levels that are higher than they have been at any other time in this century. Continued surplus precipitation relative to the long-term mean will inevitably cause more landslides, and a return to conditions of the spring of 1983 would again produce devastating debris flows. Hopefully, however, the observation of Jahns (1969) that landslide activity is most pronounced during the early part of a wet cycle will be true for Utah, and the worst of the damaging landslides will be past.

LANDSLIDES IN UTAH IN 1983 AND 1984

Landslides in Utah in 1983 and 1984 produced extensive damage and caused a great deal of personal hardship for many people. The landslide in Spanish Fork Canyon near Thistle, Utah, was the first major slope failure to occur. Movement was initially noticed on April 13, 1983, (Sumsion, 1983) and in a matter of a few days, the landslide dammed the Spanish Fork River, creating a lake that flooded the town of Thistle, U.S. Highway 6 and 50, and the main line of the Denver and Rio Grande Western Railroad (D&RGW). The D&RGW spent about $40 million to reestablish its line outside the landslide area, and U.S. Highway 6 and 50 was relocated through a cut in Billies Mountain to the east of the landslide. The University of Utah Bureau of Economic and Business Research and others (1984) evaluated total direct costs of the Thistle landslide at about $200 million. In addition, numerous indirect costs were reported that impacted the uranium, petroleum, and coal industries, several types of businesses, and tourism.

In May 1983, landslide and flood problems became much more widespread. Debris flows mobilized on slopes along the Wasatch Front and flowed into populated areas. Large landslides occurred at numerous localities around the State. The landslides threatened domestic water supplies, transportation routes, communications, various energy transmission lines, and other structures.

Schuster and Fleming (in press) have summarized reported estimates of the direct costs of landsliding in Utah for the past 2 years. Estimated damages in 1983 exceeded $250 million and in 1984 were nearly $50 million (Bruce N. Kaliser, Utah Geological and Mineral Survey, personal communication, 1984). This total does not include many indirect costs, which could perhaps equal or exceed the direct costs.

Although nearly every slope movement type known occurred in Utah during 1983 and 1984, nearly all of the damages were associated with movements of two
types: (a) reactivations of large old landslides and (b) soil slip/debris flows. There were also several first-time failures of slopes that produced landslides in the topographic benches along the Wasatch Front, but although damaging, they were numerically subordinate to the reactivated large landslides and soil slip/debris flows.

Reactivations of Large Landslides

Those who have studied the areal distribution of landslides and slopes susceptible to landsliding in Utah have recognized that large parts of the State are prone to landslide activity of one type or another. In spite of this, there have been few inventories of locations of large landslides in Utah. In a statewide study of the more conspicuous landslides, Shroder (1971) documented data for about 600 major landslides in Utah, including the then inactive landslide at Thistle. More recently, we and R. B. Johnson have been studying the results of several investigations of landslides in a single small area -- Manti Canyon, on the west side of the Wasatch Plateau. These studies show that about one third of the total drainage area (11 of 34 mi$^2$) of the canyon consists of landslide deposits.

During 1983 and 1984, many large old landslides moved again (e.g., fig. 2). For most of these landslides, the only apparent cause of reactivation was an increase in subsurface water pressures resulting from the excessive precipitation. Unfortunately, critical data are lacking to predict (a) general groundwater response and, more importantly, (b) pore-pressure changes within the inactive landslides. In the past few years, there has been increasing interest in measurement of pore pressures associated with different forms and amounts of precipitation and a few data are becoming available (e.g., see Fleming and others, 1981; Wieczorek and Sarmiento, 1982; and Sangrey and others, 1984). The data are insufficient for practical application, but point to the kinds of measurements necessary for understanding the reactivations of large landslides.

It is very difficult to predict beforehand those landslides that are susceptible to reactivation by a change in subsurface water pressures. By careful mapping, the locations of previously failed slopes can be identified (e.g., Shroder, 1971). It is also possible to infer the type and rate (within limits) of past movements from the landslide deposits, and to measure the appropriate physical properties of the failed materials. The most critical unknown is the long-term behavior of water in the slope. Stabilities of slopes that are saturated to the ground surface differ by about a factor of two from those of equivalent slopes in which the water table is located below the potential failure surface. In addition, certain geologic situations can result in groundwater conditions that produce even larger differences (Terzaghi, 1950). Piecemeal reactivations of portions of large landslides are also common; these can further complicate stability analysis.

Mobilization of Debris Flows

Debris flows comprise the other common form of slope movement that occurred in Utah during 1983 and 1984 (Fig. 3). The debris flows mobilized
Figure 2. View of the Seeley Creek landslide, which is located on the east side of the Wasatch Plateau near Joe’s Valley Reservoir. This complex landslide/earthflow is typical of large landslides in Utah that experienced renewed or accelerated movement in response to rising subsurface water pressures. The Seeley Creek landslide has been intermittently active in recent years, but in 1983 rapid movement occurred throughout its length of more than a mile. At the head of the landslide, movement occurred predominantly by sliding. In the process of sliding, the materials were remolded and flowed down a long narrow channel to a fan-shaped apron in the valley of Seeley Creek. The upper part of the landslide complex is as much as 1000 feet wide; the long section of flow movement is 135 to 150 feet wide; and the displaced fan-shaped apron is nearly 1500 feet wide. The toe of the landslide shows disturbed mature spruce trees that had not previously been severely displaced during their lifetimes. Upstream from the toe is a broad flat area consisting of pond deposits from prehistoric blockages of Seeley Creek by previous episodes of landslide movement. Thus, the rapid movement of the Seeley Creek landslide that occurred in 1983 is unprecedented during historic time.
Figure 3. The debris-flow scars shown here are in Fairview Canyon on the west side of the Wasatch Plateau. The scars produce landscapes that are typical in those parts of Utah that experienced severe debris-flow problems in 1983 and 1984. Most debris flows occurred in 1983 in response to rapid melting of a near-record snowpack, but others were mobilized during heavy summer thunderstorms and during melting of the 1984 snowpack. Two conditions were evident as essential for debris-flow mobilization: abundant water and initiation as a slip-type landslide. The favored habitats for the sources of debris flows were small swales or valleys that concentrated subsurface water flow.

The debris flow on the left side of the view, which is known locally as the White Pine debris flow, occurred on May 13, 1984, during melting of the heavy 1984 snowpack. Flows mobilized as episodes from retrogressing slip failures at the head of the gully, producing slugs of debris that traveled downslope at as much as 1 foot per second. The debris accumulated on the surface of State Highway 31, where it continued to move as much as a few feet a minute. The debris temporarily blocked the highway and Cottonwood Creek.
from soil slips (relatively small landslides), and flowed as much as 4 miles
down channels, sometimes into populated areas. In 1983, and to a lesser
extent in 1984, there was a distinct association between the receding snow
line and the occurrence of debris flows. During the summer of 1983 and
throughout 1984, many debris flows were triggered by thunderstorms that
introduced surplus water into hillslopes already containing unseasonally large
amounts of water.

Empirical studies of debris-flow mobilization have shown that the water
responsible for triggering the slope movements can be divided into two
components: (a) antecedent moisture and (b) precipitation (rainfall or
snowmelt) intensity. Campbell (1975) clearly demonstrated the influence of
both components in a study of debris-flow mobilization in the Santa Monica
Mountains, California. He found that 10 inches of seasonal rainfall
established enough antecedent water in the hillslopes that debris flows could
be triggered by a storm that delivered precipitation at 0.25 inches/hour.
Other investigators around the world have identified similar precipitation
conditions for triggering debris flows. For example, Lumb (1975) by
statistical studies determined that in Hong Kong disastrous debris flows
can be triggered by storms in which precipitation exceeds 3.9 inches in one day,
if total rainfall over the preceding 15 days exceeds 13.8 inches.

The significance of the requirement for rapid introduction of large
amounts of water can be appreciated in terms of the amount of water necessary
to saturate a column of soil equal in thickness to a typical slope failure.
For a soil slip to occur and a debris flow to mobilize, it is generally
necessary that the slope be saturated. For a surficial deposit with 25
percent porosity, the amount of water necessary for saturation is one fourth
the height of the soil column. Thus, for a typical 6-foot-thick slope
failure, 1.5 feet of water must be contained in the soil. Very few
meteorological events can introduce this amount of water over a short period of
time. Some water is supplied by the antecedent precipitation and more water
is delivered through interflow of subsurface water. There is also a clear
association between the site of initiation of a debris flow and topographic
form. The great majority of debris flows occur in concave topography ranging
from subtle swales to the floors of small valleys. The concave topography
concentrates subsurface water flow.

During the summer of 1983 and throughout 1984, several debris flows in
Utah were triggered by thunderstorms. These thunderstorms were not unusual in
terms of intensity or duration, but, because large amounts of water already
were contained in the hillslopes, the water necessary to trigger movements was
reduced.

The factors that control initiation of debris flows are so complexly
interrelated that generalizations about future debris flows are not
particularly reliable. Predictions about hazards based on one observation
often are contradicted or modified by other observations. For example, future
debris flows that occur in drainages that experienced debris flows in 1983 or
1984 might be expected to travel farther downslope and into populated areas
because the first episodes of flow cleared the channels of debris, thus
creating more-efficient channels for future flows. On the other hand, there was a tendency for first episodes of debris flows to be larger than subsequent flows and, in general, larger flow events travel farther than small events. Thus the two observations result in conflicting predictions of how far a debris flow will travel.

Also, the Utah debris flows invariably have mobilized from soil slips. In some cases, the entire volume of the soil slip was transformed into a flow and the site of initiation became an empty scar in a swale on a hillslope. However, there were numerous examples of debris flows that were mobilized from only parts of soil slips, leaving partly detached masses on the hillslopes above communities along the Wasatch Front (Wieczorek and others, 1983). Are these partly detached masses likely to become the sites of future debris flows? Just the fact that the soil slip already has occurred satisfies one part of the requirement for mobilization to a debris flow. However, the other part, saturation of the soil-slip debris, may be difficult to achieve after the slope has failed. Movement of the soil slip may have improved the internal drainage of the hillslope to the point that saturation becomes more difficult than for adjacent, unfailed slopes.

These types of issues have major implications for officials who must make decisions to reduce risk to people and structures during a landslide crisis. Certainly, a return to conditions of the spring of 1983 would produce more debris flows. Unfortunately, specific answers to the most important questions, such as "when, where, and how big?" are impossible without a better understanding of hillslope hydrology and debris-flow processes.

OPPORTUNITIES FOR RESEARCH

The extent of landsliding in Utah in 1983 and 1984 has produced unprecedented damages. Conversely, the landsliding has resulted in opportunities in Utah for significant research that have not existed during this century. Important studies are underway by various State and Federal agencies and academic institutions. However, there currently are more opportunities for important research than there are people available to conduct the research, and many important tasks are being ignored. In general, the needed research can be divided into three categories: (a) basic research of landslide and debris-flow processes to better understand the mechanics of slope failures, (b) documentation of types and locations of failures that have occurred, and inventories of previously failed slopes that have not yet reactivated, and (c) research to learn strengths and weaknesses of the 1983 and 1984 disaster responses.

Research of the first type will require instrumentation and monitoring of several slopes in different materials in different parts of Utah. In support of this instrumentation and monitoring, it will be necessary to measure the physical properties of the hillslope materials and to conduct detailed mapping of certain attributes such as landslide cracks, joints, and fissures in natural materials, and geometric configuration of susceptible materials for selected landslides and previously unfailed slopes. The second type of research is largely a mapping effort to illustrate the locations of potential
problems and to provide a framework to apply information about landslide processes. Data from these types of studies are important for extrapolating findings from areas that have been monitored to other areas. In addition, the information is necessary background for disaster planning and response. The third type of research is extremely broad in scope and related to the general subject of disaster warning and response. One of the very positive aspects of the landslide and flood disaster in Utah was the well coordinated and effective response to the problems. The lessons learned from the events of 1983 and 1984 will be of value not only for the next Utah disaster but also for other parts of the United States.

The Governor’s Conference on Geologic Hazards (Utah Geological and Mineral Survey, 1983) held in Salt Lake City on August 11-12, 1983, was comprised of 38 working groups that produced many recommendations for research of the types indicated above. Those recommendations provide an excellent shopping list for selecting or identifying studies that could contribute to an understanding of the disaster.

CONCLUSIONS

The current wet cycle in Utah has produced a costly landslide disaster. Associated with the disaster are opportunities to conduct significant research that have not existed within Utah’s recorded history. Most of the opportunities for geologists and engineers are related to the behavior of subsurface water. Groundwater response to snowmelt and rainfall in previously failed slopes and first-time failures of slopes that mobilize into debris flows are among the most critical problems that are amenable to field study.

REFERENCES


Description and Evaluation of Slope Hazards

SYNOPSIS OF GEOLOGIC PHENOMENA:
WET CYCLE OF 1981 TO PRESENT

by Bruce N. Kaliser

ABSTRACT

Study of the geologic events related to the wet cycle began in spring 1982 with a project being identified in the fall of that year. In May of 1982 a rapid earth flow occurred in granular materials on the benchland of Salt Lake County. It was the clear result of above normal runoff in a small intermittent drainage which provided high infiltration to the local perched groundwater regime. In the winter of 1982-83 aerial reconnaissance was undertaken which identified problems first with low benchland terrain, then at higher elevations. Video tape footage was taken of several of the problem areas in Utah, Salt Lake, Davis, and Weber Counties prior to the crisis period of late spring 1983. The great Thistle landslide started in early April 1983, and was followed by numerous smaller, shallow debris slides in May and June in northern and central Utah. Landslides of all types emerged in diverse geologic terrain.

Effects to Utah's population was significant from high groundwater. Flooding of basements, individual wastewater systems, municipal sewage systems and sanitary landfills resulted. Contamination of shallow aquifers was facilitated and crop yields were significantly reduced from root damage. Sedimentation was aggravated by the volumes of loose earth transported in debris flows and debris floods. Reservoir lifetimes have been reduced and carrying capacity of irrigation systems diminished. Erosion has weakened many types of facilities, particularly drainage structures, at considerable cost.

Moisture sensitive soils are grossly effected; some of these effects are either delayed or are not yet readily apparent.

Local ground collapse has occurred from piping, man-created underground voids and unengineered fill settlement.

An attempt is being made to put a cost estimate on damages from each of these phenomena.

Bruce Kaliser is with Utah Geological and Mineral Survey.
PROTOTYPE INSTRUMENTATION AND MONITORING PROGRAMS
FOR
MEASURING SURFACE DEFORMATION ASSOCIATED WITH LANDSLIDE PROCESSES
by: Michael K. McCarter and Bruce N. Kaliser

ABSTRACT
Extensometers and inclinometers were deployed in three areas of the Wasatch Front to measure surface soil deformation during the 1983 winter-spring transition. Resulting displacement vs. time plots disclose similar response patterns for each site which may be useful in identifying high risk periods for gross instability or debris flow development.

INTRODUCTION
During the spring of 1983, melting of an unusually heavy snow pack resulted in numerous debris flows and other forms of rapid slope movement in the mountainous terrain of northern and central Utah. Because of the high potential for reoccurring movement, geotechnical monitoring of recognized landslide areas appeared to be a reasonable step to help mitigate adverse impacts on down-slope communities.

Carl Terazagi (1950), well-known pioneer in soil mechanics and slope stability, expressed the opinion that few, if any, landslides occur without warning. This warning may be in the form of adverse hydrologic, lithologic, topographic, or meteorologic conditions, or warnings of a more immediate nature which are manifested in the displacement history of the affected slope. Displacement histories have been successfully used in the mining industry to help ensure safe working conditions (Kennedy et al., 1969; Ko and McCarter, 1976; McCarter, 1976; Larocque, 1977; Campbell and Shaw, 1979). Application of this technology may provide practical benefits for communities located in the path of potential debris flows. For this reason, the Utah Geological and Mineral Survey and the University of Utah engaged in a joint effort to deploy several monitoring devices on the Weber Bench in Weber County, in Pudd Canyon in Davis County, and in Reynolds Gulch in Salt Lake County. The goal of this effort was to provide quantitative data on climatic conditions and relatively long-term slope movement for several sites and to evaluate the possible existence of precursory events indicating a high potential for landslide activity.

Michael K. McCarter is a Professor of Mining Engineering, University of Utah and Bruce N. Kaliser is the Chief of the Hazards Section, Utah Geological and Mineral Survey.
Numerous devices have been developed to measure processes associated with slope failure. Some are commercially available while others must be constructed to fit individual site requirements. One of the major challenges in this project was to develop devices capable of measuring soil deformation at or near the surface during the winter-spring transition. Specifically, equipment must be robust enough to survive heavy snow loads while remaining sensitive to soil movement. The following paragraphs will provide a brief description of the transducers used to detect slope movement, equipment deployed to measure climatic conditions, and systems used to collect data.

**EXTENSOMETERS**

Prototype extensometers were constructed using two different designs. The most successful is referred to as the Rudd extensometer (see Figure 1). It consists of a 3.7 m (12 ft) length of 2.54 x 5.08 cm (1 x 2 in) rectangular steel tubing and a 1.3 m (51 in) length of 1.9 x 1.9 cm (0.75 x 0.75 in) steel bar. As shown in the figure, one end of the tube is anchored to the uphill (relatively stable) side of a fracture using a 1.07 m (42 in) length of standard 2-inch steel pipe. The tubing is attached using a bracket which allows the extensometer to pivot in a vertical plane. The bar is positioned within the tubing at the downhill end and is free to move in or out of the tubing on roller guides. The exposed end of the bar is attached to a bracket and pipe similar to the uphill anchor.

**FIGURE 1** Rudd Extensometer
The lower surface of the bar contains a milled groove which provides a longitudinal recess for a 1 m (39 in) precision gear rack. This rack serves to rotate a 32 D.P. spur gear as the rod slides in or out. This gear is held in contact with the rack by a spring and rotates a 10-turn, 2K ohm, precision potentiometer. This device provides a change in resistance proportional to the change in position of the bar relative to the end of the tube. The diameter selected for the spur gear provides an output of approximately 2 ohms per millimeter. The data collection system, however, incorporated the potentiometer as a voltage divider, and the ratio of the wiper arm potential to the applied voltage was, therefore, nearly equal to the extension in meters.

The spur gear and potentiometer are housed in a weatherproof enclosure mounted on the end of the tube. This enclosure also contains an inclinometer (Humphrey CPI7-0601-1 pendulum potentiometer) with the same electrical characteristics as the 10-turn potentiometer previously described. The inclinometer has a range of ±45°, and the mounting configuration permits measurement of extensometer attitude from horizontal to vertical in a downward direction about the pivot. Capability of measuring extension and attitude permits calculation of horizontal and vertical components of movement for the downhill anchor relative to the uphill anchor. The device is sensitive to a change in position of as little as 2 mm (0.08 in) over the 1 m (39 in) range.

The Reynolds extensometer is very similar in design to the Rudd extensometer but with several notable exceptions. The overall length, 4.6 m (15 ft), is longer, and the housing used to protect the potentiometer and facilitate electrical connection is positioned at 1.5 m (5 ft) from the downhill end. This housing is attached by welding the enclosure directly to the tubing after suitable openings are machined in the tubing to accept the potentiometer and drive linkage.

The Rudd extensometer was found to be much easier to service in the field and less susceptible to damage caused by rough handling in transit to the installation site. The Reynolds extensometer is more compact, but the welded construction and machined openings in the tube, along with the additional length, severely limit the ability of this device to withstand the snow loads encountered during this study. All of the extensometers constructed with the Reynolds design experienced excessive bending at the enclosure, and it was necessary to reinforce the welded area with an additional length of tubing which was attached in the field after readings disclosed the bending problem.

Five extensometers were constructed using the Reynolds design, and four were constructed using the Rudd Design. Eight of the nine survived to produce data during the spring snow melt. Midwinter maintenance was necessary and only three devices survived without some bending.

INCLINOMETERS

Individual inclinometers were used only at one site (Rudd Canyon) to detect gross movement. Some were buried at shallow depth to detect pro-

*Identification of brand names in this paper does not imply endorsement.
gression of slide boundaries in an uphill direction while others were mounted on the surface below the potential source of slide debris. Surface mounted inclinometers were intentionally placed where they would be destroyed or disrupted in the event of a debris flow immediately above the installation.

In all cases, inclinometers consisted of the Humphrey pendulum protected by a suitable enclosure. Readings from each inclinometer were expressed as the ratio of the wiper contact potential to the applied voltage. Positive changes in the ratio indicated a rotation of the inclinometer housing in a forward or downhill direction about the pendulum pivot. Negative changes indicated the reverse motion. The range was ±45° with respect to the direction of gravity with a detection threshold of approximately 10 min of arc.

Buried inclinometers provided consistent data throughout the test period. The attitude of surface mounted inclinometers, however, was affected (up to 7.8°) by snow creep, saturation of surface soil, and accumulation of debris. The surface mounted inclinometers also exhibited an undesirable sensitivity to temperature changes. This sensitivity did not affect data accuracy for this project but may complicate design considerations should the same device be used with other data collection systems.

WEATHER STATIONS

Only two monitoring sites (Rudd Canyon and Weber Bench) were equipped with instrumentation to monitor precipitation, and only one (Rudd Canyon) was equipped to continuously measure temperature. All other climatic data were obtained from nearby established weather stations.

The Weber Bench installation included a U.S. Weather Bureau rain gage which was manually read following precipitation events. Thermometer readings were obtained at the time of instrument readings, but maximum and minimum daily temperatures, along with precipitation data, were obtained from the Hill Field Weather Station, approximately 7.2 Km (4.5 mi) away.

The Rudd Canyon installation was equipped with a Qualimetrics P501-AE rain gage and a YSI 44004 thermistor which provided continuous data from May 5 through June 30. On April 14, a snow survey was conducted to establish the water content of the snow pack in the vicinity of the weather station.

The sources of meteorological information for Reynolds Gulch were the Argenta Station, located approximately 2 Km (1.2 mi) down canyon and 244 m (800 ft) lower in elevation, and the Brighton Station, located approximately 7.3 Km (4.5 mi) up canyon and 180 m (590 ft) higher in elevation. The Argenta Station provided precipitation data while the Brighton Station was used for temperature measurements. In addition, snow surveys were conducted especially for this study by the Salt Lake Water Department to provide direct measurements of the water content in the snow pack at the instrument site.

DATA COLLECTION

Three different methods were employed to collect data. Direct interrogation was used at Weber Bench since the site was located near a residence
and the homeowner was willing to read the instruments on a daily or more frequent basis. Remote interrogation was employed at Reynolds Gulch where a 670 m (2200 ft) cable was used to activate a stepping relay and sequentially couple each of the transducers to measuring equipment. Finally, radio telemetry was used at Rudd Canyon to relay information from the field site to the Davis County Sheriff's Dispatch Center. The following paragraphs will provide a brief outline of each system.

DIRECT INTERROGATION

Data acquisition was accomplished by connecting a 1.5 V alkaline battery across the resistive element of each displacement transducer. The potential of the wiper contact and applied voltage were then measured relative to the negative side of the battery using a 3½ digit portable multimeter. The two readings were used to calculate the ratio to 3 significant figures. The alkaline battery, multimeter, and cable connectors were incorporated in a single package for ease of operation, and the resulting data were independent of temperature and repeatable within ±2 mm (0.08 in) and ±10 min of arc for the extensometers and inclinometers, respectively. Direct interrogation was used at both Rudd Canyon and Reynolds Gulch until communication links were established.

REMOTE INTERROGATION

Following an earlier attempt which failed due to severe weather conditions, a 4-conductor cable was successfully installed on March 31, 1984, between the field site in Reynolds Gulch and a check point near the main road in Big Cottonwood Canyon. This cable enabled activation of a 12 VDC, stepping relay and sequentially established electrical contact with each extensometer and two internal reference resistors used to check system calibration. The equivalent circuit diagram is presented on Figure 2. As shown, all potentiometers were connected in parallel and the wiper contact of each was connected to one of 12 contacts on the relay. The first contact provided the potential applied to the potentiometer array by the 1.5 V battery located at the check point. This voltage \(V_a\) was less than the battery voltage because of the cable resistance which was not constant but a function of ambient temperature. All unused contacts were connected in common to the ground side of the parallel combination of potentiometers and reference resistors. The indicated potential \(V_g\) of this point in the system was above battery ground by an amount related to the resistance of the cable. Assuming an infinite impedance for the multimeter, the ratio of wiper contact potential to applied voltage, corrected for temperature and cable resistance, is given by:

\[
p_1 = \frac{V_1 - V_g}{V_a - V_g}
\]

where \(V_1\) is the indicated wiper potential for a given potentiometer.

Data acquisition was not continuous, and each point on the displacement vs. time record represents a single interrogation result. The frequency of
visits to the check point was a function of rate of movement inferred by connecting a straight line between consecutive readings. The field data show that readings were made from the check point with a precision of about ±2 mm (0.08 in).

**RADIO TELEMETRY**

The telemetry system established at the Rudd Canyon site consisted of two basic components: a multiplexer and radio transmitter. The multiplexer was designed and constructed especially for this application by the Earth Science Laboratory, University of Utah Research Institute. The multiplexer consisted of a sequencer, A to D converter, modulator, and ancillary circuitry for signal amplification and data serialization. The sequencer served to excite each measuring device and connect the return signal to an analog to digital converter. The digital data were then serialized and presented to a modulator where the value of each data bit was converted to a high or low frequency. The data string was then broadcast continuously by a field transmitter.

The horizontal distance from the Rudd Canyon site to the Davis County Sheriff's Office is approximately 1.6 km (1 mi). Given this distance and the rugged terrain, economy and reliability favored selection of radio communication over other alternatives. The high relief of the mountain front, however, prevented a direct line of sight to Farmington, and to overcome this difficulty, a repeater was established at the Lagoon Stadium 3 km (1.9 mi) westward from the field transmitter.
The base station employed a 145 mw Motorola transmitter and a directional antenna. A Bearcat 250 Scanning Radio was used to receive the signal at the repeater, and the signal was rebroadcast by a transmitter identical to the base station but operating at 11.55 MHz lower in frequency. This signal was, in turn, received by a second scanner at the Sheriff's Office.

The multiplexer and base station transmitter were powered by five, 1100 ampere-hour, Carbonaire batteries. The current drain was approximately 80 mA, providing an estimated life of 13,750 hours or 573 days. The repeater operated on 12 VDC provided by a 110 VAC power supply with a battery backup.

Figure 3 illustrates the basic components of the telemetry system including the "real-time" monitoring capability provided by the C64 computer incorporated in the system. The signal received from the lagoon repeater was demodulated and processed by the computer. Processing included comparing the status of each instrument to an upper and lower limit. If the reading was above the upper limit or below the lower limit, an audible alarm was activated and the offending device was identified by a reverse video image. The dispatchers on duty responded to the alarm by immediately notifying emergency personnel and contacting designated individuals at the Utah Geological and Mineral Survey and/or the University of Utah.

The video display consisted of a listing of 12 channels representing the 11 transducers and a system checking device. This display included the channel designation, lower limit, upper limit, and current value. In addition, the temperature, precipitation, battery voltage, battery current, reference voltage, Julian date and time were displayed below the channel tabulation.

FIGURE 3 Rudd Canyon Monitoring System
New values for all parameters were updated 3 times per minute, and a sample of all values was printed every 10 minutes. Individual readings were accumulated in the memory, and average values printed every hour. The averages were also stored and automatically transferred over telephone lines to a PDP 11/34A computer located at the University of Utah. The hard copy provided the only record for 10-minute intervals and established a backup for hourly averages in the event of power outages and consequent loss of memory. Power failures were infrequent and did not substantially interfere with system operation.

The PDP 11/34A stored the hourly averages in a two-dimensional array defined by device number and time. Auxiliary software permitted review and plotting (video image or hard copy) of any desired device for any desired window in time. This capability permitted ongoing analysis of trends and resetting of limit values to prevent triggering of alarms due to predictable, cumulative trends. Initial thresholds were set at approximately ±5° for the inclinometers and approximately ±1.0 cm (0.4 in) for extensometers. Diurnal variation in readings appeared to be well within 0.5% of the indicated value.

FIELD SITES

The following paragraphs will describe each of three sites selected for instrumentation and resulting data acquired during the 1984 winter-spring transition. The three sites include the Weber Bench in South Ogden, Reynolds Gulch located in Big Cottonwood Canyon east of Salt Lake City, and Rudd Canyon east of Farmington.

WEBER BENCH - SITE DESCRIPTION

The unstable area is located on a north-facing terrace above the Weber River. The material is of late Quaternary age and consists of lacustrine silts and fine sands with little clay. Aerial photographs and field inspection disclose a well-defined headwall scarp and indistinct toe. The current slide is approximately 490 m (1600 ft) wide by 180 m (600 ft) long and is situated in elevation between 1340 m (4400 ft) and 1390 m (4560 ft).

Two extensometers were positioned 67 m (220 ft) apart near the northwestern extremity of the headwall. One instrument was located across the headwall scarp where the fracture passed through the crawl space under a house. The other was located on the same tension fracture, outside and to the east of the home. The installation provided an excellent opportunity to compare the readings from an extensometer located in a protected, nearly constant temperature environment with one exposed to the elements.

WEBER BENCH - FIELD DATA

Figure 4 represents a summary of individual readings taken between February 17 and June 9. As can be seen, temperatures basically remained below freezing until about March 6. Warmer temperatures began melting the snow cover, and by March 24, nearly all traces of snow had disappeared. The displacement history for the outside extensometer shows a slight downward
trend (contraction) for this same period which is probably due to reduction of the snow load on the extensometer. This trend is interrupted by a distinct inflection in the curve, indicating reactivation of the dormant slide, which occurred on March 24. By April 1, homeowners situated in the toe area began to notice widening of fractures in pavement and foundations. The continuing upward trend of the curve indicates a more or less uniform rate of movement until about April 13, at which time there is a noticeable decline in the rate of movement.

FIGURE 4 Meteorologic and Extensometer Data for the Weber Bench Site
continuing through May 1. This apparent reduction is due to development of a sympathetic headwall fracture and subsequent displacement of the uphill anchor. The extensometer was repositioned on May 14, and the dotted line of the graph connects coincident points on the displacement history curve.

The inside extensometer was not installed until April 13. Its curve, however, is notably more regular than the one developed by the outside extensometer. The smooth appearance is due to the fact that the transducers used for the inside device are approximately ten-times more sensitive than those used for the outside device. The irregularities in the outside curve, for the most part, reflect the ±2 mm (0.08 in) repeatability for the device.

The precipitation history is cumulative from the beginning of the water year. It is presented here to allow the reader to infer approximately how much water was contained in the snowpack. The onset of fracture separation, recorded on March 24, is apparently the result of rapid melting of the snow cover which probably contained less than 33 cm (13 in) equivalent water. In addition, the record shows that a substantial amount of rain which began to fall at the end of May [6.9 cm (2.73 in) from May 30 to June 8] is not reflected in the displacement history. The precipitation estimates are based on data accumulated at the Hill Field Weather Station and may not precisely represent this specific site due to local relief and canyon effects.

Slide activity is characterized by a period of little or no movement (dormant), followed by a period of increasing displacement, up to about 4 mm (0.16 in) per day, and then a return to the dormant state. No debris flow or rapid slope failure occurred at this site, nor was any expected.

REYNOLDS GULCH - SITE DESCRIPTION

Reynolds Gulch is a north-trending tributary of Big Cottonwood Creek and is situated approximately 14.5 km (9 mi) from the mouth of the canyon. It was the site of an earlier debris flow which occurred in June 1983. The source area for this event is located on a west-facing mountain slope (slopes range from 26° - 30°) just below the 2438 m (8000 ft) elevation. The disturbed area consists of two superimposed slide zones with detached masses near the toe and above the 1983 headwall (Figure 5). Material covering the slope includes a well-developed organic soil and at least 3 m (10 ft) of rocky colluvium. The fine fraction of the colluvium is plastic and very slippery when wet. Colluvium is derived from Mississippian formations, but no bedrock outcrops were observed in the area, and the depth of cover is uncertain. Numerous springs and seeps are located along the northern margin and drain ground of higher topographic relief to the northeast.

Relative locations for the five extensometers established in the slide area are shown on Figure 5. Devices indicated as E2 and E5 were placed across the north lateral scarp of the slide to detect potential reactivation of debris remaining in the disturbed zone. E4 was located across a fracture bounding a detached block on the south side near the headwall, and E1 and E2 were placed across fractures well above the headwall but in the path of potential uphill progression of the zone of evacuation.
FIGURE 5 Reynolds Gulch Landslide Area, Big Cottonwood Canyon
REYNOLDS GULCH: FIELD DATA

The extensometer sites were established on November 17, 1983 and preliminary readings were made on this date and on November 30. Heavy storms prevented access to the site until January 7, 1984. Readings on this date disclosed an apparent extension of 70 mm (2.76 in) on E5 with little or no movement at the remaining installations. On January 21, the site was visited and the snow removed from E5. Inspection disclosed excessive bending of the extensometer tube. Damage was attributed to snow creep and the fact that this extensometer was positioned at an oblique angle to the down-slope vector. All other devices were located nearly parallel to this vector and, therefore, were not subjected to transverse loading. In order to insure proper functioning of the extensometers, a short length of structural tubing was bolted to the existing tube to form a composite beam with greater depth. In addition, snow was periodically removed to limit the load supported by the span. This procedure was employed for all extensometers except E4. This site was left undisturbed so that the full effect of snow loading could be assessed.

Figure 6 summarizes climatic and deformation history during the winter-spring transition. As can be seen the temperatures were predominantly below freezing up until about May 9. (Temperatures shown on this figure are those for the Brighton Station.) Subsequent to this date the temperatures were essentially above freezing. Between May 10 and May 20, rapid melting of the snow pack occurred as indicated by the snow surveys conducted on site for these dates. Anomalous rates of movement at both E1 and E3 were detected subsequent to May 12. No significant movement was indicated by either E2, E4 or E5. Interrogation of E3 disclosed a progressive increase in the rate of deformation beginning at 0.7 mm/hr (0.03 in/hr) on May 13, up to a maximum of 7.2 mm/hr (0.28 in/hr) recorded on May 20. Fractures in the snow were observed from the air on May 16, confirming reactivation of the slide. Interrogation of E1, however, disclosed a decreasing rate beginning on May 16, and continuing to May 20. On this date the rate of movement was negative indicating contraction of the extensometer which suggested that the uphill anchor for E1 was no longer stationary. On May 23, E3 was repositioned to provide additional range, and at the same time, conditions at E1 were investigated. Inspection disclosed that the fracture at E3 had extended northward isolating a block of ground which included both anchors for E1. Upon leaving the area at 7:30 a.m., masses of earth and vegetation measuring several cubic meters in volume were observed sliding down the southern flank of the upper slide. This event was the initiation of a small debris flow, the runout of which was largely confined to the preexisting landslide scar.

The deformation history disclosed by Figure 6 is very similar to that shown in Figure 8. Prior to melting of the snow pack, little or no fissure separation is indicated. Onset of movement lagged significant reduction in the snow pack by at least two days. Abrupt cessation of movement followed the debris flow also by about two days. The period of active fissure separation spanned approximately 13 days from May 12 to May 25. No further separation was measured at E3 even though significant precipitation occurred during the first part of June [10.7 cm (4.2 in) from May 30 to June 6].
FIGURE 6 Meteorologic and Extensometer Data for Reynolds Gulch
RUDD CANYON - SITE DESCRIPTION

Rudd Canyon is located immediately east of the community of Farmington and extends eastward into the Wasatch Range. The source area responsible for the 1983 debris flow is located at a prominent inflection in the main drainage of the canyon where a wedge of unconsolidated material, likely an ancient landslide mass, is situated (perched) at an elevation of about 2100 m (6925 ft) (Kaliser, 1983). The upper surface of the wedge dips as little as 5° to 7° to the west while undisturbed slopes in the source area typically range from 27° to 38°. The unconsolidated wedge consists of permeable, granular soils derived from the metamorphic rocks of the Farmington complex. These soils present ideal conditions for infiltration of melting snow and development of high piezometric pressures.

As shown on Figure 7, a series of incipient slumps are located above and to the north of the headwall and extend westward along the northern boundary of the scar. A less conspicuous, continuous tension fracture extends from the headwall area northward in a circular arc terminating above an area of natural seeps and flowing springs. This fracture defines the slump block which contains at least 6100 m³ (8,000 yd³) (Vandré, 1983) and perhaps significantly more (Wieczorek et al., 1983).

A topographic depression exists in a nearly straight line between the sites marked E3 and T3. Precipitation falling on the slopes above E3 and on or above the slump block flows to this drainage, either on the surface or underground, and feeds the perennial springs in the vicinity of T3, T1 and T2 located on Figure 7. Near-surface springs were observed (May 12, 1984) discharging directly into headwall fractures located at the apex of the drainage. Similar conditions were also observed in the spring of 1983 (Machette, 1983) and will probably reoccur in the future.

On November 16, 1983, installation of eight earth movement detection devices was begun and completed on November 19. Three extensometers (designated E1 through E3) and two inclinometers (designated T4 and T5) were placed in the upper area of the scarp. T4 and T5 were buried at a depth of approximately 30 cm (12 in) immediately behind the headwall. These devices were positioned to detect potential uphill progression of the 1983 scarp. E3 was positioned across a fissure defining the most prominent incipient slump along the north boundary of the landslide scar. Two extensometers, E1 and E2, were placed across the fracture defining the northern boundary of the slump block.

The remaining instruments were positioned near the bottom of the scar. One inclinometer, designated as T3, was buried in the ground above active springs on the north side of the canyon. The second, indicated as T1, was placed on the surface below the springs and approximately 0.6 m (2 ft) above the stream channel. The third, indicated as T2, was also placed on the surface approximately 1.8 m (6 ft) above the stream channel and about 40 m (130 ft) downstream from T2. Both T1 and T2 were intentionally placed in the channel so they would be swept away in the event of a debris flow.
FIGURE 7 Rudd Canyon Landslide Area, Farmington, Utah (1983 Slide Boundaries after Machette, 1983)

T = INCLINOMETER
E = EXTENSOMETER
RHOD CANYON - FIELD DATA

Figure 8 summarizes data telemetered from the base station located at the helipad and subsequently plotted by the PDP 11/34A at the University of Utah. As indicated, temperatures remained near or below freezing until Julian date 129 (May 8) which marks the beginning of a significant warming trend. Visual estimates indicated that south-facing slopes were nearly clear of snow by May 12, and most of the snow located at the helipad and bench area melted between May 4 and May 16. The water equivalent in the snow at the helipad was measured at 66.3 cm (26.1 in) on April 18.

At 10:33 p.m., on May 15 (Julian date 136), Davis County dispatch reported an alarm originating from E3. Printed data indicated a cumulative downslope movement of 7 mm (0.28 in) between 4:00 p.m., May 15 and the time of the alarm. By 1:00 a.m., May 16, the displacement was 11 mm (0.43 in) and by 5:47 a.m., it was 19 mm (0.75 in). From May 16 to May 18, the rate of separation at E3 averaged 1.5 mm/hr (0.06 in/hr) and increased to 2.0 mm/hr (0.08 in/hr) for period May 19 to May 21. Thereafter, the rate began to decline.

At 12:16 p.m., May 23, Farmington dispatch received alarms from T1 and T3. Radio contact was made with a nearby Forest Service helicopter and a request was made to inspect the area. The helicopter arrived in time for personnel to confirm a debris flow issuing from the spring area at the lower limit of the slide area. The initial flow from the slide area was described as very small, but the volume of the flow increased substantially as debris continued down the canyon.

Immediately following the alarms, two Farmington City personnel were dispatched. One individual reported to the aqueduct road and the other to the debris basin below the road. The debris flow was first sighted from the road. At the debris basin, clear water was observed, and then for a period of about 30 seconds, a cessation of all flow occurred. Following this event, a 2 m (6 to 8 ft) wall of debris was observed followed 2 to 3 minutes later by a 3.5 m (10 to 12 ft) wave of coarser material. According to the Davis County Sheriff's log, 6 minutes elapsed from the time of the alarms to sighting of the debris flow from the aqueduct road. An additional 6 minutes elapsed from sighting at the road to a report of debris in the basin. Duration of surging was for a period of approximately 1 hour. A new surge was sighted from the aqueduct road at 3:30 p.m.; at 3:36 p.m. an alarm was received from T2. There was a decrease in water flow at 4:04 p.m. followed immediately by a debris flow surge 4.6 m (15 ft) high. By 4:12 p.m. only muddy water was flowing.

Evaluation of printed data confirms that the alarm thresholds for T1 and T3 were exceeded in the printout interval 12:16 p.m. to 12:26 p.m. The record clearly shows that T1 was disturbed first followed by progressive failure of the bank above the springs in which T3 was buried. This failure process continued at least 30 minutes before the bank collapsed. Printed data also confirm an alarm from T2 for the interval 3:36 p.m. to 3:46 p.m.

Field inspection on May 24, disclosed continued movement in the vicinity of extensometer E3, but no visual indication of movement was evident at the
FIGURE 8 Meteorologic and Extensometer Data for Rudd Canyon

- **Temperature**
  - Temperatures range from approximately 0°C to 40°C, with fluctuations throughout the timeline.

- **Precipitation Equivalent Water**
  - Precipitation levels are shown with a range from 0 to 15 mm (inches).

- **Fracture Separation**
  - Fracture separation shows a gradual increase over time, with several marked events:
    - **May 15:** An event with a separation of approximately 5 mm.
    - **May 23:** The event is marked with a separate line indicating a significant increase in fracture separation.
    - **June 1:** Another marked event shows a slight increase in fracture separation.

The timeline is marked with Julian dates from 1984, with notable dates indicating significant events related to the phenomena being studied.
FIGURE 9 Comparison of Extensometer Readings for the Rudd Canyon Site

**E1**

- 80 (mm)
- 70
- 60
- 50
- 40
- 30
- 20
- 10
- 0

- 126 130 134 138 142 146 150 154 158 162

**E2**

- 40 (mm)
- 30
- 20
- 10
- 0

- 126 130 134 138 142 146 150 154 158 162

**E3**

- 450 (mm)
- 350
- 250
- 150
- 50

- 126 130 134 138 142 146 150 154 158 162

**JULIAN DATE 1984**

- MAY 15
- JUNE 1
remaining installations near the top of the slide. Inspection of the lower area confirmed the sequence of events as reported above. The inclinometer T1 was located very near the stream immediately below the spring issuing from the colluvium. In this position it was the first to encounter a discharge of debris from the springs. T3 was located approximately 26 ft behind the crest of the steep slope above the springs and responded to subsequent failure of the bank. Apparently, the successive flows were not high enough to reach T2 until the 3:36 p.m. episode. T2 was removed abruptly as indicated by the position of the connecting electrical cable.

It is significant to note that rainfall of 3.8 cm (1.5 in) in a 9-hour period, as recorded in the telemetry data, had no noticeable effect on fissure separation or slide reactivation.

Field estimates indicated that 460 m³ (600 yds³) of earth were removed from the north slump block in the vicinity of T3. The volume of material deposited in the basin as of May 24 was approximately 9150 m³ (12,000 yds³). These figures indicate that 95% of the debris originated from the stream channel where it had been accumulating over the preceding 12 months (Kaliser and McCarter, 1984).

Telemetry data disclose no movement at either T4 or T5. Data from E1 and E2, however, show a response similar to E3 but much less pronounced (see Figure 9). All three curves disclose a displacement rate transition beginning about May 15. The prominence of the transition appears to be a function of distance to the drainage feature previously identified. As indicated, E3 is located near the apex and within this feature, and it displays the most prominent transition. Abatement of motion at E1 and E2 is not obvious until about June 1. The trend towards improving stability at E3, however, is well-developed prior to this date. These observations suggest that the time required for recovery is, in part, related to the volume of the affected mass.

CONCLUSIONS

The objectives of this project were to explore the usefulness of surface deformation measurements in identifying precursory events leading to gross slope failure, to develop instruments capable of surviving the severe climatic environment present in mountainous terrain, and to determine if such instruments can be maintained during the winter-spring transition.

Reactivation of surface extensional fractures is undoubtedly related to a decrease in effective soil strength precipitated by rapid infiltration of snow melt. If the reduction in effective strength is related to a rising phreatic surface, as it appears to be, installation of piezometers may be a more direct method for detecting deteriorating conditions. Strategic placement of these instruments, however, is not obvious. Observations regarding discharge points relative to debris slides, spring fluctuations, soil stratigraphy, and slide morphology made during this study clearly indicate complexities in groundwater distribution that will not be easily comprehended, particularly over any appreciable areal extent. A comprehensive program to identify typical groundwater regimes associated with debris flow source areas would facilitate devel-
opment of the best strategy for deployment of piezometers. In the meantime, monitoring surface deformation appears to be a viable technique for defining the critical period in which debris flows are most likely to occur. Data from two of the three sites indicate that the beginning of the critical period is defined by the transition from little or no movement to active displacement across extensional fractures. Once displacement returns to the pre-transition rate, danger of a debris flow is apparently over for the season.

The Rudd extensometer developed for this study operates satisfactorily as long as the span is limited to 12 feet, the trend of the extensometer parallels the downslope vector, and the compacted snow cover does not exceed 4 feet at 43% density. Maintenance is possible but difficult in remote mountainous terrain and is warranted only where monitoring information is needed for scientific purposes or where it is essential to provide added protection for down-slope communities.

ACKNOWLEDGMENTS

The success of this effort is, in large measure, due to dedicated people in various agencies including the Division of Comprehensive Emergency Management (CEM), Wasatch National Forest, Utah Engineering Experiment Station, University of Utah Research Institute (UURI), Salt Lake County Commission, Salt Lake Water Department (SLWD), Davis County Commission, Davis County Sheriff's Office and City of Farmington. The authors would like to extend sincere appreciation to the students in the Department of Mining Engineering, University of Utah, specifically Robert Cameron, Jess Kelley, Charysse Menig and Rex Simpson for their dedication to this project. We would also like to acknowledge the active participation of Robert Kistner, formerly of CEM, and Dan Schenck, SLWD. Dale Green and Steve Olsen, UURI, also deserve special recognition for development of innovative telemetry and data processing equipment vital to the success of this project.

REFERENCES


MULTIVARIATE ANALYSIS OF LANDSLIDE-RELATED VARIABLES IN DAVIS COUNTY, UTAH

by Robert T. Pack

ABSTRACT

During the spring snowmelt period of 1983, over ninety landslides occurred along the Wasatch Front from Kaysville on the north to Bountiful on the south. One hundred and fifteen locations which produced landslides prior to and including 1983 are identified in this same area and compared to one hundred and fifteen randomly chosen locations which have not produced landslides. From this extensive inventory, eight landslide-related variables are identified. Data collected for these variables at each of the two hundred and thirty locations include those which can be delineated from color and near-infrared aerial photography, geologic maps, and topographic maps currently available for the area.

A discrete discriminant analysis based on the full multinomial classification model is used to develop a classification scheme capable of distinguishing between areas of higher and lower landslide potential solely on the basis of variables present at a site. Continuous variables are broken down into discrete classes and included in the model. The resulting classification scheme is capable of correctly classifying eighty-eight percent of the known landslide sites on the basis of four of the original eight variables. The four variables, which include slope, vegetation, terrain type, and slope morphology, were then mapped over a one square mile subset of the study area. By digitizing the map information and inputing it into a computer, the classification scheme is automatically applied to the map area to produce a computer-generated landslide potential map.

INTRODUCTION

Unusually heavy snowmelt during the spring of 1983 triggered over 90 landslides in a 46 square mile area of Davis County within a four-week period. Many of these landslides, commonly referred to as "debris slides" (Varnes, 1978) or "soil slips" (Campbell, 1975), came to rest only a short distance downslope from the zone of initiation. However, some of them mobilized into debris flows which transported debris including large boulders...
DESCRIPTION AND EVALUATION OF SLOPE HAZARDS

up to 5 miles downslope (Pack, 1984). One landslide in a particularly critical location initiated a debris flow which subsequently inundated several square blocks of the town of Farmington destroying several houses.

Although over 90 landslides initiated, only a few occurred in locations where subsequent debris flows were of consequence to residents of Davis County. The objective of this study is to develop and apply a method for delineating areas according to landslide potential. Meeting this objective involves four major tasks: (1) inventorying existing landslide areas and identifying important landslide-related variables, (2) collecting data on these variables for both landslide and non-landslide locations, (3) developing a classification scheme capable of distinguishing between areas of lower versus higher landslide potential on the basis of these landslide-related variables, and (4) using the classification scheme to delineate a landslide potential map. Figure I shows the landslide inventory area (sampling area) used in developing the classification scheme and the one square mile test area where the landslide potential map is delineated on the basis of the classification.

The first section of this paper will present the results of a statistical analysis of eight landslide-related variables. The classification scheme developed by a discriminant analysis of these variables will be presented in the next section. The final section will then present a landslide potential map based on the classification.

LANDSLIDE-RELATED VARIABLES

In past studies, both static and dynamic geomorphologic, geologic, hydrologic, topographic, vegetative and climatic variables have been attributed to landslide occurrence (Simons and Ward, 1979; Roth, 1983; Nilson et al., 1976; Campbell, 1975). In this study, only static time-independent variables are considered in landslide potential delineation.

On preliminary inspection of 115 landslide sites in the 46 square mile study region, qualitative variables identifiable from aerial photographs deemed important include (1) across-slope morphology, (2) down-slope morphology, (3) drainage order, (4) terrain class, and (5) vegetation class. Important quantitative variables measurable from 1:24000 scale USGS topographic maps included in the analysis are (1) elevation, (2) topographic aspect, and (3) slope gradient.

Another 115 sites were randomly chosen across the study area in order to sample the natural distribution of these variables. By comparing the distributions of variables at landslide sites to the distributions found associated with the randomly chosen non-landslide sites, the bias imparted to the analysis by the particular characteristics of the study area can be accounted for.

Past studies have shown that local geology is an important variable in determining landslide locations (Pack, 1984). However, the entire study region lies in an area mapped as an undifferentiated high-grade metamorphic
FIGURE 1 Index map showing the location of the landslide inventory area and mapping area. Each dot represents one landslide site.
complex (Bryant, 1979) which does not allow differentiation of geology between sites without extensive site-specific investigation. Geology, therefore, is not included in the analysis.

Analysis of Qualitative Variables

Data on 5 classes of down-slope morphology were collected for 115 landslide sites and 115 randomly chosen non-landslide sites. The classes are described as (1) sharp break in slope, (2) rounded break in slope, (3) uniform slope, (4) concave slope, (5) sharply concave slope. Figure 2 shows a histogram comparing the frequencies of the two types of sites for the given classes. Inspection of this figure indicates that landslide sites are much more frequently located at breaks in slope than non-landslide sites but that the majority occur on uniform slopes. Upslope from a break in slope, the less steep area may serve as an area of increased groundwater recharge during snowmelt or rainfall (Vandre, 1983). This may explain the more frequent occurrence of landslides at breaks in slope when coupled with the fact that the piezometric groundwater level is naturally closer to the ground surface on the steeper portion of the slope below the break.

Classes for across-slope morphology include: (1) strongly concave contour shape (V-notched gully), (2) concave contour shape (swale), (3) straight contour shape, (4) convex contour shape (rounded ridge), and (5) strongly convex contour shape (sharp ridge). Figure 3 shows a histogram of frequencies for this variable and indicates that landslides occur more frequently on concave slopes (swales and gullies) and less frequently on ridges than the randomly chosen non-landslide sites. Studies in other regions but in similar topography indicate that concave slopes are more likely to concentrate groundwater interflow (flow parallel to bedrock) during rainfall or snowmelt which results in elevated pore water pressures (Pierson, 1980; Anderson and Burt, 1978). Steeper slopes which tend to be more susceptible to landsliding are also found in sharply concave areas where gully walls are oversteepened.

Three stream order classes were also considered in the analysis: (1) 0th order -- upper swales and gullies of the watershed, (2) 1st order -- middle reaches of the watershed below the intersection of at least two 0th order gullies, (3) 2nd order -- lower reaches of the watershed below the intersection of at least two 1st order gullies. Sites were assigned to one of these classes depending on their proximity to a 0th, 1st or 2nd order stream. Figure 4 shows that landslide sites occur only a little less frequently near 0th and 2nd order streams and somewhat more frequently near 1st order streams than the stable sites.

Terrain in the study region is broken down into 5 possible categories including (1) over 50% rock outcrop (rock cliffs), (2) between 10% and 50% rock outcrop (scattered bedrock), (3) rubbly colluvial veneer between approximately 1 and 3 feet deep, (4) deeper sandy colluvium and silty residual soil, and (5) Lake Bonneville terrace sands and gravels. Figure 5 shows that none of the landslide sites occur in rock outcrop areas nor on Lake Bonneville terraces and only a few occur in rubbly areas. The majority of
FIGURE 2  Down-slope shape histogram

FIGURE 3  Across-slope shape histogram

FIGURE 4  Stream order histogram

FIGURE 5  Terrain histogram
Landslides occur in deeper colluvial and/or residual soils. Pack (1984) found that the failure surface of 9 out of 12 landslides examined in the field is associated with a discrete zone of clay formed from residual soil and weathered bedrock (schist and gneiss). This could explain the more frequent occurrence of landslides in areas with deeper soils where more weathering has taken place.

Six classes of vegetation in the study area were considered: (1) exposed bedrock (no vegetation), (2) grasses and herbs, (3) low shrubs, (4) high shrubs and deciduous trees, (5) scattered coniferous trees, and (6) coniferous trees. Landslides were found to occur most frequently in areas with high shrubs and deciduous trees (see Figure 6). Interestingly, many more non-landslide sites are located in areas of herbs and grasses than landslide sites. This is perhaps a reflection of the fact that this kind of vegetation generally prefers dryer (more xeric) sites on shedding slopes (Comeau et al., 1982). On the other hand, deciduous trees such as quaking aspen prefer wetter (more hygric) sites where higher pore water pressures within the soil are more likely.

Analysis of Quantitative Variables

For the 230 sites, the average slope gradient over a 200 foot drop in elevation was measured on 1:24,000 scale 7 1/2 minute quadrangle maps for the area. Figure 7 is a histogram comparing the average slope gradient subdivided into 2 degree classes for landslide and non-landslide sites. This figure shows that landslide sites generally occur on the steeper slopes of the study area. All landslide sites occurred on slopes steeper than 18 degrees with over one-half on slope gradients of between 28 and 34 degrees. Slopes above 40 degrees do not have landslides associated with them as they are generally too steep to accumulate soil over bedrock. The marked difference between the histogram shapes of landslide and non-landslide sites shown in Figure 7 is a reflection of the direct relationship between increasing slope gradient and decreasing slope stability and the inverse relationship between increasing slope gradient and decreasing soil depth.

FIGURE 6 Vegetation histogram.
Elevations were also measured in feet above sea level as the value of the nearest 40 foot contour line to the headscarp of landslide sites and to the randomly selected non-landslide points delineated on 1:24,000 scale quadrangle maps. Figure 8 is a histogram comparing the elevations divided into 200 foot intervals for landslide and non-landslide sites. This figure shows that almost one-half of the landslides occurred within the 6800 to 7400 elevation range while only 23 percent of the non-landslide sites fell within this range. The zone of maximum snow melt production occurred at the snowline which was within the 6800 to 7400 foot elevation range at the time of unseasonably high temperatures of late May, 1983, and is probably at least partially responsible for the large number of landslides found within this zone (Pack, 1984). It is also possible that the long-term climatic conditions which are strongly affected by elevation may have lead to more rapid soil formation, soil weathering, and soil shear strength reduction within this zone.

The aspect of slopes on which the 230 sites are found was measured as the azimuth of lines drawn perpendicular to contour lines found on 1:24,000 maps of the area. Figure 9 is a histogram showing aspect divided into 20 degree intervals for landslide and non-landslide sites. This figure shows little distinction between the distribution of the two types of sites on the basis of aspect. The fact that the study area lies on the west-facing side of the Wasatch Front explains why very few east-facing slopes in the 30 to 130 degrees azimuth range are present.

DISCRIMINANT ANALYSIS

This section will discuss the use of the variables described above in classifying sites according to relative landslide potential. A common statistical procedure used in the development of an optimal classification scheme is referred to as a discriminant analysis. In past studies, linear discriminant analysis procedures have been applied in the study of landslide-related variables in cases where the variables can be assumed to have continuous normal distributions (Jones et al., 1961; Reger, 1979; Pillsbury, 1976; Carrara, 1983). The three quantitative variables analyzed in this study appear to be nearly normally distributed. However, in order to include the five qualitative variables in this type of analysis, it is necessary to convert them into a normally distributed continuous variable form. Unfortunately, none of the five qualitative variables can be safely assumed to have normal distributions in this particular case. An alternative is to convert the continuous quantitative variables into discrete ones for use in a discrete discriminant analysis which has no normal distribution assumption.

In this study, a discrete discriminant analysis using the multinomial classification model (Goldstein and Dillon, 1978) is employed. To the author's knowledge, no prior landslide-related studies have used this complete model, though a variation of it called the 'matrix approach' has been employed by Degrass and Romesburg (1980) in studying regional landslide-susceptibility.
FIGURE 7  Slope gradient histogram

FIGURE 8  Elevation histogram

FIGURE 9  Aspect histogram

FIGURE 10 State probabilities $f(x)$ and 90% confidence intervals associated with each of five classes of across-slope shape.
To illustrate the use of the multinomial classification model in discriminating between two populations (landslide vs. non-landslide), the most simple type of analysis where only one variable is used (one-dimensional analysis) will first be demonstrated. In referring to Figure 3 which is a histogram of across-slope shape, it is found that 115 of the 230 sites are landslide sites. The 115 sites can be thought of as a binomial random variable $N(x)$ with expected value $N(x) + N(x)$ where $N(x)$ is the intuitive estimate of the a priori probability given by $N1/N$, and $N(x)$ is a non-parametric estimate of the class-conditional density at $x$ given by $N1(x)/N1$. At $x$, the discriminant score is defined as $g(x) = N(x) + N(x)$ for $x=1,2$ and the state probability $P(x)$, i.e., the probability that a given sample found in $x$ is a landslide site, is $P(x)/g(x)$. $P(x)$ is a parameter describing the binomial distribution of $N(x)$ and has well known properties. Taking the class V-NOTCH in Figure 3 as an example, $P_1 = 115/230 = 0.50$, $P_2 = 17/115 = 0.1478$, $P_3 = 8/115 = 0.0696$, and since $P_1 = P_2$, $P_3 = 0.1478/(0.1478 + 0.0696) = 0.68$. $P(x)$ can be interpreted as meaning, given a site occurs in a V-notch gully, it has a 68% probability of being a landslide site.

Associated with the state probability $P(x)$ is a confidence interval which is a reflection of the total number of individuals (sites), in this case 25, used in estimating the probability. Using confidence interval tables for binomial random variables given by Burstein (1971) a 90% confidence interval for the given state probability is 0.51 to 0.85. In another example, suppose that a site is chosen on a uniform slope where we find that $P(x) = 0.6$ as determined from 83 individuals. In this case the 90% confidence interval is 0.5 to 0.7. As is intuitively expected, the larger the number of individuals which fall into a particular class, the smaller the associated confidence interval becomes. Figure 10 shows the state probabilities and confidence intervals associated with each of the five classes of across-slope shape.

An intuitive sample-based classification rule for this model is to assign a site to the landslide class if $P(x)$ for the given site characteristic is greater than 0.5. If $P(x) < 0.5$, the site is assigned to the non-landslide class and if $P(x) = 0.5$ the site is randomly assigned. To test the success of this classification rule in the case where the one variable 'across-slope shape' is used, Figure 10 is used to classify each of the 230 sites by specifying that if a site is located on a V-notch, concave, or uniform slope where $P(x) > 0.5$, it should be classified as a landslide site, and if it is located on a convex slope or sharp ridgecrest where $P(x) < 0.5$, it should be classified as a non-landslide site. It was found that 69 of the 230 sites were misclassified using this method which is equivalent to a 70% success rate.

Variables which have state probabilities close to both 1.0 and 0.0 are the best for classification. For example, if all 115 landslide sites occurred on concave slopes and all 115 randomly chosen non-landslide sites occurred only on other types of across-slope shapes, a 100% classification
success rate would result. In discretizing the three continuous variables, slope gradient (SLOP), elevation (ELEV) and aspect (ASP), it is important to ascribe class boundaries which produce state probabilities close to 0.0 and 1.0. At the outset, it was found that all attempts to subdivide ASP into discrete classes resulted in state probabilities very close to 0.5. This variable was therefore never considered further in the analysis. Figure 11 shows the shapes of the approximated normal distributions for SLOP at both landslide and non-landslide sites. From this figure we see that by assigning a class boundary at 15 degrees a state probability near 0.0 results for the <15 degree class. By further subdividing it at 25 degrees we separate out the 15-25 degree class where P(x) is very low from the above 25 degree class where P(x) is quite high. The 35 degree subdivision was somewhat arbitrarily chosen to produce a second 10 degree class interval of 25-30 degrees. The discretization of SLOP therefore results in a 4 class discrete variable. Figure 12 shows the approximated normal distribution shapes for ELEV. Using a procedure identical to that described above for SLOP, class boundaries at 6000 and 7800 ft were assigned resulting in a 3 class discrete variable.

Each of the seven discrete variables, when employed separately in a one-dimensional classification scheme, result in the following percentages of sites classified correctly:

- **Terrain**: 73%
- **Across-slope shape**: 70%
- **Slope gradient**: 70%
- **Vegetation**: 69%
- **Down-slope shape**: 65%
- **Elevation**: 63%
- **Stream order**: 61%

Using the simple one-dimensional classifier results in relatively low percentages classified correctly as shown above. However, the same principles described above can be used to develop n-dimensional classifiers which are capable of producing much better results by using all variables simultaneously.

Figure 13 schematically shows how the expansion of classification procedure to include three variables increases the number of states for which state probabilities P(x) are calculated. The classification rule based on the values of P(x) remains identical to that described for the one-dimensional case. In the multi-dimensional case, however, unique combinations of variable classes (states) which tend to increase landslide potential will result in values of P(x) much closer to 1.0 than would ever be found with a one-dimensional analysis. Likewise, unique combinations seldom found associated with landslides will result in values of P(x) closer to 0.0. Thus the percentage of sites classified correctly is increased.

The goal of this study is to identify the optimal classifier which is best able to classify sites on the basis of site variables. Goldstein and Dillon (1978) present measures of optimality for a given classification which include (1) actual error, (2) apparent error, (3) discriminant score, (4)
FIGURE 11 Approximated normal distributions for slope gradient for landslide and non-landslide sites.

FIGURE 12 Approximated normal distributions for elevation for landslide and non-landslide sites.

FIGURE 13 The proliferation of states when the dimensionality of the classification procedure is increased.

FIGURE 14 Percentage of sites classified correctly by the best classifier in each dimension.
percent classified correctly and (5) significance of probabilities; the reader is referred to their book for a mathematical description of each. For 7 variables, 126 classifications are possible with various variable combinations in 0 to 7 dimensions. For each possible classification the measures of optimality described above were calculated. The most successful classifier based on the best combination of variables was then selected for each dimension. Figure 14 shows the results of this analysis as a plot of percentage of sites classified correctly for the best classifier in each dimension.

Figure 14 shows that in 7 dimensions using all the variables, 96% of the sites were classified correctly. However, because of the large number of states possible in this classification, only an average of 1.7 sites were used in the calculation of state probabilities. This leads to unreasonably wide 90% confidence intervals for the state probabilities. In one dimension, the best classification correctly classified 73% with an average of 112 sites used in the calculation of each state probability. The 90% confidence intervals in this case are very narrow. Figure 14 shows that with less than 4 dimensions, the percentage classified correctly begins to drop significantly. It was therefore decided that a 4-dimensional classification scheme with a success rate of 88% and an average of 6.3 sites used in the calculation of each state probability would be a good compromise. This optimal 4-dimensional classification includes the variables (1) terrain, (2) across-slope shape, (3) slope gradient, and (4) vegetation. It is probably not coincidental that these four variables produce the four best one-dimensional classifications.

LANDSLIDE POTENTIAL DELINEATION

Section 21, T. 3 N, R. 1 E, of the Bountiful Peak 7 1/2' Quadrangle was chosen for detailed mapping of the four variables chosen for use in the landslide potential classification scheme developed above. Terrain and vegetation maps were delineated from infrared aerial photography and are shown as Figures 16 and 17. Across-slope shape and slope maps were derived from contours of the Bountiful Peak Quadrangle map and are shown as Figures 18 and 15.

These maps were then digitized by computer and stored on disk in the form of a 100 x 100 element array. Each of the 10,000 elements of the array represents a 52.8' x 52.8' area located on the map and contains a number assigned to the class present at that location. The numbers contained in each of the 4 elements corresponding to identical locations on each of the 4 maps are then input into an algorithm which then automatically calculates the state probability according to the multinomial model. The computer then stores the probability in the corresponding element of the array which represents the derived landslide potential map. This process is continued until each of the 10,000 elements has been assigned probabilities. Figure 19 is a computer plot of the results of this analysis where the dark areas represent areas of higher landslide potential (Px>0.5) and the white areas represent areas of lower landslide potential (Px<0.5). Also plotted in black are the actual locations of slides which are pointed to by arrows.
FIGURE 19 Landslide potential map plotted by computer showing in dark areas of higher landslide potential. The arrows point to actual landslide locations delineated in black.
In every case the landslide sites are located within areas delineated as having a greater potential for landsliding. It is important to note that the state probability \( P(x) \) does not represent the probability that a landslide will occur within the area of the element. With the sampling procedure used in this analysis, 115 out of 230 sites were purposely chosen as landslide sites. The a priori sample probability of a landslide is therefore 0.5. This value is then used in computing the state probabilities \( P(x) \). On the other hand, the combined area of landslides across a 46.3 square mile region is only 0.0113 square miles. This results in an a priori spatial probability of \( 2.45 \times 10^{-4} \) which is defined as the probability that any one site will have a landslide given that landslides occur entirely at random across the area. Nowhere in this analysis is a particular time-frame implied.

**CONCLUSIONS**

The classification scheme developed using the multinomial model is successful in correctly classifying 88% of the landslide sites solely on the basis of 4 important landslide-related variables. The landslide potential map produced by the classification is successful in delineating, by an objective numerical procedure, areas which intuitively 'appear' to be landslide-prone based on experience.

The small number of samples used in calculating some of the state probabilities gives a wide 90% confidence interval. Reducing the number of classes of some of the variables is justified and could help solve this problem by reducing the number of states without reducing the success rate of the classification scheme.

With little effort, the analysis could be extended to determine the spatial probability of a landslide(s) occurring within any given watershed segment. This would be particularly useful in ranking watersheds according to the number of landslides they are likely to produce at some time in the future.

The results of this study are specific to the Wasatch Front in Davis County and cannot be extended to other regions without further data collection and statistical analysis. However, the simple objective method presented should be considered as an alternative means to analyzing multivariate landslide data and delineating landslide potential in other areas.

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APPLICATIONS OF INFRARED AND OBLIQUE AERIAL PHOTOGRAPHY
FOR DETECTION AND DELINEATION OF GEOLOGIC HAZARDS

by Woodrow L. Higdon

ABSTRACT

Photography as a technical tool has been available for many years, but has never been specifically used as an investigative tool by geologist trained in professional photography. The application of specialized techniques, films, and knowledge can significantly improve detection and delineation capabilities for various types of geologic hazards. The effective application requires multiple professional skills in an individual rather than several individuals as the situation presently exists. The end result is ground and aerial photography in black and white, color, and infrared that is specifically generated for existing or suspected geologic conditions. The directions and angles of views are determined by a pre-shooting geologic review of available information on the area, and an on site visual analysis during field shooting. The ability to understand and evaluate geologic data, and to recognize geologic features in the field is essential to the generation of useful and informative imagery. Professional photographic skills applied by geotechnical professionals can significantly improve these areas of information, as well as other areas of information in land use projects.

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Prediction and Protection for Slope Hazards

UTAH'S LANDSLIDES OF 1983 AS A MAGNITUDE-FREQUENCY EVENT WITH A FINITE RETURN PROBABILITY

by Andrew E. Godfrey

Landslides are traditionally viewed as individual events occurring in a closed system and the landslide activity on the Wasatch Plateau in central Utah during 1983 is viewed as an extraordinary event, expected only in exceptionally wet years. An alternative approach is to adapt the fluvial hydrologist's magnitude and frequency calculations for flood events to the prediction of landslide probabilities. This paper explores a possible method for calculating landslide magnitude and frequency across the entire Wasatch Plateau. To do this, the more traditional closed system approach is replaced by one in which hillsides subject to potential landsliding are viewed as open systems.

Preliminary results from this study indicate that mass movements in the most unstable areas of the Wasatch Plateau, such as those underlain by the North Horn Formation, can occur with equal probability in any given year. However, the total land area involved is only a small percentage of the entire Wasatch Plateau. Examples of these landslides are the one in Bulger Canyon (1971) which involved 10 acres and the one above Manti (1974) involving about 300 acres. In contrast, the wet year of 1983 produced a higher percentage of area involved in mass movement on the Wasatch Plateau, about 6,500 acres. In addition, the 1983 slides occurred in areas of lower landslide susceptibility than the areas involved in drier years.

When viewed as magnitude-frequency events, the effects of mass movements in shaping landforms can be placed in proper perspective. Although the locations of specific slides cannot be predicted, since statistical analysis results in the loss of a degree of spatial resolution, these magnitude-frequency data, combined with traditional landslide hazard studies, can enhance predictions of hazards to man-made structures.

INTRODUCTION

Landslides and debris flows are often considered, along with volcanic eruptions, earthquakes, and large wildfires, as very rapid catastrophic changes that alter the normal geologic processes operating in an area. This

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viewpoint treats each happening as an individual event unrelated to other happenings in the same area, and is the one most often represented in the media. In many respects this is the "why me?" attitude.

A second approach to landslides and debris flows can be described, in general systems theory, as the closed system approach. In this approach, a landslide or debris flow is considered to have a fixed potential energy due to its elevation above some base level. Movement occurs because conditions within the mass have surpassed the threshold where shear stress equals restraining forces. Movement ceases when the forces are again brought into balance, either by sufficient movement or by application of engineering practices.

A third approach, the one to be discussed in this paper, is described in general systems theory as the open system approach. In its strictest sense, an open system is one where a state of balance of land attributes is maintained by the rate of supply of material equaling the rate of removal of material from the system, such that the entropy of the system does not increase (Leopold and Langbein, 1962). This approach has been applied successfully to river reaches by fluvial geomorphologists and has led to significant advances in that field of study because "it points to analogies in mechanics and chemistry and so permits the use of established statistical-mechanical principles and results" (Ruxton, 1968). However, even in restricted river reaches, the ideal condition of inflow just equaling outflow is seldom, if ever, achieved. Instead, researchers used the simplifying assumption that the rate of change of the system is so small in comparison to the rate of movement of material through the system that it can be ignored. Under certain circumstances, this assumption could be used in studying geomorphic processes on hillslopes. Specifically, this assumption appears valid when a large area, such as the Wasatch Plateau, is considered, and when the time span of a single landslide or group of landslides is considered in relation to the timespan of the Holocene.

Ruxton (1968) points out several advantages and limitations of an open system approach. Two of the advantages are the treatment of Le Chatelier's Principle and thresholds and metastable states. Le Chatelier's Principle holds that changes in an open system tend to be self-regulating so that the response of the system to an external change is such as to minimize the disturbance. Further, the rate of restoration is proportional to the degree of disturbance from the equilibrium point. The realization that a threshold might be involved in the formation of a landscape can lead to an understanding of that landscape since "in certain cases the response of a system to a change of an external variable may involve a threshold, or discontinuity, which separates two rather different system economies..." (Howard, 1965, p. 308). The implication of these two ideas will be discussed below, but briefly they apply to considerations of the slopes of the Wasatch Plateau under two conditions. The first condition exists for long time periods over large areas, and involves soil formation and fluvial erosion. The second condition exists for short time spans and in relatively small areas, and involves landslides and debris flows.
One limitation of an open system theory, as pointed out by Ruxton (1968), is that "the statistical methods and concepts involved in analysing open systems generally involve the divorce of each observation from its three-dimensional spatial location and from its sequence in time so losing vital geographic and sequential data. Thus, predictions from open system analyses are given in terms of probable space or time frequencies and do not allow prediction at both a moment in time and a point in space." In dealing with landslides as a magnitude-frequency event this limitation is central because it prohibits predicting where or when the next slide will occur, but it allows statistical treatment of one year's landslides as a group.

The area chosen for study is the Wasatch Plateau portion of the High Plateaus Section of the Colorado Plateau (Figure 1). Located in central Utah, this area extends from Soldier's Summit on the north through Salina Canyon on the south. The area was initially chosen for study as a follow-up to a landslide susceptibility project (Godfrey, 1978).

1983 LANDSLIDES ON WASATCH PLATEAU

Most of the Wasatch Plateau is underlain by formations that range in age from Late Cretaceous through the early Tertiary. From oldest to youngest, they are the Mancos Shale, Star Point Sandstone, Black Hawk Formation, Castlegate Sandstone, Price River Formation, North Horn Formation, Flagstaff Limestone, Colton Formation, and Green River Formation. There are also some scattered outcrops of Jurassic rocks along the western edge of the plateau. About 99 percent of the landslide activity that occurred on the plateau in 1983 involved five formations. The following descriptions of these formations are taken from Godfrey (1978):

Cretaceous

Star Point Sandstone—450 feet

Massive cliff-forming, buff sandstone, medium to fine-grained, containing some interbedded shales which produce steplike topography. This formation is quite stable; however, the more rapid erosion of the underlying Masuk Shale removes support by undercutting, which leads to block falls.

Black Hawk Formation—1500 feet

Medium-to-fine-grained, buff and gray sandstone, gray shale, coal. Although not as unstable as the North Horn Formation, the Black Hawk underlies several areas of active movement. It produces abundant colluvial material which can become unstable.

Price River Formation—600 feet

Red to gray sandstone and conglomerate with varying amounts of shale. The appearance of this formation varies with geographical location. To the northwest, it is a coarse, conglomeratic unit
Figure 1: Location and general geography of the Wasatch Plateau, Central Utah.
with minor amounts of shale. This material appears to be stable. To the southeast, the size of the conglomerate decreases and shale interbeds appear. As a result, the topography changes from vertical cliffs to staircase forms, and the stability decreases as the proportion of shale increases.

North Horn Formation--2000 feet

Buff, gray, red sandstone, gray to variegated shale, conglomeratic, some limestone. It is exposed over large areas and is extremely unstable. The shales which comprise a major component of this formation become highly plastic when wetted and are capable of extensive mass movement. Since the sandstones and limestones of this formation are not well indurated, they cannot act as stabilizing agents.

Tertiary

Flagstaff Limestone--300 to 500 feet

Gray, tan, white limestone with minor amounts of shale and sandstone. This formation is, in general, stable. However, where the slopes are over-steepened or where support has been removed from the base of a dip slope, the shale partings can form glide surfaces for movement of the limestone blocks.

In the 1978 study the Wasatch Plateau was divided into four zones of susceptibility to landsliding:

Zone 1: Unstable

Areas that are actively sliding or moving today, plus other areas in the physiographic subsection that have the same bedrock formation, climate, slope, and aspect. These are the most unstable areas and may begin to move without the impetus of human activity.

Zone 2: Relatively unstable

Areas of dormant or fossil landslides. Areas underlain by the same geologic formations as areas of Zone 1, but with either gentler slope, more southerly aspect or a drier climate; or areas with deep gullies in which streams are cutting away the toes of slopes that may lead to landslides. Human activity could increase the probability of landslips in this zone.

Zone 3: Relatively stable

Areas of fluvially-dissected slopes underlain by the more stable formations of the region; however, where there are local areas of high slope angles and local gullying, small slumps and local sloughing may occur. These areas are relatively stable, but excessive human activity could produce slope failure.
Zone 4: Stable

Flat-lying areas underlain by stable formations. No stability problems are anticipated in these areas.

The 1983 landslides of the Wasatch Plateau were mapped on 7.5 minute quadrangles from low-flying helicopters and from spot ground checks. Alan Gallegos (personal communication) mapped the slides in the area north of Salina Canyon, within the Manti-LaSal National Forest. In the Salina Canyon area, I mapped fresh head scarps but could not locate the toes of many of the slides. Consequently, both areas affected and numbers of slides are reported north of Salina Canyon, while only slide numbers are reported for the Salina Canyon area. The 1983 landslide maps were then overlain on the stability zone maps at the same scale and compared with the 1:250,000 scale geologic maps of Utah (Stokes and Madsen, 1961; Hintze, 1963, and Hintze and Stokes, 1964), and the data were tabulated in Tables 1 through 4.

The first point shown in Table I is the high percentage of slides in the North Horn Formation (80 percent). In his compilation of landslides in Utah, Shroder (1971) indicates that for the formations present on the Wasatch Plateau, about 75 percent of the landslides are associated with the North Horn Formation. Thus the 80 percent figure cited here appears representative, and the North Horn Formation should be considered the most unstable formation in the area.

A second point is the prevalence of landslides in stability zone 2. About 60 percent of all slides and 55 percent of the acres on the Manti-LaSal portion of the area are located in this zone. In contrast, Tables 1 and 2 show a significantly lower percentage in zone 1, the zone defined as areas of active or recently active movement and adjoining areas of similar bedrock, slope, aspect, and elevation. Zone 2 was defined as areas of paleo slides, or areas lacking one of the characteristics of zone 1. Possibly, areas of zone 1 were able to move during the drier climate of the 1960's and 1970's thus preventing a buildup of material poised to move. In contrast, areas of zone 2 maintained the material on slopes where it gradually accumulated, unless disturbed by human activity. When the wet winters of 1981-82 and 1982-83 put excess moisture into the surficial material, the stage was set for the activity observed in the spring of 1983.

The single slide in zone 4 was a liquefaction event that occurred in the alluvial fill of the floor of Gooseberry Valley. These types of events were not considered in the original inventory, but recent work (Everitt and Godfrey, in preparation) suggests that they are more common than originally thought, especially in irrigated fields adjacent to arroyos.

From a magnitude-frequency point of view, it appears that slides in zone 1 are high-frequency, low-magnitude events, and that conditions favorable to instability in zone 2 are low-frequency, high-magnitude events. Magnitude here is measured in terms of the total area affected. The Manti Canyon Slide is a high frequency-low magnitude event because only a small percentage of the Wasatch Plateau was affected. In contrast, during the
Table 1. Numbers of landslides by formation by stability zone for entire Wasatch Plateau.

<table>
<thead>
<tr>
<th>FORMATION</th>
<th>STABILITY ZONE</th>
<th>NUMBER OF SLIDES</th>
<th>PERCENT OF SLIDES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Flagstaff Limestone</td>
<td>3</td>
<td>15</td>
<td>4</td>
</tr>
<tr>
<td>North Horn Formation</td>
<td>134</td>
<td>186</td>
<td>22</td>
</tr>
<tr>
<td>Price River Formation</td>
<td>2</td>
<td>27</td>
<td>0</td>
</tr>
<tr>
<td>Blackhawk Formation</td>
<td>0</td>
<td>25</td>
<td>3</td>
</tr>
<tr>
<td>Star Point Sandstone</td>
<td>0</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>Other (Jurassic and Alluvium)</td>
<td>0</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Total By Zone</td>
<td>139</td>
<td>257</td>
<td>30</td>
</tr>
<tr>
<td>Percent By Zone</td>
<td>33</td>
<td>60</td>
<td>7</td>
</tr>
</tbody>
</table>
Table 2. Acres of landslide by formation by stability zone for Manti Portion of Wasatch Plateau.

<table>
<thead>
<tr>
<th>FORMATION</th>
<th>STABILITY ZONE</th>
<th>Acres of Slides</th>
<th>Percent of Acres</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Flagstaff Limestone</td>
<td>51</td>
<td>22</td>
<td>135</td>
</tr>
<tr>
<td>North Horn Formation</td>
<td>2,178</td>
<td>2,468</td>
<td>123</td>
</tr>
<tr>
<td>Price River Formation</td>
<td>0</td>
<td>194</td>
<td>0</td>
</tr>
<tr>
<td>Blackhawk Formation</td>
<td>0</td>
<td>207</td>
<td>0</td>
</tr>
<tr>
<td>Star Point Sandstone</td>
<td>0</td>
<td>14</td>
<td>0</td>
</tr>
<tr>
<td>Jurassic Undevided</td>
<td>0</td>
<td>167</td>
<td>6</td>
</tr>
<tr>
<td>Total Acres By Zone</td>
<td>2,229</td>
<td>3,072</td>
<td>264</td>
</tr>
<tr>
<td>Percent Of Acres By Zone</td>
<td>40</td>
<td>55</td>
<td>5</td>
</tr>
</tbody>
</table>
Table 1: Numbers of landslides by formation by stability zone for Salina Canyon portion of Wasatch Plateau.

<table>
<thead>
<tr>
<th>FORMATION</th>
<th>Stability Zone</th>
<th>Number of Slides</th>
<th>Percent of Slides</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Flagstaff Limestone</td>
<td>2</td>
<td>13</td>
<td>3</td>
</tr>
<tr>
<td>North Horn Formation</td>
<td>94</td>
<td>120</td>
<td>17</td>
</tr>
<tr>
<td>Price River Formation</td>
<td>2</td>
<td>16</td>
<td>0</td>
</tr>
<tr>
<td>Black Hawk Formation</td>
<td>0</td>
<td>9</td>
<td>3</td>
</tr>
<tr>
<td>Alluvium</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total By Zone</td>
<td>98</td>
<td>158</td>
<td>23</td>
</tr>
<tr>
<td>Percent By Zone</td>
<td>35</td>
<td>57</td>
<td>8</td>
</tr>
</tbody>
</table>
Table 4. Numbers of landslides by formation by stability zone for Manti portion of Wasatch Plateau.

<table>
<thead>
<tr>
<th>FORMATION</th>
<th>STABILITY ZONE</th>
<th>NUMBER OF SLIDES</th>
<th>PERCENT OF SLIDES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Flagstaff Limestone</td>
<td>1</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>North Horn Formation</td>
<td>40</td>
<td>66</td>
<td>5</td>
</tr>
<tr>
<td>Price River Formation</td>
<td>0</td>
<td>11</td>
<td>0</td>
</tr>
<tr>
<td>Black Hawk Formation</td>
<td>0</td>
<td>16</td>
<td>0</td>
</tr>
<tr>
<td>Star Point Sandstone</td>
<td>0</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>Jurassic Undifferentiated</td>
<td>0</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Total By Zone</td>
<td>41</td>
<td>99</td>
<td>7</td>
</tr>
<tr>
<td>Percent By Zone</td>
<td>28</td>
<td>67</td>
<td>5</td>
</tr>
</tbody>
</table>
spring of 1983 a large percentage of the potentially unstable portion of the Wasatch Plateau was affected. A Manti or Thistle Canyon slide might be spectacularly large and attract a lot of attention, but the numerous small slides that occurred in the spring of 1983 affected a much larger area and accelerated the erosion of the Wasatch Plateau far beyond the effects of any single large slide. We must look at events as they affect the total erosion and movement of material from a geographic area.

The comparison of single vs. multiple events is probably best seen in Table 5. The Wasatch Plateau north of Salina Canyon is about 802,600 acres, of which 396,100 acres (49.1 percent) is underlain by the North Horn Formation. Individual slides in 1969 (Slide Lake), 1971 (Bulger Canyon), and 1976 (Manti Canyon) involved only 67, 10 and 300 acres, respectively. The total of all the slides in the North Horn Formation in 1983 involved 4,769 acres.

**TABLE 5** Comparison of slide magnitudes by year.

<table>
<thead>
<tr>
<th>Year</th>
<th>Acres Moved</th>
<th>Percentage of North Horn Formation</th>
<th>Percentage of Wasatch Plateau</th>
</tr>
</thead>
<tbody>
<tr>
<td>1969</td>
<td>67</td>
<td>0.017</td>
<td>0.008</td>
</tr>
<tr>
<td>1971</td>
<td>10</td>
<td>0.003</td>
<td>0.001</td>
</tr>
<tr>
<td>1976</td>
<td>300</td>
<td>0.076</td>
<td>0.037</td>
</tr>
<tr>
<td>1983</td>
<td>4769</td>
<td>1.204</td>
<td>0.594</td>
</tr>
</tbody>
</table>

While these statistics indicate the magnitudes of the differences between the 1983 event and other more "normal" years, there are insufficient data here to construct a magnitude-frequency curve. Unfortunately, many events in wildland management go undetected unless they impact some human activity. Until sufficient data are collected on the magnitude of various landslide events, other types of data will have to be used. In this regard, annual rainfall amounts appear a likely candidate and will be treated next.

**PRECIPITATION RECORDS**

Single, large magnitude precipitation events are frequently referred to as the trigger for large magnitude landslide events (see for example Endo, 1978; and Wolman and Gerson, 1978). In contrast, Erskine's 1973 studies of landslide activity in Cretaceous shales near the Fort Randall Reservoir, South Dakota, indicated that landslide activity correlated with a 12 month cumulative surplus of precipitation over evapotranspiration. He found that both the surplus and landslide activity were highest in the spring, and that both had maxima in 1952. The 1983 landslide event in Central Utah appears, like the Randall Reservoir case, to be the result of excess moisture caused by higher than average cumulative precipitation.

In the middle 1960's the U.S. Forest Service conducted a study on Sheep Creek, a tributary of Salina Creek, to determine the increase in runoff that
could be generated by removing the vegetative cover from a test watershed of 474 acres. The aspen and brush cover were converted to a grass cover by repeated sprayings in 1965 through 1967. DeGraff (1979) reported that within one year after the third year of treatment, landslide activity increased 300 percent, from six active landslides prior to treatment to 24 active landslides after treatment. He concluded that the increased slide activity was due to increased soil moisture, since the runoff increased 4 area inches. It is unlikely that the aspen roots would have decayed enough to diminish significantly their soil binding properties in the one year following the three years of treatment. This study inadvertently confirms the importance of water in initiating landslides on areas of the Wasatch Plateau underlain by the North Horn Formation.

What, then, is the periodicity of the cumulative precipitation event that triggered the 1983 landslide event? Precipitation records have been kept at Manti, Utah for over 80 years (NOAA, 1901-1983). While this weather station is in a valley, thus receiving less precipitation than the adjacent mountains, the length of records and the location west, or upwind, of the Wasatch Plateau are important. Further, the storms producing the snow pack and increasing the moisture content of the regolith, were general in nature. For example, 6.01 inches of precipitation were recorded at Manti in September, 1982. With the exception of the weather stations south and east of a line drawn roughly from St. George to Richfield to Moab, all weather stations in the state also recorded precipitation several times above average for that month (NOAA, 1983).

To construct the magnitude and frequency of the precipitation at Manti, I used the total annual precipitation reported by the Weather Bureau. This is a January 1 to December 31 total. In addition to the one year total, a two year distribution running total was also compiled. These two data sets were then computer processed through a Log Pearson Type III computer program to obtain the recurrence interval and exceedance probability. Thus the probability of exceedance plots shown in Figure 2 are based on a January through December data set. Most of the landslides, however, occur in the late spring. Thus these data sets do not accurately reflect the moisture conditions present at the time of the landslide activity. To rectify this, a new data set based on the "slide year," July 1 to June 30, was compiled for 1982 and 1983. These "slide year" data were then plotted on the probability of exceedance diagram (Figure 2).

Figure 2 indicates that the recurrence interval for the precipitation received during the 1983 slide years is about 140 years. The recurrence interval of the two-year total received through June, 1983 is about 125 years. The multi-year total was compiled because it is probably necessary to consider antecedent moisture conditions as a contributing factor to the slide event of 1983.

Precipitation data for mountain locations have been collected by the Soil Conservation Service (1979 and succeeding annual summaries). Data for stations on the west-draining slopes of the Wasatch Plateau, where over 70 percent of the 1983 landslides were located, and having at least 20 years of
Figure 2

Magnitude and Frequency of
One and Two Year Precipitation Totals at Manti, Utah

[Graph showing precipitation totals for one and two years with 1983 indicated on the graph.]
### Table 6. Return interval and exceedence probability for precipitation gages on the west side of the Wasatch Plateau, and in Manti, Utah. Same information also shown for total annual flow past stream gage on Salina Creek near Emery, Utah.

<table>
<thead>
<tr>
<th>STATION NAME</th>
<th>RETURN INTERVAL YEARS</th>
<th>EXCEEDENCE PROBABILITY IN PERCENT</th>
<th>YEARS OF RECORD</th>
<th>GAGE ELEVATION IN FEET</th>
</tr>
</thead>
<tbody>
<tr>
<td>MANTI</td>
<td>140</td>
<td>0.7</td>
<td>81</td>
<td>5,740</td>
</tr>
<tr>
<td>G.B.R.S. HEADQUARTERS</td>
<td>5,000</td>
<td>0.02</td>
<td>52</td>
<td>8,700</td>
</tr>
<tr>
<td>G.B.R.S. MEADOWS</td>
<td>60</td>
<td>1.6</td>
<td>42</td>
<td>10,000</td>
</tr>
<tr>
<td>G.B.R.S. OAKS</td>
<td>400</td>
<td>0.25</td>
<td>40</td>
<td>7,550</td>
</tr>
<tr>
<td>GOOSEBERRY R.S.</td>
<td>250</td>
<td>0.4</td>
<td>35</td>
<td>8,400</td>
</tr>
<tr>
<td>FARNSWORTH RES.</td>
<td>30</td>
<td>3.5</td>
<td>27</td>
<td>9,900</td>
</tr>
<tr>
<td>BEAVER DAMS</td>
<td>55</td>
<td>1.8</td>
<td>23</td>
<td>8,000</td>
</tr>
<tr>
<td>G.B.R.S. MAJORS</td>
<td>100</td>
<td>1.0</td>
<td>21</td>
<td>10,240</td>
</tr>
<tr>
<td>SALINA CREEK</td>
<td>200</td>
<td>0.5</td>
<td>20</td>
<td>8,720*</td>
</tr>
</tbody>
</table>

*Mean basin elevation from Blackmore and Others (1983).
record, were processed similarly to the Manti data. The results of this analysis are shown in Table 6 and Figure 3, along with the data from the Manti rain gage.

Also shown in Table 6 and Figure 3 is the recurrence interval of the total annual flow for the 1983 water year past the stream gages on Salina Creek near Emery (data from USGS, 1964-1983). This gage record was investigated for two reasons. First, it is the only gage on a stream, with no diversion, draining the west side of the plateau. Second, it is the closest currently-operating gage to the Sheep Creek vegetative manipulation project, mentioned above, where DeGraff (1979) noted that landslide activity increased 300 percent when vegetative manipulation increased runoff 4 area inches in 1968. Total flow past the Salina Creek stream gage was about five and a quarter area inches above average. This suggests that an increase of runoff of between 4 to 5 inches could be a threshold for initiating landslide activity.

Data in Table 6 suggest two things about the precipitation event. First, for the higher elevation stations, those above about 9500 feet, the 1983 precipitation has a lower recurrence interval than that of the lower elevation stations. This could result from generally higher precipitation at higher elevations, and suggests that the 1983 precipitation event was more of a low and mid elevation phenomenon than a high elevation one. Second, the weighted mean average of the low and mid elevation stations gives an exceedance probability of 0.5 percent in any one year. There is some scatter of the recurrence interval for mid and low elevation stations, but this should be expected when such relatively short observation periods are extended to such a low frequency event as 1983 precipitation.

The 200 year recurrence interval for the 1983 precipitation is about double the 94 year recurrence interval of rockfalls reported for the Southern Alps of New Zealand by Whitehouse and Griffiths (1983). Gardner (1980) also suggested about a 100 year frequency for rock avalanches in the Canadian Rocky Mountains. However, Whitehouse and Griffiths suggest that earthquakes dominate over precipitation as a triggering mechanism.

In summary, it appears that for the Wasatch Plateau of central Utah, total annual precipitation can be used to indicate the recurrence interval of landslide events. The main reason for this is the importance moisture content of the regolith plays in slope stability. This moisture content can be increased to above the stability threshold by either increasing the precipitation or by decreasing evapotranspiration.

CONCLUSIONS

The open system theory approach to the landslide event of 1983 on the Wasatch Plateau leads to three general conclusions. The first of these is that paleo slides often attributed to the wetter climate of the Pleistocene may in fact be much younger. Given the presence of Lake Bonneville adjacent to the study area, it seems reasonable to assume, as Gardner (1980, p. 284) did for rock slides in the Canadian Rockies, that landslides were more
Figure 3: Location of precipitation and stream gaging stations with recurrence interval of 1983 precipitation and runoff event.
frequent at the end of the Pleistocene and the beginning of the Holocene than they are at present. The point here is that lacking data from stratigraphic or absolute age dating techniques, a degree of uncertainty should be expressed in assigning time of movement of slides. The events of the current wet period have aptly demonstrated that Holocene climates are capable of producing large areas of landslide activity.

A second conclusion from this study, mentioned above, is that the processes of weathering and erosion that operate in dry cycles work to set the stage for the processes that operate during the wet cycles. During dry periods weathering works to disintegrate the bedrock and build a soil mantle on marginally stable slopes. As weathering progresses the amount of disturbance required to cross the instability threshold decreases. This disturbance can come either in the form of human induced interference, such as road cuts, or in the form of changed climatic conditions, such as a significant increase in precipitation. The system envisioned is similar to that depicted by Karcz (1980, p. 220-223 and Figure 4), where transient steady states are separated by successive instabilities as a threshold is crossed. In our case the transient steady state would be the period of soil formation during dry periods, while the successive periods of instability would be the wet periods of landsliding.

This generalization must be considered in the context of rather broad areas such as the outcrop area of the North Horn Formation on the Wasatch Plateau. Environmental conditions at specific locations can cause "spontaneous" instability to occur during dry periods in undisturbed locations. The Bulger Canyon slide is a good example of one of these random events.

A second point about the wet and dry cycle processes is that once instability of the type occurring in the North Horn Formation has been initiated, it can take several years before stable conditions return. Preliminary estimates from the Salina Canyon area indicate more and larger slides present in 1984 than were present in 1983. A possible reason for this is that, in addition to the already high soil and ground water levels carried over from 1983, the numerous open earth-fissures permitted a significantly higher than average percent of overland flow to enter the soil mantle. Thus the events of 1983 contributed to continued landslide activity in 1984. This process has the potential to continue for several years, even if precipitation returns to normal conditions.

The third conclusion is that the magnitude-frequency approach allows quantification of hazard probability. As pointed out in the introduction, the statistical methods used in analyzing open systems divorce observations from their spatial and temporal location. Thus predictions cannot be made for specific locations at specific times. However, the probability of a given event occurring in a given area in a specific time period can be determined. An example from hydrology is the 1 percent chance in any one year of the one-hundred year flood occurring on a floodplain.

While the probability figures given here must be considered highly speculative because of the meager data base, they do give an indication of
the order of magnitude of the risk involved for capital investments on the Wasatch Plateau. For point locations, such as campgrounds or homesites, placed on the North Horn Formation, the risk would be the percentage of area involved in sliding (1.2) times the probability of the event (0.5) or a 0.006 percent chance of damage in any given year. For linear features, such as roads, pipelines, and electric lines, the probability is significantly higher. In addition to occupying a larger area than most point locations, two other characteristics of most linear features work to increase the risk. The first characteristic is simply the soil disturbance required to construct these features. The second is that these features tend to run along the slope contour perpendicular to the direction of slide movement. Thus not just the area occupied by the feature, but the entire slope area, from ridge crest to valley bottom, that is crossed by the feature must be considered. Given these considerations, I would speculate that the risk of damage is about equivalent to the probability of the 1983 event, or about one-half of 1 percent.

REFERENCES


CALIFORNIA SOIL SLUMPS AND DEBRIS FLOWS

Robert A. Hollingsworth
G. S. Kovacs

ABSTRACT

Debris flows generated by soil slumps which occur during heavy rain storms present a greater risk of death and injury to residents of California than do all other types of slope failures combined. These failures also cause tens of millions of dollars of damage to public and private structures during years when extremely heavy rain storms occur.

Development in the hillside areas of California accelerated following World War II, and will continue in the future due to the decrease in available undeveloped flatland space and the desire of many people to live in the hillside areas. This development has resulted in structures being located on or below potentially hazardous natural slopes. During the lifetime of a hillside structure (50–100 years), several heavy rain storms are likely which could result in debris flows. U.S.G.S. Professional Paper 851, Soil Slips, Debris Flows, and Rainstorms in the Santa Monica Mountains and Vicinity, Southern California, by Russell H. Campbell (1975), presents a very good discussion of the causes and mechanics of the soil slips and resulting debris flows which occurred in Southern California mountains during the period of heavy rains from January 18 to January 26, 1969.

Campbell discusses the value of a warning system to advise residents, through the news media, when storm conditions have reached a point where debris flows are likely if high intensity rainfall continues. Also, he recommends the establishment of a system to evaluate the potential for debris flows occurring at individual sites. The purpose of this system is to provide residents with information concerning the potential hazard of remaining in their residences during heavy storms.

Robert A. Hollingsworth is a Project Geologist and Engineer, Kovacs-Byer and Associates, Inc., Studio City, California; and G. S. Kovacs is the Principal Engineer, Kovacs-Byer and Associates, Inc., Studio City, California.
At the present time, there has not been an attempt to implement either of the above systems. However, Hollingsworth and Kovacs (1981) have presented a procedure which allows the assessment of the potential for soil slumps occurring at individual sites. The procedure is based on readily measurable physical properties of the slope which enables widespread application.

The prediction method is based on the observation of over one hundred soil slumps and debris flows which occurred on natural slopes in the Santa Monica Mountains of Southern California during the period of heavy rains of February 13 to February 16, 1980. The method was tested in the San Francisco Bay Area of Northern California following the heavy rains of January 3 through January 5, 1982 to determine its applicability to other areas.

Information on the potential for soil slumps occurring can be used to guide recommendations for the placement of protective devices. Poured retaining walls, deflection walls, stem walls, debris basins, sidehill drains, and debris fences are the most commonly recommended protective devices.

MECHANICS OF SOIL SLUMPS AND DEBRIS FLOWS

Soil slumps and accompanying debris flows result from mass failures of the soil or colluvium overlying the bedrock on natural slopes. These failures occur during periods of intense rainfall. Campbell (1975) states, "the minimum conditions for failure would seem to be an initial period of enough rainfall to bring the full thickness of the soil mantle to field capacity (the moisture content at which, under gravity, water will flow out of the soil zone as fast as it flows in), followed by rainfall intense enough to exceed the infiltration rate of the parent (bedrock) material underlying the soil mantle"...

Once the rainfall intensity has exceeded the infiltration rate of the bedrock, a perched water table will begin to form in the soil zone. The effect of this perched water table on the shear strength of the soil may be illustrated by reference to Terzaghi's formula for resistance to shear (Lambe and Whitman, 1969).

\[ S = c + (p - hw) \tan \phi \]

\[ S = \text{Shearing resistance per unit area} \]
\[ c = \text{Cohesion per unit area} \]
\[ p = \text{Pressure due to the weight of solids and water} \]
\[ h = \text{Piezometric head} \]
\[ w = \text{Unit weight of water} \]
\[ \phi = \text{Angle of internal friction} \]

As the piezometric head increases, the friction component \((p - hw)\tan \phi\) decreases. The magnitude of the decrease can be
greater than one-half the dry strength. The cohesion which results from intergranular air-water surface tension is also reduced as water replaces air in the interstices. The combination of these results leads to a drastic reduction in the shear resistance.

The formation of a perched water table is aided by several surface conditions of the slope which serve to increase the infiltration rate. These include abundant shallow-rooted vegetation, a thin mulch of dead plant debris, animal burrows, and desiccation cracks in the soil (Campbell, 1975). The formation of a perched water table can also be aided by a concentration of drainage. Such a concentration occurs in gullies, swales, and other small depressions in topography into which sheet flow from the surrounding area is directed.

Once the shearing force exceeds the shear strength of the soil, a soil slump is initiated. Deformation of the mass changes the soil from a rigid body to a viscous fluid. As the material moves downslope, it flows together and moves down established drainage paths in relatively narrow streams. The speed of the debris flows depends on the viscosity of the debris, the gradient of the slope or channel, and the height of the slope.

The speed of debris flows varies considerably and can often be inferred from the damages. Damage caused by high velocity impact attests to the fact that many flows can move at avalanche speeds of up to 40 ft./sec. In other cases, damage occurs when a weak point in the structure fails due to lateral pressure from debris. In these cases, the flows ooze down the slope at speeds probably not much less than one ft./sec. (Campbell, 1975).

As stated in the foregoing discussion, the minimum conditions for failure are enough rainfall to bring the soil zone to field capacity, followed by rainfall sufficiently intense to exceed the percolation rate into the bedrock. The intense rainfall must last for a sufficient period of time to allow establishment of a perched water table. Campbell's evaluation of the storm of January 18-26, 1969, suggests that, in the greater Los Angeles area, these minimum conditions are reached when a site has a total of about 10 inches of rain followed by rainfall intensities of greater than 0.25 in./hr. These estimates were based upon data collected from a limited number of rainfall stations which may not be representative of the actual rainfall in areas where soil slumps occurred. These minimum conditions were, however, exceeded in the 1979-1980 storms in Southern California and were greatly exceeded in the 1981-1982 storms in Northern California.

PREDICTION METHOD

The 1981 study involved examining, in detail, approximately one hundred sites in the Santa Monica Mountains where soil slumps
and debris flows occurred on natural slopes during the heavy rains of February 13-16, 1980. Evaluation of each failure included the determination of soil type, bedrock type, dimensions of the failure, and the extent of any damage to structures caused by debris flow. The failure dimensions measured were the length, width, depth, scarp heights, and slope angles before and after failure. Representative samples of the soil involved in failures which occurred in each soil type and slope condition were collected and tested. Liquid limit, plastic limit, and partial wet sieve analysis were performed on the samples. These tests were performed in accordance with the ASTM standard procedures.

Based on the field observations and the soil test results, a method for assessing the potential for soil slumps occurring on natural slopes was developed. The method involves evaluating three factors for each site: the slope gradient factor, the concentration factor, and the soil-bedrock factor.

Slope Gradient Factor

The slope gradient is one important factor influencing the potential for failure with the steeper slopes being more susceptible to soil slumps. Table I assigns numerical values for slope gradient.

Concentration Factor

Every slope failure observed on natural slopes occurred in areas where the slope drainage became concentrated. Slope failures on man-made compacted fill slopes were not evaluated by this study. The concentration factor is used to evaluate the contribution of concentrated drainage to the potential for failures. Table 2 presents guidelines for assigning a concentration factor. The ordering of re-entrant gullies is intended to parallel the system of stream ordering. A first order stream is one which is not fed by any other stream, and a second order stream is fed by two or more first order streams (Strahler, 1969). Figure 1 illustrates the assignment of concentration factors to specific drainage plans. A slope with a concentration factor of 1 or 0 should be identified by field observation.

<table>
<thead>
<tr>
<th>Slope Gradient (Degrees)</th>
<th>Slope Gradient Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 15</td>
<td>0</td>
</tr>
<tr>
<td>15 - 25</td>
<td>1</td>
</tr>
<tr>
<td>26 - 32</td>
<td>2</td>
</tr>
<tr>
<td>33 - 40</td>
<td>3</td>
</tr>
<tr>
<td>41 - 63</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 1. Slope gradients and the corresponding slope gradient factor.
Table 2. Description of slope conditions and the corresponding concentration factor.

<table>
<thead>
<tr>
<th>Type of Drainage on Slope</th>
<th>Concentration Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Planar Slope</td>
<td>0</td>
</tr>
<tr>
<td>Gentle Swale</td>
<td>1</td>
</tr>
<tr>
<td>Swale</td>
<td>2</td>
</tr>
<tr>
<td>1st Order Re-Entrant Gully</td>
<td>3</td>
</tr>
<tr>
<td>2nd Order or Higher Re-Entrant Gully</td>
<td>4</td>
</tr>
</tbody>
</table>

Figure 1. The map is a portion of the Santa Monica Mountains Topo Sheet #38, Scale=1:3000. The dashed lines enclose different drainages. The circled numbers indicate the concentration factor for each drainage.

Soil-Bedrock Factor

The soil type present at each site is determined utilizing the Unified Soils Classification System. A partial sieve analysis consisting of determining the percentage of material passing the #40 and #200 sieves is performed on each sample. The liquid and plastic limit analyses are performed on the samples possessing a sufficient quantity of fine material to allow a reliable test.

Figure 3 shows the percent passing the #40 and the #200 sieves plotted versus slope angle for the Santa Monica Mountains samples tested. Figure 3 shows the variation in the boundary between failure and no failure for the Santa Monica Mountains samples and the San Francisco Bay Area samples. Based upon these graphs, the finer grained the soil the greater the range of slope angles at which failure is likely.
Figure 2. Percent soil passing the #40 and #200 sieves versus slope angle for the Santa Monica Mountains samples.

Figure 3. Illustrates the variation in the boundary between failure and no failure for the Santa Monica Mountains and the San Francisco Bay Area samples.

Figure 4 shows the liquid limit and plasticity index plotted versus the slope angle for the Santa Monica Mountains samples. The graph for the liquid limit again reveals that the finer grained the soil sample and hence the higher the liquid limit, the greater the range of slope angles at which failure is likely. The plasticity index did not prove to be a good indicator of the potential for failure.
Figure 4. The liquid limit and plasticity index versus slope angle for the Santa Monica Mountains samples.

The bedrock type is an important factor controlling the composition of the soil or colluvium. For the Santa Monica Mountain study, the bedrock types and formations likely to produce various soil types were identified. This was not done for the San Francisco Bay Area Study. Table 3 illustrates the assignment of the soil-bedrock factor for the Santa Monica Mountains samples. Table 4 illustrates the assignment of soil factors for the San Francisco Bay Area samples. Though the soil tests for a particular sample may not all indicate the same factor, by combining the results it should be possible to assign a reasonable factor.

The soil factor was revised slightly for the San Francisco Bay Area samples to provide better correlation with the test results. However, the table devised for the Santa Monica Mountains when applied to the San Francisco Bay Area samples produced satisfactory results.

Table 3. Soil test results and correlated bedrock types used to determine the soil-bedrock factor for the Santa Monica Mountains.

<table>
<thead>
<tr>
<th>Soil Bedrock Factor</th>
<th>Soil Type</th>
<th>% Passing #200</th>
<th>% Passing #40</th>
<th>Liquid Limit</th>
<th>Parent Bedrock Material</th>
<th>Bedrock Formation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SW</td>
<td>0-20</td>
<td>0-30</td>
<td>NA</td>
<td>Granite, Slate</td>
<td>Granite, Santa Monica Slate</td>
</tr>
<tr>
<td></td>
<td>GM</td>
<td></td>
<td></td>
<td></td>
<td>Granite, Slate</td>
<td>Santa Monica Slate</td>
</tr>
<tr>
<td></td>
<td>SM</td>
<td></td>
<td></td>
<td></td>
<td>Granite, Slate</td>
<td>Topanga, Slate</td>
</tr>
<tr>
<td>2</td>
<td>SM</td>
<td>20-35</td>
<td>50-70</td>
<td>40</td>
<td>Slate, Granite, Conglomerate Sandstone</td>
<td>Topanga, Slate</td>
</tr>
<tr>
<td></td>
<td>SC</td>
<td>35-55</td>
<td>70-85</td>
<td>40</td>
<td>Interbedded Sandstone and Shale</td>
<td>Topanga, Slate</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Shale, Mudstone, Shale</td>
<td>Topanga, Slate</td>
</tr>
<tr>
<td>4</td>
<td>MI</td>
<td>55-100</td>
<td>85-100</td>
<td>40</td>
<td>Silurian, Mudstone, Shale</td>
<td>Topanga, Slate</td>
</tr>
<tr>
<td></td>
<td>MH</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Topanga, Slate</td>
</tr>
<tr>
<td></td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Topanga, Slate</td>
</tr>
<tr>
<td>Soil Factor</td>
<td>% Passing #200</td>
<td>% Passing #40</td>
<td>Liquid Limit</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>------------</td>
<td>----------------</td>
<td>--------------</td>
<td>-------------</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0 - 20</td>
<td>0 - 40</td>
<td>NA</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>20 - 35</td>
<td>40 - 55</td>
<td>15 - 25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>35 - 50</td>
<td>55 - 70</td>
<td>25 - 35</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>50 - 100</td>
<td>70 - 100</td>
<td>35</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4. Soil test results used to determine the soil factor for the San Francisco Bay Area.

Potential for Failure

The potential for soil slumps occurring at individual sites is determined by summing the slope gradient factor, the concentration factor, and the soil-bedrock factor as shown on Table 5. Sites with a high or extreme rating should be closely evaluated during site development or the development of a warning system.

<table>
<thead>
<tr>
<th>Potential for Failure</th>
<th>Sum of The Three Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nil</td>
<td>1 - 3</td>
</tr>
<tr>
<td>Low</td>
<td>4 - 6</td>
</tr>
<tr>
<td>High</td>
<td>7 - 9</td>
</tr>
<tr>
<td>Extreme</td>
<td>10 - 12</td>
</tr>
</tbody>
</table>

Table 5. The potential for soil slumps occurring at individual sites based upon the sum of the three factors.

The prediction method described is intended to provide a practical method of determining the potential for soil slumps occurring on natural slopes. The method is based on easily measured slope characteristics and soil properties. The method is not intended to model the mechanics of soil slumps or debris flows.

RESULTS

The total of the factors for the majority of the Santa Monica Mountains failures was 7 to 12 or high to extreme. The total of the factors for the San Francisco Bay Area failures was 6 to 8, or on the boundary between the low and high potential rating. A few San Francisco Bay Area failures had a low rating and none of the failures had an extreme rating.

This difference was primarily the result of differences in topography and rainfall intensity between the two areas. The topography in the majority of the area observed in the San Francisco Bay Area was more mature than in the Santa Monica Mountains. The gentler, mature topography results in flatter slope gradients and lessens the contribution from concentrated drainage. These factors lead to a lower probability for failure. However, these...
factors were offset by the extremely heavy rains. Rainfall totals and intensities were approximately double those that Campbell suggested were necessary for initiation of failure. The extremely heavy rainfall was responsible for failures occurring on some sites with low potential.

The data obtained from the San Francisco Bay Area indicates that the prediction method is applicable to other areas. It should be noted that, with one exception, the San Francisco Bay Area sites with a low potential rating did not produce damaging debris flow. Therefore, when using this method, prime attention should be paid to those sites with high or extreme ratings.

PROTECTION METHODS

The prediction method allows the potential for soil slumps occurring at individual sites to be evaluated. This knowledge should be utilized not only in the development of a warning system, but also to guide recommendations for providing devices to protect proposed and existing structures from damage resulting from debris flows. The type, design, and placement of these devices should be guided not only by the potential for failures occurring but also by other physical characteristics of the slope, such as height and area of drainage. The following presents specific designs for protective devices: however, it should be noted that not every site can be safely developed even if protective devices are included in the design.

Retaining Walls

Retaining walls are the most commonly recommended device for protecting structures. Observations of both retaining walls which failed and those which performed well were made during this study. The following discussion on retaining wall design and construction is based on these observations. The causes of the retaining wall failures can be roughly divided into two groups: failures which result from inadequate construction methods, and failures which result from inadequate design.

Retaining walls constructed of concrete block are the most common type employed at this time. Observations of failures of this type of wall indicate that a preferable method of construction would be to utilize poured concrete walls rather than block walls. The advantages of poured concrete walls are: poured walls do not have planes of weakness such as exist between concrete block and mortar; proper placement of steel in poured walls to resist tensile stresses is more feasible than in block walls and the bond between the steel and the concrete is superior in poured walls. The second problem is that of providing reasonable design criteria for the portion of the retaining wall which is subject to forces resulting from debris flows. Observations and analyses
of failed walls indicate that the portion of a retaining wall which is subject to impact from a debris flow should be designed for an equivalent fluid pressure of 125 pounds per cubic foot.

**Deflection Wall**

A deflection wall is a retaining wall which is placed at an angle other than 90 degrees to the direction of the slope. The purpose of such a positioning is to decrease the perpendicular component of the impact force which acts to cause a failure of the wall and to increase the parallel component which acts to direct debris along the wall (Figure 5). Directing debris parallel to the wall will decrease the possibility of the wall being overtopped.

The deflection walls should be designed and constructed as other retaining walls. These deflection walls should be utilized wherever it is desired to direct debris around a structure. When deflection walls are utilized to create a level rear yard, the necessary amount of level rear yard area must be determined. The desired level area will vary depending on the slope conditions and must be evaluated for each site. However, at no point should the wall be closer than six feet to the structure, and the level area enclosed should be no less than 80 percent of the area which would have been created by placing a retaining wall so as to create a uniform 15-foot level rear yard setback.

Should the desired angle of the deflection walls be such that the area enclosed by the retaining walls is greater than the desired level area, an additional retaining wall may be constructed parallel to the residence to enclose the recommended level area. The remainder of the deflection walls which extend upslope from this parallel wall may be constructed as grade beams. The grade beams should extend two feet below the slope surface and be provided with three feet of freeboard and a paved "V" drain on the upslope side (Figure 6).

**Stem Walls**

If debris flows over the freeboard provided on the rear yard retaining walls and fills the rear yard, the debris which rests against the rear wall of the structure exerts a lateral load on this wall. This lateral load can result in the failure of a weak point of the wall, such as a window, thereby allowing debris to enter the structure. To reduce the possibility of this type of failure occurring, the footing or retaining wall which supports the rear wall of the structure can be extended three feet above the level rear yard. This extension should be designed for an equivalent fluid pressure of 90 pounds per cubic foot (Figure 7). Doors may still be provided along the rear wall of the structure; however, these doors should be provided with a moveable barrier.
Debris Basins

A commonly employed method of tract development has been to place residences on the ridge tops and in the canyon bottoms, leaving the remainder of the slopes in their natural condition. In the Santa Monica Mountains, numerous, small, steep re-entrant gullies and small canyons which trend transverse to the major canyons are present. The mouths of these canyons, though often narrow, pro-
vide a larger level area for building than the noses of adjacent ridges, thus, residences were often located in these areas.

The narrow mouths of these canyons are often ideally suited for small debris basins. Construction of these basins requires that the wall of the basin be positioned so that the ends of the wall may be founded in bedrock. Also, the catchment area of the basin should be sufficiently large to collect the maximum anticipated amount of debris for any one year. The amount of debris will vary with the size of the drainage area, the thickness of the regolith, and the steepness of the canyon and must be evaluated for each site. Access to the basins should be provided for cleanout.

Sidehill Drains

Sidehill drains are designed to be used in conjunction with other devices. The purpose of these drains is to control surface drainage and erosion on slopes. By controlling the surface drainage, these devices are intended to prevent the formation of surficial failures. These drains would likely fail if impacted by a high velocity debris flow and should not be used to control that type of failure. Figure 8 illustrates a typical sidehill drain design.

Debris Fences

Debris fences are designed to be utilized in conjunction with other devices to provide protection for structures. The typical debris fence design is shown in Figure 9. The purposes of these fences are to retard the rate at which debris flows down the slope, to catch a portion of the debris, and to break up the flowing mass, thereby allowing the escape of any air which is trapped under the flow. Such trapped air could serve to reduce the friction between the debris and the slope resulting in an increased rate of flow. Debris fences often fail during a debris flow; however, if the mode of failure is a bending of the fence rather than an intact movement of the fence downslope, the fence will generally still serve its purpose.

The placement of the debris fences has a tremendous effect on the ability of the fence to retard the rate of the flow. If possible, small debris fences should be constructed in the area of concentrated drainage. These fences will serve to collect the debris from a small slump near its point of inception and prevent the formation of a more extensive slump. Fences placed at the toe of a steep slope will probably fail due to the high speed at which the debris flow will be moving at this location. Debris fences should be examined periodically and cleaned or repaired as necessary.
REFERENCES:


475 LANDSLIDES IN THE CITY OF PACIFICA, CALIFORNIA

By Vincent N. Pascucci, A.M. ASCE

Heavy rains triggered over 475 landslides in the City of Pacifica, California in early January, 1982. A number of landslides occurred where dense residential development existed at the base of steep natural hillsides. The City retained our geotechnical services to respond to the emergency needs of the citizens. At many sites landslides had partially inundated homes with debris. One slide had completely destroyed several houses and buried three children.

It was necessary to identify potential landslide risk areas. Air photo interpretation, topographic maps and air reconnaissance were used to plot areas of potential high landslide risks onto orthophoto maps to develop temporary landslide hazard maps.

Most slope failures were shallow and occurred on natural topographic swales with slope angles ranging from 1:1 to 2:1 (horizontal to vertical). The failure prone colluvium and residual soil overlies various types of bedrock including Juro Cretaceous Franciscan greywacke, altered volcanic rock (greenstone) and Paleocene sandstone and shale (turbidite sequence). Colluvial and residual soils involved in failure were essentially cohesionless clayey, silty sand with a trace of gravel. Geometry of most of the slope failures was controlled by tension cracks and soil-bedrock contact. A majority of the slope failures were classified as complex landslides and resulted in debris/earth flows. Samples were collected from representative soil horizons and tested for soil properties and strength characteristics. Representative soils involved in debris/earth flows had cohesion values less than 400 psf (19 kPa) and average angle of internal friction of 35 degrees.

The engineering properties were utilized to back calculate the factor of safety against sliding before failure. Total saturation and reduced cohesion because of tension cracks was sufficient to cause a reduction in the factor of safety to within 10 percent of 1.0.

Vincent Pascucci is a Project Engineer with Howard F. Donley & Associates, Redwood City, California.
Initial mitigation measures consisted of constructing retaining walls, drainage systems, impact walls and revegetation. Future mitigation measures included timber cribwalls, baffles to retard soil flow, and containment walls.

INTRODUCTION

The City of Pacifica is located in northwestern San Mateo County, approximately 10 miles south of San Francisco, California as shown in Figure 1. Communities are built within the valleys and hillsides of the Santa Cruz Mountains.

In response to the emergency situation a method was developed to identify potential landslide risk areas. The results of
the initial study method were used to save lives and improve the community's emergency response to hillside failure. Following the emergency response nine of the slope failures were studied in detail to investigate the relationship between geology, soil engineering properties, mechanics and mode of failure.

The slope failures were shallow usually less than 10 ft. (3.0 m) deep and occurred on natural topographic swales with slope angles ranging from 1:1 to 2:1 (horizontal to vertical). Colluvial and residual soils involved in failure ranged in thickness from 18 in. (457 mm) to 12 ft. (3.7 m) and were essentially cohesionless clayey, silty sand with a trace of gravel. Fifty-eight percent (58%) of the soil samples tested from the nine slope failures contained 40 to 60 percent sand, 20 to 40 percent silt and 10 to 30 percent clay.

Geometry of most of the slope failures was controlled by tension cracks and the soil-bedrock contact. The landslides appeared to occur irrespective of bedrock type. The landslides were classified as complex that included rotational/translational sliding followed by debris/earth flows. The failure prone material was tested for engineering properties that were utilized to back calculate the factor of safety against sliding before failure.

Three of the nine slopes studied were repaired by constructing retaining and impact walls, installing drainage systems, regrading the slopes and revegetation. Future mitigation measures to reduce the hazards from some of the slopes failures include timber cribwalls, baffles to retard soil flow and containment walls.

GEOGRAPHIC SETTING AND CLIMATE

The City of Pacifica is located within the northern portion of the Santa Cruz and Montara Mountains that are characterized by interior valleys and highlands. These valleys originate at Sweeney Ridge, a prominent northwest trending topographic feature located approximately 3 mi. (4.8 km) inland. The drainage divides which separate the valleys are characteristically flat-topped with moderately dissected margins that contain numerous subtle linear swales and more well developed first-order, tributary drainages. Elevations range from sea level to approximately 1,200 ft. (366 m) above sea level.

The climate of Pacifica is characterized by dry, mild summers and moist, cool winters (Wagner, 1961). The average annual temperature is 50 degrees Fahrenheit. Average rainfall is 25 in. (635 mm) per year, most of which occurs during the months of January, February and March. Rainfall is accompanied by periods of high intensity averaging 0.20 to 0.25 in. (5 to 6 mm) per hour.
EMERGENCY RESPONSE

A major weather system dropped an estimated 6 to 8 in. (150 to 200 mm) of rain during a 30-hour period in Pacifica, California. Considerable antecedent precipitation had fallen in this area prior to January. Thus the stage had been set such that this particular storm triggered more than 475 slope failures within the City. A number of landslides occurred where dense residential development existed at the foot of steep, natural hillsides.

In the evening of Monday, January 4th, the community was faced with an emergency situation. Hillside residents began calling the City's Building Official for advice and evaluation of landslides. Our initial involvement began on January 12. This included responding to phone calls from citizens and making site inspections with the City's Building Official. At many sites landslides had partially inundated homes with debris and completely destroyed several houses and buried three children. After several inspections of the slope failures, it became apparent that natural slopes originally believed to be relatively safe had, in fact, failed in a manner heretofore unrecognized by the geotechnical profession. We realized that this was more than a simple matter of localized failures and that the problem involved potential hazards within the entire community. It became necessary to identify, within a very short period of time, potential landslide areas and identify residents that might be impacted by a slope failure. The emphasis was solely on life threatening perceptions as opposed to potential property damage.

Preparing Potential Landslide Hazard Maps

In order to identify potential landslide risk areas, U.S. Geological Survey (USGS) photos of the community were used to identify existing landslides. These photos had been taken within four days after the storm. Our professional staff also examined the community from the air and matched topographic features of existing landslide areas with other hillslope areas within the City. The air photos and topographic maps were used to plot areas of potential high landslide risks onto orthophoto maps with a scale of 1 in. equal to 400 ft. The City's address directory was used to identify addresses and names of residents below these potential high risk areas. These maps were to serve as temporary landslide hazard maps. Within two weeks of the January 4th storm, the City of Pacifica issued public notices to these residents advising them of the perceived hazard.

Public Notices

The public notices issued January 16, 1982 suggested that residents should evacuate their homes during heavy rains. It explained that additional rain of one-half inch or more could
trigger more slides. Each individual resident was asked to use his own judgment in making the decision as to when to evacuate. On May 5, 1982 a final public notice rescinded the January notice after a 30-day long-range weather forecast from the National Weather Service Forecast Office which stated that precipitation for the month of May was predicted to be below "normal" or less than one-half inch.

GEOLGY

Franciscan rocks are part of the California Coast Range Province that is characterized by particular kinds of rocks and structures. The bedrock present within the study area is the Jurassic-Cretaceous eugeosynclinal assemblage - the Franciscan rocks (Bailey, 1966). Folds, thrust faults, steep reverse faults, and strike-slip faults developed as a consequence of Cenozoic deformation, some of which is continuing today.

The formation is a disorderly assemblage of various characteristic rocks that have undergone unsystematic disturbance and lacks ordinary physical, spatial, and temporal coherence. The rocks include deep water sediments and mafic marine volcanic material, all of which are locally accompanied by masses of serpentine. Franciscan greenstone with bodies of sandstone, serpentine and limestone is predominant underneath the slope failures. Franciscan rocks are commonly weathered and form soil and colluvial horizons. The degree of weathering is known to be more pronounced in the more altered rocks; however, delineation of relative weathering in Franciscan terrain was not available in the literature.

GEOTECHNICAL ENVIRONMENT

A summary of the geotechnical environment is given for each slope failure in Table 1. Listed are the parent bedrock and description (thickness, color, soil type) of the colluvial and residual soil.

Bedrock Conditions

Franciscan greenstone is the predominant bedrock at the studied slope failures. The bedrock was usually intensely sheared, fractured, and weathered. Spacing of the fractured rock measured at Grand Teton, Terra Nova and Oddstad was between 1 to 4 in. (2.5 to 10.1 cm). The fractures exposed at Grand Teton were lined with light grey clay.

The predominant sheared and weathered greenstone at Grand Teton had inclusions of hard serpentine. The bedrock at Big Bend and Yosemite had inclusions of sandstone. These inclusions were termed as "knockers" or "float." Hard massive greenstone
TABLE 1  Summary of geotechnical environment

<table>
<thead>
<tr>
<th>Study Area</th>
<th>Parent Material</th>
<th>Residual Soil (3)</th>
<th>Colluvial Soil (4)</th>
<th>Topsoil (5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grand Teton</td>
<td>Serpentine &amp; Greenstone</td>
<td>2-feet Clay with Rock</td>
<td>5-feet Grey Gravelly Silty Sandy Clay to Clayey Silt with Gravel</td>
<td></td>
</tr>
<tr>
<td>Brookhaven</td>
<td>Greenstone</td>
<td>4-feet Yellow-brown Clayey Sand with Gravel and Rock Fragments</td>
<td>2-feet Greyish-brown Sandy Clay to Clayey Sand with Gravel</td>
<td></td>
</tr>
<tr>
<td>Big Bend</td>
<td>Greenstone</td>
<td>2-inches Brown Gravelly Sandy Clay</td>
<td>3-feet Brownish-yellow Clayey Gravelly Sand with Cobbles</td>
<td></td>
</tr>
<tr>
<td>Moana</td>
<td>Greenstone</td>
<td>6-feet Dark Brown Clayey Sand</td>
<td>4-feet Dark Grey Silty Clay</td>
<td></td>
</tr>
<tr>
<td>Highway 1</td>
<td>Sandstone, Shale, Serpentine &amp; Greenstone</td>
<td>2-feet Dark Brown Clayey Sand</td>
<td>3-feet Dark Grey Silty Clay</td>
<td></td>
</tr>
<tr>
<td>Woodlawn</td>
<td>Greenstone</td>
<td>3-feet Dark Grey Silty Clayey Sand with Gravel to Gravelly Silty Sandy Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Terra Nova</td>
<td>Greenstone &amp; Limestone</td>
<td>1-foot Brown Sandy Silty Clay</td>
<td>1 to 2-feet Greyish to Grey Brown Clayey Silty Sand</td>
<td></td>
</tr>
<tr>
<td>Yosemite</td>
<td>Greenstone &amp; Sandstone</td>
<td>2-feet Clayey Sand</td>
<td>3-feet Greyish-brown to Black Silty Clayey Sand</td>
<td></td>
</tr>
<tr>
<td>Oddstad</td>
<td>Greywacke</td>
<td>11-feet Dark Brown Clayey Silty Sand with Gravel</td>
<td>2-feet Dark Grey to Greyish-brown Clayey Sand</td>
<td></td>
</tr>
</tbody>
</table>

"knockers" were sometimes found in the weathered greenstone bedrock.

The bedrock condition at Highway 1 road cut consisted of a mixture of fractured and sheared sandstone, shale and serpentine with inclusions of hard, subrounded greenstone knockers. This
appeared to be the only studied slope failure with a mixture of bedrock. The bedrock at Oddstad consisted of Franciscan greywacke and was only exposed downslope from the source area.

**Soil Conditions**

Soil cover usually consisted of either residual, colluvial or topsoil. Residual soil found at seven study areas varied in thickness from 2 in. (10.1 cm) to 6 ft. (1.2 m). The soil gradation varied between clayey sand to sandy clay. Gravel and rock fragments were part of the soil mixture at Grand Teton, Brookhaven and Big Bend.

Colluvial soil seemed to be the dominate soil unit in the study areas. The thickness of the colluvium layer was generally between 2 to 5 ft. (0.6 to 1.5 m) with the exception of 11 ft. (3.3 m) at Oddstad. Gradation of the soil varied at the seven slope failures where colluvium soil was found. The soil was usually a mixture of sand, silt and clay with occasional gravel.

Topsoil was only found at five of the nine study areas. The thickness of the topsoil varied from 1 to 3 ft. (0.3 to 0.9 m). The topsoil usually consisted of a mixture of sand, silt and clay.

**TOPOMORPHOLOGY**

Slope angle plays an important role in the distribution of flows as shown by Figure 2. The nine slope failures mapped in detail all lie within a slope angle range of 1:1 to 2:1. This slope angle range is suggested by Campbell (1975) as being most common for debris/earth flow occurrence. Shallower slopes also contained flows but invariably their toes coincided with an artificial or natural steepening of slope (e.g., Moana, Highway 1 Road Cut and Terra Nova High School).

The slope characteristics were generally similar. The preponderance of flows occurred near the head of first order drainages (swales) sometimes represented only by a subtle inflection in the slope which, in transverse profile, showed less than 3 ft. (0.9 m) of relief. The nine studied slope failures consisted of the source area and main track. The source area was usually U-shaped and varied in length from 20 to 400 ft. The track was longer than the source area. The length of the track was about three to twenty times the width. The slope failures could be depicted as long and narrow chutes of flowing debris and mud.

**LANDSLIDE CLASSIFICATION**

The types of landslides that occurred in the nine slope failures can be classified as the following: (1) slide-debris flow; (2) slump-earth flow; and (3) solifluction. The first two
are classified as complex landslides (Varnes, 1978), and occurred at all the slope failures, except Grand Teton. Type 1 landslide occurred at Brookhaven, Yosemite and Oddstad. Debris/earth flows or debris avalanches are thought to have been initiated by translational or rotational sliding that subsequently disaggregated into a flowing mass of soil and water. The flows/debris avalanches occurred on both natural slopes and on oversteepened cut slopes underlain by soil (e.g., Highway 1 and Moana slope failures).

A solifluction landslide was mapped at Grand Teton. This type of failure was peculiar in that failure resulted in a rippled ground surface without distinct boundaries. This type of failure can be considered a creep process that is capable of generating a debris flow or a rotational type of slide as movement proceeds to locally oversteepen the slope.

**LANDSLIDE GEOMETRY**

Tension cracks appeared to have controlled the configuration of the source areas. The source areas were U-shaped and varied in size from 540 ft² (50 m²) to 9000 ft² (836 m²) for Highway 1 and Oddstad slope failures, respectively. Grand Teton had a source
area of 13,000 ft² (1208 m²) although the source area did not become a flow.

Debris flow source areas clearly exposed the geometry of the basal rupture surface. The rupture surface occurred at or within 2 ft. (0.6 m) of the soil-bedrock contact and was often subparallel to the adjacent natural ground surface. Bedrock "windows" were commonly exposed on the floor of the source area and springs were observed discharging from fractured rock or soil.

Rupture surface profiles were very irregular, some having topographic relief of as much as 2 ft. (0.6 m). Locally, steep reaches of the rupture surface were coincident with rootlined tension cracks exposed in the adjacent flanks of the source area. This relationship may represent headscarps of individual landslide blocks that failed retrogressively and probably simultaneously.

SOIL CHARACTERISTICS

Soils encountered were either formed in place from the underlying bedrock (residual soil) or were gravity transported from an upslope source (colluvial). Source areas generally exposed less than 2 ft. (0.6 m) of medium plastic topsoil underlain by homogeneous, granular soil. Engineering properties of the respective landslide soils were comparable as shown in Table 2. Figure 3 illustrates the grain size distribution of soils involved in the nine slope failures. In general, excluding the extreme data points, the representative soils involved in debris/earth flows contained 30 to 80 percent sand, 20 to 60 percent silt and 10 to 45 percent clay. This compares well with values of the shear strength parameters where cohesion was commonly less than 400 psf (19.1 kPa) and the average angle of internal friction was 35 degrees. Table 3 lists the ranges of the soil properties for topsoil, colluvial and residual soils. The values shown include data from the nine study areas.

Hydrometer readings of the minus 200 portion of the soils shows a higher percentage of silt than clay. The clay fraction appears to be higher for residual soil compared to colluvium and topsoil. The average percentage of silt (33%) was similar for both colluvium and residual soil. Average percentage of clay was 15 percent and 16 percent for colluvial and residual soil, respectively.

SLOPE STABILITY ANALYSES

An effective stress analysis for each of the nine slope failures was made utilizing the method of slices for an infinite slope with a pre-failure geometry as illustrated in Figure 4. Shearing resistance was evaluated by applying the equation developed by Terzaghi (1950):
### TABLE 2 Engineering properties of soil samples

<table>
<thead>
<tr>
<th>Sample depth in feet (2)</th>
<th>Dry density in pounds per cubic foot (3)</th>
<th>Moisture content (4)</th>
<th>Atterberg limits (5)</th>
<th>Gravel size at a percentage (6)</th>
<th>Sand size at a percentage (7)</th>
<th>Clay size at a percentage (8)</th>
<th>Grain size distribution (9)</th>
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</thead>
<tbody>
<tr>
<td>6.2</td>
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<td>15</td>
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<td>50</td>
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<tr>
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<td>19.0</td>
<td>41</td>
<td>2</td>
<td>25</td>
<td>100</td>
<td>15</td>
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<td>108</td>
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<tr>
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<td>331</td>
<td>7</td>
<td>53</td>
<td>40</td>
<td>20</td>
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<td>102</td>
<td>19</td>
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<td>32.9</td>
<td>28.4</td>
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<tr>
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<td>32.9</td>
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<td>7</td>
<td>53</td>
<td>40</td>
<td>20</td>
</tr>
</tbody>
</table>

**Note:**
- 1 ft = 0.305 m
- 1 cfs = 16.38 km
- 1 kbf = 4448 kg

**FIGURE 3** Grain size distribution
TABLE 3  Properties of overburden soil

<table>
<thead>
<tr>
<th>Engineering Property</th>
<th>Residual</th>
<th>Colluvium</th>
<th>Topsoil</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range (2)</td>
<td>Average (3)</td>
<td>Range (4)</td>
</tr>
<tr>
<td>Silt</td>
<td>16-40</td>
<td>33</td>
<td>25-40</td>
</tr>
<tr>
<td>Clay</td>
<td>2-48</td>
<td>16</td>
<td>6-28</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>10-31</td>
<td>19</td>
<td>5-20</td>
</tr>
<tr>
<td>Cohesion in pounds per square foot (kPa)</td>
<td>0-600 (0-24)</td>
<td>256</td>
<td>70-500 (2-24)</td>
</tr>
<tr>
<td>Angle of Internal Friction</td>
<td>35</td>
<td>28-50</td>
<td>38</td>
</tr>
</tbody>
</table>

\[ S = LC (P - hw) \tan \phi \]  \hspace{1cm} (1)

Parameters used in Equation 1 are for saturated soil samples. The water table is at the ground surface and the failure surface is along the bedrock-soil contact.

The Factor of Safety (FS) was calculated for each of the nine slope failures from the equation:

\[ FS = \frac{S}{W \sin \theta} \]  \hspace{1cm} (2)

in which \( S \) = shearing resistance from Equation 1; and \( W \sin \theta \) = component of total weight, oriented parallel to the rupture surface.

In some cases the stability analysis illustrated that the buoyancy effect from total saturation was not enough to cause landsliding. However, lowering the cohesion value by assuming a tension crack located at the headscarp and extending from the ground surface to the basal rupture surface was sufficient to cause a reduction in FS to within 10 percent of 1.0, a value representative of failure under the given conditions and method of analysis.

FIGURE 4  Slope stability analyses
MECHANICS AND MODE OF FAILURE

Intense rainfall during the January storm was the triggering mechanism for the slides. Campbell (1975) suggests that at least 10.5 inches (267 mm) of antecedent rainfall is necessary to bring the soil to field capacity (water draining from soil at same rate as it is added) following an average dry season. Campbell illustrates that a critical rainfall intensity of 0.20 to 0.25 inches (5 to 6 mm) per hour is necessary to trigger flows once field capacity is satisfied.

Kesseli (1943) postulates that slope stability imbalance occurs during intense rainfall if water percolates into overburden soils at a rate greater than percolation into the subject bedrock. This condition develops a temporary, perched water table in the soils. The height of the water table increases with continued rainfall until the entire prism of overburden soil is totally saturated. After saturation occurs, all excess rainfall is distributed as surface runoff and a downslope seepage force develops in the soil mass. In response to pore pressures that develop, the soil tends to float, effectively decreasing the forces resisting movement.

This model postulated by Kesseli and applied to the mechanics of debris flows in Southern California by Campbell (1975) suggests that debris flow are initiated by sliding. This seems to be a reasonable model to apply to the mechanism of debris/earth flows in Pacifica, based upon rupture surface geometry, the presence of remnant slide blocks exposed in the source area of many slope failures and the results of the stability analyses.

Once a saturated, granular soil mass becomes mobilized by sliding on a hillside, it commonly disaggregates into a debris/earth flow or sometimes a debris avalanche. The first stage occurs when FS = 1.0 and the slide mass begins sliding along or just above the soil-bedrock contact. High pore pressure caused from permeability differences in soil and bedrock and tension cracks are the primary controls. As the slide mass moves, plastic deformation in the formerly rigid mass occurs as internal shearing causes an instantaneous reduction of strength (Campbell, 1975). Further disaggregation reduces the strength of the slide block initiating flowage of soil and water over the original ground surface. As the flow moves downslope it accelerates. If the slide breaks into separate blocks, one surge may override another as the debris descends a slope or separate surges may occur.

MITIGATION MEASURES

Three slope failures (Grand Teton, Brookhaven, Big Bend) were repaired by the City of Pacifica. The Grand Teton slope was repaired by constructing a retaining wall at the top of the slope.
and removing the colluvium/residual soil down to bedrock on the slope below the wall. A drainage system and an impact wall was constructed at the base of the slope. The slope was revegetated to prevent erosion and "slipouts."

The Brookhaven landslide was repaired by cutting the slope to bedrock and placing a combined impact and retaining wall at the toe of the slope. The angle of the cut slope was about 1.3:1. A drainage system consisting of a concrete V-ditch was constructed along the top.

A drainage system consisting of surface and subsurface drains was constructed down the slope at the Big Bend landslide. Future mitigation measures include a timber railroad tie cribwall at the face of the scarp. This cribwall serves as a retaining wall to control landslide failure. A second phase consists of installing sets of "dywidag" bar baffles for retarding soil flow. Each group of bars have chicken wire-mesh wrapped around three bars. A steel fence system would be constructed further downslope to also retard the soil flow. Finally an impact/containment wall would be constructed at the base of the slope.

These impact walls are designed for static loads in excess of 125 pounds per cubic foot as is normally done in the Los Angeles area. Upslope material and type of potential debris should be considered when designing these impact walls. The wall is designed to fail at a controlled point to relieve the excess pressure built up behind the wall.

Several other mitigation measures were presented to the City of Pacifica to protect downslope property and prevent future slides. (1) Deflection walls are used to withstand the impact of slide debris and divert the debris away from endangered structures. (2) Surface swales collect runoff to lessen the amount of water entering the ground. Three points should be considered when using surface swales, deterioration of the swale, maintenance and blockage. (3) Erosion control prevents the start of gullying which could concentrate water to unstable areas. (4) Subsurface drainage system helps to prevent the development of seepage forces. (5) Gunite applied to the slope prevents infiltration of water. Considerations for this method include applying the gunite to the top of the slope, all vegetation on the slope must be removed, soil creep, weathering and maintenance. (6) Slope modification can include such things as grading to buttress fills at the toe.

ACKNOWLEDGMENT

A number of individuals, firms, and organizations cooperated with and assisted HOWARD-DONLEY ASSOCIATES, INC. in this investigation. Appreciation is expressed to Joel Baldwin, Terry Howard and Howard Donley for providing initial documentation.
Special thanks are extended to the City of Pacifica for their support throughout the project.

REFERENCES


Identification and Runout of Debris Flows

IDENTIFICATION OF DEBRIS FLOW AND DEBRIS FLOOD POTENTIAL ALONG THE WASATCH FRONT BETWEEN SALT LAKE CITY AND WILLARD, UTAH

by Gerald F. Wieczorek, Stephen Ellen, Elliott W. Lips, Susan H. Cannon and Dan N. Short

ABSTRACT

In late May and early June of 1983, rapid melting of an exceptionally heavy snowpack in the Wasatch Range triggered numerous debris flows from hillsides, some of which traveled down the main stream channels and beyond the mouths of the canyons; others, diluted by extremely high runoff in the channels contributed large quantities of debris to flooding. Reconnaissance along the Wasatch Front in mid-June revealed many hillsides with freshly-developed scarps and cracks having small offset. These areas of incipient, partly-detached landsliding could mobilize into debris flows from future episodes of rapid melting of snowpack similar to that which occurred in the spring of 1983 or from intense rainfall during summer convective storms. Evidence of historic and prehistoric debris flows were used in concert with an empirical model of debris-flow runout from partly-detached landslides to develop a technique for rating the potential for debris flow and debris flood from canyons along the Wasatch Front between Salt Lake City and Willard, Utah.

We evaluated the potential for a debris flow from a partly-detached landslide to reach the canyon mouth by estimating its potential travel distance through comparison to debris flows that reached canyon mouths in this area during the spring of 1983. Observation and theoretical considerations indicate that for channels of more-or-less similar cross-section, and for materials of similar properties, the ability of a given debris flow to sustain movement depends upon its volume and the gradient of the channel. We used the estimated volume (15,500 m$^3$) of the main debris-flow scar in Ward Canyon as a standard of comparison for major-size canyons and used a volume of 3,600 m$^3$ of the debris-flow scar in Hornet Creek as a standard for smaller canyons that are locally called half-canyons. In drainages with volumes of partly-detached landslides exceeding these standards, the potential for debris flow reaching beyond the canyon mouth was rated as very high. Where volumes of partly-detached landslides were less than these standards, the potential for debris flood was rated very high because of the likely contribution of large quantities of sediment to the bedload of the stream during periods of flooding.

Gerald F. Wieczorek, Stephen Ellen, Elliott W. Lips, and Susan H. Cannon are with U.S. Geological Survey and Dan N. Short is with Los Angeles County Flood Control District.
Mapping of prehistoric debris flow and alluvial fan deposits as well as documentation of debris flows and debris floods in this area has existed long before the conditions brought by rapid snowmelt in the spring of 1983. Where more than one historic or prehistoric debris flow was recognized beyond the canyon mouth, the drainage was rated as having a high potential for recurrent debris flow. In a similar manner we evaluated potential for recurrent debris flood where alluvial fans at canyon mouths suggest a succession of past debris floods. We assigned relative potential for debris flow as very high, high, moderate or low and for debris flood as very high, high or low in 22 major canyons and numerous half-canyons, between Salt Lake City and Willard.

Based on the experience of the Los Angeles County Flood Control District with hydrologic design of debris basins and with design of other mitigative measures we made recommendations for the areas beyond the mouths of canyons for those drainages with a very high potential for either debris flow or debris flood. As a consequence of these recommendations, the Federal Emergency Management Agency, in cooperation with the state and local governments (as of November 1983) has constructed a debris basin below Rudd Canyon, has funded three other debris basins and has discussed as many as 16 other potential debris basins.
IDENTIFICATION AND RUNOUT OF DEBRIS FLOWS

FACTORS INFLUENCING DEBRIS-FLOW RUNOUT

by Elliott W. Lips, Gerald F. Wieczorek, and H. Brad Boschetto

ABSTRACT

Of the thousands of debris flows that occurred in the spring of 1983 in Utah some traveled only a few tens of meters down the hillsides, while others traveled several kilometers before coming to rest on alluvial plains beyond the mouths of canyons. Since the identification of runout distance is necessary in evaluating the potential hazard of a debris flow, an explanation for this wide range of distances is essential for the evaluation. Previous investigators have identified at least four primary factors that effect the distance a debris flow will travel: 1) volume of material in the flow, 2) gradient of the path down which the flow travels, 3) geometry of the channel in which the flow is confined, and 4) composition of the material making up the flow; however, no method exists for relating these factors to runout distance.

We have developed a method of quantifying the relationships between those parameters that were observed to have the greatest effect on debris-flow runout. This method is based on data collected during detailed investigations of eight debris flow deposits located in Sanpete County (Birch Springs and Crooked Creek near Fountain Green, two unnamed creeks near Lower Gooseberry Reservoir, South Fork of North Creek near Mount Pleasant, and Little Clear Creek near Indianola), in Utah County (Pole Canyon near Santaquin) and in Davis County (Ward Canyon above Bountiful). At these sites we took measurements and made calculations for volume of material from the source area, gradients and cross-sections of the channel, width and depth of deposit and length traveled. Samples were taken along the flow path and analyzed for grain size. While at these sites we also mapped distinctive features of the flows on enlargements of recent aerial photographs which we later used for control on stereo pairs in the PG-2 plotter to produce topographic maps of the debris-flow sites. Analyzing these data we developed an empirical model that considers the length of runout to be a simple function of several easily measurable variables. Although we view this model as valid for the sites we investigated, we realize the need to test it on a larger statistical sample before widespread application. By determining the volume of material likely to mobilize, gradient and geometry of the channel, and grain size of the material, the length of debris-flow runout could be determined for other canyons.

Description and Behavior of Debris Flow

RUDD CREEK DEBRIS FLOW

by Bruce C. Vandre

ABSTRACT

By studying the past, we hope to learn for the future. However, we need to recognize that past natural events may significantly change the conditions affecting future events.

During the spring of 1983, a debris flow initiated in the upper portions of Rudd Creek and its effects terminated at an elevation approximately 2400 feet below in Farmington, Utah, causing extensive property damage. Photogrammetric maps of the channel area, before and after the debris flow, were prepared and compared for geotechnical evaluation. The map changes were interpreted in the light of site inspections, geotechnical concepts, and published knowledge regarding debris flows.

This paper presents the geotechnical study findings and interpretations, discusses the site changes which may affect future events, attempts to delineate debris flow hazards in terms of landslide occurrence, channel conditions, and flood plain conditions. Other landslide and flood plain debris flow events are compared to the Rudd Creek event.

INTRODUCTION

A newspaper headline may read as follows: "Arson Causes $1 Million Damage." From a technical viewpoint, this statement is an over-simplification relative to cause and consequence. Precisely, an arsonist started a fire; however, other factors including building composition, building condition, and fire department response time contributed to the damage amount. The initiator of events is frequently the focus of attention. The importance of the event is generally determined by the consequences. A $1000 damage amount from a fire would not be expected to be a newspaper headline.

Landslides, debris flows, and debris floods involve a chronology of conditions and consequences as indicated by Table 1. The combination of several

1 Bruce C. Vandre is a Geotechnical Engineer with the Intermountain Region, USDA Forest Service.
conditions can be hazardous, such as topography, geology, and hydrology. Consequences from one event can also be hazardous relative to a subsequent event. Landslides can be hazardous for flood plain debris flows or debris flows. Debris flows or floods can be hazardous for property values or public safety. However, not all landslide debris flows or debris floods are hazardous. Additional conditions may need to exist to create hazards. During the springs of 1983 and 1984, the number of major damaging flood plain debris events compared to the number of landslide occurrences in Davis County, Utah, was less than 5 percent. Therefore, conditions in addition to landslides need to be delineated relative to flood plain debris flows and debris floods.

**TABLE 1 Hazard Chronology**

<table>
<thead>
<tr>
<th>Hazard Conditions</th>
<th>Hazard Consequences</th>
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</thead>
<tbody>
<tr>
<td>Topography</td>
<td>Landslides</td>
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<tr>
<td>Geology</td>
<td>Debris Flows</td>
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<tr>
<td>Surface Water Supply</td>
<td>Debris Floods</td>
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<td>Landslides</td>
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<td>Channel Gradient</td>
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<td>Streamflow Volumes</td>
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<td>Floodplain Gradient</td>
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<tr>
<td>Debris Flows</td>
<td>Property Damage</td>
</tr>
<tr>
<td>Debris Floods</td>
<td>Personal Injury</td>
</tr>
</tbody>
</table>

Since the consequences and causes of debris flows and debris floods can be different, safety problems may result if they are lumped together or considered to be synonymous.

Although both springs, 1983 and 1984, had large volumes of snowmelt for many days, most landslide activity occurred within a few days. Delineating only heavy snowpack as a landslide hazard appears to be an oversimplification.

Physical conditions after landslides and debris floods or flows can change. These changed conditions can result in changed criteria for hazard delineation. For example, conditions initiating landslides may differ from those needed to reactivate landslides or cause flood plain damage.

Shortly after the Rudd Creek debris flow event, the USDA Forest Service performed a geotechnical assessment of conditions (Vandre, 1983). Photogrammetric maps of the channel area, before and after the debris flow, were compared. The map changes were interpreted considering field observations, geotechnical concepts, and published information regarding debris flows. The Rudd Creek debris flow locations were also visually inspected from the ground and from a helicopter during the springs of 1983 and 1984.
This paper attempts to use the Rudd Creek experience to delineate debris flow hazards for the Wasatch Front in Davis County, Utah. Other 1983 and 1984 snowmelt events within the vicinity have also been considered. The conditions examined in this paper include: past history, landslide occurrence, drainage channel flow conditions, and flood plain conditions. Attempts are made to distinguish debris flood hazards from debris flow hazards.

PAST HISTORY

The recent landslide activity does not appear to be associated with pre-existing landslide activity. Approximately one hundred landslide locations were mapped on aerial photos along the Wasatch Front in Davis County prior to 1983 (Olson, 1981). Approximately 70 1983 landslide locations were inventoried during helicopter surveillance (Winkelaar, 1983). Less than 5 percent of the recent landslides occurred at locations previously mapped. Only a few of the 1983 landslides were noticeably reactivated during 1984.

Uniformitarianism is a working concept in geology. Events in the past, present, and future are assumed to be similar. Much of the usefulness of hazard delineation depends upon the ability to predict the timing and magnitude of events. Geologic history does not provide this perspective. An ancient debris flow deposit appears to occur at the mouth of Rudd Creek; however, the conditions of initiation and deposition are not subject to interpretation. It is not known whether this deposit was associated with rain, snowmelt, or dry weather.

Historical experience can be more useful for prediction purposes. Events can sometimes be correlated with conditions. For example, the recent Rudd Creek debris flow, which is now history, can be associated with rapid, abundant snowmelt and landsliding activity. However, the historical records in Utah report that most debris flows and debris floods in the Rudd Creek vicinity were associated with thunderstorms and erosion. The recent Rudd Creek event is somewhat anomalous with respect to this record.

Current observations indicate debris flow deposits can be built up from several events. The initial event in 1983 deposited approximately 80 percent of the material reaching Farmington. Approximately 20 percent of the material deposited in Farmington during 1983 and 1984 was associated with several small flows following the initial event. The building up of deposits indicates that geologic maps delineating boundaries of debris flow deposits can be misleading with respect to perceptions of magnitude of events. The aerial extent of debris flow deposits can be mistakenly associated with with magnitude of debris flow deposits.

LANDSLIDE OCCURRENCE

Photogrammetric Evaluations

The Rudd Creek debris flow that reached Farmington in 1983 appears to have been initiated by a landslide occurring at an elevation approximately
2400 feet above the flood plain. Figure 1 indicates estimates of landslide volumes relative to elevation. These estimates were taken from photogrammetric maps before (USDA Forest Service, 1983a) and after (USDA Forest Service, 1983b) the 1983 event. Field observations of debris locations along the flanks indicate that an initial slope movement of 6,000-7,000 cubic yards occurred.

FIGURE 1 Rudd Creek Landslide Volume Estimates

Geologic Setting

Both geology and topography are lumped together in the following discussion. The Rudd Creek landslide activity occurred in the vicinity of a physiographic boundary, as portrayed by Figure 2. The Weber Valley erosion surface has been described by Eardley, occurring along the Wasatch Front in Davis County at an elevation above the Rudd Creek slope break (Eardley, 1944). This surface was reportedly uplifted by block faulting.

Most of the other landsliding activity in Davis County in 1983 and 1984 also appears to occur within an elevation range in close proximity to the terminus of Eardley’s erosion surface. Landslide frequency relative to 200-foot contour intervals are plotted on Figure 3. The elevations in this figure were estimated during helicopter surveillance and are in close agreement with subsequent aerial photo inventories (Winkelaar, 1983). The elevations of head areas of perennial drainages have also been plotted on this figure. Most landslide activity occurs within the elevation range of 6800-7400 feet, which also is the elevation range at which frequency of drainage heads noticeably increases. Above elevations 7600 feet, drainage
head elevations can be correlated with watershed divides. The increase in frequency of drainage heads at elevation 6800 feet could be explained by groundwater discharge. A zone of groundwater discharge can be explained by an uplifted piedmont boundary.
Slope breaks are frequently associated with conditions creating groundwater barriers or confinement, such as shallow bedrock, resulting in springs. A bench area provides infiltration opportunities for surface water and can be a recharge location for groundwater development. Many of the other 1983 and 1984 landslides, including East Layton and Upper Rudd Creek, also have occurred below slope breaks (Vandre, 1983b).

Table 2 provides some insight as to water supply needs for the development of adverse groundwater conditions relative to slope stability. For a continuous downward wetting front to develop from infiltration, the surface water supply needs to equal the soil permeability (Lomb, 1975). The first column on the table indicates the permeability coefficient range common to the landslide soil classification units (GM-GC). The second column converts permeability coefficient units to snowmelt infiltration units. The third column indicates the time required for a soil layer to become saturated from the ground surface to a 5-foot depth. Assumptions used in making these calculations included the soil was 90 percent saturated prior to infiltration and had a porosity of .5. If a barrier to vertical flow occurs at this depth, groundwater pressures may develop in a downslope direction. Inspection of this table indicates a need for water ponding to allow the development of surface saturation for significant depths. If there is much runoff, typically occurring snowmelt water supply rates are not adequate for the more permeable soils ($10^{-5}$ and $10^{-4}$). The duration of water supply (88 days) needed for saturation of the less permeable soils is also not common.

<table>
<thead>
<tr>
<th>SOIL PERMEABILITY (CN/SEC)</th>
<th>WATER SUPPLY (IN/DAY)</th>
<th>DAYS FOR SATURATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>$10^{-5}$</td>
<td>3</td>
<td>8.8</td>
</tr>
<tr>
<td>$10^{-4}$</td>
<td>3.6</td>
<td>8.9</td>
</tr>
<tr>
<td>$10^{-3}$</td>
<td>10.4</td>
<td>8.1</td>
</tr>
</tbody>
</table>

Groundwater occurrence in steep slope and shallow soil cover environments is frequently intermittent, temporary, and responsive to infiltration events. The concept of long term buildup and rises in water table may not apply.

The groundwater observed discharging from the Rudd Creek landslide scarps appears to be shallow, localized, and perched. Springs are discharging from the ground surface upslope from the flank scarp. The depth of discharge at the flank scarp locations is estimated to be 6-8 feet below the ground surface. The depth of discharge for the head scarp area is approximately 20 feet below the pre-existing bench ground surface. The groundwater appears to be discharging from weathered rock or in the vicinity of a rock-colluvium interface.
The removal of material from the Rudd Creek landslide is expected to reduce groundwater confinement and improve groundwater drainage. According to local residents, streamflow volumes during the summers of 1983 and 1984 appear to be significantly greater than past recollections. Groundwater drainage can rapidly reduce excess pore pressures, as experienced during the installation of horizontal drains at other locations (Vandre, 1975). Ground cracking adjacent to and upslope from the head scarp area is believed to be a reaction to a common groundwater condition, and not in a response to ground movement or removal of downslope support. If the groundwater regime is connected throughout the cracked area, movement can result in improved drainage and reduce overall groundwater pore pressures at distant locations. The 1984 crack monitoring activities indicated that movement significantly decreased after material sloughed from the flank scarp. The flank scarp is located 300-400 feet downslope from the crack area (McCarter, 1984). Possibly, the changed condition of improved groundwater discharge can reduce landslide reactivation hazards at some locations.

1983 and 1984 Water Supply

Snowmelt was the apparent water source driving much of the landslide activity. During 1983, the initiation of landslide activity appeared to be associated with the location of the snowline. Figure 4 indicates equivalent inches of water of snowmelt per day for May 21 to June 5 for both 1983 and 1984. The left bar applies to 1983 and the right bar applies to 1984. The total equivalent inches of water from snowmelt for the graphed period of time in 1983 was approximately 20 inches, as compared to 22 inches in 1984. The values plotted on these figures are based upon daily snow telemetry data from upper Farmington Canyon at an elevation of 8000 feet (USDA Soil Conservation Service, 1984). The snow telemetry data was adjusted (+1/2 inch) assuming a temperature 6 degrees warmer at the Rudd Creek site and using typical snowmelt coefficients calculated for the Farmington site. Snow course data for a 20-year period indicates that the snowpack generally melts within a 1-month period at an average daily rate of .7 inches (USDA, Soil Conservation Service, 1974). The available water from snowmelt from May 26-30, 1983, was approximately 2 to 2-1/2 inches each day. The Rudd Creek debris flow initiated May 30, 1983. Reactivation of the Rudd Creek landslide did not occur during this time period in 1984, although water supply was higher.

The East Layton debris flow occurred May 14, 1984. The daily snowmelt at this landslide elevation on May 12 and 13 is estimated to have averaged 2-1/2 inches for both days. The location of the snowline during the East Layton landslide debris flow is not known. The south-facing slope setting suggests the snowline could have been in the vicinity of the landslide location on May 13.

Water supply quantities are plotted on Figure 4 for a 24 hour period because lower evening or freezing temperatures could interrupt water supply beyond this time period. Both the East Layton and Rudd Creek debris flows were preceded by 5 to 6 days of significant snowmelt supplying a total accumulation of 9 to 11 inches of water.
Empirical relationships developed by others indicate approximately 4 inches of rainfall for a 24 hour period would be a threshold value for rainfall-initiated shallow landslides and debris flows (Caine, 1980). The lower snowmelt threshold values for water supply in Davis County could be explained by the lower runoff conditions associated with topographic benches.

McCarter monitored significant increases in fracture separation at Rudd Creek starting May 15, 1984, although no debris flow activity occurred. The debris flow activity in 1984 associated with the scarp sloughing, previously mentioned, occurred 8 days later.

CHANNEL CONDITIONS (ELEVATION 6700-4500 FEET)

Photogrammetric Evaluations

Debris flow depth, channel gradients, and scour depths were evaluated by comparing September 1980 (USDA Forest Service, 1980a) and June 1983 (USDA Forest Service, 1983c) aerial photos and mapped 100-foot contour intervals. Photogrammetric estimates are as follows:

1. The average channel bottom elevation was lowered approximately 8 feet after the debris flow.

2. 16,000 to 24,000 cubic yards of material is estimated to have been removed from the channel bottom.

3. Maximum scour depths approached 15 to 20 feet.
4. Bedrock was exposed by scour at most locations at elevations above 4700 feet.

5. Debris could be observed to override and be deposited on the bank of the inside curves at some locations.

6. The average channel gradient is 36 percent.

7. Minimum or zero elevation changes of channel bottom occurred at the locations adjacent to bank landslides or tributaries.

8. The maximum debris flow depths were approximately 20 feet in the vicinity of the landslide (30-35 percent channel gradient) and 8 feet (20 percent channel gradient) near the flood plain.

The flow depth-gradient observations are anomalous to standard hydraulic relationships. Accordingly, greater flow depths would be associated with flatter gradients. Possible explanations for this anomaly include higher concentrations of solids near the landslides or higher water content and mixing near the discharge point of the channel. The channel debris flow depth just below elevation 5700 is approximately equal to the estimated depth of the landslide, suggesting initially a possible plug type flow.

Limiting Equilibrium

Using a two-dimensional infinite slope model and assuming seepage parallel to the slope gradient, most granular stream bed material would be expected to slide when slope gradients exceed 35 percent. Many existing stream channel reaches have gradients in excess of 40 to 50 percent, including Rudd Creek. The stability of the material in these channels subjected to water flow can be explained by frictional resistance along the sides of the bed material. Two-dimensional analysis only considers frictional resistance along the bottom.

The initiation of scour of the bed material can be evaluated in terms of limiting equilibrium of an infinite slope. Forces acting on a three-dimensional rectangular section of bed material include the gravity driving forces, a drag force caused by the debris flow over the bed materials, frictional resistance along the bottom and frictional resistance on the sides of the bed material. According to this model, safety with respect to channel materials sliding increases with increases in depth of channel material and decreases with increased channel width.

Critical debris flow depths for varying channel gradients are plotted on Figure 5. Calculation assumptions include a 5-foot depth of bed material, a 20-foot wide channel, a drag coefficient of .5, and a material friction coefficient of .8. This figure indicates that channel scour can be initiated on gradients steeper than 30 percent by minimum debris flow depths. The debris flow drag forces tend to offset the frictional resistance to sliding on the sides of the channel material at this gradient.
Intuitively, streamflow volumes are expected to determine if landslide debris will continue moving in drainage channels and in what manner. The capacity of streams to transport sediment is expected to increase as the water flow volume increases. The difference between debris floods and debris flows is the ratio of solids to liquid. Debris floods have comparatively higher water contents and, therefore, would be expected to be associated with high stream flows. Debris flows have relatively lower water contents and, therefore, would be expected to be associated with lower stream flows. A critical relationship between streamflow volume and landslide volume is expected to exist relative to the occurrence of floodplain debris flows. The concentration of solids can be diluted so that the debris flow movement mechanism no longer exists.

On Figure 6, stream flows are plotted relative to landslide volumes and stream gradients. Accordingly, streamflow values above the gradient lines could have adequate water volumes to dilute landslide volumes, so that debris will not reach the flood plain in the form of a debris flow. The theory of this model is that water volume available for dilution can be estimated by dividing the travel distance between the landslide source and the flood plain by the difference in flow velocity between the stream and the landslide, and then multiplying this quotient by the stream flow. An estimated high velocity difference tends to be conservative with respect to estimating minimum stream flows for dilution purposes. By dividing this water volume by landslide volume, a dilution factor can be calculated. A critical dilution factor was calculated for Stone Creek which involved a 19,000 cubic yard landslide almost being transported by 120 cfs streamflow to the flood plain. Stone Creek’s dilution factor was then used to calibrate calculations used to
develop Figure 6. Other values used in developing this figure included a travel distance between elevations 7000 feet and 4500 feet as a function of stream gradient and an assumed difference of flow velocity of 16 feet per second. The debris flow hazards delineated by this figure are low streamflow volumes relative to stream gradients and landslide volumes. The quantities on this figure are believed to have ballpark significance only. These relationships will not be expected to apply for conditions in which streambed material would be unstable. During 1983 and 1984, streams with relatively flat gradients tended to have flows exceeding those plotted. Most of the streams having relatively steep gradients were observed to have flows less than those plotted.

**FIGURE 6** Critical Streamflow Volumes for Debris Flow Dilution

![Diagram showing critical streamflow volumes](image)

Although Figure 6 indicates low stream flows could be a hazard relative to flood plain debris flows, high stream flows can be a hazard relative to debris floods. It is not only the quantity of debris that distinguishes debris floods from typical sediment transport during floods, it is also the transport of boulders. Figure 7 indicates the minimum flow volumes in a 20-foot wide channel needed to initiate movement of a 2-foot diameter boulder. Research results regarding dam overtopping were used to calculate the threshold flows plotted on this figure (Oliver, 1967). According to this figure, flow volumes above the curve could initiate movement of a 2-foot diameter boulder. Most snowmelt runoff streamflows in Davis County during 1983 were considerably below the curve. The 1983 Stone Creek-Bountiful debris flood was apparently caused by blockage of a drainage structure which resulted in a water impoundment. Flow volumes associated with the breach of this blockage have been estimated from high water marks to approach 3,300 cfs (Lindskov, 1984).
The debris flood hazard delineated by this figure is high streamflows associated with cloudbursts or stream blockages.

**FLOOD PLAIN (BELOW ELEVATION 4500 FEET)**

Photogrammetric Evaluations

Comparison of photogrammetric maps of the flood plain before (Davis County, 1982) and after the Rudd Creek debris flow indicated the following (USDA Forest Service, 1983d):

1. Approximately 65,000 to 75,000 cubic yards of material was deposited as a debris fan in Farmington.

2. An additional 7,000 to 14,000 cubic yards of debris was deposited approximately 1/2 to 1 foot thick below the toe area of the fan.

3. Deposition occurred on gradients of 10 percent or flatter.

4. The runout distance on the flood plain (10 percent or less gradient) is approximately 35 to 45 percent of the elevation distance between the head of the slide (elevation 6900 feet) and the point of discharge onto the debris fan (elevation 4500 feet).

The runout distance and elevation difference (potential energy) relationship is approximately the same for a higher elevation debris flow in Rudd Creek and the East Layton debris flow, as indicated on Table 3. This empirical relationship appears to have some potential for prediction purposes. This relationship is believed to be limited to debris having a similar nature such as a CH-CC soil classification and a runout angle of 6-7 degrees.
TABLE 3 Runout Distance on a 10 Percent or Flatter Gradient Related to Elevation Drop of Debris Flow

<table>
<thead>
<tr>
<th>DEBRIS FLOW</th>
<th>CHANNEL GRADIENT</th>
<th>RUNOUT DISTANCE ELEVATION DROP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Rudd</td>
<td>.28</td>
<td>.42</td>
</tr>
<tr>
<td>Flood Plain</td>
<td>.36</td>
<td>.36</td>
</tr>
<tr>
<td>Rudd</td>
<td></td>
<td></td>
</tr>
<tr>
<td>East Layton</td>
<td>.32</td>
<td>.36</td>
</tr>
</tbody>
</table>

By subtracting the estimated landslide volumes above elevation 6700 feet (19,000 cubic yards) and the estimated channel bottom erosion volumes (16-24,000 cubic yards) from the estimated volume of debris deposited in Farmington (72-89,000 cubic yards), the origin of 37-46,000 cubic yards of materials remain unaccounted for. This volume of material apparently originated from the erosion and undercutting of the channel banks.

A debris basin was constructed at the mouth of Rudd Creek prior to the spring of 1984. Approximately 12-14,000 cubic yards of debris were deposited in the basin during 1984 (Maxwell, 1984). Approximately 600 cubic yards of the volume came from the Rudd Creek landslide scarp (McCarter, 1984). The remainder of the material apparently came from the channel area below.

SUMMARY AND CONCLUSIONS

Landslides may have been the "arsonist" during the springs of 1983 and 1984. The preceding discussion attempted to focus on additional conditions which can affect the occurrence of flood plain debris flows or floods.

Focusing on one or two links in a causation chain has been described as an oversimplification. However, simple does not always have a negative connotation. One of the critical elements in good geotechnical practice is the ability to recognize and understand relationships. Simple quantitative relationships can provide perspective regarding parameter sensitivity and, if we are lucky, perspective regarding threshold values. When it is unlikely that we will achieve complete understanding, it is better to be approximately correct rather than precisely wrong.

Simple calculations can provide insight regarding:

1. The significance of bench topography for groundwater development.
2. Drainage channel material additions to landslide debris flows.
3. Streamflow dilution of landslide debris.
4. Threshold water flow volumes moving boulders.
Specific hazards and consequences can be delineated as follows for the Wasatch Front, Davis County:

**Hazard (Condition)**

- Heavy snowpack with snow line between elevations 6700-7400 when 24 hour average temperature (maximum and minimum) in Salt Lake City is above 70 degrees.
- Streamflows exceeding 500-1000 year recurrence interval or stream blockage resulting in similar flows.
- Flood plain locations within 1000 feet radius of flood plain apex below drainage channels with gradients steeper than 25 to 30% with landslides occurring near the channel.

**Consequence**

- Landslide initiation
- Flood plain debris flow consequences
- Flood plain debris flow consequences

When the occurrence of landsliding activity is as frequent as 1983 and 1984, opportunities for exceptions to generalizations, experience, and logic are expected to exist.

Estimates of magnitudes of debris flow events appear to be highly speculative. Recent experience indicates the volumes of deposits on the flood plain may be controlled more by the channel conditions than landslide volumes.

Landslide movements can be considered to have a purpose. One purpose is to provide groundwater pressure relief. The 1983 and 1984 landslide occurrences probably have reduced groundwater confinement at these locations. A reduction in potential for groundwater pressure development is expected to reduce or change the hazards associated with landslide reactivation.

**REFERENCES**


Davis County (1982). Planning Map No. 18, scale 1" = 200', contour interval 2' and 5', based on April 1982 photography.


USDA Forest Service (1983a). Rudd Creek Canyon before debris flows, scale 1" = 200', contour interval 10', based on September 1980 photography at 1:6,000 scale, Geometronics, Intermountain Region, Ogden, Utah.

USDA Forest Service (1983b). Landslide Area, Rudd Creek, scale 1" = 50', contour interval 2', based on June 21, 1983, photography at 1:6,000 scale, Geometronics, Intermountain Region, Ogden, Utah.

USDA Forest Service (1983c). Rudd Creek Canyon after debris flows, scale 1" = 200', contour interval 10' with 5' supplementals, based on June 21, 1983, photography at 1:6,000 scale, Geometronics, Intermountain Region, Ogden, Utah.


EFFECTS OF SLURRY COMPOSITION ON DEBRIS FLOW DYNAMICS, RUDD CANYON, UTAH

by Thomas C. Pierson

ABSTRACT

Surface velocity, horizontal velocity distribution, flow depth, and slurry composition (sediment concentration and particle-size distribution) were measured for a surging, channelized debris flow on June 5, 1983, at a site in lower Rudd Canyon in Farmington, Utah. Peak discharge (24 cubic meters per second), peak depth (2.2 meters), and peak sediment concentration (in excess of 88 percent by weight) occurred at the head of the flow, but peak velocity did not. The peak surface velocity of 4.5 meters per second occurred in the middle of the flow, after sediment concentration and mean particle size had decreased. The flow became progressively more dilute behind the head, making the transition from debris flow to hyperconcentrated streamflow between 70 and 75 percent solids by weight. This transition was marked by the loss of competence to suspend gravel and by the onset of significant turbulence and channel downcutting.

Coarse particles (cobbles and boulders), present only at the head of the flow, played a key role in flow behavior. When mixed in with the finer grained slurry, the large particles increased shear strength by a factor of 10 and caused a rigid plug to form in the flow. When concentrated in the steep-fronted leading edge of the main surge, the interlocking clasts acted as a dam. This bouldery front provided sufficient additional resistance to intermittently impede the flow, causing ponding of slurry behind it.

The expected dependence of fluid velocity on flow depth was significantly complicated by variations in flow resistance due to changes in sediment concentration and by backwater effects of the boulder front. For a given depth (slope and channel roughness held nearly constant), debris flows (at around 80 percent sediment by weight) moved at a higher velocity than more dilute flows. This is interpreted to be due to a decrease in flow resistance caused by dampened turbulence.

Thomas C. Pierson is a geologist with the U.S. Geological Survey (Water Resources Division) at the Cascades Volcano Observatory in Vancouver, Washington.
INTRODUCTION

A series of debris flows, occurring over a week of rapid spring snowmelt in 1983, caused severe damage to homes and property situated on the debris fan at the mouth of Rudd Canyon in Farmington, Utah (Fig. 1). Slope failures, triggered by high pore-water pressures in the colluvial hillslope soil, initiated the flows, which traveled 2.9 km to the canyon mouth. Deposits from these flows totaled approximately 80,000 m³ and covered 7.2 ha; roughly 81 to 84 percent of this material was scoured from the channel and incorporated into the flows (Wieczoerek and others, 1983).

Debris flow is here defined as flowage of a coherent, single-phase mixture of poorly sorted sediment and water, whereby the water and the silt-clay fraction form a pore fluid that becomes trapped within the framework of coarser grains. In other words, the sediment itself is flowing, transporting the water. Such a mixture possesses internal shear strength and can be modeled as a Bingham plastic fluid (Yano and Daido, 1965; Johnson, 1970). If too much water is added to the mixture, the pore fluid is able to escape, and a mixture with two independent phases evolves—muddy water and coarser particles. In this case, the water is the flowing medium, transporting the sediment, and it is termed hyperconcentrated streamflow (Beverage and Culbertson, 1964). These authors proposed that this transition occurs at a sediment concentration of about 80 percent by weight (wt percent).

The third largest debris flow to come out of the small, steep Rudd Creek watershed in 1983 occurred at 10:30 a.m. on June 5 and had a peak discharge of 24 m³/s. Samples of the flowing slurry were collected, and flow depth, surface velocity, and the horizontal velocity distribution were recorded intermittently over the duration of the flow. The debris flow arrived at the canyon mouth in four surges. Sediment concentration was highest at the bouldery flow front of the second surge and decreased during flow recession.

The objectives of the study were to obtain debris-flow velocity and velocity-distribution data in this field setting, in order to correlate these data with the potential controlling variables: depth of flow, particle-size distribution, and sediment concentration. This was undertaken as part of a broader study to define rheologic characteristics of channelized debris flows from a variety of field settings.

METHODS

Data collection at Rudd Creek utilized time-lapse photography, 16-mm motion-picture photography, and hand sampling of the passing flow with wide-mouth plastic jars. Photographs were taken looking down across the channel from a bank 11 m above the channel bottom (Fig. 2), 55 m upstream from the head of the debris fan (canyon mouth). Samples were collected at the canyon mouth.

Sequences of still photographs were taken using a 35-mm camera with motor drive, which could shoot at the rate of 3.1 frames per second. The camera also had an hour-minute-second digital data back that imprinted times.
FIGURE 1 Location of study area on west flank of Wasatch Range in Farmington, Utah.
FIGURE 2  Study site in lower Rudd Creek Canyon. Channel cross section at camera position (upper left), plan view (middle), and longitudinal profile (lower right). Channel boundaries shown are high mudline positions from a previous debris flow.
on the individual frames. This allowed correlation of flow dynamics with sampling times. The camera was handheld by the operator and rapid-fire sequences of 4 to 6 frames were taken at intervals of 15 seconds to several minutes as the flow passed below. Distances and angles between the camera and channel were later measured and additional photographs were made with a survey rod for scale in the channel. After corrections were made for the oblique angles, distances traveled between frames were scaled for individual clasts or twigs suspended in the debris slurry. This allowed measurement of surface velocity at different points across the flow. Horizontal velocity distribution could then be plotted. Velocity errors by this method were estimated to be on the order of 10 to 15 percent for this flow, but they could be improved by more precise measurements.

Motion-picture photography was used to record the general nature and appearance of the flow, including turbulence at the surface and the apparent viscosity of the mixture (informal, ordinal ranking). The camera was mounted on a tripod and activated by the operator. When the film was exposed, a new roll was loaded, and filming continued for the duration of the flow.

Material properties of the slurries were obtained from samples collected intermittently by reaching out from the edge of the flow with the container and dipping into the moving flow. The containers were wide-mouth plastic jars, having a 3.6-L capacity. Jar openings were 95 mm in diameter. This procedure biased the sampling against clasts larger than about 9 cm in diameter, but except for the bouldery flow front and peak flow, the coarsest particles were finer than this. Filled jars were labeled with time collected (referenced to the time at which that part of the flow passed the photo site) and sealed in the field with screw lids and tape. Samples were then returned to the lab for sediment concentration analysis (by weight), sieving, pipette analysis, and estimation of shear strength and competence. Statistical grain-size parameters (Folk and Ward, 1957) were computed. The less inclusive graphic sorting \( G_c \) and graphic skewness \( S_k_c \) were used because high clay contents prevented accurate assessment of the fine tails of the distributions.

Shear strength was computed for debris-flow slurries in the field and in the laboratory, using equations provided by Johnson (1970, 1984). These methods contain the simplifying assumption that the slurry can be modeled at field scale as a homogeneous plastic substance.

In the field, strength was estimated for fine grained "matrix" slurry that spilled out of the channel during peak flow (overbank deposits of relatively uniform thickness) by application of the critical thickness formula:

\[
k = \frac{T_c Y_d \sin \delta}{h}
\]  

where \( h \) is the plastic yield strength in \( \text{dyn/cm}^2 \), \( T_c \) is the thickness of the slurry lobe, assumed to be at critical thickness, \( Y_d \) is the unit weight of the slurry, and \( \delta \) is the inclination of the surface on which the lobe is resting.
Shear strength can also be estimated from the relative width of any rigid plug that forms on the flow surface. The equation for a semi-elliptical channel is:

$$k = \frac{(W/2) \gamma_d \sin \beta}{(W/2D)^2 + 1}$$

where \(W_p\) is the width of the plug, \(W\) is the width of the channel, and \(D\) is depth of flow.

In the laboratory, shear strength was estimated for a number of different sediment concentrations using a sediment sample collected from the debris-flow phase of the June 5 flow. Starting with an air-dry sample in a container, water was incrementally added to the sediment while mixing was done by hand. After each increment was added and mixed in, the container was weighed to determine sediment concentration, then nearly spherical pebbles of different sizes were carefully placed on the surface of the debris mixture (breaking the surface tension). The depth to which the pebbles sank into the mixture and remained was measured. These values were then applied to the equation:

$$k = 0.219 \ h \ (\gamma_h - n \ \gamma_d)$$

where \(h\) is the diameter of the clast, \(\gamma_h\) is the unit weight of the clast, \(n\) is the volume fraction of the clast submerged in the debris, and \(\gamma_d\) is the unit weight of the slurry.

Suspension competence of this mixture was also estimated by simply noting the largest size class held in suspension at each concentration. The general appearance of the mixture and its apparent fluidity were noted and comparisons were made with the field photographs, in order to estimate sediment concentration during the flow when no samples were taken.

RESULTS

The June 5 debris flow arrived at the study site in four surges that occurred within 15 minutes of each other. Discharge returned to near normal rates only minutes after each surge, but dilution back to normal sediment concentrations took much longer (hours). Variations in stage, surface velocity, and sediment concentration with time are summarized in Fig. 3. Flow appeared to be laminar while sediment concentrations (\(C_s\)) stayed in the upper debris-flow range (>80 wt percent), but turbulence began to appear when concentrations dropped below 80 wt percent. Reynold's numbers could not be computed because viscosity was not measured. Slurry temperature measured from samples shortly after collection was 12°C.

Flow Characteristics

Prior to the arrival of the debris flow, streamflow in the channel was about 15 cm deep and discharge was on the order of 0.1 m³/s. The debris
FIGURE 3 Variation in stage, thalweg surface velocity, sediment concentration by weight, and silt and clay content with time for June 5 (10:30 a.m.) debris flow, measured at study site.
Flow arrived as a small preliminary surge (A, Fig. 3) of viscous, fine-grained slurry about 30 cm deep and supporting a few small pebbles.

About a minute later the main surge (B) arrived (Fig. 4a). Peak discharge was 24 m$^3$/s, computed from velocity and cross-sectional information at the observation site (Fig. 2). It had a steep boulder front about 2 m high and 4 m long, which acted as a kind of moving dam holding back the flow, owing to the frictional resistance encountered by the front in the channel. Boulders in this frontal plug averaged 30 to 40 cm in diameter and were interlocking. Spaces between the boulders were only partly filled with matrix material, and although unsampled, the sediment concentration was estimated to have been well over 90 wt percent. Without the interstitial fluid, intergranular friction within the boulder plug and sliding friction between the plug and the channel bed were high. Stage peaked behind the front and then began to quickly recede. Velocity peaked with the main surge at just under 3 m/s and then decreased for about 30 seconds to 1.5 m/s as sediment concentration decreased to about 88 wt percent. Velocity picked up again as sediment concentration decreased further to about 82 wt percent.

A minor surge (C) of only 40 cm depth occurred 3.5 minutes after the main surge (Fig. 4b). This surge, however, did not have a bouldery front, and although it was not sampled, it did not appear from the movies that sediment concentration changed. This surge front moved rapidly down channel (4.3 m/s), at times turbulently, and it overtook the slower flow in the channel (1.3 m/s). Flow velocity had decreased immediately prior to the arrival of this surge, possibly in response to damming of the flow upstream. Flow velocities remained over 3 m/s following this surge as sediment concentration continued to decrease. One additional minor surge (D) occurred 12 to 13 minutes after the main surge, but its front was not recorded.

At sediment concentrations above 80 wt percent, flow appeared to be completely laminar; the slurry had a distinct viscous appearance and was competent to carry up to cobble- and boulder-size clasts in suspension (Figs. 4, 5a). The debris-flow surface was gently undulatory due to the formation of faint arcuate compressional wrinkles and ridges in the shape of downstream-pointing "U"s on the flow surface. As sediment concentration fell below 80 wt percent, these compressional features disappeared and minor turbulence began developing around obstructions, such as large boulders in the streambed (Fig. 5b). At concentrations of 75 to 76 percent, some pebbles were still in suspension and the flowing sediment-water mixture was still a debris flow, but surface turbulence was increasing and oblique standing waves began to develop. Also at this point, the flow surface began to take on a distinct shiny, wet appearance due to the expulsion of pore water to the surface. Pore-water expulsion marks the beginning of the breakdown of the coherent debris-flow slurry.

At concentrations between about 75 and 70 wt percent, flow was transitional between typical debris flow and streamflows. Turbulence was beginning to develop over the whole flow surface at approximately 74 to 75 wt percent, with vigorous splashing in some places and with only very fine pebbles in suspension. At about 71 to 73 wt percent, turbulence was well developed over
A. Bouldery front of the main debris-flow surge (B). Bulk sediment concentration behind boulder front estimated to be about 90 percent by weight. Front velocity is 1.3 m/s.

B. Turbulent front of debris-flow surge C. Note the lack of concentrated coarse clasts. Sediment concentration is about 61-82 wt percent. Front velocity is 4.3 m/s.

FIGURE 4 Types of flow fronts on surges of the June 5 debris flow. Flow is from right to left.
A. Apparent laminar flow of fully developed debris flow (surge B); sediment concentration is 81 wt percent, surface velocity is 2.8 m/s. Note the lack of turbulence around large stationary boulders in the channel.

B. Onset of turbulence in dilute debris flow (tail of surge C); sediment concentration is 75-77 wt percent, surface velocity is 3.0 m/s. Turbulence at far right is over a short, steep riffle.

FIGURE 5 Flow type as a function of sediment concentration. Flow is from right to left.
the entire flow, although it was obvious from the smooth, rounded wave crests that the fluid still had considerable viscosity as compared to water (Fig. 5c). The fluid was still totally opaque, but only particles of sand size and finer were in suspension. When sediment concentration dropped below 70 wt percent, the flow surface took on a highly agitated, choppy appearance (Fig. 5d), and the transition was completed. Thus, the transition from debris flow to hyperconcentrated streamflow occurred at a less sharply defined and lower concentration threshold than Beverage and Culberston (1964) proposed.

Grain-Size Variations

Only one major trend was observed in grain-size distributions of the sediment-water mixture as the flow was sampled: average grain size (d50 and M2) decreased (Table 1). This trend occurred together with the gradual dilution of the slurry and loss of suspension competence, which resulted in a relative decrease of the gravel fraction and an increase in the fines (Table 1, Fig. 3). Sorting (σG) was variable, improving within the debris flow and hyperconcentrated flow ranges as concentration decreased, but becoming more poorly sorted again for normal streamflow. Sediment concentration is closely related to mean grain size (Fig. 6), and a plot of sorting against mean grain size clearly differentiates the flow types (Fig. 7).

Although the change in grain size and sorting with time can be seen in the envelopes of particle-size curves (Fig. 8), a more graphic illustration is provided by size frequency histograms (Fig. 9). All sediment samples were fine-skewed (Table 1), with a tendency for more dilute samples to be more strongly fine-skewed.

Shear Strength

Velocity profiles of the surface flow (Fig. 10) revealed that a rigid plug formed at the head of the surge when the flow mixture contained a large percentage of cobbles and boulders. However, the plug disappeared or became very small when the flow did not contain large clasts. This suggested that the coarse particles added a significant amount of shear strength to the slurry. To test this, shear strength for the slurry that had a rigid plug was computed using Johnson’s plug-width equation (Eq. 2), yielding a value of $1.25 \times 10^4$ dyn/cm$^2$. Shear strength was then computed for a lobe of finer grained slurry (fine pebbles and finer) on the debris fan using Johnson’s critical thickness equation (Eq. 1). As a spillover lobe, it was probably deposited at or near peak flow, so the absence of the coarse clasts would be the only major difference between this slurry and the slurry forming the rigid plug. The result was $1.23 \times 10^3$ dyn/cm$^2$, an order of magnitude less than when boulders were mixed in. In other words, the coarse clasts provided about 90 percent of the shear strength for the plug flow, assuming that these slurries can be macroscopically modeled as homogeneous plastics.

Shear strength was also computed for the laboratory sample of slurry at different water contents using Johnson’s “emergent boulder” equation (Eq. 3). These results were plotted against sediment concentration (Fig. 11), which
C. Well developed turbulence in flow transitional between debris flow and streamflow (tail of surge C); sediment concentration is 71 wt percent, surface velocity is about 2.5 m/s.

Highly agitated turbulent streamflow; sediment concentration is approximately 20 wt percent, surface velocity is estimated to be 1.0-1.5 m/s.
Table 1. Parameters of particle-size distribution, following the terminology of Folk and Ward (1957), for debris flow (D), transitional (T), hyperconcentrated (H), and normal streamflow (N) parts of the June 5 debris flow and runout.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Median Grain Size</th>
<th>Mean Grain Size</th>
<th>Sorting Coefficient</th>
<th>Skewness</th>
<th>Bilt and Clay</th>
<th>Bilt and Clay</th>
</tr>
</thead>
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<tr>
<td>Time</td>
<td>Collected Sample</td>
<td>Sediment Conc.</td>
<td>$d_{50}$ (µm)</td>
<td>$M_G$ (µm)</td>
<td>$G_G$ (µm)</td>
<td>$Sk_G$ (µm)</td>
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<tr>
<td></td>
<td>(min. after 10:30 a.m.)</td>
<td>(pct by wt)</td>
<td>(µm units)</td>
<td>(µm units)</td>
<td>(µm units)</td>
<td>(pct &lt; 0.062 mm)</td>
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<tr>
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<td>0.2</td>
<td>3.50</td>
<td>+.14</td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>3.1</td>
<td>83</td>
<td>-0.8</td>
<td>3.45</td>
<td>+.13</td>
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<tr>
<td></td>
<td>c</td>
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<td>3.70</td>
<td>+.22</td>
</tr>
<tr>
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<td>75</td>
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<td>2.85</td>
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<tr>
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<tr>
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<td>2.6</td>
<td>2.65</td>
<td>+.43</td>
</tr>
</tbody>
</table>

1Phi (Φ) units are a measure of grain diameter equal to the negative logarithm (base 2) of the diameter in millimeters.
FIGURE 6  Relation between sediment concentration by weight and mean grain size. Phi (\(\phi\)) units are a measure of grain diameter equal to the negative logarithm (base 2) of the diameter in millimeters.

FIGURE 7  Relation between sorting (\(\sigma_G\)) and mean grain size (\(M_d\)), showing separate fields for debris flow, transitional flow, and streamflow (hyper-concentrated and normal) samples.
FIGURE 8 Envelopes of cumulative grain-size curves for debris flow, transitional flow, and streamflow (hyperconcentrated and normal). Curves bracket all sample data: 3 debris-flow, 5 transitional-flow, and 4 streamflow samples (Table I). Sediment concentrations by weight ($C_w$) are indicated.
FIGURE 9 Size-frequency histograms for a debris flow sample (a), two samples (d and e) from transitional flow, and one sample (i) of hyperconcentrated streamflow (Table 1). Sediment concentration by weight (C_w) and sorting coefficient (σo) are indicated. Note the marked improvement in sorting. Total clay content (undifferentiated) is shown as a single size class.
FIGURE 10 Measured horizontal velocity profiles for flow with sediment concentrations $C_w$ fully within the debris flow range. Profiles show a rigid plug developed (top) when a high concentration of boulders was mixed into the slurry and no discernible rigid plug (middle and bottom) when the boulders were absent. Edge of left bank hidden by terrace edge, so distance to left bank is estimated.
FIGURE 11 Relation between shear strength and sediment concentration for reconstituted debris-flow sample (h) with varying water contents. Strength determined experimentally in the laboratory using Eq. 3.
when extrapolated suggests that the shear strength of the mixture essentially disappears at 70 to 72 wt percent solids. In other words, this is the threshold at which this material stops behaving as a Bingham plastic and presumably becomes a Newtonian fluid (assuming that the relation between applied shear stress and rate of strain is linear). From the field observations, this threshold also corresponds to the onset of major turbulence.

**FACTORS AFFECTING VELOCITY**

Velocity is a function of flow depth, channel slope, and hydraulic roughness of the bed in steady, uniform open-channel flow of water, as represented in the well known Manning equation (for metric units):

\[ V = \left( \frac{1}{n} \right) R^{0.67} S^{0.50} \]

where \( V \) is mean velocity of the fluid, \( n \) is the coefficient of roughness, \( R \) is hydraulic radius (approximately equal to mean depth for wide channels), and \( S \) is energy slope (equal to channel slope for uniform flow). Flow in lower Rudd Canyon was not uniform or steady, the fluid was not clear water, and only surface velocity could be measured. However, one might expect at least some consistent relation between hydraulic radius (or depth) and velocity for similar stages at a single station, because \( n \) and \( S \) would presumably stay fairly constant.

Such was not the case for the June 5 debris flow. Two different debris surges, separated by only 5 minutes, exhibited gradually varied flows behind their fronts that behaved very differently from each other. The plot of hydraulic radius versus maximum surface velocity is shown for each surge (Fig. 12). Flow on the recession of surge B, though deeper, was significantly slower than surge C. Surge B had a higher sediment concentration in the matrix slurry, and it contained more and larger gravel-size particles. Furthermore, a hysteresis effect also occurred in surge B, with flow on the rising limb slower for a given hydraulic radius than on the more dilute recessional limb. These results appear to suggest that a dilute slurry will flow faster than a more concentrated one. However, the denser, more concentrated slurry in B was dammed behind a slowly moving frontal plug of boulders, and a "backwater" was formed for an undetermined distance upstream of the front; this also would have acted to slow the flow. Therefore, it is not possible here to separate the backwater effects from the sediment concentration effects on velocity, at least for sediment concentrations greater than 82 wt percent.

Sediment concentration, nevertheless, can be demonstrated to have an effect on the velocity of this debris flow. Flow of more dilute mixtures, which was not affected by surge on backwater effects, does show a relation between surface velocity and sediment concentration when hydraulic radius is held within a narrow range (Fig. 13). This plot shows a steady increase in velocity with increasing concentration up to 80 wt percent and then a decrease. It is not certain, however, whether the velocity decrease at 82 wt percent is due to measurement error or to increased internal friction, which must, at some point, begin to be a factor as the slurry becomes more concentrated.
FIGURE 12 Relation between hydraulic radius and thalweg surface velocity for two debris-flow surges (B and C). Arrows indicate the sequence of measurement. Dashed arrows are on the rising limb of the surge, solid arrows are on the recession.

FIGURE 13 Relation between velocity and sediment concentration at the measurement site for flow not influenced by surge or backwater effects and limited to a range in hydraulic radius from 0.10 to 0.27 m.
It is hypothesized that, for maximum flow efficiency of an extremely concentrated sediment-water mixture, there is an optimum sediment concentration. If too dilute, turbulence will increase flow resistance and inhibit flow. Although the data set for the June 5 debris flow is incomplete, it appears that the highest velocity flow occurred at concentrations of 80 wt percent or greater.

CONCLUSIONS

The transition from turbulent hyperconcentrated streamflow (assumed to be a Newtonian fluid) to apparently laminar debris flow (a non-Newtonian fluid possessing shear strength) occurred over the sediment concentration range of 70 to 75 percent by weight. At approximately constant depth (hydraulic radius), slope, and channel roughness, the fully developed debris flow had a higher velocity than the hyperconcentrated flow. This is believed to be due to the decrease in flow resistance caused by the turbulence-damping effect of the extremely high sediment load. When debris flows develop, however, the normal flow resistance experienced by the fluid in the channel can be augmented by the creation of boulder plugs at the front of the flow, which act like sliding dams and allow backwater to build up behind the obstruction. Shear strength of the June 5 debris-flow slurry (assuming it behaves as a Bingham plastic fluid) is an order of magnitude greater with the coarsest particles (cobbles and boulders) mixed in than it is without them. Formation of a rigid plug at the head of the flow was due to frictional strength provided by interlocking coarse particles rather than to any change in cohesive strength in the slurry.

REFERENCES


BEHAVIOR AND EFFECT OF DEBRIS FLOWS ON STREAMS IN THE OREGON COAST RANGE.

by Lee E. Benda

ABSTRACT

Debris flows in the Oregon Coast Range can affect morphology of stream channels and aquatic ecosystems. Steep first- and second-order channels periodically flush stored sediments by debris flow activity. Debris flows that enter low-gradient streams may strongly influence channel morphology for years.

The length debris flows attain is controlled by stream basin and channel morphology as well as by the material properties of the debris flow. Severe erosion by debris flows is confined to channel slopes >10°, although transport occurs in third-order channels with slopes as low as 4°. Deposition of debris flows occurs mostly at stream junctions. Reduced channel slopes, increased channel widths, and changes in flow direction increase the likelihood of deposition at tributary junctions. Debris flows that enter third-order channels do not encounter sharply angled tributary junctions along their path. These debris flows usually originate from major drainage divides.

Deposits of debris flows that resist fluvial erosion for long periods occur in upper regions of basins that have small drainage areas. Volume of deposits ranges from 1000 to 8000 m³ and include large volumes of woody debris. With increasing watershed area, debris flow deposits are likely to erode quickly and to leave large boulders at the deposition site.

Effects of debris flow on aquatic ecosystems, in particular the salmonid fishery, depend on whether debris flows travel along second-order streams or enter third-order channels. In addition, deposition effects on channels depend on where the debris flow occurs in a basin.

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Debris flows are a significant erosion process in certain areas of the Coast Range in western Oregon. Small landslides originating in upper drainages during winter storms enter first-order channels and become mobile, highly viscous debris flows. As the debris flows move rapidly down preexisting drainages, they dramatically increase in size by incorporating large volumes of channel material and colluvium from streamside toeslopes. Deposition of inorganic and organic material by debris flows may dominate local channel morphology for centuries and influence aquatic ecosystems.

In this paper the definition of debris flows according to Varnes (1978) will be used. Debris floods, although dominated by hydrodynamic forces, may occur as downstream extensions of debris flows and will be considered as part of the debris flow process.

Major effects of debris flows on aquatic ecosystems include emplacement of large deposits of debris (Swanson and Lienkaemper 1978), channel scour or redistribution of gravels (Swanston 1980), and reduced capability for dispersed retention of organic material. These effects concentrate biological processes at debris flow deposition sites (Sedell and Dahm 1984).

The effect of debris flows on anadromous fish habitats in mountain ecosystems of the Pacific Northwest is a major concern to land managers. Barriers to fish migration, scour or resorting of spawning gravels, and sediment deposition in spawning and rearing habitat are potentially negative consequences of debris flows in fish bearing streams.

Debris flows are, however, a natural process and play an important role in the morphology of channels. Large wood and boulders, which are often delivered to channels by debris flows, are an important structural element of fish habitat in mountain streams.

The frequency of debris flows in the Pacific Northwest has increased as a result of timber management practices. Studies in the Coast and Cascade Ranges of Oregon reveal that high percentages of the total number of debris flows in a basin originate from roads and clearcuts: 93 percent (Morrison 1975) in the Coast Range; and 76 percent (Swanson and Lienkaemper 1978) and 71 percent (Swanston and Swanson 1976) in the Cascade Range.

Resource managers need to understand debris flow processes and their links to channel morphology and aquatic ecosystems. This knowledge will aid in assessing the effects of increased debris flow activity within managed watersheds. Our research was designed to clarify the behavioral characteristics of debris flows and help managers to control or mitigate effects on streams. The objectives were (1) to define the transport and depositional behavior of debris flows, and (2) to determine the depositional effects on channel morphology and anadromous fish habitat within a basin in the Oregon Coast Range underlain by Tertiary sandstone bedrock.
STUDY AREA

Knowles Creek is a 52-km² basin located within the Coast Range of western Oregon (fig 1). The basin is underlain by uplifted marine sandstones of the Tyee/Flourney formation (Baldwin 1964). Alluvium and weathered colluvium stored in upper order channels are the primary components of debris flows. Hillslopes are steep, 30-45°, and merge directly with stream channels. Valley bottoms in the upper two-thirds of the basin are narrow and have only minor terrace and floodplain development. Annual precipitation of 160 cm supports dense stands of Douglas-fir (Pseudotsuga menziesii) and western hemlock (Tsuga heterophylla). The Knowles Creek basin has a long history of logging: Clearcutting and road construction began intensively in the 1950's and continue to the present.

METHODS

Debris flows were located using aerial photo analysis and field reconnaissance. Twenty-six debris flows were located on aerial photos taken in 1968, 1972, 1979, and 1982. Field inventory revealed an additional 39 debris flow sites. Field data were collected on 55 of the 65 flows.

FIGURE 1. Study area site in western Oregon.
Data collected from debris flows varied depending upon the age and condition. Dendrochronological methods were employed to determine age of debris flow deposits (Leopold et al. 1964). Using standard surveying techniques, 16 debris flows ranging from 1 to 14 years old were measured for volumes of initiating failures, material entrained along the path of flow, and inchannel deposits. Slopes of erodional and depositional areas, channel geometry, velocities of debris flows, and depth of material stored in channels prone to debris flows also were measured. Information such as location in basin, length of flow track, average slope of track, and stream junction angles along the track were obtained from aerial photos and topographic maps for the majority of older events if the points of initiation and deposition were known.

Streams were ordered by the Strahler system (1952). Topographic depressions located in the uppermost headwall regions of the basin were considered to be 0 order.

RESULTS

TRANSPORT OF DEBRIS FLOWS

Sixty-five debris flows that occurred over a period of 300 years were inventoried in Knowles Creek basin. Fifty-five of the debris flows were 1 to 30 years old, the remaining 10 were between 200 and 300 years old. Debris flows were usually initiated by small planar failures (average volume 600 cubic meters) located near ridge crests. Seventy-four percent of the debris flows traveled along entire segments of second-order channels; 54 percent of these continued into lower gradient third-order streams.

Debris flows in Knowles Creek basin varied greatly in length. The distance a debris flow traveled was related to the angle of the second-order stream junction encountered along the path of flow. Second-order stream junction angles are defined here as the horizontal projection of the angle between a second-order and a higher order stream.

Debris flows were more likely to be deposited at second-order junctions with angles between 70-90°, thereby limiting transport to between 400-800 m of first- and second-order channels (fig. 2). This type of debris flow was more likely to originate from interfluvial ridges where second-order tributaries intersect higher order streams. Deposition by debris flows at second-order junctions comprised 34 percent of the total population of events in Knowles Creek basin.

Lengths of 1600 m were attained by debris flows that entered third-order channels after their passing through second-order junction angles 25° (fig. 2). These long flows traversed entire third-order stream channels and 90 percent of them originated from heads of major drainage divides.

The variation in lengths within these two groups of debris flows (see fig. 2) was due primarily to differences in lengths of stream order segments;
FIGURE 2. Relationship between distance traveled by debris flows and stream junction angles as encountered enroute.

This variability increased with increasing stream order in Knowles Creek basin. Thus, debris flows that traveled along third-order channels varied more in length than flows that remained in second-order channels.

Debris flows that traveled less than 400 m comprised 26 percent of the total population. These flows either deposited along steep, first-order channels or entered large, fourth- and fifth-order streams and were transformed into debris floods.
Stream junctions in general were important in controlling length of debris flows; 80 percent of all debris flows in Knowles Creek basin stopped at tributary junctions. A combination of factors, such as reduced slope, increased width, and change in flow direction, promote deposition of debris flows at stream junctions.

Channel slopes are also a major factor in controlling erosion and deposition of debris flows. Debris flows consistently displayed reduced erosion rates below channel slopes of 10°. Significant deposition began at channel slopes less than 60°, although debris flows that entered third-order channels flowed farther before depositing.

Debris flows generally scour channels with slopes >10° to bedrock. The material entrained along these first- and second-order channels comprised the majority of volume of debris flow deposits that ranged from 1000 to 8000 cubic meters. Debris flows that traveled along lower gradient, third-order channels exhibited reduced rates of erosion, though sorting of channel material such as wood and gravel was often severe. Discontinuous levees of debris flow material were also deposited along third-order channels.

DEPOSITION AND EFFECTS ON CHANNEL MORPHOLOGY

Debris flow deposits were found in channels and on floodplains along the valley floor and on toeslopes adjacent to streams. The majority of the deposits were confined to channels and were composed of a matrix of inorganic and organic debris. Large organic debris such as whole trees and rootwads, were located at the leading edge of deposits and were followed by unsorted, inorganic material ranging in size from silt to boulders up to 2 m in diameter. Debris flows that traveled along third-order channels distributed their deposits over longer stream reaches than debris flows that stopped and deposited en masse at second-order junctions.

Because of the large volumes of material involved, long-term valley floor landforms, such as levees and terraces, are often formed by debris flow deposits. In addition, large boulders contained in deposits of debris flows are resistant to weathering and fluvial transport and may reside at depositional sites for decades to centuries, thereby influencing channel morphology. Debris flow deposits may reach 2-4 m in thickness, exceed 100 m in length, and completely fill the channel.

Erosion of in-channel deposits of debris flows, both immediately following deposition and through time, is a function of stream discharge, volume of deposit, deposit composition, and channel morphology. Stream discharge based on drainage area in Knowles basin is an important factor governing the stability of deposits. The percentage of recent, <5-year-old deposits remaining in channels is a function of drainage area (fig. 3); erosion increased with increasing drainage area above the deposit (correlation coefficient = 0.95).
Sixty percent of debris flows in Knowles Creek were deposited directly within habitat of coho salmon (Oncorhynchus kisutch), steelhead trout (Salmo gairdneri), and chinook salmon (O. tsawytscha). Coastal cutthroat trout (S. clarki) also reside in the basin.

The extent of spawning and rearing habitat used by anadromous fish within the basin has been identified by population surveys and habitat inventories. The entire mainstem of Knowles Creek and the lower reaches of several tributaries produce anadromous fish. Productive habitat diminishes where stream slopes exceed 4°.

**DISCUSSION**

Significant variation in length of debris flows in Knowles Creek basin results from a combination of geomorphic factors such as junction angles, channel geometry, and channel roughness, as well as variations in material properties of the flow caused by water and material content. Variation in length of stream order segments is also a factor.

Debris flows that deposit at sharply angled (70-90°), second-order junctions encounter decreasing channel slopes, increasing channel widths, abrupt slope changes between the contributing and receiving streams, and an abrupt change in flow direction at the tributary junction. These factors favor deposition at tributary junctions.
Debris flows that travel longer distances within third-order channels do not encounter sharply angled second-order junctions. Rather, these debris flows travel relatively unimpeded through second-order junctions with angles \( \leq 25^\circ \).

Deposition by debris flows into larger streams is governed by the magnitude of the stream discharge encountered at that point. With increasing discharge, deposits are likely to be eroded. Deposits that fill channels and resist erosion occur in the upper regions of basins.

Debris flow deposits that enter fish-bearing streams are a major concern to fishery biologists. Inchannel deposits of debris flows are one of the most obvious effects and are usually perceived to be the most threatening to fish habitat because of the potential to block or hinder migration. Baker (1979) documents the ability of a debris flow deposit to stop migration of anadromous fish in a basin in the Oregon Coast Range.

Deposits of debris flows affect fish habitat in the immediate area of deposition. Everest and Maehan (1981) report decreased spawning and rearing habitat and decreased fish biomass immediately below debris flow deposits. Their data show a 90-percent reduction of salmonid biomass in small streams and a 55-percent reduction in large streams. They also found, however, increased fish populations in addition to increased rearing and spawning habitat within the pools created above certain debris flow deposits. These pools produced underyearling coho at rates 10 times greater than reaches with no debris flow pools.

Not all effects on fish habitat from debris flows are centered around the immediate site of deposition. Debris flows that enter large streams lose their viscous properties and may move downchannel as a debris flood. Being heavily loaded with debris, in particular large organic material, debris floods have the potential to scour spawning gravels, damage rearing habitat, and disturb adjacent stream banks.

Large organic debris and boulders in channels serve as sites for temporary retention of fine organic materials that route through fluvial systems (Swanson et al. 1982). Debris accumulation areas are generally dispersed throughout a stream reach and enable microinvertebrates and macroinvertebrates to break down organic material, a process that is important in the aquatic food chain. Debris flows tend to remove large wood and boulders from scour and transport zones, thus eliminating dispersed biological processing, while concentrating these activities in zones of deposition. Debris flows may actually reduce the capacity of streams to retain and process medium to fine organic material.

The effects on channel morphology and habitat vary with the type of debris flow. To understand the interaction between debris flows and fish habitat in Knowles Creek basin, the behavior of debris flows was viewed in context of the habitat available to anadromous fish.
Debris flows that travel along third-order channels create large deposits in regions of small drainage area. This type of debris flow erodes the longest reaches of channels and could create a barrier to fish migration. These debris flow deposits are, however, generally near the upstream limit of productive fish habitat. Channels that may be affected by long debris flows can be identified by locating headwall areas with small second- to third-order junction angles of less than 25°.

Debris flows 400-800 m in length that deposit at tributary junctions involve second-order channels entering higher order streams. The stability of the deposit is in large part a function of stream energy encountered at the stream junction at the moment of deposition. The deposit will either remain relatively intact, breach and retain some integrity, or disperse downstream, depending upon its position within the drainage basin.

The different types of debris flow deposits at second-order junctions will affect channel morphology and stream ecosystems in various ways. In regions of small drainage area there is a potential for deposits to dominate local channel morphology for decades to centuries and, at least for a time, block or hinder fish migration.

With increasing drainage area, deposits breach during emplacement or soon thereafter but retain sufficient material to create large pools. These are the type of productive pools that Everest and Meehan (1981) refer to in their study.Degraded habitat and reduced fish populations may also occur in the immediate vicinity of this type of deposit (Everest and Meehan 1981).

Debris flows that enter large streams farther down in the basin encounter larger volumes of water and become incorporated into the stream flow. The inorganic and organic material is then transported downstream by hydrodynamic forces with the large organic debris being carried the farthest. Large logs and root wads tumbling in the current could scour spawning gravels and cause mortality of incubating salmonid embryos. The existence of coho and chinook salmon eggs in gravels during the most active period of debris flow occurrence requires careful consideration of the erosive activity created by this type of event.

Knoles Creek is structurally deficient in large inorganic and organic material, thus lacking in fish habitat diversity. Past logging operations removed most of the large woody material from the stream. Debris flow deposits contain large wood and boulders and often form pools, thus certain debris flow deposits can actually create habitat in structurally deficient streams. It seems likely that debris flows have historically played an important role in maintaining habitat diversity in these steep, sandstone streams.

REFERENCES


OBSERVATIONS ON SLOPE FAILURES ASSOCIATED WITH THE RAINSTORMS OF 1978, 1980, AND 1983 IN SOUTHERN CALIFORNIA

by P. M. Merifield

Widespread damage from slope failures resulted from rainstorms during February and March 1978. Two periods of rainfall culminating in the intense storms of February 9-10 and March 3-4, 1978, brought over 8 inches to some locations in less than 24 hours. Six storms during 9 days in February 1980 brought over 20 inches of rain to mountain and foothill areas. Most of the damage and fatalities were due to debris flows and debris flooding; thirteen people perished in debris flooding of the small community of Hidden Springs in the San Gabriel Mountains. Much of the flood damage in mountain and foothill areas was associated with burned watersheds, which produced an order of magnitude more debris than unburned areas, causing some recently constructed debris basins to overflow. Both man-made and natural unburned slopes suffered damaging slope failures. Debris flows were largely restricted to slopes vegetated with grasses or other ground cover lacking deep root systems. In general, unburned slopes with deep-rooted vegetation were not characterized by debris flows, debris flooding and other shallow slope failures.

The winter of 1982-1983 also brought greater than normal precipitation to southern California. But rainfall was distributed over the season, and short-duration, high-intensity storms comparable to 1978 and 1980 were lacking. Debris flows and debris flooding were not prevalent, but 1983 was characterized by deep-seated landslides; unusually high ground-water levels were a major contributing factor.

Recommended mitigative measures include stricter enforcement of existing grading codes, stability analyses of surficial failures, regional studies by state and local governments to identify hazardous areas, site-specific investigations by private consultants when properties change hands, and increased efforts to educate the public about landslide and flood hazards.

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INTRODUCTION

Coastal southern California is largely foothills and mountains reaching elevations of more than 3000 m. Youthful topography is maintained by uplift along active faults, although Taylor (1982) estimates the erosion rate to be 1 m per thousand years. Coastal mountains create desert conditions in eastern California, but a Mediterranean climate prevails in the coastal and near-inland areas. Because snow is restricted to relatively small areas, snow melt is not a significant factor in debris flows and debris flooding. Precipitation, which occurs almost entirely in the winter months, averages about 15 inches in the coastal plains (for example, at Los Angeles Civic Center) but more than twice that in some mountain and foothill areas. Upland slopes are commonly 1-1/2 horizontal to 1 vertical or greater. The soil mantle, which is generally thinner than 1 m, is therefore near the angle of repose. During dry periods unconsolidated colluvium accumulates to a depth of about 3 m in ravines.

Sediment moves downslope principally during infrequent heavy rainstorms. Vegetation consists largely of chaparral: shrubs that grow primarily in winter and form a nearly continuous canopy over the slopes. Chaparral is highly flammable: extensive wildfires are common during the dry season, and especially difficult to control when fed by dry Santa Ana winds from the Mojave Desert. The production of sediment increases enormously in burned areas when heavy rains follow wildfires within one to three years. During the past quarter century, urban development has encroached into hillside areas, and hazards to residential properties from debris flows (including mudflows), debris flooding, and landslides have greatly increased.

OVERVIEW OF 1978 AND 1980 STORMS

Annual rainfalls of one and one-half to twice the average of 15 inches are not uncommon. If we use the Los Angeles Civic Center for comparison, rainfall records since 1877 show nine years with rainfall between 26 and 38 inches. The records also show that high rainfall years tend to be clustered. Thus the recent above-average years of 1978, 1980 and 1983, spaced closely in time, are similar to the periods 1878-1891, 1934-1941, and 1965-1969.

In recent years, three storms stand out prominently for both intensity and severity of damage: the February 9-10 and March 4-5, 1978, storms and the mid-February storm of 1980. Figure 1 shows the rainfall figures for Mt. Wilson in the San Gabriel Mountains, which commonly records among the highest precipitation of stations in southern California because of its elevation and location. The storm culminating on February 9-10 began five days earlier. On the night of February 9-10, more than 8 inches of rain were recorded at some localities in 24 hours. One station, Crystal Lake in the San Gabriel Mountains, recorded 13 inches on February 10. Several southern California stations exceeded 100-year return periods of high-intensity rainfall in early 1978 (Pyke, 1982). However, these rainfall intensities for a given 24-hour period are not unique; on New Year's Eve and New Year's Day, 1933-34, Pasadena recorded 12 inches, and on January 25, 1969, over 20 inches were recorded at some stations in the San Gabriel
Mountains. The highest rainfall recorded in southern California for a single 24-hour period was 26.12 inches at Hoeegees station in the San Gabriel Mountains on January 22, 1943.

The other heavy rainstorm of 1978 occurred from February 27 to March 4, 1978; Mr. Wilson recorded more than 24 inches during the six-day period (Fig. 1). The rains of March 4 were particularly intense; La Crescenta, at the foot of the San Gabriel Mountains, received 6.79 inches of rain on that day.

In mid-February 1980 a series of six storms occurred over a nine-day period. Mt. Wilson received over 30 inches of rain in this storm (Fig. 2). Some stations in the San Gabriel Mountains recorded up to 13 inches in 24 hours. Topanga Canyon, a residential community in the Santa Monica Mountains, received 17 inches during this period (Fig. 3). The Topanga Canyon Station recorded 8.3 inches in 24 hours on February 17, and one rain gauge recorded 8 inches of rain in a 10-hour period. At Bel Air, 5 miles away, 3.5 inches of rain fell in the same 24-hour period. This exemplifies the wide variation in precipitation as these storms moved across southern California in relatively narrow belts.

STORM DAMAGE

The City of Los Angeles reported damage at 3,102 addresses during the 1978 winter storms. Los Angeles County, in a less comprehensive survey of unincorporated areas, reported damage at 1,796 individual street addresses (Weber et al., 1979). Thirty-eight storm-related deaths and $220 million in damages were attributed to the 1978 storms (Brooks, 1982).

Much of the flood damage in mountain and foothill areas occurred in burned watersheds, which produced far more debris than unburned areas (see Shurrman, Slosson and Yoakum, this volume). In burned areas several debris basins—even some recently constructed—overflowed, though in unburned areas, the basins proved quite adequate (Brooks, 1982). Extensive areas of the San Gabriel Mountains had been burned over in preceding dry periods. The canyons and foothills within south-facing watersheds were thus vulnerable when the intense storms struck. In 1978 and 1980, 688,000 cubic meters of the 1,720,000 cubic meters of debris deposited in the debris basins operated by the Los Angeles County Flood Control District were attributed to burned watersheds (Davis, 1982). Zachau, Rubio, and Shields Canyons, all within burned watersheds, overtopped their debris basins. Debris flooding in Shields Canyon was particularly bizarre. The watershed in Shields Canyon above La Crescenta is only about 0.7 sq km with slopes averaging 1:1. In the 1960’s new homes were constructed in the canyon above an older debris basin. A smaller debris basin was constructed above the new development along with a concrete channel to convey flow through the development to the old basin (Davis, 1982). During the intense storm of February 9-10, the smaller basin filled to capacity and debris overtopped the structure. The channel, which was too narrow to accommodate the debris up to boulder size, became clogged and flow was diverted out of the channel and onto the street. The high volume of flow and the steep gradient of the street provided the debris flood with the capacity to transport automobiles parked along the street. Because the street made a
Fig. 1  Daily rainfall, December–March, 1977–1978, Mt. Wilson station, NOAA Climatological Data.

Fig. 2  Daily rainfall, December–March, 1979–1980, Mt. Wilson station, NOAA Climatological Data.
right-angle turn just above the old debris basin, the debris flood left the street and dammed up in the front yard of a single-family residence (Fig. 4).

In terms of lives lost, the event at Hidden Springs on the Middle Fork of Mill Creek in the San Gabriel Mountains was the costliest result of the February 1978 storms (see also Shuurman, Slosson and Yoakum, this volume). A wildfire in July, 1977 burned the entire 10-sq-km watershed of the Middle Fork of Mill Creek. The small resort community and thirteen persons were swept away by an estimated 4.5-m-deep flow of water and debris produced by the rains of February 8-10, 1978.

Debris flows damaged the largest number of houses and caused the greatest dollar losses during the 1978 storms. Grass-covered slopes were particularly susceptible. Extensive areas of the urbanized foothills are underlain by Miocene marine shales which weather to clay-rich soils that support primarily shallow-rooted grasses (Fig. 5). Debris flows originating on these slopes did extensive damage to residences lacking adequate protective devices or sufficient setbacks. Even where structures were undamaged, yards were commonly inundated with debris, requiring costly removal.

Debris flows on wooded or chaparral-covered slopes originated chiefly in colluvium-filled ravines. Relatively small failures starting high on the slopes near the heads of natural drainage courses, possibly as landslides, often became enormous debris flows by the time they reached the base of the slope. One such failure in a tributary of Benedict Canyon above Beverly Hills incorporated hundreds of cubic meters of ravine-filling material as it moved more than 300 m downslope. Homes constructed in the 1960's at a cul-de-sac on Liebe Drive, which lacked adequate provision for such an event, were severely damaged (Fig. 6). The same homes were again damaged in 1980 when additional debris flows, initiating along the natural flanks of the ravine, overtopped newly constructed flood control facilities.

Landslides involving bedrock were also numerous, with movement generally beginning days to months following the storms. Initial movement of the Bluebird Canyon landslide in Laguna Beach, for example, began in October 1978.

Thirty deaths were attributed to the storms of February 1980; damages in southern California were estimated at $400 million. The 1980 rains caused even more damage than those of 1979, primarily because concentrated, heavy rainfall occurred over a larger area (Weber, 1980). One hundred eleven homes were destroyed and 1,350 were reported damaged (Weber, 1980). The foothill town of Monterey Park on the east edge of the Los Angeles basin suffered extensively from debris flows. Many slopes (natural and manmade) in older residential areas planted with common iceplant failed; new man-made slopes with shallow-rooted ground covers were also susceptible to failure (Weber, 1980).

The rains and runoff were particularly severe in Topanga Canyon, washing out the principal roads and isolating the canyon community for several days. Low bridges and imprudent construction practices by local
Fig. 3  Daily rainfall, December-March, 1979-1980, Topanga station, NOAA Climatological Data.

Fig. 4  Effects of debris flooding, Shields Canyon, February 1978.
Fig. 5  Surficial failures on grass-covered slopes, foothills of the San Gabriel Mountains at Big Tujunga Wash, 1978. (Photo courtesy of H.F. Weber, Jr.)

Fig. 6  Debris flows originating at the head of ravines, Santa Monica Mountains. (Photo courtesy of City of Los Angeles, Dept. of Building and Safety.)
homeowners impeded high discharge and caused flood damage when natural channels overflowed. In Riverside County, the San Jacinto River overflowed, and portions of the town of San Jacinto near the river banks were flooded. Lake Elsinore, receiving waters from the San Jacinto River, rose to unprecedented heights, inundating shoreline houses and mobile homes. Many deep-seated landslides occurred as previous slides were reactivated or accelerated.

Greater than normal precipitation also occurred in southern California during the winter of 1982-1983 (over 30 inches at Los Angeles Civic Center). But rainfall was distributed over the season, and short, intense storms comparable to 1978 and 1980 did not occur. A histogram of daily rainfall for Topanga is shown in Fig. 7. Debris flows and debris flooding were uncommon, but 1983 was characterized by deep-seated landslides caused primarily by unusually high ground-water levels.

The monitoring of more than 30 water wells in southern California since 1976 (Merifield et al., 1984) indicates that after the severe drought of 1976-1977, ground-water levels generally crested during 1980 or 1981. Since these highs, the levels of most wells have been maintained not far below their crests. It seems reasonable to speculate that ground-water levels in many parts of southern California have also remained high up until 1984 and this has been a major factor in deep-seated landsliding. It is in fact the only plausible explanation for landslides that have become active in 1983 and 1984 such as the Flying Triangle landslide on the seaward-sloping flank of the Palos Verdes Peninsula (Ehlig, 1982a) and the Big Rock landslide of Malibu (Evans, 1984). Movement of a portion of the ancient Flying Triangle slide near its head was first noted in 1980. In late 1983 or early 1984, however, signs of movement were noted over a much larger area, at least 400 m x 1300 m, even larger than the ancient slide as mapped by Woodring et al. (1946). Movement of the Big Rock Mesa slide, involving 150 acres and 45 million cu m of rock, was not clearly established until mid-1983. The homes on these slides (30 in the Flying Triangle area and more than 300 in the Big Rock area) are on private sewage disposal systems, each introducing 300 to 500 gallons per day of effluent into the subsurface.

GRADING CODES

The first grading codes for the City of Los Angeles were established in 1952. Before 1952, geological and soil engineering input into residential construction was not required and rarely sought. Los Angeles County established its first grading code in 1957, Orange County in 1962. Deficiencies in these early codes have periodically been corrected; major revisions in the City of Los Angeles code were made in 1963. In 1974, California state law required that all building-permit-granting agencies within the state adopt Chapter 70 of the Uniform Building Code (UBC) as a minimum; some agencies, notably the City of Los Angeles, maintain even stricter standards. The California Division of Mines and Geology (CDMG) has also issued a series of guidelines outlining the elements that should be covered in geologic reports related to development (Slosson, 1984). Codes now in effect specify investigations to assess the potential for bedrock failures. These investigations contain an evaluation of the effect
1982-1983 TOPANGA
37.20 INCHES

Fig. 7 Daily rainfall, December-March, 1982-1983, Topanga station, NOAA Climatological Data.
of individual sewage disposal systems on known or potential bedrock landslides, although introducing sewage effluent into known landslides is generally not specifically prohibited.

Of particular significance to the following discussion is the treatment of surficial failures in the codes. Prior to 1979, grading codes in use did not specifically require an analysis of the stability (either quantitative or qualitative) of surficial failures on natural slopes; they were, however, being performed by the more prudent professionals. In 1979, following the severe damages wrought by the 1978 storms, the City of Los Angeles required analysis of surficial failures. For slopes exceeding 2:1, a minimum factor of safety of 1.5 against surficial failure was specified, to be derived by calculations for an infinite slope with seepage parallel to the slope. The use of other methods is subject to approval by the Department of Building and Safety. The minimum assumed depth of soil saturation is 3 ft (0.9 m) or depth to firm bedrock. Soil strength parameters are to be derived from the testing of representative samples under conditions approximating saturation. The required stability is to be achieved by appropriate mitigating measures.

Slope vegetation requirements for man-made slopes are set forth in Chapter 70 of the UCB; the City of Los Angeles has the same requirements. Slope vegetation should consist of trees with a minimum spacing of 20 ft (6 m) or shrubs spaced 10 ft (3 m) or a combination of the two. Vegetation on natural slopes is not covered in the code. In practice, natural vegetation is commonly removed for fire protection, or replaced by ornamental species.

Mitigation of water and debris flooding is called for in general terms in Chapter 70 of the UCB. Specific provisions are absent, and mitigating measures are expected to fall within "good engineering practice." In reality, the Los Angeles County Flood Control District and the Army Corps of Engineers have assumed responsibility for protecting most urbanized areas by constructing dams and debris basins.

DISCUSSION

The conclusions that can be reached from observing surficial failures during the rainstorms of 1978 and 1980 reinforce those of Campbell's (1975) study of slope failures from the 1969 rainstorms in the Santa Monica Mountains. Prerequisites to debris flows are slopes of 2:1 or greater and a sufficient thickness of soil, colluvium or poorly consolidated artificial fill. Both artificial fill slopes and natural slopes are susceptible to surficial failures. Many fill slopes constructed before updated grading codes were in effect were poorly compacted or placed on improperly prepared surfaces. Stability analyses of fill slopes constructed under modern grading codes have generally been performed on hypothetical arcuate failure surfaces within the fill, but not on the surficial slab. Well-compacted slopes become less well consolidated in time owing to physical and chemical weathering processes and the activity of organisms. Natural slopes in southern California also rapidly undergo change as new soil continually forms, creeps and accumulates in ravines.
Prior rainfall sufficient to bring the soil to field moisture capacity (7 to 10 inches), followed by a rainstorm with an intensity of greater than 0.2 inches per hour, are the critical failure criteria (Campbell, 1975). Slosson and Krohn (1982) observed that surficial failures are most severe when heavy rains (over 7 inches in all) occur for five or more days and especially when the most intense rainfall occurs near the end of the storm period.

Shallow-rooted vegetation such as grasses and iceplant are not effective in preventing surficial failures; in fact such ground covers probably promote soil failures by increasing the infiltration of surface water, thereby leading to "a more rapid and thorough saturation of the soil mantle" (Campbell, 1975).

Detailed studies in the San Dimas experimental forest near Glendora (Rice et al., 1969) have demonstrated that soil failures are 3 to 5 times more frequent for grass covered slopes than for brush covered ones; the minimum angles for failure were less for grassy cover than for most chaparral vegetation.

In another study near Cincinnati, Ohio, tree roots increased the factor of safety against shallow sliding ninefold (the sliding surface in this case was the contact between bedrock and colluvium). The average shear strength contributed by tree roots penetrating the contact was determined to be about 5,900 N/m² of the shear surface, whereas the average strength contributed by residual friction was about 720 N/m². In forested areas, colluvium-mantled slopes were stable up to 35°, whereas deforested slopes were subject to sliding on 12° to 14° slopes (Riestenberg and Sovonick-Dunford, 1983).

Fatalities in residential areas usually occurred when debris flows suddenly burst from steep, soil-mantled slopes above the rear yards of homes built close to the base of the slope. Most slopes were vegetated only with grasses, iceplant or other shallow-rooted ground covers. Swales or ravines concentrating flow toward the residence produced an even greater hazard.

The effect of strengthened grading codes has been extremely encouraging. A survey of 37,000 sites graded before 1963 in the City of Los Angeles showed 2,790 failures or 7.5 percent, whereas 36,000 sites graded after 1963 had 210 failures or 0.6 percent (Slosson and Krohn, 1982). Until recently, stability analyses were not performed on man-made or natural slopes to investigate the potential for surficial failures. The City and County of Los Angeles now require that the debris flow potential be assessed as part of the geological investigation. Some other permit-granting agencies, however, do not even require geological studies. The vegetation requirements in Chapter 70 of the UBC are not generally adhered to. Hydromulching of fill slopes with grass seed is common practice. Moreover, it is questionable whether the code, which calls for trees at 20-ft (6-m) spacing or shrubs at 10-ft (3-m) spacing, is adequate. More important is that the slope be covered with a continuous canopy of trees or shrubs with root systems sufficient to bind the surficial unconsolidated materials to firmer substrate.
The reason that burned watersheds yield far greater amounts of water and debris runoff than unburned areas is that the vegetal canopy and some of the shallower root systems are destroyed, which results in greater erosion by rain impact and sheet wash. More creep also produces thicker ravine fill for movement during ensuing rains (Simpson, 1969). The intense heat of wildfires produces a hydrophobic layer just below the surface, an impervious waxy layer formed from organic matter in the soil that is believed to contribute significantly to increased runoff (Cleveland, 1972).

Debris basins in burned areas performed poorly (Brooks, 1982) because the facilities were evidently not designed to accommodate large debris flows resulting from extensive wildfires followed closely by unusually heavy rain. Many privately engineered flood control provisions in residential tracts were underdesigned or had design flaws. Furthermore, drainage provisions are commonly neglected. Developers of housing tracts are required to install bench drains, collection basins, berms and other devices to convey water quickly away from slopes. When the homes are finished, responsibility for maintaining the devices passes to the homeowners, who seldom make a coordinated effort to maintain devices common to several properties. The devices become clogged, and fail to function when the need arises.

The enormous increase in the number of deep-seated bedrock slides since the winter of 1978 can be attributed not only to perched water but also to a general rise in the water table, especially for the largest, deepest slides. The effluent from private disposal systems augments the natural rise in water table. Costly dewatering systems have been successful in suppressing movement of the Abalone Cove slide on the Palos Verdes Peninsula (Ehlig, 1982b) and are being installed at Big Rock in Malibu, but a long-term solution must also involve public sewer systems.

The 1978 and 1980 rainstorms have made geologists and soil engineers much more aware of the hazards of surficial failures. We now know how to identify potentially unstable slopes; the codes need to be both updated to reflect this knowledge and enforced. Stability analyses for surficial failures should be performed on both natural and engineered fill slopes affecting new development. Measures to mitigate the surficial failure problem are also available. Debris fences and deflection walls (see Hollingsworth, R. A., this volume) are cost-effective. Planting deep-rooted vegetation as densely as possible is even less costly. Multiple rows of closely spaced fast-growing trees such as eucalyptus and pine will form a barrier across the slope.

For existing developments, special slope stability studies should be conducted in sensitive areas. The California Division of Mines and Geology has recommended hazard mapping in selected areas (Weber, 1982). Site-specific investigations by private consultants when property ownership is transferred plays an important role, for this alerts sellers and buyers to potential geologic hazards that can usually be minimized or abated at modest cost in comparison to the value of the property. Correction of drainage, improvements in vegetation cover, and addition of deflection walls and debris fences are common low-cost mitigation measures. Further
educating the public on slope hazards, including the maintenance of mutually owned drainage devices and the planting of proper slope vegetation, is valuable and relatively low in cost.

In summary, implementing the following recommendations will reduce property damage and loss of life such as those experienced in recent years.
1. Enforce existing grading codes.
2. Perform stability analyses for surficial failures on natural soil-mantled slopes and engineered fill slopes.
3. Plant a dense cover of deep-rooted trees and shrubs on artificial fill slopes, and maintain natural vegetation or equivalent fire-resistant plants on natural slopes.
4. Design debris basins and flood control channels to take into account the increased debris produced by burned watersheds.
5. Strictly prohibit the discharge of effluent by densely spaced private disposal systems into known landslides or potentially unstable slopes.
6. Follow the guidelines recommended for preparing engineering geologic reports in CEMG Note Number 44.
7. Perform hazard mapping in selected areas and continue research on the mechanics of slope failures under the direction of appropriate government agencies.
8. Have private consultants perform site-specific investigations when properties change hands.
9. Increase efforts to educate the public on slope maintenance.

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The erosion rate of a watershed can be dramatically increased due to the fire/flood sequence that often plagues California and other semi-arid to arid areas subject to brush fires and seasonal rainfall. As a result, fire-affected areas will suffer more extensive damages related to debris flows. Case histories in Southern California have shown that erosion may be increased by a factor approaching fifty for the first year following a significant brush or forest burn. High temperature wild fires: 1) change the physical properties of the soil profile causing concentration of coarse-grained particles in the top few inches of the soil profile; 2) develop a wax-like aliphatic hydrocarbon water-repellant zone at two to three inches below the ground surface which inhibits infiltration of rainfall; 3) loosen the outer few inches allowing gravity-related dry ravel and rapid erosion from flow-water during periods of high intensity rainfall; and 4) destroy the litter and brush which acts as a series of natural micro debris basins.

During the torrential rains of 1978 and 1980, the bulking factor for some burned Southern California watershed streams ranged from 200 percent to 500 percent. Precipitation runoff intensity for these same burned watersheds appears to have reached approximately 100 percent for the 15 to 60 minute period of rainfall. This almost sudden surge of sediment-laden flow was then superimposed upon nearly full bank flow creating the "flash flood" and "debris flows" which caused death and destruction.

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DESCRIPTION AND BEHAVIOR OF DEBRIS FLOWS

INTRODUCTION

Several factors combine to produce exceptionally damaging debris flow/flood conditions following brush fires. These factors caused by high temperature fires include: 1) Changes in physical properties of soils; 2) Reduction of the thickness of water absorbing soil layers by production of a wax-like water repellant zone two to three inches below the ground surface; 3) Drying out and loosening the surface soils so that dry ravel can occur on steeper slopes; and 4) Destruction of vegetation and ground litter which have a significant effect on water absorption and reduction of erosion, all of which reduce runoff concentration time and increase erosion. Research by Rowe (29, 30) and others (12, 13) has shown that erosion may be increased 35 to 48 times resulting from high intensity runoff during the first year after a fire. A case history is used to illustrate the interplay of these fire related factors in developing catastrophic events following a heavy storm.

RELATED FACTORS

Research by the United States Department of Agriculture (31) and others (1,19,20,29,30,36,37,38) lucidly indicates that erosion following a wildfire is increased dramatically as a result of removal (fire loss) of the vegetation and its associated litter. The vegetation (grass, brush, trees and litter) not only provides a canopy to protect the soil from rainfall impact, but also utilizes or absorbs some of the rainfall. In addition, litter acts as multiple micro-debris basins on a slope retarding the effects of sheetflow.

High temperature brush fires (400°F) change the infiltration capacity near the surface and the permeability of the subsurface soils (12,13,36,37,38). As the nature of the soil profile is altered, some people feel that fine-grained materials tend to move downward through the soil profile causing a shift in particle-size distribution with the near-surface soils becoming coarser grained and more porous while densifying the lower soils by way of an increase in the percentage of fines. It is suggested that this change in particle-size distribution reduces the permeability of the lower, denser soils.

The writers believe that this theory is most likely only as the result of freeze thaw cycles. Since the arid and semi-arid slopes of southern California seldom experience freeze-thaw conditions, the writers suggest as an alternate that partial fusion of clay particles on the surface occurs as the result of high fire temperatures. This forms a thin glaze on the surface which initially tends to shed water, but with time after the fire, this glaze...
breaks up into small pieces and takes on the physical properties of a lightweight granular soil which is highly erodable (see Figure 1). This, we believe, explains why there seems to be a higher percentage of granular soil near the surface after a fire. Certainly this concept could form the basis for an important research project that could be performed by the Forest Service in their controlled burn areas, i.e., profiling soils and performing gradation and particle type analyses before and after the burns.

Further, a water repellent layer develops two to three inches (5.1 to 7.6 cm) below the surface as a result of concentration of wax-like aliphatic complexes of long chain hydrocarbons derived from the burning of vegetation (see Figure 1). The origin of this repellent zone is associated with the burning of litter and overlying vegetation with the vaporized hydrocarbons moving downward in the soil and condensing at a depth of two to three inches (5.1 to 7.6 cm) below the surface where the soil is cooler. The ensuing coating of the soil particles produces a subsurface water repellent layer (12,13,36,37,38).

The end product of the development of the water repellent layer is to reduce the storage capacity of the soil mantle by 20 times or more. Thus, the remaining storage space within the surficial soils (2 to 3 inches, 5.1 to 7.6 cm) rapidly fills during a storm causing runoff to begin sooner and the runoff rates to be much higher. Therefore, one inch (25mm) of rainfall may have the same effect on a recently burned watershed as five to six inches (127 to 152mm) of rainfall would have on an unburned watershed.

Following a fire, gravity becomes a dominant factor affecting the surficial soils; dry ravel occurs, moving loose materials downslope onto the lower portions of the slopes and into the channels (3). Later, during the ensuing winter rains, the loose material on the lower portions of the slopes begin to wash downslope into the channels and become involved in the mass sediment transport process.

Because of the lack of vegetation and litter to retain the loose soils, the erosion rate is greatly exaggerated, and the bulking factor increases. The greatest increase in erosion and sediment transport occurs after the soils have been saturated from many days of antecedent rainfall and then impacted by torrential rain associated with high peak intensity rainfall (squall). As runoff water velocity increases, erosion starts to occur and soil particles become part of the flowing mass. With soil particles in suspension, the bulk unit weight of the fluid increases. This allows the fluid to pick up larger particles which, in turn, increases the density and so on. This, along with the increased frictional affect of the sediment carrying flow, explains why some very large boulders are found quite a distance from their original resting place after a flood.
DESCRIPTION AND BEHAVIOR OF DEBRIS FLOWS

A. Before Fire

B. Immediately after Fire

C. Sometime after Fire

FIGURE 1. SOIL PROFILE SEQUENCING
Fire Preceding The Storm

The years of 1975-1976 and 1976-1977 were dry years, recording less than 50 percent of the average annual rainfall in 1975-1976 and approximately 80 percent of the average annual rainfall in 1976-1977 (35) (see Figure 2). This sequence of dry years provided an abundance of extremely volatile, dry brush necessary for the wild fires that are typical to the Southern California foothills and mountains. On July 24, 1977, a person visiting the open firing range on United States Forest Service property of the watershed of the Middle Fork of Mill Creek (see Figure 3) fired a black powder, muzzle-loading rifle. According to the report by the fire warden, "Kleenex" was used for wadding rather than the required non-flammable cloth wadding. The "Kleenex" smoldered and subsequently caught fire, igniting the tinder dry watershed of the Middle Fork of Mill Creek. The fire was controlled only after the entire 2440 acres (10 km²) of watershed was burned (see Figure 4).

February 8-10, 1978 Storm

The winter of 1977-1978 produced the third greatest annual rainfall of the past century in Los Angeles (see Figure 2). Only in 1883-1884 and 1889-1890 were higher rainfall totals recorded. One storm, lasting for three days from February 8 through February 10, 1978, produced extensive floods, flash floods, mudflows/debris flows, and landslides. Several burned areas north of Los Angeles in the path of the storm cell produced disastrous debris flows and flash floods. The most spectacular one, Hidden Springs, will be used for illustration purposes herein, but other areas experienced similar devastation. The following discussion of events from the (National Oceanic and Atmospheric Administration) report entitled "Report on the Southern California Floods, Flash Floods, and Mudslides of February 8-10, 1978" (26) provides a reasonable account of the amount of rainfall as well as the effects of the storm, particularly in Mill Creek on which Hidden Springs is located (see Figure 4).

"A monstrous storm, one of the worst in recent southern California history, brought death and destruction to the Los Angeles area and neighboring counties on February 8-10, 1978... "An estimated 700 persons were driven from their homes throughout southern California. At least 100 houses were damaged by mud and rock slides in the Sunland area alone. Entire neighborhoods were isolated by either road closures or floods, several bridges were washed out, and dozens of schools were closed. Power outages affected more than 500,000 people. Twenty lives were lost. Damage exceeded $83 million ($43 million in the Los Angeles area and another $40 million in the San Joaquin Valley was agricultural. Eight counties were declared Federal disaster areas..."
INTENSITY OF RAINFALL, STORM OF 2-10-78

FIGURE 3: REGIONAL MAP AND STORM PATH OF FEBRUARY 10, 1978
DESCRIPTION AND BEHAVIOR OF DEBRIS FLOWS

RAINFALL GAGING STATIONS

FIGURE 4. MIDDLE FORK AND MILL CREEK WATERSHEDS ABOVE HIDDEN SPRINGS
"Late on Wednesday, February 8, heavy rains began over Los Angeles Basin and adjacent counties to the north and continued until the morning of Friday the 10th. Nearly 4 inches fell at Los Angeles Civic Center, but much heavier amounts fell in surrounding hills and mountains. Mt. Wilson, Lake Arrowhead, and Tujunga Canyon reported as much as 12-16 inches... Although the ground was saturated and reservoirs were full from earlier rains (Los Angeles had received 16 inches of rain from the winter season up to February 7, double the normal for the date and greater than the seasonal normal of 14 inches), the key to the flooding was the extremely heavy, short duration of rainfall on February 10. For example, Haines Canyon in the Tujunga Drainage recorded 1.4 inches in 5 minutes at 1:30 a.m. on the 10th. Bakersfield received 3.00 inches in 24 hours— the greatest such total in the 100-year record. As a result, there was widespread flooding, some flash flooding, and mudslides. However, most of the rainfall amounts were at or below those associated with the '10-year storm.' Even the 9-inch in 24-hour, the 3.9-inch in 6-hour, and 1.6 inch in 1-hour rainfalls near Hidden Springs were within the limits of the expected 10-year storm. This means that, given the proper antecedent conditions, similar flooding could be expected several times during an average lifetime.

"Hardest hit was the tiny community of Hidden Springs about 20 miles north of downtown Los Angeles in a canyon of the San Gabriel Mountains. Hidden Springs is a resort/fishing village located on Mill Creek. At 2 a.m. on Friday the 10th (all times PST), a 15-foot wall of water described as a 'big wave' swept over the community carrying 13 residents to their death. Ten of these died when the wave hit a lodge located on Mill Creek. A fire had broken out in the lodge and the volunteer fire department, consisting of several men and a pumper, were fighting the fire when the 'big wave' hit. The pumper was found four miles downstream several days later. Three people were swept to their death when a nearby triplex was hit by the wave.

"To indicate the short duration of the wave, one man who was trapped in the wreckage of the lodge was not drowned. Survivors indicated the wave rose in seconds and subsided in seconds sweeping everything before it—houses, cars, trucks, and people. They said they had never seen Mill Creek rise so fast.

"The sudden onslaught of this 'big wave' suggests temporary damming upstream from Hidden Springs at the confluence of Middle Fork and Mill Creek. Middle Fork flows through two culverts under the Angeles Forest Highway before joining Mill Creek. Local residents indicated one culvert and a section of road were washed out as water flooded over the highway, possibly contributing to the 'big wave' that surged down Mill Creek. The watershed above Middle Fork had been extensively burned the previous August and most likely this led to the rapid runoff and accumulation of debris beyond the culvert. It is also possible, of course, that debris dams formed
on Mill Creek above Middle Fork, although there did not appear to be any damage a short distance above the junction. Another factor contributing to the damage was altering of the Mill Creek streambed. The stream meanders a bit in the canyon bottom and had been "bowed out" in some places to make more level ground available for building. Of course, when the flood came the stream tended to follow the more direct path, destroying man-made objects in its way."

The following meteorological and topographic factors influenced precipitation and runoff in the Middle Fork and Mill Creek watersheds:

1. Figure 3 shows the storm cell path of the high intensity rainfall that was the source of torrential precipitation causing the flood (26).
2. This storm cell moved from the Santa Monica Bay area north-easterly and was forced to rise as it passed over the high ridge northerly of Mill Creek (maximum elevation about 6000 feet (1830 meters). Topographic control or orographic effects brought about an increase in precipitation over the Mill Creek area.
3. The high ridge area of Mount Gleason may have caused a stalling of the storm further increasing precipitation.
4. Air temperatures dropped below 32°F (0°C) in the vicinity of Mount Gleason.
5. The watershed of the Middle Fork of Mill Creek consisted of 2440 acres (10 km²) relatively steep canyon and ridge topography with natural slopes averaging 20° to 30° but with some portions steeper than 45°. (see Figure 4).
6. Flow from the 2440 acre (10 km²) watershed was concentrated in the narrow, relatively steep gradient channel of the Middle Fork of Mill Creek. Middle Fork joined the main channel of Mill Creek approximately one half mile (0.8 km) above the village of Hidden Springs.

Further complicating the runoff factor of the Middle Fork of Mill Creek is the highly sheared, coarse-grained nature of the bedrock and overlying soil. The bedrock is a highly sheared cataclastic anorthosite, which had been intensely fractured by past faulting. The soil produced is a coarse-grained gravel with some clay fines. Most of the gravel-sized fragments are individually nearly rectangular-shaped feldspar crystals. Soil thicknesses range from 6 to 10 inches (15.2 to 45.7 cm) on hillside slopes. These soils are very erodible under non-burned conditions and are subject to a dramatic increase in erodibility following a fire.
Findings At Mill Creek

Review of the affected area, discussion with Wells and others (1982), research of fire/flood storm patterns, topography, and rock/soil type(s) strongly infer that:

1. At least 8 to 10 inches (203 to 254 mm) of precipitation had fallen prior to the peak 1-hour rainfall at 1:00 a.m. on the 10th of February. A peak precipitation of about 1.5"/hr. (33 mm/hr.) with a 15 minute peak of 0.7 inches (18 mm) occurred between 1:00 a.m. and 2:00 a.m. (see Figure 5). Although this was a high intensity storm, it was considered a 10-year storm for the area.

2. The high intensity precipitation between 1:00 a.m. and 2:00 a.m. on February 10, 1978 caused a quick increase in runoff which was unimpeded by vegetation and litter. This sudden increase in runoff, in turn, caused a dramatic increase in velocity and bulking of the sediment-laden flow. Supportive evidence of this increase in velocity was shown by the viewing of small boulders 2 to 5 feet (.6 to 1.5 meters) in diameter which were put into motion by the high velocity stream flow. Data by Aimoto (2) and others (15, 23) indicate the velocity necessary to move these boulders to be at least 20 to 30 feet/second (6.2 to 9.2 meters/sec.). Additional sediment in suspension increased the density of the fluid causing greater buoyancy and erosion. The erosion rate was dramatically increased as a result of the fire/flood factor causing the bulking factor to be 200 percent to 500 percent.

3. Approximately 380,000 c.y. (290,488 cm) of sediment was eroded from the 2440 acre (10 km²) watershed during the short period of high intensity rainfall constituting more than double the maximum sediment production, as per Rowe (29), for the first year after a burn.

4. The runoff during peak rainfall was 100 percent of rainfall plus up to 500 percent bulking. This surge of sediment-laden flow was added directly to the nearly full bank flow creating a "flash flood" which was superimposed on the existing flow in Mill Creek causing a quick rise of the water level to flood stage. The abrupt increase in flood conditions caused a sudden wall of water (wave bore) and a series of roll waves (11) gravity waves (5,6,9,) that inundated the small community of Hidden Springs. The first wave bore or surge reached Hidden Springs approximately 30 minutes after the peak rainfall fell in the Middle Fork.
DESCRIPTION AND BEHAVIOR OF DEBRIS FLOWS

L.A.C.F.C.D. RAINFALL GAGES

FIGURE 5. RAINFALL HYDROGRAPH - FEBRUARY 8-10, 1978
Conclusive evidence showed that there was no damming effect with ensuing failure of the dam (highway) as is suggested by the NOAA report (26). Additionally, the roadway of Angeles Forest Highway was not washed out, and the culverts were in-place following the flood. The high intensity rainfall and related "flash flood" was a flood event caused and controlled by the natural setting, geologic environs, rainfall/storm cell patterns, fire/flood sequence, and the dramatic increase in erosion and runoff attributable to these factors. The structures damaged and people killed were located within the flood plain of Mill Creek.

OTHER AREAS

Other San Gabriel Mountain areas that experienced fires followed by major mudflow caused damages included the Schwartz and Zachau Canyon areas. Refer to Figure 3. Debris basins in both of these canyons were overwhelmed by the magnitude of debris generated by runoff from the February 8-10, 1978 storm. Both 'drainage' areas were burned in the Mill fire of November 1975. Following the fire were three relatively dry years; consequently, vegetation had not reestablished itself well. The rainfall in these areas had an intensity about twice that of the Hidden Springs area, but these were much smaller watersheds. The bulking factor was in the order of 300 to 500 percent with spectacularly large pieces of stone being transported to the debris basins. Because the material generated by the fire/flood sequencing exceeded the storage capacity of the basins, some material passed over the dams and on down stream into residential areas. However, the debris was of the finer fraction, and the damage was much less than would have been expected if the basins had not been installed.
REFERENCES/BIBLIOGRAPHY


31. San Dimas Experimental Forest Staff, "Fire-Flood Sequences... on the San Dimas Experimental Forest," California Forest and Range Experiment Station, U.S. Department of Agriculture, Technical Paper #6, 1954

ABSTRACT

During the early 1970's, the U.S. Geological Survey, in cooperation with the Nevada Bureau of Mines and Geology, initiated a project of environmental and geologic mapping of 7-1/2-minute topographic quadrangles in Nevada. Part of this project involved the delineation of geohydrologic hazards including those related to water-borne flood debris. Flood-magnitude and flood-frequency relations were determined using basin characteristics, streamflow measurements, channel geometry, and available streamflow records for nearby streams. These relations were in turn used to estimate the magnitude of the 100-year flood. (The 100-year recurrence-interval flood is the one most commonly used by local planners for flood-hazard prediction and planning.) Onsite field assessment of debris hazards was related to specific characteristics of the drainage basin and to topography of probable depositional areas. Debris-hazard areas were generally delineated as severe (many large boulders), moderate (a few boulders, but most fine-grained sediment), or light (almost all fine-grained sediment).

The Washoe City Quadrangle, along the east front of the Sierra Nevada between Reno and Carson City, was the first quadrangle mapped. Flood and related debris-flow hazards along Ophir Creek, a drainage basin of about 4 square miles, were evaluated as part of this effort. A peak flow of 2,000 cubic feet per second for the 100-year flood was estimated for the creek near its canyon mouth, less than 1/2 mile above an already existing real-estate development. The map also delineated debris-hazard areas downstream from the flow-estimate site, in the area where housing developments would most likely increase in the future.

No flooding occurred for about a decade after the mapping. But about noon on May 30, 1983—a clear hot day following several days of intensive melting of a record snowpack—a mass of rock and finer grained sediment with associated vegetal cover and snowpack, of 40- to 50-acre extent, moved quickly down the steep southeast face of Slide Mountain. Some of this mass slide into Upper Price Lake, a small 4- to 5-acre pond on upper Ophir Creek. The rapid debris movement into the lake displaced the lake contents and those of a much smaller, adjacent downstream pond, and swept the cumulative

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contents (20-30 acre-feet), down the steep canyon of Ophir Creek (25-percent grade). This flood wave gouged debris from the canyon floor and walls, and increased in momentum as it gained mass downstream. After 8 to 9 minutes of travel time at an average velocity of 18 to 20 miles per hour (25-30 ft/sec), the mass arrived at the site of the flow estimate of a decade earlier. The approximately 30-foot-high flood wave of fluid debris is estimated to have had a flow rate of about 50,000 cubic feet per second—25 times greater than the previously estimated magnitude of a 100-year flood. Downstream, the mass killed one person, injured several others, destroyed or severely damaged five homes, buried a highway to a maximum depth of 9 feet, and damaged or destroyed considerable other property (including several vehicles and some livestock). Heaviest damage was caused by debris that included numerous boulders having diameters as great as 12 feet.

The disaster has allowed comparisons between the predicted and actual consequences of flooding and related debris movements. Consequences of the rapid release of a relatively small quantity of impounded water also improved both the general knowledge of flash flooding and the understanding of hazards resulting from failure of small dams.
Debris Flow Theory

MECHANISMS ASSOCIATED WITH UTAH'S 1983 SLIDES AND DEBRIS FLOWS

by Roland W. Jeppson

PRECIPITATION

Climate of Utah

During approximately three months in the spring of 1983, the State of Utah, with a population of approximately 2 million people sustained direct damages in excess of 250 million dollars from landslides, debris flows, debris floods and flooding. These natural disasters were so widespread and extensive that 22 of the 28 counties of the State were declared national disaster areas. To fully understand the significance of these events it is important to understand the typical climate of Utah.

Utah lies in a relative arid portion of the western United States, and many portions of the State have chronic water shortages. The mean annual precipitation, when spread uniformly over the entire state, is 13 inches. The average for the U.S. is about 30 inches. Therefore, during the average year Utah's precipitation is about one-third of the national average and is considerably less than the potential evapotranspiration which varies from slightly less than 18 inches in the high mountains to over 36 inches in the desert areas. Utah's topography of mountains and valleys, as well as other climatic factors, cause abrupt differences in precipitation amounts. This abrupt variation is exemplified by contracting the amount of precipitation on the valley floor some 40 miles west of Salt Lake City with that on the headwater areas of Little Cottonwood Canyon, 20 miles south and east of Salt Lake City. The valley floor at an elevation of 4,100 feet receives only 4 to 5 inches during the average year, whereas, the high Little Cottonwood Canyon area at an

1This research was done through an Agreement between Utah State University and Forest Sciences, Intermountain Forest and Range Experiment Station, Contract/Grant No. 12-11-204-3. Some additional financial support was provided by the National Science Foundation.

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elevation of 11,000 feet receives 50 inches.

In addition to rapid areal variations, large seasonal variations are caused by two main factors: (1) the topography and (2) the changes in general atmospheric circulations. Winter precipitation is associated mainly with large frontal air masses that originate in the Pacific Ocean and sweep across the State from the northwest. As these moist air masses are lifted by the mountains, the moisture is deposited. These frontal systems are most vigorous during the winter months and are the dominate influence on the northern Utah climate, but are less important in southern Utah. Generally, the principal source of summer moisture is the northward flow of warm moist air masses from the Gulf of Mexico. These movements create thunderstorms that may occur in any section of the State, but generally the areas of heaviest activity lie along the southern slopes of the Uinta Mountains and the eastern slopes of the Wasatch Range, with an area of extensive thunderstorm activity throughout southeastern Utah. These summer storms are reduced in frequency and intensity on the western slopes of the Wasatch Range and in the northwestern part of Utah.

Figure 1, taken from the Hydrologic Atlas of Utah (Jeppson et al., 1968), shows 30 year average monthly precipitation amounts for the natural climatic zones that have been designated by ESSA Weather Bureau. Note from this figure that the northern portions of Utah in the North Central Zone and the Northern Mountains Zone receive more precipitation during the winter months and the least during the summer months of July, August and September, whereas southern Utah in the Dixie, South Central and Southwest Zones receives as much or more precipitation during the summer months as during winter months. The northward flow of Gulf of Mexico storms create these summer storms which also effect the Uinta Mountains, or eastern portions of the Northern Mountains Zone.

The Wasatch Front in the Northern Mountains Zone and the Wasatch Plateau in the northern portion of the South Central Zone experienced the extensive slides and flooding during the spring of 1983. The eastern portion of the North Central Zone contains the vast majority of the population of the State in the valleys at the toe of the Wasatch Front Mountains.

Analysis of Precipitation Data

The floods and debris flows during 1983 originated from the Wasatch Front and Wasatch Plateau Mountains that traverse approximately parallel to the line dividing the North Central and Northern Mountains Zones down into the South Central Zone of Figure 1. None of the climatic zones provide ideal coverage of these mountain areas hath because the dividing lines between the zones do not delineate the area in question, and the majority of the weather records are in the valleys and not the mountains.
Figure 1. Average monthly precipitation for natural climatic zones in Utah.
The North Central Zone best represents the accumulations of precipitations immediately prior to and causing the flooding in the Spring of 1983, despite that its coverage of the actual area producing the floods is less than desirable since the gages are in the valleys west of the mountain front.

Figure 2 contains a bar graph of average precipitation amounts using all stations in this zone for water years, October 1 through September of the designated year, over the 52 year period from 1932 through 1983. Lines on this figure also give the average, or mean precipitation, over this 52 year period and confidence band that contains 90 percent of the possible events assuming they follow a normal distribution. Thus there is a 5 percent probability that an event will be larger than the top dashed line on this graph and a 5 percent chance that an event will be smaller than the lower such dashed line. This confidence band was obtained by adding and subtracting 1.65 times the standard deviation from the mean of these same data.

An examination of Figure 2 shows that average accumulated precipitation during the 1983 water year is the largest in this 52 year record. The next largest is 1982. The manner in which the precipitation accumulated during these two consecutive record setting water years is significant. By February, the 1982 water year had accumulated the largest amount of precipitation by this date in the 52 year record. Year 1982 retained the distinction of the largest in the record through the end of March, at which time 1983 rose from the 9th largest to the 3rd largest in the record. Only 1982 and 1952 exceeded it in amount. The above normal amounts of precipitation during April of 1983 placed the 1983 water year as the largest in the record from this date through the remaining months of the 1983 water year.

In examining the amounts of precipitation that actually occurred during any designated month rather than the precipitation accumulation from October 1 to that date, it is interesting to note that years 1982 and 1983 did not dominate as consistently largest precipitation months. Only during March, July and September did 1982 produce the largest amount of precipitation and the the month of August, 1983 produced the largest amount. However, during the first 8 months of the 1983 water year through May, the ranking of 1983 and its probabilities of being exceeded are: 10(.192), 21(.404), 14(.269), 32(.615), 12(.231), 4(.077), 13(.250), and 6(.115) respectively. What these results indicate is that during any given month the amount of precipitation during the 1983 water year was not unusually large, but rather only modestly above normal. However, consistent month after month above normal precipitation resulted in accumulated record large amounts. During most years, record large monthly precipitation amounts are likely preceded and/or followed by less than normal monthly amounts. This trend of month after month of modestly above normal precipitation, with a few record setting months was not limited to
DEBRIS FLOW THEORY

FIG. 2. ANNUAL PRECIP. AVERAGED FOR STATIONS IN THE NORTH CENTRAL ZONE.
just the North Central Zone but extended throughout most of the State.

The conclusion that can be drawn from the precipitation records is that taken on a month by month basis the precipitation preceding the Spring of 1983 was not particularly unusual. Of significance is that the wet cycle persisted month after month. This persistency is extending thus far into the 1984 water year, and its effect is evident by other indicators of the climate including the unprecedented rise in the level of Great Salt Lake. Obviously the carryover effect of a preceding 1982 high water year also contributed to the occurrence of 1983.

Another possibly even more significant factor in causing the widespread slides, debris flows and floods of 1983 was that temperatures remained unseasonably cool during the spring until toward the latter part of May. Then suddenly temperatures abruptly returned to normal or above, resulting in unusually high rates of snowmelt. The snowpacks were unusually ripe at this time having just accumulated the precipitation from several heavy spring storms. The result was that snowlines rapidly receded up the mountain slopes in late May and early June. Sequential video tapes by the news media showing the mountain snowline reveal this rapid upward recession at about the elevations where the mountain slides occurred that resulted in the extensive damages to such towns and cities as Farmington and Bountiful at the toe of the mountains. It was the unusually rapid application of water from melting snows to mountain soils already containing more than normal water contents that resulted in the many shallow slope failures.

SLOPE INSTABILITY, THE ORIGINATOR OF DEBRIS FLOWS

Mountain Slides of 1983

The extensive damages to towns and cities during the 1983 spring floods were due more to mountain landslides in creating large water surges accompanied by large amounts of debris if not by a debris flow directly. In larger streams the slides created temporary dams that upon failing sent surges of water and earthen materials downstream. Roadway fills, whose culverts had insufficient capacities, were washed out and added to the debris. Such a large number of landslides over a very short time period have not been experienced in the Great Basin since man settled the area. From infrared photo that were made from flights that covered the entire west facing slopes of the Wasatch Mountains, Robert Pack, a graduate student at USU inventoried over 90 significant mountain landslides. Figure 3 identifies those in the area from Farmington south
Figure 3. Distribution of "spring 1963" landslides from Bountiful to Farmington, Utah.
to Bountiful. Many more, probably numbering as many as 1000, occurred in the Wasatch Plateau Mountains of central Utah.

A typical 1983 slide might be referred to as a shallow soil slip, and is characterized as a debris slide. The depth to the failure plane is around ten feet or less. The slides that were examined by field visits have failure plane at or slightly above the soil-bedrock interface. Landslides in the slide areas vary between 29 and 46 degrees from the horizontal. From slides that were visited a thin zone of silty and clayey soil below sand and gravel has been identified as the shear surface. Mica-rich rocks are abundant in the slide areas. Rhotite schist appears to be associated with many, if not most, of the slide materials. A slide in Holbrook Canyon, east of Bountiful, and identified as # B4 on Figure 3 show that the failure surface was a clayey layer that weathered from muscovite-rich pegmatite bedrock to form a weaker shear plane. This material is classified as CL by the Unified Soil Classification System. Laboratory tests on disturbed samples of material taken adjacent to failure surfaces of some of these slides are given in Tables 1 and 2. Table 1 provides a description of sites where the samples were obtained as well as the plastic and liquid limits under the column of Atterberg limits, and Table 2 provides the grain size distributions for the samples identified in the first column of Table 1. The average grain size distribution, with plus and minus standard deviations from such determination from 65 soil samples from the mountain slide areas are shown in Figure 4. Note the soil samples contain only a small amount of clay.

Soil Water's Influence on Stability of Slopes

Masses of soil on inclined surfaces are subjected to shearing stresses on nearly all internal surfaces because gravitational forces act to force surfaces to be horizontal as occurs in a body of water or other Newtonian fluids which moves continuously in response to shear stresses. When the shear stresses exceed the soil's resistance to shear over some sufficiently large internal area, then failure occurs. Methods for analysis of the stability of slopes against failure are included in books dealing with Soil Engineering, and require that the materials internal friction angle \( \phi \) and its cohesion \( c \) be known.

Soil water is the variable that causes an otherwise stable slope to fail. The reduction in stability by soil water is due to two important effects: 1. the introduction of seepage forces in the downslope direction, and 2. the reduction or complete loss of the soil's cohesive strength or its apparent cohesive strength.

The effect of seepage forces is well understood and can be quantified in engineering slope stability analyses if a solution of the groundwater flow system is available. The second effect is less well understood and
Table 1: Description of sites where soil samples were collected from and Atterburg limits.

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>ATTERBURG LIMITS</th>
<th>DESCRIPTION WHERE SAMPLE WAS TAKEN FROM</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SAMPLES TAKEN FROM THE RICK CREEK SLIDE AREA (JUNE 24, 1983)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RC-1</td>
<td>31.54</td>
<td>Collected 5 ft from scarp at head, approximately 7-8 ft vertical distance from the original ground surface clay material overlying bedrock.</td>
</tr>
<tr>
<td>RC-2</td>
<td>35.59</td>
<td>Soil collected from debris 30 ft below the head of the slide.</td>
</tr>
<tr>
<td>RC-A1</td>
<td>28.42</td>
<td>Organic layer collected from south side of slide about 10 ft below the head of the slide.</td>
</tr>
<tr>
<td>RC-A2</td>
<td>30.50</td>
<td>Collected 6 ft below the ground surface on the scarp face. A silty-clay sand with few rocks in it. This sample is also stained red due to iron oxidation. Highly weathered gneissic bedrock is present just below the sample depth. In place dry bulk density determined as 1.45 g/cc.</td>
</tr>
<tr>
<td><strong>SAMPLES TAKEN FROM THE NORTH HARD CANYON SLIDE AREA (LOWER PART OF NORTH FORK OF CANYON) (JUNE 23, 1983)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NR-1</td>
<td>20.01</td>
<td>Collected 10 ft below scarp head within 3 inches of bedrock which is extensively weathered banded gneiss.</td>
</tr>
<tr>
<td>NR-2</td>
<td></td>
<td>Collected from the west flank, 40 yard from the head of the slide in the debris outflow.</td>
</tr>
<tr>
<td>NR-3</td>
<td></td>
<td>Collected 8 ft below original ground surface on the East side of the scarp.</td>
</tr>
<tr>
<td><strong>SAMPLES TAKEN FROM THE FARMINGTON CANYON SLIDE AREA (JUNE 17, 1983)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FC-1</td>
<td>24.01</td>
<td>Collected from lower west slide, soil from slide face.</td>
</tr>
<tr>
<td>FC-2</td>
<td>24.06</td>
<td>Collected from the lower west slide, soil from slide face.</td>
</tr>
<tr>
<td>FC-3</td>
<td>24.56</td>
<td>Collected from lower east slide, at a distance 10 ft horizontally from head of failure, near seepage face.</td>
</tr>
<tr>
<td>FC-4</td>
<td>24.85</td>
<td>Collected from lower east slide, soil collected 50 ft horizontal distance from head, near seep.</td>
</tr>
<tr>
<td>FC-A1</td>
<td>25.78</td>
<td>Collected 14 ft below the ground surface at top bottom of the scarp of the north west finger of the slide. This location is about 40 yards below pirometer bank 2A and 2B (horizontal distance), and just below the Farmington lunch spot. No bedrock was observable at this location. Water was seeping out of the bottom of the scarp to the left of the sample site. The sample was a light brown sandy material.</td>
</tr>
<tr>
<td>FC-A2</td>
<td>29.41</td>
<td>Collected 10 ft below the ground surface, 4 ft vertically above sample FC-A1. This sample had more clay in it from its appearance than FC-A1 and a larger number of cobbles, pebbles ranging in size from 1 inch to 3 inches in diameter.</td>
</tr>
<tr>
<td>FC-A3</td>
<td>29.41</td>
<td>Collected 3 ft below ground surface, 1 1/2 ft below top soil (organic layer). A sandy soil with a comparable amount of cobbles and pebbles by appearance to FC-A2, but less clay by appearance.</td>
</tr>
<tr>
<td>FC-81</td>
<td>25.85</td>
<td>Collected 3 ft below the ground surface on the scarp face of the east branch of the slide. The soil was sandy by appearance with pebbles, cobbles and stones ranging up to 8 inches in size. Some of the stones were more angular than observed at other sampling sites. Undisturbed sample was obtained. The in place dry bulk density equals 1.62 g/cc.</td>
</tr>
<tr>
<td>FC-82</td>
<td>28.78</td>
<td>Collected 7 ft below the ground surface. A highly weathered bedrock is exposed sporadically at this level. The soil is sandy by appearance, large stones are absent, pebbles and cobbles are few in number, a general homogeneous soil. Undisturbed sample was obtained. The in place dry bulk density equals 2.21 g/cc.</td>
</tr>
</tbody>
</table>
Table 1. Continued.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>FC-3</td>
<td>Collected 25 ft below the ground surface. The soil is fine grained, probably evolving from the underlying bedrock which is exposed about 4 ft below the sampling site. Undisturbed sample was obtained. The in place dry bulk density equals 2.12 g/cc.</td>
</tr>
<tr>
<td>FC-4</td>
<td>Same location as sample FC-1 but an undisturbed sample was obtained. The in place dry bulk density equals 2.36 g/cc.</td>
</tr>
<tr>
<td>FC-5</td>
<td>Same location as sample FC-2 but an undisturbed sample was obtained. The in place dry bulk density equals 2.00 g/cc.</td>
</tr>
<tr>
<td>FC-6</td>
<td>Same location as sample FC-3 but an undisturbed sample was obtained. The in place dry bulk density equals 2.13 g/cc.</td>
</tr>
</tbody>
</table>

Samples collected from the Ward Canyon Slide Area (June 19, 1983)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>MC-1</td>
<td>Collected from scarp at head, interface between top soil and weathered bedrock.</td>
</tr>
<tr>
<td>MC-2</td>
<td>Collected at head, interface of bedrock.</td>
</tr>
<tr>
<td>MC-3</td>
<td>Tensep from pleurometer bank III.</td>
</tr>
</tbody>
</table>

Samples collected from the Ward Canyon Slide Area (June 18, 1983)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>MC-A1</td>
<td>Organic horizon approximately 2 ft deep. Soil collected approximately 1 ft from original ground surface.</td>
</tr>
<tr>
<td>MC-A2</td>
<td>Sand and rock material collected approximately 4 ft from original ground surface. Sample has same large blocks, and gravel.</td>
</tr>
<tr>
<td>MC-A3</td>
<td>Sandy material collected approximately 8 ft from original ground surface. Nonsiliceous material.</td>
</tr>
<tr>
<td>MC-A4</td>
<td>Weathered bedrock collected approximately 15 ft from original ground surface. At the site where these MC-A series of samples were collected from the Organic horizon was approx. 3 ft thick. For the next 3 ft of depth sand and rocks occurred. For the next 8 ft of depth to 14 ft the material consisted of sand and gravel. The next 5 ft of depth to 19 ft consisted of weathered bedrock producing a sandy-gravely-randy material.</td>
</tr>
</tbody>
</table>

Samples collected from the Debris Deposit in Farimont from the Rudd Creek Debris Flow (June 23, 1983)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Description</th>
</tr>
</thead>
</table>
| PFD-1A | This series of samples were collected on 1E, between 5th North and 4th North. The samples were collected on the west side of the deformed layer of the deposits. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. The deformed layer of the deposits is 1 ft thick. 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Table 1. Continued.

- **Sample FC-81**: Collected 25 ft below the ground surface. The soil is fine grained, probably evolving from the underlying bedrock which is exposed about 4 ft below the sampling sites. Undisturbed sample was obtained. The in-place dry bulk density equals 2.12 g/cc.
- **Sample FC-C1**: Same location as sample FC-A1 but an undisturbed sample was obtained. The in-place dry bulk density equals 2.36 g/cc.
- **Sample FC-C2**: Same location as sample FC-A2 but an undisturbed sample was obtained. The in-place dry bulk density equals 2.00 g/cc.
- **Sample FC-C3**: Same location as sample FC-A3 but an undisturbed sample was obtained. The in-place dry bulk density equals 2.35 g/cc.

- **Samples Collected from the Handy Canyon Slide Area (June 15, 1983)**
- **Sample MC-1**: Collected at the head of the slide, interface between top soil and weathered bedrock.
- **Sample MC-2**: Collected at the head of the slide.
- **Sample MC-3**: Topsoil from piezometer bank (1).

- **Samples Collected from the Handy Canyon Slide Area (June 18, 1983)**
- **Sample MC-A1**: Organic horizon approximately 1 ft deep. Soil collected approximately 1 ft from original ground surface.
- **Sample MC-A2**: Sand and gravel material collected approximately 4 ft from original ground surface. Sample has some large rocks and sand as well.
- **Sample MC-A3**: Sandy material collected approximately 8 ft from original ground surface. Non-cohesive material.
- **Sample MC-A4**: Weathered bedrock collected approximately 15 ft from original ground surface. At the site where these MC-A series of samples were collected from the organic horizon was approximate 3 ft thick. For the next 3 ft of depth sand and rocks occurred. For the next 9 ft of depth to 14 ft the material consisted of sand and gravel. The next 5 ft of depth to 19 ft consisted of weathered bedrock producing a sandy-gravelly-rocky material.

- **Samples Collected from the Debris Deposit in Farmington from the Rued Creek Debris Flow (June 23, 1983)**
- **Sample TDF-1A**: This series of samples were collected on 1E, between 5th North and 6th North. The samples were collected on the west side of the street in vertical sections on each different layer of the deposited debris. The debris was 8 ft thick at this location. 1A was taken 1 ft from the top of the flow. The debris was gray, sandy-silty with cobbles. 1B was taken from the middle layer of the debris flow, 5 ft below the top. The debris was gray and a little coarser than the debris of TDF-1A. 1C was taken from the bottom of the debris flow, 7 ft below the surface of the deposit. The debris at this depth is also gray.
- **Sample TDF-1B**: This set of samples was collected between 5th N and 6th N, and 1st E and 2nd E, on a side street in the city of Farmington. This set is a vertical cross section of the debris flow where it crossed the street. The flow was 10 ft from top to bottom. Three slightly different layers of debris were present; one sample was taken from each. 2A was taken in the top layer of debris, 1 ft below the surface. A fine grained sand with numerous cobbles and organic material existed here. 2B was taken in the middle layer, 6 ft from the top surface. An organic black debris with cobbles existed here. 2C was taken from the bottom of the debris deposit. A black organic debris with cobbles was present at this depth.
- **Sample TDF-1C**: This set of samples was collected between 4th N and 5th N and Main Street and 1st East. The sample set was taken from a vertical cross section next to a house where the debris had been excavated. This sample set was the farthest from the mouth of Rued Creek, and thinnest in depth being about 3 feet thick. Layering was very difficult to determine. 3A was taken 1 ft below the surface. A gray debris with pebbles and cobbles existed at this depth. 3B was taken 2 1/2 ft below the surface. A gray debris similar to that at 3A existed here. 3C was taken 3 1/2 ft below the surface.
- **Sample TDF-4A**: These samples were collected at the outlet of Rued Creek into Farmington. 4A was taken from the front of the first debris flow at the top of the city water tank. 4B was taken from the final debris flow.
Table 1. Continued

| Samples Taken from Various Positions and Depths Within the Farmington Debris Deposit |  
| FD-1 | 22.1 | This series of samples, i.e. FD-1 through FD-7 are taken from different locations and depths within the debris fan deposited within the north-east portion of the city or Farmington |
| FD-2 | 23.7 |  
| FD-4 | 23.0 |  
| FD-5 | 25.0 |  
| FD-6 |  
| FD-7 |  

<p>| Samples Taken from the Willard Debris Flow (June 8, 1983) |
| NCZ-1 | Collected from the 4th debris flow in the top materials. |
| Table C. Laboratory determined grain size distributions for 65 soil samples taken from 1983 slide areas or debris flows. Soil sample designation is identified in column 1 of Table 1. |
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FIGURE 4. AVERAGE GRAIN SIZE DISTRIBUTION CURVE FROM 65 SOIL SAMPLES TAKEN FROM THE 1983 MOUNTAIN SLIDE AREAS.
more difficult to quantify but is believed to be of considerable significance in the shallow mountain slides of 1983. It is associated with the surface tension of water, and that under unsaturated conditions the soil water is under negative pressures equal to several atmospheres if the soil is dry. These negative pressures result in soil particles contacting adjacent soil particles with sizable normal stresses, and result in what might be thought to be cohesion. When the soil becomes saturated the negative pressures disappears and are replaced by positive pressures, and this apparent cohesion of the unsaturated material becomes zero. Both the effect of capillary soil water tension and seepage forces are discussed below.

The stability of a slope is reduced by seepage forces because the shear strength must resist the dissipation of the hydraulic gradient $dh/ds$ causing the water flow as well as gravity's downward pull on it. A simple example of a cohesionless soil on an infinite two dimensional slope helps illustrate this reduction in stability. In the sketch below the forces on an element within such a infinite slope with an angle $\alpha$ from the horizontal is shown. In this illustration assume the soil is saturated to the surface so the element's buoyant weight is $W_b = \gamma_b \cdot s \cdot d \cdot \cos \alpha$, $\gamma_b$ is the buoyant specific weight and equals the total specific weight $\gamma$ of the mass including the water in its voids minus the specific weight $\gamma_s$ of the water or $\gamma_b = \gamma - \gamma_s$. The normal force $N = W_b \cdot \cos \alpha = \gamma_b \cdot s \cdot d \cdot \cos \alpha$. The seepage force equals $s \cdot d \cdot (dh/ds)$ for a constant hydraulic gradient $dh/ds$ throughout the area $s \cdot d$. For the above case with the soil saturated to the surface the hydraulic gradient $dh/ds = \gamma \cdot \sin \alpha$. Consequently, the driving forces are the sum of this seepage force plus the component of the buoyant weight in the direction of the slope or

$$\text{driving force downslope} = s \cdot d \cdot \gamma \cdot \sin \alpha + \gamma_b \cdot s \cdot d \cdot \sin \alpha \cdot \cos \alpha.$$ (1)

This driving force must be resisted by the force $F$ which at incipient motion equals the normal force $N$ times the tangent of the internal friction angle $\phi$, or
\[ F = sd' \cos \phi \tan \phi = sd'y_w \sin \alpha + \gamma_b sd \sin \alpha \quad \ldots \quad (2) \]

After simplification Eq. 2 becomes,

\[ \tan \phi = \left( \frac{\gamma_b}{\gamma_w} \right) \tan \alpha \quad \ldots \quad \ldots \quad (3) \]

To illustrate the above described significance that seepage forces have in reducing the stability of a slope, assume that a soil typical of the slide soils is cohesionless and on an infinite slope. It has an internal friction angle of 40°. Without seepage forces this material would be stable on slopes up to 40° from the horizontal. The specific gravity of the soil grains equals 2.65 and its porosity (volume of voids divided by total volume) equals 40 percent. The soil's specific dry weight then equals 0.6 \times 2.65 \times 62.4 = 99.2 \text{lb/ft}^3, and its total saturated weight is \( \gamma_c = 99.2 + 0.4(62.4) = 124.2 \text{ lb/ft}^3 \). From Equation 3 the maximum angle that this soil could be placed on without failure equals,

\[ \alpha = \tan^{-1} \left( \frac{\gamma_b}{\gamma_w} \tan \phi \right) = \tan^{-1} \left( \frac{(124.2-62.4)}{124.2} \tan 40^\circ \right) \]

or a reduction of 17.3 degrees. Since local hydraulic gradients in saturated mountain swales would be expected to be larger than the sine of the slope angle, \( \alpha \) further reduction of the slope would be needed for stability of this soil.

Typical slopes in the mountains along the Wasatch Front in Davis county, where the 1983 slides occurred are 30 degrees or more from the horizontal. Without added stability from plant roots, cohesion or three-dimensional stabilizing effects, failure will occur in these areas upon being saturated. A reasonable conclusion is that during normal snowmelt periods the drainage systems and intake capacity of soils in these mountains are adequate so that most of the volume remains in an unsaturated state, even during the most rapid snowmelt.

The second important effect that was referred to previously is that as a soil is saturated it losses apparent cohesion as negative pressure of the soil water reduces to zero. This effect is not very significant in large slides because the normal stresses due to the deep overburden are large, but for shallow soil slips becomes more important. The relationship of this negative pore pressure to the unsaturated water content of the soil depends upon the distribution of void interstices. As the water content of the soil is reduced the water recedes to ever smaller interstices, has smaller radii of curvature in wetting the soil and its pressure reduces according to the well established equation.
in which \( R_1 \) and \( R_2 \) are radii in orthogonal directions, and \( s \) is the water surface tension (which equals 0.514 \( \text{lb/ft}^2 \) for water at 40°F). These radii cannot be measured and therefore measurements of the soil water tension versus its water content, or degree of saturation are used to establish its pressure-saturation relationship.

The effect of negative water pore pressures probably is not equivalent in all respects to an equal normal stress between soil grains caused by overlying materials in providing it shear strength. However, negative pore pressures do cause soil grains to adhere together and resist motion due to shear forces. This added ability to resist shear forces is a factor in preventing failure of mountain soils on slopes of 40 degrees or larger, and for shallow depths of 10 feet or less may provide an equal or greater ability to resist shear than the weight of the overlying materials especially when it is dry. As the water content rises to complete saturation this additional ability to resist shear is lost. This effect in addition to the reduction of stability of a slope by seepage forces means that saturation of a material causes a considerable loss in its ability to resist failure.

Another effect of saturating a sloping soil mass is buoyancy. Based on simplified theory, water buoyancy of the soil grains does not reduce stability because the actuating forces in the direction of a possible failure surface are reduced proportionately to the normal forces which are based on the buoyant weight under complete saturation and this sloping soil surface completely submerged in a level body of water will stand theoretically at the same angle of repose as will a dry noncohesive soil. This angle equals the internal friction angle \( \phi \) of the soil. However, as any localized movement occurs in the saturated soil, and if the particles tend to form into a more compact configuration, then the normal stresses between soil grains are reduced proportionately to the local increases in water pressure and the soil's resistance to shear is reduced. If this process involves considerable amounts of soil mass and is visible by a reduction in soil volume after failure it is referred to as liquefaction. Such localized occurrences of positive pore pressures help explain why shallow mountain slides often are transformed into debris flows that exhibit properties more like a fluid than a solid.

These combined considerations reinforce the conclusion that mountain soils in the 1983 slide areas, especially when they exist at slopes angles of 40 degrees and larger are not completely saturated, and will fail should complete saturation occur.
MEASURED AND SURMISED MOUNTAIN GROUNDWATER CONDITIONS

**Piezometer Data**

Groundwater in the mountain watersheds not only plays the dominate role in instability of slopes and is the variable causing debris flows, but it is the source of the vast majority of streamflow. The exceptions are rainfall on stream surfaces during high intensity rainfall, or very rapid snowmelt when the rates of application exceeds the infiltration capacity of the soil in local areas, and the overland flow does not subsequently infiltrate into the soil before reaching a stream. No information was available about mountain groundwater conditions in slide areas prior to, or during, the spring of 1983. To acquire some information about these conditions, a number of hand driven piezometers were installed in six slide areas at several depths and locations in a crash program during the first half of June 1983. Table 2 identifies these installations. It was recognized at the time of installation that the most interesting groundwater occurrences had already taken place, but it was hoped that these piezometers would provide some information about hillside groundwater recession. At this time snowpack still closed the mountain roads to vehicle access, and with limited budgets it was practical to utilize only very limited helicopter services. Therefore, these installations required hiking several miles over the steep rugged terrain carrying the piezometer pipe, driving equipment, etc. These conditions also limited the size, type and number of soil samples that could be brought out for testing. Furthermore, the installations of these piezometers were not guided by any prior knowledge of strata most appropriate to monitor. The piezometers were installed in nests of about 3 at different depths with the deepest one at the greatest depth allowed by the hand driver and the resistance encountered. Since drilling did not occur there was no way to seal the piezometers from making connections with adjacent layers to where their tips were, and therefore it is not certain whether the readings represent the groundwater conditions at the piezometer's tips, or some composite mountain water table level. These piezometers were visited periodically to record their water table levels or to determine whether any water table level did exit within the depth of the installation. Small pieces of foam were placed in plexiglass tubes that were inserted in the piezometer pipes hoping that they would adequately adhere to these tubes and clearly mark the highest level of water reached in that piezometer since the last time it was visited. However, this method of recording high water table levels proved to be unreliable.

Despite these limitations the recorded water table levels in these piezometers, where any water tables were found, indicate that a complex groundwater system exists in the mountain slide areas. The majority of the piezometers from their installation throughout the remainder of the
spring and summer showed no water tables. This fact supports the earlier conclusion that saturated conditions to the surface of these mountain soils is rare. Rather the water movement in these materials is dominantly unsaturated flow. Table 3 provides the depths from the surface to the water table from these piezometer readings. The second column of these data provide the date of the reading and the third column gives the time of the day when the reading was taken. Negative values for water table depths indicate that the water level is above the ground surface.

The slide identified as #74 on Figure 3, in Ward Canyon provides some interesting water table levels. The piezometer identified as 1A in Table 3 showed a drop of 3.11 feet on June 21, the first time it was visited after being installed a couple of days earlier. It was visited by two different field crews that day, the first time at 10 a.m., and later at 4 p.m. Because of this rapid drop in the water table in the afternoon a crew monitored its water level every one-half hour on June 30. At this later date a drop of 0.45 feet was recorded from 11 a.m. to 2:30 p.m. It is believed these rapid fluctuations in groundwater level are associated with rather small changes in total water content in the profiles, and are caused by relatively small time dependent changes in the groundwater flow, rather than large differences. However, there likely is a rapid spatial variation in the hydraulic properties of the materials. Unsaturated flow phenomena in heterogeneous media, as described later, can explain these recorded changes.

In late summer on September 5 after rainstorms, another piezometer, #4A in Ward Canyon, at a depth of 15.5 feet and immediately upslope from the slide failure surface, and which had previously been dry when visited, contained an artesian water level 2.40 feet above the ground surface. Yet the near vertical failure surface just downslope from the piezometer showed no emerging water. The land slope here is 30 degrees from the horizontal.

At the same time, September 5, piezometers in the Ricks Creek slide area at all depths were dry or nearly dry, as was the case for most of them since their installation, but water was emerging from numerous seepage faces in the slide area and forming a number of streamlets. Furthermore, the surface upslope from the Ricks Creek slide was wet on September 5, with the appearance that the materials were saturated to the surface. Vegetative growths imparted the same impression. Near identical conditions existed at the Ricks Creek slide area from the time of the piezometer installation. One might suspect that the piezometers terminated in a impervious layer, but the water never came into the piezometers as might be expected if this were the case. Selected slug tests of the piezometers also verifies they were adequately connected to relatively permeable materials. The conclusion is that even in this area that was wet on the surface with evidence of surface flow present particularly in areas exposed by the slope failures, very limited
Table 3. Readings from piezometers installed adjacent to mountain slides during first portion of June 1983. The first column represents the distance in feet from the ground surface to the water table in the piezometer. The second and third columns give the date and time of the reading respectively.

<table>
<thead>
<tr>
<th>Ward Canyon, Piezometer 1A (depth below G.S. = -9.2')</th>
<th>Ward Canyon, Piezometer 3A (depth below G.S. = -2.3')</th>
<th>Farmington, Piezometer 10 (depth below G.S. = -15.3')</th>
<th>Rick Creek, Piezometer 1A (depth below G.S. = -6.2')</th>
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<tr>
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<table>
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**Field Notes:**
1. June 24, 1983, 9:00 a.m.: First portion of June 1983. The first column represents the distance in feet from the ground surface to the water table in the piezometer. The second and third columns give the date and time of the reading respectively.
saturated conditions exist at and near the ground surface. Rather the water is moving under unsaturated conditions.

Without sufficient data giving both the hydraulic properties of the soils and the soil’s water pore pressures these limited recorded water tables are difficult to utilize in fully describing the groundwater flow system. This difficulty arises because hydraulic heads recorded within a flowing system must be interpreted in connection with a full description of the water movements. Otherwise water table levels can be, and usually are, misleading, and will result in incorrect conclusions.

Recent development of instrumentation and micro processors that are capable of performing under the adverse environmental conditions in remote areas of a mountain watershed over several months now allow a comprehensive in-situ pore pressure monitoring program to be undertaken. Their installation, both because of the costs of drilling in remote mountain locations, and the number necessary to fully describe the flow system, will be very costly, however.

Solutions to Unsaturated One-Dimensional Flows

The lack of detailed data, giving the hydraulic properties and their variations are not available, and therefore a detailed two- or three-dimensional solution of possible groundwater flow systems in these slide areas would of necessity be academic. Rather than do this in the following paragraphs some solutions to over simplified situations are provided to point out the need for much more comprehensive data.

The solution of unsaturated flow in porous media is governed by Darcy's Law, as is saturated flow. However, the hydraulic conductivity now becomes a function of the soil water content. The common method is to define the hydraulic conductivity as the product of the saturated hydraulic conductivity and a relative hydraulic conductivity, $K_r$ that is a function of soil water pressure (tension), or

$$K = K_s K_r$$

Therefore, relationships are needed between the water content, or degree of saturation, and the relative hydraulic conductivity, $K_r$ with the soil water pressure. A number of parameterized empirical equations have been proposed to define these relationships. For use herein, the Brooks-Corey equations are selected among other reasons because of their simplicity. These equations are:
\[ S_e = (p_b/p)^\lambda \quad \ldots \ldots \quad (6) \]

and

\[ K_r = (p_b/p)^{2+3\lambda} \quad \ldots \ldots \quad (7) \]

in which \( p_b \) is the bubbling pressure, or air entry pressure and is a negative value equal to the tension when a soil first begins to desaturate significantly, \( \lambda \) is a pore size distribution exponent which is large for soils of uniform grain size, and smaller for well graded materials, and \( S_e \) is the effective saturation given by,

\[ S_e = (S - S_r)/(1 - S_r) \quad \ldots \ldots \quad (8) \]

In which \( S_r \) is the residual saturation and represents the saturation a soil will retain under very large negative soil water pressures. To remove this additional water requires oven drying. Equation 7 can be obtained from Equation 6 from the Burdine theory, 1953.

Equations 6 and 7 fit data from the drainage cycle of many soils well, and if the near unity saturation region is ignored, since under imbibition \( p_b \) has no meaning, they also fit the imbibition cycle of many soils reasonably well by using \( p_b \) from the drainage cycle or modifications of it. General time dependent unsaturated groundwater flow problems require numerical solutions of the nonlinear partial differential flow equation that use equations such as 6 and 7 to define needed pressure-saturation and pressure-relative hydraulic conductivity relationships. However, solutions to a number of simplified one-dimensional steady-state situations are easily possible. At most they require the numerical solution of an ordinary differential equation.

Figure 5 provides a solution to steady-state downward infiltration through a homogeneous soil based on knowing the parameters in the Brooks-Corey Equations and the rate of application of water on the surface. For the example on this figure \( \lambda = 1.8, S_r = 0.2 \), and the water is applied on the surface at a rate equal to \( 0.1 \) times the saturated hydraulic conductivity. The soil's saturation \( S \) equals \( 0.64 + 0.2 = 0.84 \). From this saturation and the bubbling pressure \( p_b \) the soil water pressure can be computed.

If the rate of application equals the saturated hydraulic conductivity, i.e., \( v/K_0 = 1 \), then the saturation equals 1 also, regardless of the values of \( \lambda, p_b \), and \( S_r \). The soil water pressure will be atmospheric, or zero, throughout the entire soil column. Piezometers installed in this soil would show no water table at any depth even though
Figure 5. Relationship of the ultimate steady state soil saturation to the rate of infiltration application, the pore size distribution exponent in the Brook's-Corey equations and the residual saturation.
the entire profile is saturated to the surface. An erroneous conclusion might be that the soil is dry. In reality, in this special case the hydraulic gradient exactly equals the gravitational gradient so that there is zero pressure gradient, and the pressure is zero (i.e. established by atmospheric pressure on the surface) throughout the material as shown by the hydraulic head, pressure head and elevation head functions on the right side of Figure 6.

Nonhomogeneities, even if only small, in a soil which contains near steady-state downward flow would result in water tables being recorded at, or above positions where the hydraulic conductivity is smaller. If the smallest hydraulic conductivity occurs at the surface, then the entire profile will be unsaturated unless water is ponded on the surface to some sufficient depth. Erroneous conclusions might be that perched water tables exist in such situations, where the smaller conductivities are below the surface or that the soil is dry if the smallest conductivity is in the surface layers, rather than the true picture that capacity, or near capacity downward flow is occurring through a material with slight variations in hydraulic conductivity. The first portion of Figure 7 illustrates such a situation for which the hydraulic conductivity varies according to

$$K_0 = K_1 (1 + (1/K_2) \sin(2n z/H)) \ldots \ldots (10)$$

in which $K_0$ is the saturated hydraulic conductivity, and $K_1$ and $K_2$ are constants. If it is assumed that the small negative pressures do not cause the soil to desaturate, i.e. they are between pds and 0, then a solution to this flow situation can be obtained in closed for as,

$$h = \frac{q/K_1}{2 \pi} \left[ C_0 \tan^{-1} \left( \frac{\tan(\pi z/H) + 1/K_2}{(1 - (1/K_2)^2)} \right) - C_1 \right] \ldots \ldots (11)$$

in which $C_0$ and $C_1$ are constants and $q/K_1$ is the dimensionless rate of application on the surface. This solution for $h$ as well as the pressure and elevation heads are shown in the latter portion of Figure 7, for $K_2 = 5$. Solutions for other values of $K_2$ are given in Table 4. Note that small positive pressures develop in the upper portion of the soil, but negative pressures occur throughout most of the profile, even though the soil is completely saturated throughout with a ponding to a small depth of 0.021 H feet on the surface.

A solution to a variation of the above simplified problem is given in Table 5 that allows the one-dimensional flow to become unsaturated. The Brooks-Corey equations are used in defining the unsaturated properties of this problem with $p_h = 0.8$ ft of pressure head, $\lambda = 1.5$, and $S_r = 0.15$. The
Figure 6. Downward flow in a homogenous porous media. The rate at application on the surface equals the saturated hydraulic conductivity. All piezometers show zero water levels, yet the entire soil is saturated, at a zero pressure.

Figure 7. Downward flow in a nonhomogenous porous media whose saturated hydraulic conductivity varies according to Eq. 10. Except for a small depth near the surface the entire profile is under negative soil water pressures $q = K_1$ and $K_2 = 5$ in Eq. 10.
Table 4. Solution to vertical movement of water through a saturated porous medium whose hydraulic conductivity varies according to Eq. 10. The solution to this situation is given by Eq. 11 and is illustrated by Figure 29.

<table>
<thead>
<tr>
<th>Dimensionless position</th>
<th>Variations of dimensionless hydraulic and pressure heads</th>
</tr>
</thead>
<tbody>
<tr>
<td>z/H</td>
<td>K_L = 20</td>
</tr>
<tr>
<td></td>
<td>h/H p/(Ym)</td>
</tr>
<tr>
<td></td>
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<td>0.000</td>
<td>0.000 0.000</td>
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<td>0.050</td>
<td>0.050 0.000</td>
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<tr>
<td>0.100</td>
<td>0.099 -0.001</td>
</tr>
<tr>
<td>0.150</td>
<td>0.147 -0.003</td>
</tr>
<tr>
<td>0.200</td>
<td>0.195 -0.005</td>
</tr>
<tr>
<td>0.250</td>
<td>0.242 -0.008</td>
</tr>
<tr>
<td>0.300</td>
<td>0.290 -0.010</td>
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<tr>
<td>0.350</td>
<td>0.339 -0.012</td>
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<tr>
<td>0.400</td>
<td>0.386 -0.014</td>
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<tr>
<td>0.450</td>
<td>0.435 -0.015</td>
</tr>
<tr>
<td>0.500</td>
<td>0.485 -0.015</td>
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<tr>
<td>0.550</td>
<td>0.535 -0.015</td>
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<tr>
<td>0.600</td>
<td>0.586 -0.014</td>
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<td>0.750</td>
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<td>0.899 0.000</td>
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<tr>
<td>0.950</td>
<td>0.951 0.001</td>
</tr>
<tr>
<td>1.000</td>
<td>1.001 0.001</td>
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Table 5. Solution to a one-dimensional, steady-state, unsaturated flow through heterogeneous porous media with K(x)=2-(sin x)2/0.110. Parameters used in the Brooks-Corey equations are: L = 5, p = 0.8 ft, S = 0.15. Depth of profile equals 15 ft, surface saturated at a pressure of p = 0.8 ft, and the flux applied at the surface equals 2100.0 ft/s. Saturated hydraulic conductivity on surface and bottom of profile.

<table>
<thead>
<tr>
<th>z/H</th>
<th>h</th>
<th>p/Ym</th>
<th>satur-</th>
<th>water</th>
<th>relative</th>
<th>saturated</th>
<th>total</th>
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<td>-0.800</td>
<td>0.000</td>
<td>0.000</td>
<td>0.450</td>
<td>0.100E+01</td>
<td>0.200E-04</td>
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<td>0.216E+04</td>
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<td>0.200E-04</td>
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<td>0.974</td>
<td>0.438</td>
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<td>0.231E+04</td>
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<tr>
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<td>0.963</td>
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<td>0.417</td>
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<tr>
<td>0.45</td>
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<td>0.925</td>
<td>0.416</td>
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<td>0.453E+04</td>
<td>0.200E-04</td>
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</table>

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saturated hydraulic conductivity is specified equal to,

\[ K_0 = (2 + \sin(\frac{\pi z}{H})) \times 10^{-5} \text{ fps} \]

The depth of the profile is equal to 15 feet, and the application of flux to the surface is specified as \( q = 2 \times 10^{-5} \text{ fps} \), or equal to the saturated hydraulic conductivity at the top and bottom of the profile, and the soil's porosity was specified equal to 0.45. The above specification for the saturated hydraulic conductivity gives an increase of 50 percent in its value at the center over that at the top and bottom of the profile.

The solution shows that unsaturated flow exist throughout the entire profile except at the very surface, and the pressure heads are all negative and less than the bubbling pressure head. Should a less permeable material such as edderoc occur at some depth, then a saturated condition would exist unless there is sufficient lateral drainage to remove any such saturated accumulation.

Table 6 provides a solution to a problem similar to the last example except that water in the amount of \( 2 \times 10^{-6} \text{ cfs per square foot of cross-sectional area per unit length between depths 2.5 and 5.0 ft is assumed to be added to simulate in an approximate manner by this one-dimensional solution the effect of lateral inflow from an adjacent area into this profile. Now note the zone of saturated flow begins at a depth of \( 4.75 \times 10^{-15} = 7.125 \text{ ft} \), well below where the added flow starts. Yet the positions below 7.5 ft are passing a flux equal to 1.5 times the saturated hydraulic conductivity on the top and bottom of the profile.

Obviously without recognizing that flow is occurring and what this flow pattern consists of, along with some knowledge of the soil's varying properties, would likely result in erroneous conclusions.

In a field situation flow is never steady-state, and is distorted from being vertical by edderock and other boundary conditions so that it is three-dimensional in space, and consequently much more difficult to describe. Without an extensive data base that describes soil water conditions in time and space as well as soil hydraulic properties in considerable detail correct understanding of the complex subsurface hydrology of the watershed is not likely.

The above discussion makes it clear that the limited piezometer data, with lack of information about subsurface hydraulic properties, cannot be used to define subsurface flow patterns in the instrumented slide areas. A much more extensive data base is needed, and most particularly during the snowmelt period when manual reading of the data would be most difficult, if not impossible, because the instruments would still be covered by snow. With the importance of mountain groundwater to understanding why slides occur as well as its importance to the water
### Table 6. Solution to steady-state, one-dimensional problem similar to that in Table 5, except that the downward flux is increased linearly from $z/D=0.167$ (2x2.5 ft) to $z/D=0.300$ (2x5.0 ft) by an amount equal to 23.0 ft/s. $f=1.5$, $p=-0.8$, $S=0.15$, the surface flux equals 3100 fps, and it is maintained saturated at a value $p$.

<table>
<thead>
<tr>
<th>$z/D$</th>
<th>$h$</th>
<th>$q/r$</th>
<th>$S$</th>
<th>$W$</th>
<th>$K$</th>
<th>$K_s$</th>
<th>$K_{sc}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.033</td>
<td>-0.305</td>
<td>-0.305</td>
<td>0.392</td>
<td>0.447</td>
<td>0.2025E-04</td>
<td>0.2105E-04</td>
<td>0.9623E+00</td>
</tr>
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<td>0.067</td>
<td>0.189</td>
<td>-0.011</td>
<td>0.983</td>
<td>0.442</td>
<td>0.2023E-04</td>
<td>0.2203E-04</td>
<td>0.9163E+00</td>
</tr>
<tr>
<td>0.100</td>
<td>0.064</td>
<td>-0.817</td>
<td>0.974</td>
<td>0.438</td>
<td>0.2022E-04</td>
<td>0.2309E-04</td>
<td>0.9755E+00</td>
</tr>
<tr>
<td>0.133</td>
<td>1.178</td>
<td>-0.022</td>
<td>0.966</td>
<td>0.435</td>
<td>0.2020E-04</td>
<td>0.2407E-04</td>
<td>0.9393E+00</td>
</tr>
<tr>
<td>0.167</td>
<td>1.685</td>
<td>-0.815</td>
<td>0.977</td>
<td>0.440</td>
<td>0.2024E-04</td>
<td>0.2500E-04</td>
<td>0.9896E+00</td>
</tr>
<tr>
<td>0.200</td>
<td>2.192</td>
<td>-0.808</td>
<td>0.987</td>
<td>0.444</td>
<td>0.2028E-04</td>
<td>0.2588E-04</td>
<td>0.9362E+00</td>
</tr>
<tr>
<td>0.233</td>
<td>2.696</td>
<td>-0.802</td>
<td>0.997</td>
<td>0.449</td>
<td>0.2031E-04</td>
<td>0.2669E-04</td>
<td>0.9855E+00</td>
</tr>
<tr>
<td>0.267</td>
<td>3.217</td>
<td>-0.783</td>
<td>1.009</td>
<td>0.450</td>
<td>0.2743E-04</td>
<td>0.2743E-04</td>
<td>0.1000E+00</td>
</tr>
<tr>
<td>0.300</td>
<td>3.757</td>
<td>-0.742</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2809E-04</td>
<td>0.2809E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.333</td>
<td>4.321</td>
<td>-0.679</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2866E-04</td>
<td>0.2866E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.367</td>
<td>4.909</td>
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<td>1.000</td>
<td>0.450</td>
<td>0.2914E-04</td>
<td>0.2914E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.400</td>
<td>5.523</td>
<td>-0.477</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2951E-04</td>
<td>0.2951E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.433</td>
<td>6.164</td>
<td>-0.336</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2978E-04</td>
<td>0.2978E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.467</td>
<td>6.833</td>
<td>-0.167</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2996E-04</td>
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</tr>
<tr>
<td>0.500</td>
<td>7.534</td>
<td>0.034</td>
<td>1.000</td>
<td>0.450</td>
<td>0.3000E-04</td>
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<td>1.0000E+00</td>
</tr>
<tr>
<td>0.533</td>
<td>8.234</td>
<td>0.234</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2995E-04</td>
<td>0.2995E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.567</td>
<td>8.937</td>
<td>0.437</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2978E-04</td>
<td>0.2978E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.600</td>
<td>9.645</td>
<td>0.645</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2915E-04</td>
<td>0.2914E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.633</td>
<td>10.361</td>
<td>0.861</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2914E-04</td>
<td>0.2914E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.667</td>
<td>11.088</td>
<td>1.088</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2866E-04</td>
<td>0.2866E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.700</td>
<td>11.828</td>
<td>1.329</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2809E-04</td>
<td>0.2809E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.733</td>
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<td>1.584</td>
<td>1.000</td>
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<td>0.2743E-04</td>
<td>0.2743E-04</td>
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</tr>
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<td>0.767</td>
<td>13.360</td>
<td>1.860</td>
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<td>0.450</td>
<td>0.2699E-04</td>
<td>0.2699E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.800</td>
<td>14.159</td>
<td>2.159</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2588E-04</td>
<td>0.2588E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.833</td>
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<td>1.000</td>
<td>0.450</td>
<td>0.2505E-04</td>
<td>0.2505E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.867</td>
<td>15.840</td>
<td>2.840</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2407E-04</td>
<td>0.2407E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.900</td>
<td>16.731</td>
<td>3.231</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2309E-04</td>
<td>0.2309E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.933</td>
<td>17.661</td>
<td>3.661</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2208E-04</td>
<td>0.2208E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>0.967</td>
<td>18.635</td>
<td>4.139</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2105E-04</td>
<td>0.2105E-04</td>
<td>1.0000E+00</td>
</tr>
<tr>
<td>1.000</td>
<td>19.658</td>
<td>4.658</td>
<td>1.000</td>
<td>0.450</td>
<td>0.2000E-04</td>
<td>0.2000E-04</td>
<td>1.0000E+00</td>
</tr>
</tbody>
</table>

### Table 7. Slug tests performed on piezometers in the Ward Canyon and Rick's Creek slide areas. The tests retained water to the top of the piezometer for a 30 minute period prior to the test. The amounts given in the table are the amount of water required to keep the pipe full to its top for an additional 20 minute period thereafter.

<table>
<thead>
<tr>
<th>Piezometer volume</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A 10 ml</td>
<td>5 gallons of water were poured thru the piezometer, and it was not possible to fill the pipe.</td>
</tr>
<tr>
<td>1B</td>
<td>Completely dry in 12 min., 15 sec. Enough water could not be measured and poured into piezometer to record the amount it would have received in 20 minutes.</td>
</tr>
<tr>
<td>2A 580 ml</td>
<td></td>
</tr>
<tr>
<td>3A 140 ml</td>
<td></td>
</tr>
<tr>
<td>5A 130 ml</td>
<td></td>
</tr>
<tr>
<td>5B</td>
<td></td>
</tr>
</tbody>
</table>

### DEPLAS FLOW THEROY
resources of an arid State like Utah a couple of selected areas should be properly instrumented and the data telemetered to a valley station for recording and analysis.

In order to obtain a rough estimate of the saturated hydraulic conductivities of the material in which the piezometer were installed a few slug tests were done. In these tests the piezometer pipe (1/2 inch galvanized) was saturated with water to its top for 30 minutes before readings were taken. Then the pipe was keep full to its top for a 20 minute period and the amount of water added recorded. The results of these slug tests are contained in Table 7. In the case of piezometers 38 and 58 in Ward Canyon the hydraulic conductivities were so large that it was not possible to perform the tests. All tests indicate that the piezometers are in materials with reasonably large hydraulic conductivities.

The writer's current speculation is that saturated flows in the mountain swales, even during normal or even other wet water years is generally confined to relatively small areas upstream from small streamlets that would be visible during the spring runoff period, or after significant rainfall. During the spring of 1983 the snowmelt rates were sufficient in many areas to saturate the slopes that contained considerably more than the usual water contents that during other wet years were prominately unsaturated. These unusual levels of saturation caused outward hydraulic gradients and loss of strength from reversal of the usual negative pore water pressure to positive ground water pressures and resulted in numerous shallow soil failures. These slope failures were accompanied by a rapid release of groundwater, which in a number of cases combined with the sliding mass adding a few percent to its water content and transformed the slide into a debris flow.

**SIMULATION OF RUDD CREEK'S DEBRIS FLOW**

**Background of Event.**

A damaging debris flow occurred on May 30, 1983 from the Rudd Creek Canyon east of Farmington, which has a drainage area of only 0.6 square miles. During normal years this drainage provides a small stream that home owners at its canyon mouth have described as just adequate to water their lawns. A slide in the upper portion of this canyon was responsible for initiating the debris flow that scoured the streambed as much as 6 feet deep down to bedrock and deposited more than 90,000 cubic yards of material within a 3 by 3 block area of the northeast end of the city.
The debris fan deposit in Farmington was mapped in the field by John Finnie, a graduate student at USU to estimate the volume deposited. The procedure used was to measure and/or estimate depths of the deposit at all locations where these depths could be reasonably determined from house walls, car wreckage, sign posts, trees, excavations underway and other clues in conjunction with a base topographic map with a 2 foot contour that had been prepared for the area prior to the fan deposit. These depths were plotted on aerial photographs. From these maps the deposit cross-sections were determine and the volume of debris calculated to be 96,600 cubic yards.

The total deposit consists of materials from several debris flows. The first debris flow occurred Monday evening on Memorial Day, May 30, 1983 with several subsequent debris flows taking place during the next few days. The last such secondary event occurred on Sunday, June 5, 1983. The first debris flow contained considerable organic material, and subsequent flows consisted of much larger fractions of finer material that covered the fan from previous flows. The chief of police of Farmington provides the observation that the first debris flow pushed rocks and trees at its front. This debris flow had a velocity slower than a man could walk. He substantiates this observation by describing how occupants of one house saw the debris flow lobe approaching and ran to their neighbors house about 200 feet away, to warn them, but returned to their own house to retrieve some belongings. This observation is born out by other eye witnesses. Another homeowner describes a surging forward motion and he removed one of his vehicles from its onslaught, and indicated he could have saved a second vehicle in his garage had he not been prevented from doing so by an individual concerned for his safety.

Another large debris flow occurred from a canyon 35 miles north of Farmington over a farm yard at Willard, Utah. The quantity of debris deposited at Willard was mapped at 43,000 cubic yards. At this site a debris catchment basin was constructed during the 1930's. Owners of the land inundated report this debris came as four separate flows; two occurred on Sunday, May 29, and two on Monday, May 30. The second debris flow came while a bulldozer was working in the upper fan channel area to clean it and redirect the channel flow. The bulldozer operator reported that he "floated the machine down the channel on top of the debris flow".

Such natural hazards have not been experienced in Utah for some time. However, during the 1920's and 1930's the Davis County area along the Wasatch Front frequently experienced what were termed "torrential floods in Northern Utah." Large quantities of flood debris inundated farm lands and often destroyed dwellings in the communities. Pack (1923) gives an excellent description of two such debris flow in August 1923, that impacted heavily on the same towns of Farmington and Willard, in which boulders weighing 50 tons were transported. These earlier historic events
were associated with high intensity rain storms rather than snowmelt, by-in-large, however. A special Flood Commission was appointed by the Governor of Utah in 1930 that made a number of recommendations, including construction of catchment and debris basins. A major conclusion of the commission was that overgrazing had depleted mountain watersheds of needed vegetation. Eventually the entire mountain watersheds along the Wasatch Front were purchased by public funds and placed under the management of the U.S. Forest Service with grazing of vast areas completely restricted. The Davis County Experimental Watershed was established in Farmington. As a result of these actions considerable climatic and hydrologic data are available for the area, as stream gages were established on all significant streams in this area, and a number of special studies were undertaken. Eventually as funding for basic data and research diminished, stream gaging stations were discontinued, so that during the floods of 1983 none of the Wasatch Front streams were being gaged from Salt Lake City north to Ogden River.

Debris flows such as the one that occurred from Ruff Creek in 1983 are not uncommon and are responsible for considerable property loss and even loss of life on a national and worldwide basis. A few destructive examples are cited below. In 1919 in Java a volcanic ejection of water in a crater lake created a massive debris flow that buried 131.2 square kilometers (50.7 square miles) and in so doing destroyed 104 villages and killed 5,110 people. In 1934 shortly after midnight on New Year's Eve as people were ending their celebrations a debris flow in the La Canada Valley, Los Angeles County, California destroyed 400 homes and took 40 lives. In more recent times a debris flow was triggered in Nelson County, Virginia in 1969 by the rains from Hurricane Camille that killed about 150 people and caused tens of millions of dollars worth of damages. In Tanzania heavy rainfall initiated about 1000 debris flows. The May 18, 1980 eruption of Mount Saint Helens in the State of Washington caused a debris flow in the East Fork Pine Creek that had maximum flowrate of 29,000 cubic meters per second (1,024,000 cfs). In November of 1972 heavy rainfall caused a debris flow that inundated Big Sur, California. In the northwest of the U.S. debris flows occur regularly, as do they in some areas of southern California. Debris flows exist in virtually every state of the U.S. where there is significant topographic relief, and since the mid 1970's these hazards have become an almost annual event in one or more of the western states.

Mechanics of Debris Flows

Debris flows commonly originate in a steep mountain area as a landslide that is transformed into a slurry capable of travelling long distances on milder slopes. They consist of nonhomogeneous mixtures of water and earthen materials whose properties vary considerably. The
specific gravity of a typical debris flow is 2 (twice as heavy as water). Debris and mud flows are distinguished from landslides and other mass movements by their mobility and inherent fluid characteristics, but not necessarily their mass densities. The hydraulics of these flows is particularly difficult because the fluid properties needed to define flow characteristics depend upon the rate of shear occurring in the flow, i.e., the substance is a non-newtonian fluid. Prior to the 1983 Utah debris flow occurrences, a research project at the Utah Water Research Laboratory was funded by the Office of Water Resources and Technology to study the mechanics of Debris Flows. The results of this research are contained in two reports by DeLeon and Jeppson, 1982, and Jeppson and Rodriguez, 1983. An objective of this research was to develop a practical means for predicting depths and velocities of such debris flows. A computer program was developed that computes these quantities, and also allows a time dependent solution to be obtained for such flows based on some assumed fluid properties of the materials in these flows. The methods used in obtaining the computer solution are contained in the above reports. Herein the computer program is utilized to simulate what the Rudd Creek Debris flow might have consisted of. Sufficient data is not available regarding flowrates, etc. to claim that it represents the actual occurrence. However, in this simulation a major contribution to the downstream channel flows is from channel scour rather than from flow at the upstream end of the channel, since it is estimated that less than 20 percent of the volume of debris deposited in the Farmington fan had its origin in the slide.

The computer solution consists of two components. The first component provides the depths, velocities, flowrates, areas, etc. of the debris flow at a specified number of output stations based on having steady-state, but nonuniform, flow throughout the entire channel. Comparing these quantities with those of an equivalent water flow allows differences between the two fluid flow to easily be contrasted. Figures 8 and 9, taken from DeLeon and Jeppson, 1982, show how much greater the debris flow depths are than an equivalent volumetric flow of water. Figure 8 shows the relationship of the ratio of debris flow depth to water flow depth to bed slope and flowrate, and Figure 9 adds the effect of Mannings n for water flow, under the assumption that a uniform flow exist. The second component of the solution describes the time dependent characteristics, such as depth, velocity, flowrate, etc. at each output station as well as the velocity and position of the front surge or lobe of the debris flow.

In describing the needed physical properties of Rudd Creek Channel it was assumed that its shape is adequately defined by a trapezoidal cross-section and its varying size can be defined by a linear interpolation between the following four positions along its length: 0, 3000, 6000 and 6450. The canyon mouth is at position 6000 ft, and the remaining 450 ft represents its spreading over the fan beyond the canyon.
Figure 8. Ratio of debris flow to water flow depths in a wide rectangular channel related to bedslope and rate of discharge. Results of water flow are for Mannings $n = 0.035$.

Figure 9. Ratio of debris flow depth to water flow depth related to bedslope and Mannings $n$ (for water flow only) as obtained for the problem shown in plan and profile views at the top.
mouth. The major concern of the simulation is what occurs in the channel upstream from the canyon mouth, and therefore the simulation was terminated soon after the debris reached the 6450 ft position. At these positions the channel has the following characteristics:

<table>
<thead>
<tr>
<th>Position along channel (feet)</th>
<th>0</th>
<th>3000</th>
<th>6000</th>
<th>6450</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom slope</td>
<td>.43</td>
<td>.40</td>
<td>.235</td>
<td>.003</td>
</tr>
<tr>
<td>Bottom width (ft)</td>
<td>4.0</td>
<td>5.0</td>
<td>6.0</td>
<td>200</td>
</tr>
<tr>
<td>Side slope</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The steady-state component of the solution specified that at the upstream end of the channel, i.e. the position just downstream from the slide that caused the Rudd Creek debris flow, that the volumetric flowrate, \( Q = 100 \) cfs. Due to channel scour additional flowrates of 275 cfs and 300 cfs were picked up uniformly between positions 0 - 3000 ft, and 3000 - 6000 ft respectively, resulting in a volumetric flowrate of 675 cfs at position 6000 ft and beyond.

The results of this steady-state solution are shown in Figure 10. The steady-state depths are referred to as the "normal depths" on Figure 10, and at the beginning of the channel where \( Q = 100 \) cfs, this depth is 2.33 feet, and at the canyon mouth at position 6000 ft the depth equals 6.21 feet. As the channel rapidly widens over the northeast portion of Farmington, the depth abruptly increases to 10.55 ft. This abrupt rise in depth with decreasing channel slope, and widening is a characteristics of the solution of debris flow that is vastly different from that of water flows, and helps explain why in actual debris flows they stop, and possibly start again, as they emerge onto flatter benches beyond the canyon mouths. At the 6000 ft position the steady-state velocity is 8.82 fps, whereas at position 6450 ft the velocity equals 0.29 fps, i.e. a slow walking pace.

The time dependent component of this simulation is shown in Figure 11, that shows the advancing position of the debris surge every 20 seconds as well as the depths upstream therefrom. In this time dependent solution it was assumed that at time zero (the beginning of the time dependent solution), that steady-state conditions existed up to position 1881 ft and that the advancing debris was at this position with a downstream water flowrate of 3 cfs. The scour of the channel bottom is assumed to add to the flow as in the steady-state solution. Therefore, a flowrate of 275 cfs is picked up uniformly between positions 0 and 3000 ft if the debris flow passes any position within this reach where the additional flow enters. Likewise as the debris flow advances between 3000 ft and 6000 ft...
FIG. 10. PLAN AND PROFILE VIEWS OF RUDD CREEK CHANNEL USED IN COMPUTER SIMULATION.
FIG. 11. POSITION OF DEBRIS FLOW EVERY 20 SECONDS OBTAINED FROM SIMULATION #2 OF RUDD CREEK.
an ever larger portion of the additional 300 cfs is added to the debris' volumetric flowrate. This channel scour component of the debris flowrate does not vary with time, however, after the debris lope has passed a position in the channel. The flowrate at the position 0 ft, however, has been specified to change with time according to that given below, i.e. it is initially increased from 100 cfs to 200 cfs, and then decreased to 40 cfs.

<table>
<thead>
<tr>
<th>time(sec)</th>
<th>0</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
<th>100</th>
<th>120</th>
<th>140</th>
<th>160</th>
<th>180</th>
<th>200</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flowrate,</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>220</td>
<td>240</td>
<td>260</td>
<td>280</td>
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The time for the surge to travel from position 1881 ft to position 6000 ft at the canyon mouth is 620 seconds = 10.3 minutes. The average speed of the debris surge is 6.64 fps = 4.53 mph.

REFERENCES


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ANALYSIS PROCEDURES FOR DEBRIS FLOW MOVEMENT

by Jey K. Jeyapalan

ABSTRACT

While the art of calculating the factor of safety of a slope against sliding and the science of measuring shear strength properties of various types of geological materials are supported by extensive research and practice, very little progress has taken place for predicting the characteristics of debris movement. This paper will review a few procedures for characterizing the rheological behavior of debris and for classifying the flow into various types of segments. Different types of analyses procedures and associated mechanics of debris movement will also be discussed in this paper. Suitable laboratory procedures for measuring material parameters will be presented. A few case histories will be used to illustrate some of the salient features of the phenomenon of debris movement. In addition, a movie of debris flow in motion will be shown as part of this presentation.
ABSTRACT

One-dimensional debris flow (or mudflow) simulation is based on a general viscoplastic fluid model and the basic concepts of open-channel hydraulics. The previously developed uniform mudflow formula is applicable to wide channels only. Extending this formula to a more general case of one-dimensional debris flow in a channel with section of arbitrary geometric shape requires that it be semi-empirically reformulated by replacing the flow depth by the hydraulic radius. Among the rheological parameters yet to be redefined in terms of the hydraulic radius are the yield-stress index (i.e., the relative strength of the yield stress against the bed shear), the Heddström number, and the Bingham (or yield) number. A combination of the Darcy-Weisbach equation with this semi-empirically reformulated uniform mudflow formula further enables one to express the Darcy-Weisbach resistance coefficient in terms of the generalized Reynolds number and the redefined yield-stress index (or alternatively Heddström or Bingham number). In practice, however, generalized Manning’s n reformulated by varying it with the rheological parameters and the hydraulic radius is more useful in a full range of laminar and turbulent mudflow. Unlike in wide channels, the momentum and energy correction factors for nonuniform distribution of mudflow velocities over an arbitrary channel section cannot be theoretically evaluated; they are only estimated through velocity measurements in the laboratory or field. The form of the one-dimensional model for mudflow is found to be identical to that for clear-water flow; therefore, the clear-water model can be applied to the mudflow simulation by using the empirically determined momenta (or energy) correction factor and resistance coefficient for mudflows. Experience in modeling debris flows following the May 18, 1980 eruption of Mount St. Helens shows the practical usefulness of this semi-empirically reformulated uniform mudflow formula.

INTRODUCTION

In a previous study (Chen, 1989b), it has been shown that a generalized viscoplastic fluid model, which is a combined form of the "power-law" and Bingham plastic fluid models, is capable of describing the time-independent
rheological properties of flowing cohesive and noncohesive granular materials. The further analysis of this rheological model for plane gravity flow has proved that the extended Magnold [1954] relation is identical to the Drucker-Prager [1952] criterion. This essentially establishes the generality and applicability of the model, at least, in the two-dimensional case. For uniform mudflow in a wide channel, the theoretical expressions of the uniform flow formula and the flow parameters, such as the resistance coefficient and the momentum and energy coefficients for nonuniform velocity distribution, can be directly obtained from the generalized viscoplastic model upon substitution of the theoretical expressions of the shear stress and the yield stress from the equation of motion. To attain similar theoretical expressions for a more general case of one-dimensional (1-D) debris flow in a channel with section of arbitrary geometric shape, however, is presently unlikely unless one resorts to a two- or three-dimensional approach. The extension of the uniform mudflow formula developed for wide channels to that for channels with arbitrary cross-sectional shape involves indispensible empiricism, namely employing the hydraulic radius in place of the flow depth in the original formula. This type of a semi-empirical formulation of the general uniform flow formula is indeed one of the basic concepts adopted in open-channel hydraulics. The objective of the present paper is thus to reformulate the uniform mudflow formula based on such hydraulic concepts, thereby evaluating, through a critical analysis of available field data on an actual event, its practical usefulness in 1-D debris flow modeling.

UNIFORM LAMINAR MUDFLOW FORMULA

The previously derived uniform formula for laminar mudflow in wide channels [Chen, 1983a] can be reformulated in terms of the hydraulic radius as

$$v = \left( \frac{n}{n+1} \right) \left( \frac{\rho g S_0}{\mu} \right)^{1/n} \left( \frac{\tau_0}{R} \right)^{(n+1)/n} \left[ 1 - \frac{\eta}{2n+1} \left( \frac{\tau_0}{R} \right) \right]^{(n+1)/n} \tag{1}$$

in which \(v\) is the area-averaged velocity component in the longitudinal direction of flow; \(n\) is the flow-behavior index of the viscoplastic sediment-water mixture; \(\rho\) is the mass or bulk density of the mixture; \(g\) is the gravitational acceleration; \(S_0\) is the channel slope; \(\mu\) is the consistency index of the mixture; \(R\) is the hydraulic radius of flow; and \(R - r_0\) is the hydraulic radius of rigid plug corresponding to the yield stress, \(s\), defined by

$$s = \rho g S_0 (R - r_0) \tag{2}$$

Likewise, the shear stress, \(\tau\), can be expressed as

$$\tau = \rho g S_0 (R - r) \tag{3}$$

in which \(r\) is the fictitious space coordinate to be defined in such a way that \(R - r\) represents the hydraulic radius corresponding to a part of the cross-sectional area in question, as shown in Fig. 1. Around the perimeter
FIGURE 1 Definition sketch of uniform mudflow in a channel with section of arbitrary geometric shape.
of this partial cross-sectional area (i.e., the shaded area in Fig. 1b), the magnitude of \( \tau \) is supposed to be equal. Obviously, \( \tau = s \) at \( r = r_0 \) and thus, Eq. 3 reduces to Eq. 2. The bed shear, \( \tau_0 \), can be expressed by letting \( r = 0 \) in Eq. 3, namely

\[
\tau_0 = \frac{\rho g S_0 R}{2}
\]  

which agrees with the expression obtained by applying directly the force balance principle to uniform mudflow. Introduction of \( R \) and \( r_0 \) in the formulation of Eq. 1 is one of the basic concepts of open-channel hydraulics; the other concepts will be introduced later. For convenience, Eq. 1 is henceforth referred to as the uniform mudflow formula for channels with section of arbitrary geometric shape.

For clarity, the relation between \( r \) and \( R \) can be explained in more detail below: As shown in Fig. 1,

\[
r = \frac{A_0}{P_0} - \frac{A(r)}{P(r)}
\]

in which \( A_0 \) is the total cross-sectional area of flow and \( P_0 \) is the total wetted perimeter of flow, both for \( r = 0 \); and \( A(r) \) and \( P(r) \) are respectively the partial cross-sectional area and the corresponding wetted perimeter in question. Note that \( R = A_0/P_0 \) and \( R(r) = A(r)/P(r) \). Given \( \tau \), the hydraulic radius, \( R = r \), is computed from Eq. 3 so that \( r \) corresponding to \( \tau \) is

\[
r = R - \frac{\tau}{\rho g S_0}
\]

Similarly, at \( r = r_0 \), i.e. on the boundary surface of rigid plug, one obtains from Eq. 2

\[
\tau_0 = R - \frac{s}{\rho g S_0}
\]

Johnson [1970, p. 301] derived the radius of rigid plug for flow of a Bingham plastic fluid in a semicircular channel based on the assumption of a constant \( s \). Johnson's result is identical to Eq. 7 because the hydraulic radius of a semicircular section is one fourth of its diameter. However, the assumption of a constant \( s \) made in the derivation of Eq. 7 is open to debate because \( s \) in reality is not constant, except for idealized materials such as purely or ideally cohesive soil or simple plastic.

In soil mechanics, an extended form of the Mohr-Coulomb criterion for cohesive soils is

\[
s = c \cos \phi + \sigma \sin \phi
\]
in which $c$ is the cohesion or cohesive strength of soil, $\phi$ is the static angle of internal friction on a failure plane of soil, and $p$ is the hydrostatic pressure. Because $p$ varies with the depth of mudflow, so does $c$ by virtue of Eq. 8. The previously derived expression of the rigid plug thickness is applicable to wide channels only [Chen, 1983b]. To extend this expression to a more general case of 1-D debris flow in a channel with section of arbitrary geometric shape again requires the expression to be reformulated in terms of $R$ and $r_0$ as follows:

$$r_0 = R - \frac{c \cos \phi}{\rho g (S_0 - \cos \theta \sin \phi)}$$

(9)

in which $\theta$ is the angle of inclination of channel bottom. If $\phi = 0$ (i.e., for purely or ideally cohesive soil or simple plastic), it can be shown that Eqs. 7 and 9 are identical. Therefore, the hydraulic radius of a viscoplastic rigid plug, $R = r_0$, in a channel of arbitrary cross-sectional shape can be expressed by the last term in Eq. 9. In a semicircular channel, for example, since the radius of a semicircular section is theoretically twice its hydraulic radius, the theoretical radius of rigid plug, can be expressed as twice the last term in Eq. 9.

Given $p$ and $S_0$, there are three rheological parameters, $\eta$, $\lambda$, and $r_0/R$ that need to be determined for use of Eq. 1 in 1-D mudflow simulation. Evaluation of these rheological parameters can be made using a viscometer or flume, or both. During experiments, however, a constant value of $r_0/R$ should be maintained. It can be readily shown from Eqs. 2 and 4 that the parameter $r_0/R$ signifies the relative magnitude of the yield stress, $s$, to the bed shear, $t_0$, and thus can be referred to as the yield-stress index [Chen, 1983b]. Besides $r_0/R$, two other rheological parameters have been frequently cited in the literature. They are called the Knudsen number, $\kappa$, and the Bingham (or yield) number, $B$. Because $\kappa$ and $B$ are both defined in terms of $s$, which in turn is pressure-dependent and is not constant across the channel section, the values of $\kappa$ and $B$ may considerably vary over the section in question. To avoid confusion resulting from inadvertent use of such nonconstant $\kappa$ and $B$ in 1-D debris flow modeling, use of $r_0/R$ is recommended [Chen, 1983b]. More comments on the relations of $r_0/R$, $\kappa$, and $B$ through the Berc-Welsch resistance coefficient will be made later.

Given soil properties of $c$ and $\phi$ in addition to $p$ and $S_0$ ($c > 0$), the value of the yield-stress index, $r_0/R$, can be computed from Eq. 9, namely

$$\frac{r_0}{R} = 1 - \frac{c \cos \phi}{\rho g R (S_0 - \cos \theta \sin \phi)}$$

(10)

In particular, for flow of noncohesive granular materials (i.e., $c = 0$), one obtains $r_0/R = 1$ from Eq. 10 and thus noncohesive granular materials are treated as a "power-law" fluid. Accordingly, for power-law fluid, Eq. 1 reduces to
In a practical problem, for lack of data on \( c, \phi, \) and \( \eta, \) Eq. 11 with \( \eta = 2 \) was assumed and then applied to routing mudflows along the Toutle and Cowlitz Rivers from a hypothetical failure of Spirit Lake blockage on Mount St. Helens, Washington [Swift and Kresch, 1983].

For convenience, Eq. 1 can be rearranged, with the help of the Darcy-Weisbach equation,

\[
\tau_0 = \frac{\rho}{4} \frac{v^2}{2}
\]

into a more manageable form:

\[
v = \left( \frac{8g}{f} \right)^{1/2} \rho^{1/2} \omega^1/2
\]

in which \( f \) is the Darcy-Weisbach resistance coefficient, expressed in general for laminar flow as

\[
f = \frac{C}{E}
\]

Here the "constant" \( C \) and the generalized Reynolds number \( E \) in Eq. 14 are defined respectively as

\[
C = \frac{8}{(\frac{\tau_0}{R})^{\eta +1}} \left[ \frac{\eta + 1}{\eta} \right] \left[ \frac{\eta}{1 - \frac{\eta}{2n + 1}} (\frac{\tau_0}{R}) \right]^{\eta}
\]

\[
E = \frac{\rho \omega^{2-\eta} \omega^n}{\omega}
\]

Substituting \( f = \frac{C}{\rho \omega^{2-\eta} \omega^n} \) from Eqs. 14 and 16 into Eq. 13 and simplifying the resultant expression yields

\[
v = \left( \frac{C \omega}{\omega} \right)^{1/\eta} \omega^{(\eta + 1)/\eta} \omega^{1/\eta}
\]
Eq. 17 is the general uniform laminar mudflow equation in which \( C \) is semi-empirically expressed in Eq. 15 as a function only of the two rheological parameters, \( n \) and \( r_0/R \). In view of known behaviors for flow of Newtonian fluids, the value of \( C \) is also likely to vary with channel geometry and \( S_0 \). For illustration, Eq. 15 is plotted in Fig. 2, which shows the variation of \( C \) against \( n \) and \( r_0/R \) for power-law fluids, Bingham plastic fluids, and generalized viscoplastic fluids. However, the actual variation of the \( C \) value as affected by the major rheological properties of fluids and channel geometry can only be experimentally determined in the laboratory. The experimental determination of the \( C \) value will thus constitute one of the major tasks in the future mudflow research.

The \( f \) versus \( I \) Relation with Parameters Describing Yield Stress

As mentioned earlier, the relative magnitude of the yield stress, \( s \), can be described by using one of the three parameters: the yield-stress index (\( r_0/R \)), the Bedström number (\( H \)), and the Bingham (or Yield) number (\( B \)). The previously defined \( H \) and \( B \) are expressed in terms of the flow depth and thus, are applicable to wide channels only. To apply \( H \) and \( B \) to a more general case of 1-D debris flow in a channel with section of arbitrary geometric shape requires that \( H \) and \( B \) be redefined in terms of the hydraulic radius, \( R \). For Bingham plastic fluids, they are redefined as

\[
H = \frac{s^n}{\mu^2} \quad (18)
\]

\[
B = \frac{s R}{\mu Y} \quad (19)
\]

and for the generalized viscoplastic fluids, Eqs. 18 and 19 are extended to

\[
H = \frac{s^{2-n} n^{n}}{\mu^2} \quad (20)
\]

\[
B = \frac{s R^n}{\mu Y^n} \quad (21)
\]

Although use of \( H \) and \( B \) in describing the relative strength of \( s \) cannot be fully justified, as mentioned previously, it is worth establishing relations among \( H \), \( B \), and \( r_0/R \) because most readers are believed to be more familiar with conventional \( H \) and \( B \) than with \( r_0/R \) for Bingham plastic fluids. For the generalized viscoplastic fluids, incorporating Eqs. 2, 4, and 12 with Eqs. 20 and 21 yields, by virtue of Eq. 16

\[
H = n^2 - n \quad g \quad \{f \left( 1 - \frac{r_0}{R} \right) \}^{2-n} \quad (22)
\]
FIGURE 2. C value in $f = C/R$ plotted against the flow-behavior index ($n$) and the yield-stress index ($r_0/R$).
For Bingham plastic fluids (i.e., \( n = 1 \)), Eq. 22 reduces to

\[
H = B/R = \frac{f}{R} \left(1 - \frac{r_0}{R} \right) \frac{g^2}{n}
\]  

(23)

Simplifying both sides of the last equal sign in Eq. 22 yields

\[
B = \frac{f}{R} \left(1 - \frac{r_0}{R} \right) \frac{g^2}{n}
\]  

(24)

which is independent of \( n \). An alternative expression of \( B \) by virtue of Eq. 14 is

\[
B = \frac{C}{R} \left(1 - \frac{r_0}{R} \right) \frac{g^2}{n}
\]  

(25)

Similarly, \( R \) can be expressed in terms of \( C \) by substituting Eq. 25 into Eq. 22 as

\[
R = \frac{C}{R} \left(1 - \frac{r_0}{R} \right) \frac{g^2}{n} \frac{Z - n}{Z}  \]  

(26)

A comparison of Eqs. 22-26 with the corresponding expressions of \( H \) and \( B \) derived previously for wide channels [Chen, 1983b] reveals the sole difference in the expression of the yield-stress index between them. Therefore, the \( f - R \) relation versus the yield-stress index developed for wide channels can also be extended to channels of arbitrary cross-sectional shape if the yield-stress index defined for wide channels is replaced by \( r_0/R \). It can be further shown that the \( f - R \) and \( f - B \) relations derived for channels with section of arbitrary geometric shape are identical to those for wide channels: For the generalized viscoplastic fluids, both relations are

\[
\left[ 1 - \frac{B H^{1/(2-n)}}{f \cdot 2/(2-n)} \right]^{(n+1) \left[ 1 + \frac{n}{(n+1)} \frac{B H^{1/(2-n)}}{f \cdot 2/(2-n)} \right]^n} = \frac{(2n+1)^n}{f \cdot 2/(2-n)} \frac{g}{n}
\]  

(27)

\[
\left(1 - \frac{B R}{f \cdot 2/(2-n)} \right)^{n+1} \left[ 1 + \frac{n}{(n+1)} \frac{R R}{f \cdot 2/(2-n)} \right]^n = \frac{(2n+1)^n}{f \cdot 2/(2-n)} \frac{R}{n}
\]  

(28)

and for Bingham plastic fluids (i.e., \( n = 1 \)),

\[
\left(1 - \frac{B H}{f \cdot 2/(2-n)} \right)^2 \left[ 1 + \frac{4 H}{f \cdot 2/(2-n)} \right] = \frac{24}{f \cdot 2/(2-n)}
\]  

(29)
An inspection of Eq. 27 reveals that the equation breaks down at \( n = 2 \). This is attributed to the fact that the generalized \( \mathcal{H} \) is not defined at \( n = 2 \), as manifested itself in Eqs. 20, 22, and 26. The breakdown of Eq. 27 at \( n = 2 \) worsens the adequacy of using \( \mathcal{H} \) or \( B \) as a third parameter in characterizing the effect of \( s \) on the \( f - \mathcal{I} \) relation because of the nonconstant \( s \) value over the channel section, as mentioned previously. In the future, therefore, exclusive use of \( r_0/R \) as a third parameter in the \( f - \mathcal{I} \) relation is recommended.

For illustration, the \( f - \mathcal{I} - r_0/R \) relation (Eqs. 14 and 15), \( f - \mathcal{I} - \mathcal{H} \) relation (Eq. 29), and \( f - \mathcal{I} - B \) relation (Eq. 30) for Bingham plastic fluids are all plotted on log-log paper, as shown in Fig. 3, although the first relation is only recommended herein. Because these three semi-empirical resistance relations are derived for laminar flow of Bingham plastic fluids only, development of resistance formulas for other flow regimes is still required for drawing the full \( f - \mathcal{I} \) curves in Fig. 3. It has been observed from experiments that a Bingham plastic fluid in the turbulent flow regime behaves as a Newtonian fluid [Thomas, 1963; Uddo, 1970; Ooi et al., 1980]; therefore, semi-empirical resistance formulas developed for turbulent flow of Newtonian fluids may be applied to the case of Bingham plastic fluids. Experimental results obtained from numerous laboratory investigations of Bingham plastic and pseudoplastic fluids have substantiated these semi-empirical resistance formulas for turbulent flow in pipes. Extending such turbulent flow formulas from pipes to open-channels, however, requires that the formulas be again expressed in terms of the hydraulic radius. Presently available turbulent flow resistance formulas are thus briefly appraised in the following.

**TURBULENT MUDFLOW RESISTANCE FORMULAS**

The semi-empirical turbulent flow formulas for pseudoplastic fluids in pipes resemble the counterparts for a Newtonian fluid in pipes. For example, an empirical formula for the Fanning resistance coefficient, \( f_n \), developed by Shaver and Merrill [1959] is

\[
f_n = \frac{0.079}{n^5} \frac{r}{\mathcal{I}_n}
\]

in which the Reynolds number, \( \mathcal{I}_n \) and its exponent, \( c \), are respectively defined as

\[
c = \frac{2.63}{(10.5)^n}
\]

\[
\mathcal{I}_n = \frac{\rho \nu^2 n \rho}{\rho} = 8 \left[ \frac{1}{2 \left( 3 + 1/n \right)} \right]^n
\]
FIGURE 1 The Darcy-Weisbach friction coefficient ($f$) versus Reynolds number ($R$) diagram for flow of Bingham plastic fluids in channels with section of arbitrary geometric shape.
in which \( D \) is the pipe diameter and all the other symbols were previously defined. For a Newtonian fluid (i.e., \( n = 1 \)), because Darcy-Weisbach's \( f \) is four times Fanning's \( f_n \), Eq. 31 reduces to the Blasius equation:

\[
f = \frac{0.316}{\ell_n^{0.25}}
\]

in which \( \ell_n = \frac{Dw_p}{u} \) from Eq. 33 for \( n = 1 \). The Blasius equation, Eq. 34, is applicable approximately in the range of \( 750 < \ell_n < 25,000 \); therefore, one may reasonably expect that the same range of \( \ell_n \) applies to Eq. 31.

For turbulent flow of pseudoplastic fluids in smooth pipes, Metzner [1958], Dodge and Metzner [1959], and Nogue and Metzner [1963], among others, have found from experiments that the following semi-empirical formula is valid for \( 0.3 < n < 1 \) but has been extrapolated to the region where \( n > 1 \) without substantiation by experiment.

\[
\frac{1}{\sqrt{f_n}} = \frac{4}{\ell_n^{0.75}} \log \left( \ell_n \frac{f_n}{1-n/2} \right) - \frac{0.40}{n^{1.2}}
\]

in which \( \ell_n \) is defined in Eq. 33. For a Newtonian fluid (i.e., \( n = 1 \)), Eq. 35 reduces to the Kármán-Prandtl equation:

\[
\frac{1}{\sqrt{f}} = 2 \log \left( \ell \sqrt{f} \right) - 0.8
\]

For turbulent flow of Bingham plastic fluids in smooth pipes, Thomas [1963], Daido [1970], and Dai et al. [1980], among others, have shown, using their respective experimental data, the applicability of Eq. 35 for \( n = 1 \), Eq. 36, or an equivalent form thereof:

\[
\frac{1}{\sqrt{f_n}} = 4 \log \left( \ell_n \sqrt{f_n} \right) - 0.4
\]

Note that Eqs. 36 and 37 are identical by virtue of \( f = 4f_n \). Because Eq. 35 is independent of \( u \), a Bingham plastic fluid in the turbulent flow regime may be assumed to behave as a Newtonian fluid.

A semi-empirical formula for resistance to turbulent flow of both power-law and Bingham plastic fluids in rough pipes has not yet been developed. It is expected, however, that the general form of such a formula, if viable, should be able to reduce to the Kármán-Prandtl equation for turbulent flow of Newtonian fluids in rough pipes:

\[
\frac{1}{\sqrt{f}} = 2 \log \left( \frac{D}{2k^{1.74}} \right)
\]
in which $k$ is the roughness size of rough pipe.

An abundance of existing experimental evidence shows agreement with the resistance formulas such as Eqs. 31 and 35 developed for turbulent flow of pseudoplastic and Bingham plastic fluids in smooth pipes, but very little data exist in the case of open channels. Because the turbulent flow of a Bingham plastic fluid in a pipe behaves like that of a Newtonian fluid, an analogy may be drawn to extend the semi-empirical resistance formulas for turbulent flow of Newtonian fluids in smooth and rough channels to the counterparts of Bingham plastic fluids. On the basis of the hydraulic concepts, therefore, the resistance formulas for turbulent flow of Newtonian fluids in channels can be readily obtained from their counterparts in pipes. Substituting $D = 4R$ and $R_n = 4R$ into Eqs. 34, 36, and 38 yields respectively

$$f = \frac{0.223}{\frac{2}{k}}$$

$$\frac{1}{\sqrt{f}} = 2 \log \left( \frac{R}{\sqrt{f}} \right) + 0.404$$

$$\frac{1}{\sqrt{f}} = 2 \log \left( \frac{R}{k} \frac{2}{k} \right) + 1.74$$

Note that Eqs. 39, 40, and 41 are the Blasius equation for turbulent flow in smooth channels, the KÁRMAN-Prandtl equation for turbulent flow in smooth channels, and the KÁRMAN-Prandtl equation for turbulent flow in rough channels, respectively. Assuming that Eqs. 39 - 41 also apply to the case of Bingham plastic fluids, one can plot these equations in Fig. 3.

To determine the critical $R$ for transition from laminar to turbulent flow of Bingham plastic fluids in channels of arbitrary cross-sectional shape is a formidable task. A sudden transition seems unrealistic. For practical purposes, therefore, the generalized Manning formula, which is asymptotic both to Eq. 14 and the equivalent Manning formula [Chen, 1983b], can be used to describe the gradual transition of $f$ from laminar to turbulent flow. The final form of the generalized Manning formula thus consists of the original Manning formula and the generalized Manning $n$, all expressed in terms of the hydraulic radius. The Manning formula in metric system is

$$V = \frac{1}{n} R^{2/3} g^{1/2}$$

and Manning's $n$ is generalized as
in which \( n_0 \) is a specific value of Manning’s \( n \) for fully turbulent flow in a rough channel with roughness size \( k \) and channel slope \( S_0 \). Note that \( C \) and \( n_0 \) may be considered as two flow parameters needed to be evaluated after the three rheological parameters \( u \), \( n \), and \( r_0/R \) are determined. In the absence of experimental data on the variations of \( C \) and \( n_0 \) with respect to \( S_0 \) and other factors which may affect the roughness, the theoretical \( C \), as shown in Fig. 2, and the empirical \( n_0 \) for fully turbulent clear-water flow in mountain streams with gravel, cobble, and boulder beds, such as \( n_0 = 0.03 \sim 0.07 \) [Chow, 1959], may be assumed for practical purposes. Note that for flow of clear water (i.e., \( n = 1 \)), Eq. 43 reduces to the form formulated by Chen [1981].

The asymptotic behavior of \( n \) is self-evident from Eq. 43. Known the values of all the aforementioned parameters, the value of \( n \) varies only with \( R \). As \( R \) decreases, and approaches zero in the limit, the first term in Eq. 43 dominates and is equivalent to the result obtained by equating the right-hand side of Eq. 42 to that of Eq. 17, which is in fact the theoretical uniform laminar flow equation derived for generalized viscoplastic fluids. Conversely, if \( R \) increases and approaches infinity, the first term in Eq. 43 becomes increasingly negligible and the second term dominates; therefore, at \( n = n_0 \), the flow is fully turbulent. Chen [1981] has experimentally verified the validity and applicability of Eq. 43 for flow of a Newtonian fluid. Whether or not Eq. 43 is valid for flow of non-Newtonian fluids should be investigated in the future.

The uniform mudflow formula, Eq. 1, or a simplified form thereof, Eq. 17, is believed to be useful in debris flow simulation. However, it is only applicable to the range of laminar flow. In practice, therefore, one may encounter the difficulty in using Eq. 1 or 17 because it is unknown a priori whether a debris flow in question is in the range of laminar, transition, or turbulent flow. One way to circumvent this difficulty is to use the generalized Manning formula (i.e., Eq. 42 incorporated with Eq. 43), which has been made asymptotically valid in a full range of laminar and turbulent mudflow. The disadvantage of using the generalized Manning \( n \) (Eq. 43), however, manifests itself in the four parameters needed to be evaluated prior to its use. In other words, the expression of generalized Manning’s \( n \) has an additional parameter, \( n_0 \), over \( u \), \( n \), and \( r_0/R \) used in the formulation of Eq. 1 or 17. Note that both Eq. 17 and Eq. 43 have the parameter \( r_0/R \), which is expressed through \( C \), as shown in Eq. 15.

**EVALUATION OF PARAMETERS**

In the absence of equipment which is capable of measuring the local velocity of debris flow with concentration of sediment as high as 90 percent by weight or higher, one may have to resort to the simplified uniform mudflow formula (Eq. 17) for the evaluation of the three rheological parameters: \( u \), \( n \), and \( r_0/R \). In view of interdependence of one parameter on the others, the field determination of these parameters by means of Eq. 17 seems unlikely.
Experiments of running uniform mudflows in a laboratory flume are thus needed for acquiring the least squares estimates of the three parameters. Because the logarithmic transformation of Eq. 17 leads to a linear relationship between the response and the predictor variables, the parameters of a physically-based regression model, as posed in the form of Eq. 17, are intrinsically linear and thus can be estimated by the standard linear regression procedures.

Taking logarithms of both sides of Eq. 17, after the left side being rearranged into a form of $V$ divided by $R$, yields

$$ n \log \left( \frac{V}{R} \right) = \log \left( \frac{8pg}{Q_i} \right) + \log \left( S_0 R \right) $$

which is linear in $\log (V/R)$ and $\log (S_0 R)$. Therefore, given "m" observations, i.e., measured depth, $h$, corresponding to given discharge, $Q_i$, $(h_i, Q_i)$, $i = 1, 2, ..., m$ in a channel of specified bed slope, $S_0$, and roughness, the best fitted values of $n$ and $8pg/CU$ can be found by applying the method of least squares to Eq. 44. Because the same number of data points on the hydraulic radius, $R$, and the mean velocity, $V_i$, $(R_i, V_i)$, $i = 1, 2, ..., m$ can be obtained from the corresponding data on $(h_i, Q_i)$, $i = 1, 2, ..., m$, the least squares estimates of $n$ and $8pg/CU$ are readily expressed, using a linear regression analysis, as

$$ n = \frac{\sum V_i \log (h_i/R_i) - \sum (V_i/R_i) \log (S_0 R_i)}{\sum (V_i/R_i)^2 - \sum (V_i/R_i) \log (S_0 R_i)} $$

and

$$ \frac{8pg}{CU} = \frac{1}{m} \sum \log \left( \frac{h_i}{R_i} \right) - \frac{1}{m} \sum \log \left( S_0 R_i \right) $$

in which $S_0$, $i = 1, 2, ..., m$ is the channel slope for the $i$-th experiment and $\sum$ denotes the summation of "m" data values. Because the channel slope may vary from one observation to the other, the "m" data points actually used in the formulation of Eqs. 45 and 46 are $(h_i, V_i, S_0)$, $i = 1, 2, ..., m$.

The validity of Eqs. 45 and 46 is predicated on the assumption that the values of the three rheological parameters ($n$, $u$, and $r_0/R$) do not vary while the whole set of data points $(R_i, V_i, S_0)$, $i = 1, 2, ..., m$ for uniform flow of a given sediment-water mixture in the channel are collected. In reality, however, the assumption of constant $r_0/R$ does not hold whenever there is a change in $0$ or $S_0$ unless experiments are carefully designed to adjust $0$ or $S_0$, or both, in accordance with Eq. 10. This experimental procedure is essential because $r_0/R$, as shown in Eq. 10, varies with $R$ and $S_0$ (or $0$) even though $n$ and $u$ are constant for the given mixture. Therefore, in order to maintain a constant value of $r_0/R$ throughout the acquisition of the "m" data points $(R_i, V_i, S_0)$, $i = 1, 2, ..., m$, one must establish uniform mudflows with the following hydraulic radii, $R_i$, $i = 1, 2, ..., m$ for various
Varying the value of \( r_0/R \) in Eq. (47) yields another set of "m" data points \( (R_1, V_1, S_0) \), \( i = 1, 2, \ldots, m \). Repetition of this procedure will thus allow us to draw a family of parallel straight lines for various values of \( r_0/R \) on log-log paper, using \( V/R \) and \( S_0 R \) as ordinate and abscissa, respectively. The slope of the parallel straight lines is \( 1/n \). Once the least squares estimates of \( n \) and \( \rho g/C_0 \) are determined from Eqs. 45 and 46, respectively, the value of \( C \) can be computed from Eq. 15 upon substitution of the known value of \( r_0/R \) from Eq. 10 and the least squares estimate of \( n \) from Eq. 45. The \( C \) value so computed can then be used to determine the \( u \) value from the least squares estimate of \( \rho g/C_0 \), Eq. 46, provided that the specific weight of the mixture, \( \rho g \), is known.

A dimensional analysis of the generalized viscoplastic model reveals that the dimension of \( u \) is grams/cm/sec^2 - \( n \) in metric system, depending upon the value of \( n \). Therefore \( u \) for Bingham plastic fluids (i.e., \( n = 1 \)) has the same dimension as the dynamic viscosity of Newtonian fluids, viz. grams/cm/sec (or poises) and is often called the Bingham viscosity. In the following, the practical usefulness of a log \( (V/R) \) versus log \( (S_0 R) \) plot with \( \rho g/C_0 \) as a "lumped" parameter, as shown in Fig. 4, is assessed through the evaluation of \( u \), using reconstituted data on lahar peak flow after the May 18, 1980 eruption of Mount St. Helens.

Plotted in Fig. 4 are two families of straight lines representing Eq. 44 for Bingham plastic fluids (\( n = 1 \)) and dilatant fluids (\( n = 2 \)) with \( \rho g/C_0 \) equal to 1, 5, 10, 50, and 100 m^2 sec^-n, respectively. Measured or reconstituted data on lahar peak flow in Muddy River and Pine Creek [Pierson, written commun., 1982] are analyzed and the values of \( \rho g/C_0 \), \( f \), and \( n \) computed, as listed in Table 1. For comparison, these data points are also marked in Fig. 4. It is interesting to note from Fig. 4 and Table 1 that the values of \( \rho g/C_0 \) in all cases of \( n = 1 \) except one point fall in between 10 and 100 m^2 sec^-1 for Muddy River, but except two points in between 1 and 10 m^2 sec^-1 for Pine Creek. Each data point represents a possible combination of the various values of \( n \), \( u \), and \( r_0/R \) for a lahar to move through each of the measured points in Muddy River or Pine Creek. Presumably far upstream from the section where the transition from debris flow to hyperconcentrated flow took place, the values of \( n \) and \( u \) did not change drastically when a lahar with a certain sediment concentration and grain-size distribution, moved down the same valley, but the value of \( r_0/R \) might vary considerably, depending upon \( R \) and \( S_0 \) (or \( S \)), see Eq. 10. Because \( c \) and \( \phi \) are unknown, there seems no way of determining \( r_0/R \) from Eq. 10 and hence, \( C \) from Eq. 15. Because \( r_0/R \) varies from point to point, it is meaningless to draw a best fitted line to pass through all the data points for each of Muddy River and Pine Creek. In the absence of data on \( c \) and \( \phi \) which can be used to evaluate \( r_0/R \) and \( C \), one can at best roughly estimate the order of magnitude for \( u \), using the assumed \( r_0/R \) or \( C \) value.
FIGURE 4 Logarithmic diagram of uniform laminar mudflow formula for Bingham plastic and dilatant fluids, plotted with various values of a "lumped" rheological parameter, $8pg/Cu$. Data points are shown to illustrate the reconstructed flow behaviors of peak lahars measured in Muddy River and Pine Creek after the May 18, 1980 eruption of Mount St. Helens.
from the least squares estimate of $R_0/C_0$.

Assuming that $C_0$ is in the order of $10^2$ which approximately corresponds to $r_0/R = 0.4$ for Bingham plastic fluids ($n = 1$), as shown in Fig. 2, one can estimate $\mu = 0.2 - 2$ poises for Muddy River and $\mu = 2 - 20$ poises for Pine Creek. In general, the estimated values of $\mu$ for Muddy River is an order of magnitude lower than those for Pine Creek, despite physically similar lahars observed in both valleys (Piersson, oral commun., 1984). This may be attributed to the lower values of $R_0$ or $S_0$, or both, in Muddy River, thus resulting in the lower values of $\mu$ (see Table 1). Note that the $\mu$ values of a lahar in Pine Creek are strikingly in the same order of magnitude as those of debris flows in Hengxia Ravine, China, analyzed by Kang and Zhang [1980], but are one or two orders of magnitude lower than those observed in Wrightwood California. Sharp and Nobles [1953], for example, estimated the apparent viscosities of debris flows in Wrightwood, California, ranging from $2.1 \times 10^3$ to $6 \times 10^3$ poises, but Johnson [1970] later estimated its Bingham viscosity at $7.6 \times 10^3$ poises. Using the uniform mudflow equation, Smith et al. [1983] statistically analyzed the Bingham viscosities of a lahar in the North Fork of Toutle River following the 1980 Mount St. Helens eruption, and found then in the range of $3 \times 10^3$ to $4 \times 10^3$ poises, which are roughly in the same order of magnitude as found by Sharp and Nobles in Wrightwood, California. Why the viscosities of a lahar in Toutle River had at least two orders of magnitude higher than those in Pine Creek after the eruption remains to be investigated.

Apparently the Bingham viscosity of mudflow varies widely, depending mainly on the concentration of sediment (or the water content) and its grain-size distribution (Kang and Zhang, 1980). For instance, the viscosity of a volcanic mudflow containing 22% of water in Mount Bandai, Japan was determined by Iida [1938] to be $10^4$ poises, and that of an alpine mudflow containing less than 10% of water in the Tenmile Range, central Colorado was analyzed by Curry [1966] to be $3 \times 10^4$ poises. These and other values exemplify a wide range of the Bingham viscosities of debris flows in nature.

It has been demonstrated that use of the uniform mudflow equation, Eq. 17, alone cannot estimate the values of the three rheological parameters of debris flow unless a flume is precisely operated to control the yield-stress index, $r_0/R$, of various uniform mudflows produced in the laboratory environments. In the field, however, the value of $r_0/R$ varies with $R$ and $S_0$ even though the mass of a given sediment-water mixture does not alter the other rheological properties during its movement down the valley. Therefore, applying the uniform mudflow equation by a regression analysis to all data measured at various locations in a straight channel may result in the misleading values of the three rheological parameters. This may explain why the Bingham viscosities of a lahar in the North Fork of Toutle River, as analyzed by Smith et al. [1983] are two orders of magnitude higher than those in Pine Creek.

I-D DYNAMIC EQUATIONS FOR DEBRIS FLOW

It has been shown in the previous study [Chen, 1983b] that the I-D
### Table 1: Data on Larar Peak Flow in Muddy River and Pine Creek After the May 18, 1980 Eruption of Mount St. Helens.

<table>
<thead>
<tr>
<th>MEASURED LOCATION</th>
<th>VELOCITY V (m/sec)</th>
<th>CROSS-SECTIONAL AREA A (m²)</th>
<th>HYDRAULIC RADIUS R (m)</th>
<th>CHANNEL SLOPE S (°)</th>
<th>V/R (1/sec)</th>
<th>R²/R</th>
<th>Δ Δ (°C)</th>
<th>Darcy-Weisbach f</th>
<th>Manning's n</th>
<th>SPECIFIC WEIGHT ρ (g/cm³)</th>
<th>CONCENTRATION Cw (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Muddy River</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M1</td>
<td>16.4</td>
<td>1054</td>
<td>5.98</td>
<td>0.030</td>
<td>3.26</td>
<td>0.140</td>
<td>14.7</td>
<td>20.3</td>
<td>0.0399</td>
<td>0.0359</td>
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</tr>
<tr>
<td>M2</td>
<td>8.8</td>
<td>1053</td>
<td>3.57</td>
<td>0.011</td>
<td>2.01</td>
<td>0.037</td>
<td>54.2</td>
<td>108.9</td>
<td>0.0639</td>
<td>0.0374</td>
<td>2.28</td>
</tr>
<tr>
<td>M3</td>
<td>2.7</td>
<td>1518</td>
<td>4.97</td>
<td>0.005</td>
<td>0.786</td>
<td>0.016</td>
<td>43.5</td>
<td>57.0</td>
<td>0.0884</td>
<td>0.0439</td>
<td>2.28</td>
</tr>
<tr>
<td>M4</td>
<td>4.4</td>
<td>511</td>
<td>5.18</td>
<td>0.006</td>
<td>0.402</td>
<td>0.015</td>
<td>63.4</td>
<td>218.0</td>
<td>0.0662</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>M5</td>
<td>6.6</td>
<td>540</td>
<td>4.66</td>
<td>0.005</td>
<td>1.37</td>
<td>0.016</td>
<td>89.0</td>
<td>121.8</td>
<td>0.0256</td>
<td>0.0250</td>
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<td>M6</td>
<td>3.3</td>
<td>450</td>
<td>5.18</td>
<td>0.005</td>
<td>0.638</td>
<td>0.016</td>
<td>22.9</td>
<td>14.7</td>
<td>0.1599</td>
<td>0.0662</td>
<td>1.08</td>
</tr>
<tr>
<td><strong>Pine Creek</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
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<tr>
<td>P1</td>
<td>23.5</td>
<td>727</td>
<td>7.73</td>
<td>0.184</td>
<td>3.04</td>
<td>1.19</td>
<td>2.55</td>
<td>7.77</td>
<td>0.0653</td>
<td>0.0580</td>
<td>2.28</td>
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<tr>
<td>P2</td>
<td>17.7</td>
<td>1819</td>
<td>11.90</td>
<td>0.086</td>
<td>2.08</td>
<td>0.57</td>
<td>2.14</td>
<td>8.46</td>
<td>0.119</td>
<td>0.0573</td>
<td>-</td>
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<td>1257</td>
<td>10.13</td>
<td>0.085</td>
<td>2.08</td>
<td>0.57</td>
<td>2.14</td>
<td>8.46</td>
<td>0.119</td>
<td>0.0573</td>
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<tr>
<td>P4</td>
<td>13.1</td>
<td>2180</td>
<td>12.22</td>
<td>0.041</td>
<td>1.07</td>
<td>0.50</td>
<td>2.14</td>
<td>8.46</td>
<td>0.327</td>
<td>0.0820</td>
<td>-</td>
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<tr>
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<td>12.4</td>
<td>1747</td>
<td>11.99</td>
<td>0.042</td>
<td>1.04</td>
<td>0.50</td>
<td>2.14</td>
<td>8.46</td>
<td>0.327</td>
<td>0.0820</td>
<td>-</td>
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<tr>
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<td>10.9</td>
<td>1825</td>
<td>11.99</td>
<td>0.043</td>
<td>0.618</td>
<td>0.112</td>
<td>1.79</td>
<td>1.94</td>
<td>0.328</td>
<td>0.0820</td>
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<td>14.3</td>
<td>1475</td>
<td>11.10</td>
<td>0.038</td>
<td>1.28</td>
<td>0.386</td>
<td>3.26</td>
<td>4.20</td>
<td>0.1346</td>
<td>0.0231</td>
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<tr>
<td>P8</td>
<td>21.1</td>
<td>960</td>
<td>9.87</td>
<td>0.051</td>
<td>2.49</td>
<td>0.286</td>
<td>8.32</td>
<td>22.8</td>
<td>0.0489</td>
<td>0.0231</td>
<td>-</td>
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<tr>
<td>P9</td>
<td>15.2</td>
<td>990</td>
<td>6.20</td>
<td>0.038</td>
<td>2.94</td>
<td>0.135</td>
<td>2.16</td>
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<td>0.0483</td>
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<td>672</td>
<td>7.39</td>
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<td>0.169</td>
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<td>11.0</td>
<td>572</td>
<td>7.52</td>
<td>0.018</td>
<td>1.46</td>
<td>0.143</td>
<td>10.2</td>
<td>14.0</td>
<td>0.0927</td>
<td>0.0481</td>
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</tbody>
</table>

a: Estimated based on the simplified super-elevation formula V = (2g)²ΔΔ / 2ΔΔ, where ΔΔ = centerline radius of curvature.

ΔΔ = super-elevation, and ΔΔ = channel width.

b: Taken from 1:400 scale maps over 500 to 1000 feet of channel.

c: Reconstituted data.
unsteady mudflow equations for wide channel are identical to the counter-
parts of the 1-D unsteady clear-water flow equations, except for the
different values of the flow parameters such as the momentum (or energy)
correction factor and the resistance coefficient. Extending such unsteady
flow equations for wide channels to a more general case of channels with
section of arbitrary geometric shape requires again that they be semi-
empirically reformulated in terms of the cross-sectional area and the
hydraulic radius. In the simplest case of no erosion and deposit of sediment
in the transport process, the 1-D dynamic mudflow equations can be expressed as

\[
\frac{3A}{3t} + \frac{3(\Delta V)}{3x} = 0
\]

\[
\frac{3(\Delta V)}{3t} + \frac{3(\Delta V^2)}{3x} = g A S_0 - g \cos \theta \frac{2(Ah)}{3x} - g A S_f
\]

in which \(A\) is the cross-sectional area of flow; \(t\) is time; \(x\) is the space
coordinate in the longitudinal direction of flow; \(\beta\) is the momentum
correction factor; \(\bar{h}\) is the depth of the centroid of the cross-sectional area;
and \(S_f\) is the friction slope. Unlike in wide channels, the \(\beta\) value for flow
in channels of arbitrary cross-sectional shape cannot be theoretically
determined. The empirically-determined \(\beta\) value should be used instead.

The friction slope, \(S_f\), for unsteady mudflow is different from that for
steady mudflow, but for the first-order approximation, may be assumed to be
the same as for uniform mudflow as usually treated in clear-water open-
channel hydraulics [Chow, 1959]. Therefore, using the Darcy-Weisbach equation
(Eq. 13) or the Manning formula (Eq. 42), one can express \(S_f\) in terms of \(V\)
and \(n\), and then substitute it into Eq. 49 with \(F\) or \(n\) to be determined from
other means. For example, if a sediment-water mixture in question is treated as a
Mingham plastic fluid with known \(c\), \(\phi\), and \(u\), the value of \(r_f/R\) can be
calculated from Eq. 10, then the value of \(C\) from Eq. 15, and finally Darcy-
Weisbach's \(F\) from Eq. 14. Similarly Manning's \(n\) can be determined from
Eq. 43, provided the value of \(n_0\) for fully turbulent mudflow can be
reasonably estimated.

In the absence of data on \(\beta\) for mudflow in channels with section of
arbitrary geometric shape, it may be assumed unity for simplicity in analysis.
Eq. 49 upon substitution of \(\beta = 1\) and with the aid of Eq. 48 reduces to

\[
\frac{3V}{3t} + \frac{2V}{3x} = g (S_0 - S_f) - g \cos \theta \frac{2(Ah)}{3x} - g A S_f
\]

(50)

Unless there is a rapid change in the cross-sectional area, \(A\), such as at an
alluvial fan, the assumption of \(\beta = 1\) seems justified. Therefore, the
simplified 1-D dynamic mudflow equations may consist of Eqs. 48 and 50 with
\(S_f\) to be expressed in terms of Darcy-Weisbach's \(F\) or Manning's \(n\). At an
alluvial fan, however, it would be more accurate to use a 2-D depth-averaged flow model for describing the deposition process of debris flow. This problem will be taken up in a future study.

SUMMARY AND CONCLUSIONS

The previously developed uniform mudflow formula for wide channels has been extended to a more general case of channels with section of arbitrary geometric shape, using the basic concepts of open-channel hydraulics. In other words, the previous formula has been semi-empirically reformulated by replacing the flow depth by the hydraulic radius. A combination of the Darcy-Weisbach equation with this semi-empirically reformulated uniform mudflow formula has further enabled one to express the Darcy-Weisbach resistance coefficient in terms of the generalized Reynolds number and the redefined yield-stress index, \( r_0/R \). The practical usefulness of this semi-empirically reformulated uniform mudflow formula has been demonstrated through the evaluation of the three rheological parameters, \( n \), \( \mu \), and \( r_0/R \), for debris flows following the May 18, 1980 eruption of Mount St. Helens.

It has been shown that the yield-stress index, \( r_0/R \), varies considerably with the hydraulic radius, \( R \), and the channel slope, \( S_0 \) (or \( B \)) even though the mass of a given sediment-water mixture with known cohesion, \( c \), and static angle of internal friction, \( \phi \), does not alter the other rheological properties (i.e., \( n \) and \( \mu \)) during its movement down a valley. Therefore, directly applying the uniform mudflow equation statistically to all data measured at various locations in a straight channel may result in the misleading estimates of the three rheological parameters. Apparently, use of the uniform mudflow equation alone cannot estimate the values of the three rheological parameters of debris flow in the field. Another relation, such as Eq. 10, is thus needed in the determination of \( r_0/R \), which in turn can be incorporated with the uniform mudflow equation to evaluate \( n \) and \( \mu \), using a regression analysis.

To determine the rheological parameters in the laboratory, a flume can be built and precisely operated to control \( r_0/R \) of various uniform mudflows produced in the flume. It is imperative during experiments that a constant value of \( r_0/R \) be maintained for flow of a given sediment-water mixture, in which \( n \) and \( \mu \) are supposedly constant, provided that the uniform mudflow equation can be used in the least squares estimates of the rheological parameters.

For routing debris flows down a narrow valley, the 1-D dynamic equations, analogous to those for clear-water flow, can be formulated. It has been shown that the 1-D unsteady flow equations for debris flow are identical to those for clear-water flow. The difference between them reflects only in the different values of the flow parameters, such as the momentum (or energy) correction factor and the resistance coefficient. These flow parameters for debris flows in channels with section of arbitrary geometric shape can only be determined empirically. At an alluvial fan, where debris flows rapidly expand, a 2-D depth-averaged mudflow model should be used instead.
NOTATION

A  cross-sectional area of flow.
A₀  total cross-sectional area of flow.
B  Bingham number or yield number.
C  constant.
C₀  cohesion or cohesive strength.
D  pipe diameter.
D₀  Darcy-Weisbach friction coefficient.
fn  Fanning resistance coefficient.
g  gravitational acceleration.
H  Hedström number.
k  roughness size.
k₀  Manning resistance coefficient.
P₀  Manning resistance coefficient for fully turbulent flow.
Pₚ₀  total wetted perimeter of flow.
p  hydrostatic pressure.
R  hydraulic radius of flow.
Rn  Reynolds number.
Rₚn  Reynolds number defined in Eq. 33.
r  fictitious space coordinate.
r₀/R  fictitious space coordinate corresponding to the yield stress.
S₀  channel slope.
Sf  friction slope.
S₀  yield stress.
t  time.
V  area-averaged velocity component in the longitudinal direction of flow.
X  space coordinate in the longitudinal direction of flow.
E  energy correction factor for 1-D mudflow.
E₀  momentum correction factor for 1-D mudflow.
Ω  flow-behavior index.
θ  angle of inclination of channel bed.
μ  consistency index.
ρ  mass density of sediment-water mixture.
ρ₀  shear stress.
t₀  bed shear.
φ  static angle of internal friction of sediment-water mixture.

REFERENCES


Iida, K. 1938. The mudflow that occurred near the explosion crater of Mt. Bandai on May 9 and 15, 1938, and some physical properties of volcanic mud. Tokyo Imperial University, Earthquake Research Institute, Bulletin 16:658-681. (in Japanese)


ABSTRACT

The authors advance a better understanding of hyperconcentrated sediment flows, commonly referred to as debris flows or mudflows, with a fundamental investigation of the nature of fluid motion. In these flows of large concentrations of sediment, the predominant processes of energy dissipation are related to the viscous, turbulent, dispersive and yield stresses. The relative magnitude of these components largely depend on the fluid properties and whether the flow matrix consists of cohesive or noncohesive sediment. Based on experimental data, the following relationships are provided: 1) stress versus rate of strain, 2) viscosity versus sediment concentration, and 3) yield strength versus sediment concentration. These results expand our knowledge of the physical properties of hyperconcentrated flows.

The authors also review the application of fluid principles to these flows. The fundamentals of fluid mechanics are outlined for the case of hyperconcentrated flows on steep slopes with emphasis on the physical properties of non-Newtonian fluids. A theoretically sound and simplified methodology prescribes the engineering analysis for these hazard flows.

INTRODUCTION

Hyperconcentrated sediment flows are commonly referred to as mud flows or debris flows. The term hyperconcentrated, however, depicts a broader spectrum of sediment transport ranging from large concentrations of suspended sediment in streams to landslides. Sharp and Nobles (1953) refer to hyperconcentrated flows as debris flows instead of mud flows when fifty percent or more of the sediment in the flow matrix is coarser than sand. Debris flows have also been described as granular flows which are identified by the absence of fine material (silt and clays).

Hyperconcentrated flows originate in basins which can be delineated into three zones. The sediment source area is located in the uppermost region of the watershed and may be in a landslide area. The zone of sediment transport is a steep channel system in which erosion and deposition are generally in equilibrium. Finally, the alluvial fan is a depositional zone often identified by a break in the bed slope of the main channel.

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Alluvial fans in the mountain communities of Colorado and Utah have become attractive sites for development; homes, subdivisions and even entire towns are located on the fans of small watersheds. Recent growth and development trends are forcing construction to encroach on apparently inactive fans with devastating results. There has been a dramatic increase in the number of destructive and life threatening encounters with high hazard mud floods and mud flows. The annual cost incurred from these destructive events now exceeds millions of dollars. In 1984 damages in Colorado were estimated at $32 million. There is a critical need to delineate these hazard areas and develop a predictive methodology that will define the level of hazard.

The long-term objective of this research is to develop a predictive mathematical model for hyperconcentrated sediment flows that is manageable and cost effective. The model should be based on a thorough understanding of the physical processes and should predict the following flow properties at desired reach stations: peak discharge, time to peak, average velocity, average flow depth, volume of water and sediment, impact pressure, runout distances, and areas of inundation for a given event frequency.

A steep watershed model for water and sediment routing of overland flow and in open channels using the kinematic wave approximation for the momentum equation has been in use for several years. It remains to link the watershed model with constitutive equations for mud and debris flows, routing them down open channels and across alluvial fans. In addition, a complete model will require description of the mobilization processes of the eroded material which comes off the slope and enters the channels. Such processes may include rill and gully erosion, bank sloughing failure, landslide and overland flow.

The continuing research on hyperconcentrated sediment flows at Colorado State University is directed towards development of a predictive methodology. This requires a thorough understanding of the physical processes of water-sediment mixtures which can only be accomplished through a research program involving theoretical analysis, laboratory measurements and field investigation. Initially in this paper, the physical properties of hyperconcentrated sediment flows are defined, followed by a description of the shear stress relationships linked to the mechanics of hyperconcentrated flows. The proposed theoretical developments are supported by laboratory analysis from field samples. A simple methodology has been applied to 16 watersheds to evaluate the relative magnitude of internal to boundary energy losses.

DELINEATION OF HYPERCONCENTRATION SEDIMENT FLOW CATEGORIES

Hyperconcentrated sediment flows encompass a wide range of flow concentration conditions. An attempt to delineate mass wasting processes into several categories with different flow properties was initiated by the National Research Council Committee on Methodologies for Predicting Mud Flows (NRC, 1982). To refine the delineation with some physical properties of the fluid matrix, several experiments were performed at Colorado State University on mud flow samples extracted from undisturbed deposits located in Colorado. The samples were
analyzed for size fraction and silt and clay content, and the properties of the mixture were described for various water and sediment concentrations. The results were incorporated into the definitions promulgated by the NRC committee and are shown in Table I.

In nature, there exist a continuum of flow conditions and one hydrologic event may consist of several flow processes. Flow deposits, scour characteristics, and fan patterns are helpful tools in identifying the flow regimes and processes. Although the transition between the different types of flow are difficult to distinguish, mass wasting processes can be divided in four main categories: water floods, mud floods, mud flows, and landslides.

Conventional water flooding is defined as water inundation by overbank discharge. Sediment is transported through the mechanisms of suspension and rolling and saltation along the bed which depend largely on water velocity and turbulence. For water floods, standard hydrologic and sediment transport capacity methods and formulas are applicable. Water floods are not a phenomena analyzed in this paper.

Mud floods define a range of concentration from 20 to 45 percent by volume (Table I). This concentration refers to the fluid matrix and should be assumed to consist of silts, clays and fine sands only. Water floods and mud floods display inherent fluid properties, both are unable to resist shear stress without motion or exhibit any appreciable yield strength. Conventional analysis using momentum, energy and continuity equations are applicable. Sediment transport capacity equations such as Einstein and Meyer-Peter and Müller are inappropriate because higher viscosities of the mixture and lower fall velocities of solid particles invalidate the empirical constants which are based on clear water as the fluid medium. Water floods and mud floods are classified under the National Flood Insurance Program (NFIP) definition of floods by the NCR (see Figure 1).

In mud flows the sediment concentration is sufficient to support large clastic material in a quiescent condition without settling. The flow matrix exhibits a distinct resistance to motion (high yield strength). This resistance to shear stress is a pseudo-plastic flow property corresponding to high viscosities. The National Research Council (NRC, 1982) report states, "The key characteristic in differentiating between mud floods and mud flows is that a mud flow displays a combination of density and strength that will support inclusions of higher density than water, such as boulders, both during transport and when the mass comes to rest". Throughout the flow process the combination of fluid matrix density and small settling velocities keep the boulders near the surface in the absence of turbulence. In steep basins, mud flows are generated under certain conditions of rainfall and sediment availability. When unlimited supplies of sediment become available, the probability of producing a mud flow is very high for intense rainfall events. Debris flows are acknowledged as having more than fifty percent of the sediment sizes coarser than sand. Debris flows without fine materials (silt and clays) are referred to as granular flows.
### Table 1: Description of Hyperconcentrated Sediment Flow as a Function of Concentration

<table>
<thead>
<tr>
<th>Flow Type</th>
<th>Concentration by Volume $C_v$</th>
<th>Concentration by Weight $C_w$</th>
<th>Flow Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Landslides</td>
<td>0.53-0.90</td>
<td>0.75-0.96</td>
<td>Will not flow, failure by block sliding</td>
</tr>
<tr>
<td></td>
<td>0.50-0.53</td>
<td>0.73-0.75</td>
<td>Block sliding failure with internal deformation during the slide, slow creep prior to failure</td>
</tr>
<tr>
<td>Mud Flows</td>
<td>0.48-0.50</td>
<td>0.72-0.73</td>
<td>Flow evident, slow creep sustained mud flow, plastic deformation under its own weight, cohesive, will not spread on level surface</td>
</tr>
<tr>
<td></td>
<td>0.45-0.48</td>
<td>0.69-0.72</td>
<td>Begins spreading, cohesive</td>
</tr>
<tr>
<td></td>
<td>0.40-0.45</td>
<td>0.65-0.69</td>
<td>Mixes easily, shows fluid properties in deformation; spreads on horizontal surface but maintains a inclined fluid surface, large particle settling, waves appear but dissipate rapidly</td>
</tr>
<tr>
<td></td>
<td>0.35-0.40</td>
<td>0.59-0.65</td>
<td>Marked settling, spreading nearly complete on horizontal surface, liquid surface two phases appear, waves travel substantial distance</td>
</tr>
<tr>
<td></td>
<td>0.30-0.35</td>
<td>0.54-0.59</td>
<td>Separation of water on surface, two phases, waves travel easily, most sand and gravel has settled out</td>
</tr>
<tr>
<td></td>
<td>0.20-0.30</td>
<td>0.41-0.54</td>
<td>Distinct wave action, fluid surface, all particles resting on bottom in quiescent fluid condition</td>
</tr>
<tr>
<td>Water Flood</td>
<td>0.20</td>
<td>0.41</td>
<td>Water flood with bed and suspended loads</td>
</tr>
</tbody>
</table>

*This information is qualitative guideline in which the concentration refers to the fluid matrix consisting of silts, clays and fine sands. The concentration by weight is computed using 2.72 as the specific gravity for the sediment as measured in the laboratory.*
Landslides consist of downslope movement of earth by mechanisms of falling, toppling, sliding and spreading. Such earth movements may be either wet or dry. Landslides and bank slumps are an integral part of generating mud flows and mud floods in steep basins. This mechanism delivers source material to channel in brief singular events that often perturbate the channel flow hydraulics.

PHYSICAL PROPERTIES OF THE FLUID MATRIX

The presence of large concentrations of sediment induces complex processes of energy dissipation in the fluid matrix. Besides the viscous and turbulent stresses existing in clear water flows, the interaction of water and sediment, the exchange of sediment particles with the channel boundary, and the collisions of suspended particles (dispersive stress) all contribute to the dissipation of energy from the fluid matrix. Moreover, the presence of clay particles whose cohesive forces arise from hydrophilic bonding, modifies the physical processes governing the fluid flows. Hyperconcentrated sediment flows, therefore, are a function of complex interrelationships between water and sediments which require further investigation.
Conventionally, the force in fluids necessary to produce a given deformation is proportional to the rate of deformation. For a real fluid in motion relative to a rigid boundary, shear stresses develop in proportion to the rate of angular deformation. This definition implies that shear stresses will exist only in moving fluids. Figure 2 shows different relationships between shear stress and the rate of deformation.

![Figure 2. Behavior of Fluids](image)

**Newtonian Fluids** follow a linear relationship between shear stress and rate of strain in which the slope of the line is the viscosity $\mu$ of the fluid. The Bingham plastic model combines a yield stress $k$ and a linear stress-strain relationship for shear stresses in excess of the yield value. Hyperconcentrated flows are non-Newtonian, the shear stress exhibited by the flow is not proportional to a linear rate of strain. Bingham plastic and visco-plastic relationships are commonly used to describe hyperconcentrated flows. The property of a yield stress which must be exceeded to initiate motion gives rise to the plastic or Bingham nature of the fluid.

In the analysis of most rivers and streams, the sediment being transported has negligible effect on the Newtonian properties of the fluid (water). In mud flows, however, large concentrations of fine sediment alter the fluid properties, particularly viscosity and turbulence. For the case of mud flows, the 'fluid' consists of the water and fine sediment and is referred to as the fluid matrix. Mud flows generally transport large clastic material, including large boulders. The clastic material is suspended in the fluid matrix, often being rafted on or near the surface of the flow. The large concentration of fine material (silt and clay) have altered the fluid matrix.
properties of viscosity and density and, therefore, the lift, drag and buoyancy forces acting on the particle exceed that which would have been exerted by water alone. The fluid matrix consists of the fluid plus the sediment particles which will have a negligible fall velocity in a quiescent condition.

Consider the case of granular flows in which the fluid matrix is water and the sediment is virtually all noncohesive elastic material. Granular flows may be either wet or dry (Passman et al., 1980, Nunziato and Passman, 1980 and Savage, 1979). The fluid medium is water and the fall velocity of the particle is large due to the absence of fines and the corresponding small viscosity of the fluid matrix.

Concentration and flow properties should be expected to change with larger concentrations of silt and clay. Graf (1971) reported that the fall velocity of particles decreases with the addition of fine sediment to water. A small percent concentration by weight of sediment in flowing water dampens turbulent eddies (Vanoni, 1941). Bagnold (1956) further indicated that at high concentrations of sediment, the turbulence may disappear altogether. Increasing the concentration of fines has the effect of increasing both the viscosity and density of the flow. Viscosities of actual debris flow deposits have been measured in the laboratory in excess of 1000 poises (the viscosity of water is about 0.01 poises).

The sediment concentration determines the physical characteristics of hyperconcentrated sediment flows. Concentration can be measured either by weight \( C_w \) or by volume \( C_v \) with a conversion of

\[
C_w = \frac{C_v \cdot G}{1 + (G-1)C_v}
\]

where \( G \) is the specific gravity of dry sediment. A concentration of 50% by volume corresponds to 73% concentration by weight using 2.65 as the specific gravity for the sediment. Referring to Table 1, 50% concentration by volume represents a perceived limit to a mud flow with some fluid properties as determined through laboratory experiments.

It is noteworthy that Bagnold (1954), in his paper on dispersive stress theory, described flows of uniform grains with a concentration by volume of 57% as a granular paste and 52% concentration by volume as the Newtonian fluid limit. In his calculations he correctly reported that the maximum concentration for spheres is 74% by volume with a lower value of 65% for natural, reasonably rounded uniform grains. Using some data from Lamb and Whitman (1969) and Us (1983) the concentrations in Table 2 were computed. The loosest stable arrangement for uniform spheres is a simple cubic structure with a concentration of 53% by volume. The average minimum volumetric concentration of several soil types shown in this table is 54%. For impending fluid motion of the sediment, the concentrations must decrease from these minimum values given in Table 2; otherwise the sediment would move as a block. This evidence supports the delineation of flow definitions indicated in Table 1.
**TABLE 2. CONCENTRATION BY VOLUME OF GRANULAR SOILS**
(Modified After Lamb and Whitman, 1969 and Das, 1983)

<table>
<thead>
<tr>
<th>Description</th>
<th>Minimum</th>
<th>( C_v )</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform Spheres</td>
<td>0.53</td>
<td>0.74</td>
<td></td>
</tr>
<tr>
<td>simple cubic configuration</td>
<td>0.53</td>
<td></td>
<td></td>
</tr>
<tr>
<td>bodied-centered cubic</td>
<td>0.68</td>
<td></td>
<td></td>
</tr>
<tr>
<td>face-centered cubic</td>
<td>0.74</td>
<td></td>
<td></td>
</tr>
<tr>
<td>hexagonal close-packed structure</td>
<td>0.74</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uniform Inorganic Silt</td>
<td>0.48</td>
<td>0.61</td>
<td></td>
</tr>
<tr>
<td>Standard Ottawa Sand</td>
<td>0.56</td>
<td>0.67</td>
<td></td>
</tr>
<tr>
<td>Clean Uniform Sand</td>
<td>0.50</td>
<td>0.61</td>
<td></td>
</tr>
<tr>
<td>Silty Sand</td>
<td>0.53</td>
<td>0.78</td>
<td></td>
</tr>
<tr>
<td>Fine Sand</td>
<td>0.54</td>
<td>0.71</td>
<td></td>
</tr>
<tr>
<td>Coarse Sand</td>
<td>0.57</td>
<td>0.74</td>
<td></td>
</tr>
<tr>
<td>Fine to Coarse Sand</td>
<td>0.51</td>
<td>0.83</td>
<td></td>
</tr>
<tr>
<td>Micaceous Sand</td>
<td>0.45</td>
<td>0.71</td>
<td></td>
</tr>
<tr>
<td>Silty Sand and Gravel</td>
<td>0.54</td>
<td>0.88</td>
<td></td>
</tr>
<tr>
<td>Gravelly Sand</td>
<td>0.59</td>
<td>0.83</td>
<td></td>
</tr>
<tr>
<td>Gravel</td>
<td>0.63</td>
<td>0.77</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>0.54</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
In the field, higher concentrations may be possible with larger quantities of silt and clay in the flow matrix. Written accounts of mud flows describe a wide range of concentrations with maximum concentrations by weight as high as 79 to 85% (Sharp and Nobles, 1953), 60 to 78% (Pierson, 1981), 59 to 86% (Pierson, 1980), 60 to 90% (Johnson, 1970) and 91% (Curry, 1966). Any loss of water during the sampling process, however, could result in significantly higher concentrations than actually occurred during the flow events. Surges and nonuniformity in the flow concentrations also distort the measured estimates of the flow properties. It is suggested that attempts at reporting mud and debris flow events should focus on a description of the mean flow properties which will assist in developing future predictive methods.

MECHANICS OF HYPERCONCENTRATED SEDIMENT FLOWS

The predominant processes of energy dissipation and resistance to motion are a function of the viscous, turbulent, dispersive and yield shear stresses. The relative magnitude of these stresses largely depend on the fluid properties, the concentration of sediment and whether the flow matrix includes cohesive sediment. Although the initiation of motion through landslides and creeping soil failures are more properly examined through a soil mechanics approach, the hyperconcentrated flows should be analyzed in a continuum approach to describe a wide range of concentrations ranging from clear water to very viscous mud flows.

Newton's second law is applied to describe the one-dimensional motion of an incompressible water-sediment mixture. The force equilibrium per unit mass may be written as

\[ \frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} = g \sin \theta - \frac{1}{\rho_m} \frac{\partial p}{\partial x} + \frac{1}{\rho_m} \frac{\partial}{\partial y} \left( \tau - \mu \frac{\partial u}{\partial y} \right) \]  

where \( \rho_m \) is the density of the fluid mixture, \( u \) is the velocity in the downstream \( x \)-direction, \( p \) is the internal pressure, \( \tau \) is the shear stress, \( g \) is the gravitational acceleration, \( \sin \theta \) is the channel slope and \( y \) is the upward distance above the channel bed perpendicular to the flow. The left side of the equation represents the local and convective acceleration of the fluid. These terms depict the unsteadiness and nonuniformity of the flow. The right side of the equation represents the gravity, pressure, and resistive shear stress terms. In the original Navier-Stokes equation valid for Newtonian fluids, the pressure distribution can be assumed to be hydrostatic and the shear stress is a function of the viscosity \( \mu \) and of the rate of deformation

\[ \tau = \mu \frac{\partial u}{\partial y} \]  

In mud flows, however, the shear stress is a complex function of the water and sediment properties comprising the fluid matrix which limits its direct application for predictive modeling. A general equation postulated for the resistive shear stress in a water sediment mixture is
\[ \tau = k + \mu_{m} \frac{\partial u}{\partial y} + C_1 \left( \frac{\partial u}{\partial y} \right)^2 + \ldots \] (4)

where \( k \) is the yield stress, \( \mu_m \) is the viscosity of the fluid matrix and \( C_1 \) may be a variable whose magnitude depends on depth and concentration. The combination of the first two terms is referred to as the Bingham model for mud flows. The Bingham model consists of a yield stress term and a viscous stress term. This model is applicable when the applied shear stress exceeds the yield stress (\( \tau > k \)) and the resistive stress is linearly proportional to the rate of strain. Both the yield stress \( k \) and the viscosity \( \mu_m \) are functions of concentration as shown in Figures 3 and 4. The Bingham model can be used somewhat successfully to describe the motion of mud flows in smooth prismatic open channels for partially turbulent or transitional flows without energy losses due to roughness.

The \( C_1 \left( \frac{\partial u}{\partial y} \right)^2 \) term is a composite of the dispersive and turbulent stresses. It is referred to as the inertial term in the shear stress equation. The conventional representation for the turbulence stresses, in clear water is

\[ \tau_B = \rho \kappa^2 y^2 \left( \frac{\partial u}{\partial y} \right)^2 \] (5)

in which \( \rho \) is the density of clear water and \( \kappa \) is the von Karman constant. The dispersive stress arising from the collision of sediment particles as defined by Bagnold (1954) is

\[ \tau_D = a_1 \Lambda^2 \left( \frac{D_s}{\lambda} \right)^2 \left( \frac{\partial u}{\partial y} \right)^2 \] (6)

where \( D_s \) is a representative grain diameter and \( a_1 \) is a constant. The linear concentration \( \Lambda \) can be written as a function of the concentration by volume \( C_v \) and the maximum possible static concentration by volume \( C_0 \)

\[ \Lambda = \frac{1}{(C_v/C_0)^{1/3} - 1} \] (7)

In a water and sediment mixture, these two stresses can be combined in Eq. 4 since both are functions of the second power of the rate of deformation. The turbulent stresses assist in suspending particles into the flow by exchanging momentum from the fluid to the sediment particles. The dispersive stresses meanwhile, impart momentum transfer between the particles. Increasing concentration and the corresponding collisions between the particles dampens turbulence. A characteristic of turbulence is the irregular or random motion of a fluid which generates pseudo-stresses in the sense that they originate from the acceleration terms. The momentum is transferred to the boundary by viscous diffusion (vorticity). The flow is mixed through eddies and the stretching of vortices create smaller eddies and drive the interaction between eddies of different sizes. High sediment concentration dampens
$k = 0.00886 e^{13.11C_v}$

$r^2 = 0.89$

**FIGURE 3. YIELD STRESS vs. CONCENTRATION BY VOLUME**
The viscosity of clear water
\( \mu = 0.01 \) poise

\[
\mu_m = 0.650 e^{16.81 C_v}
\]

\( r^2 = 0.85 \)

FIGURE 4. VISCOSITY vs. CONCENTRATION BY VOLUME
the eddies in a cumulative manner dissipating the smaller eddies first or hampering their formation altogether. Sharp and Nobles (1953) noted this phenomena in their descriptive paper. In this fashion, the energy is sapped from the main body of the flow and expended to increase the sediment particle velocity and the height of suspension. This energy is distributed throughout the various levels of the flow and is eventually lost in the fluid mixture through viscous heat.

The turbulence and dispersive stresses lose their separate identities in a hyperconcentrated sediment flow and both stresses can be combined in the last term of Eq. 4. The stress-strain relationship given by Eq. 4 is promoted as correctly representing the behaviour of hyperconcentrated sediment mixtures. This relationship is theoretically sound since it is derived from fundamental principles in fluid mechanics, and the parameters of this function represent physical quantities. The relative magnitude of these parameters depends on the composition of the water-sediment mixture which can be described by (the concentration by weight \( C_w \) or the concentration by volume \( C_v \) and the concentration of fine material \( C_f \).

Equation 4 was tested in laboratory analysis using a rotating viscometer to measure the stress-strain relationship of a fluid matrix from a mud flow deposit. The results are shown in Figure 5. The physical properties defined by the relationship are the yield stress \( (k = 0.0108 \text{ lb/ft}^2) \), the viscosity of the fluid matrix \( (\mu_m = 0.00065 \text{ lb-s/ft}^2 = 0.31 \text{ poises}) \) and \( C_f = 0.0065 \text{ lb-s}^2/\text{ft}^2 \). The viscosity of the mixture is about thirty times larger than that of clear water. The parabolic relationship defined by regression analysis generates a better fitting curve \( (r^2 = 0.98) \) than a linear relationship between stress and strain rate \( (r^2 = 0.95) \). The Bingham model erroneously predicts a viscosity \( (9.43 \text{ poises}) \) thirty times larger than the Eq. 4.

The ratio \( R \) of the inertial stress term to the viscous stress, the last two terms on the right side of Eq. 4, is

\[
R = \frac{C_f}{\mu_m} \frac{\partial u}{\partial y}
\]

This non-dimensional ratio defines the relative magnitude of the inertial to viscous stresses in a form similar to the Rouse number for clear water turbulent flows. This ratio supersedes the use of any critical Reynolds number which is not applicable to delineate non-Newtonian flow regimes. A small value of \( R \) indicates the predominance of viscous stresses and suggest the use of a Bingham model rather than the complete solution of Eq. 4. The value of \( C_f \) is determined through laboratory analysis from Figure 3 and is a function of the sediment concentration, particle diameter, flow depth and clay concentration.

Similarly the Bingham number can be written as the ratio of the yield stress to the viscous stress.
\( \tau = 0.0108 + 0.00065 \frac{\partial u}{\partial y} + 0.0065 \left( \frac{\partial u}{\partial y} \right)^2 \)

**FIGURE 3. SHEAR STRESS vs. RATE OF STRAIN**
TABLE 3. RATIO OF INTERNAL TO BOUNDARY ROUGHNESS STRESSES $R_s$

<table>
<thead>
<tr>
<th>$C_v$</th>
<th>Flood Event Return Period in Years</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td>.36</td>
<td>2.0</td>
</tr>
<tr>
<td>.38</td>
<td>3.3</td>
</tr>
<tr>
<td>.40</td>
<td>5.3</td>
</tr>
<tr>
<td>.42</td>
<td>9.5</td>
</tr>
<tr>
<td>.44</td>
<td>17.0</td>
</tr>
<tr>
<td>.46</td>
<td>30.0</td>
</tr>
<tr>
<td>.49</td>
<td>48.2</td>
</tr>
<tr>
<td>.51</td>
<td>71.5</td>
</tr>
<tr>
<td>.54</td>
<td>88.3</td>
</tr>
</tbody>
</table>

$R_s$ values in percent, Standard Error ranged from 0.3 to 3.9%

This simplified analysis reveals the importance of the physical properties of the fluid matrix and prescribes the need for more fundamental research on mud flows. Ignoring either the viscous or friction slope term in the analysis would result in the overprediction of the velocity of the flow. There are inaccuracies in this analysis. First, the Manning's equation is only applicable for fully developed rough turbulent flows. Second, the Bingham model is not applicable for high velocity, rough turbulent flow. Mud flows and debris are inherently unsteady, nonuniform flows. On steep slopes, using the kinematic wave analogy, equation (15) should be solved using the three terms of Eq. 4 and this requires the use of $C_1$. More experimental analysis is required for the evaluation of $C_1$ and its variability with concentration, sediment size and boundary roughness. A stainless steel viscometer has been designed for this purpose.

CONCLUSION

The devastating effects of hyperconcentrated sediment flows in the past demonstrate an urgent need for a predictive methodology to define the hazard levels and to aid in the design of adequate mitigations measures and structures. Such a methodology would rely on an accurate knowledge of the physical properties of the water-sediment mixture. Research efforts must be focused on fundamental investigations involving both theoretical and experimental analysis.

This paper emphasizes the physical properties of hyperconcentrated sediment flows. Flow descriptions have been classified as a function of the concentration of sediments. Various experimental, theoretical and field data show that the maximum concentration by volume for mud flows is unlikely to be in excess of 0.50.

Basic fluid mechanics principles are recommended to describe the broad continuum of hyperconcentrated flows. A simple quadratic model (Eq. 4) is postulated, in which each term represents a well-defined physical property of the fluid. The last term of this equation
represents the inertial losses and requires further investigation. Physical meaning of this term, however, has been demonstrated to be associated with turbulent and dispersive stresses in the fluid matrix and has been verified experimentally. Empirical relationships between yield stress, viscosity and the concentration by volume were obtained from laboratory analysis of mud flow samples.

A simplified methodology, applied on 16 small steep watersheds near Glenwood Springs, Colorado, showed that at low concentrations by volume \( C_v < 40\% \), the energy losses are controlled by boundary roughness. For larger concentration, the internal energy losses rapidly become dominant. For the mathematical routing of open channel flows both a macro- and microscopic fluids approach must be applied. The macroscopic approach is needed for energy dissipation attributed to channel roughness. The real energy losses occur in the form of heat dissipation from the viscous interaction between water and sediment particles. The viscous energy dissipation is enhanced by turbulence which, in turn, is promoted by boundary roughness.

Continuing research at CSU on hyperconcentrated flows will focus on the complex physical processes of hyperconcentrated sediment flows preparing the foundation for the eventual mathematical routing of these flows in open channels.

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LIST OF SYMBOLS

- \( a_i \) coefficient of the dispersive stress equation
- \( B \) Bingham number
- \( c_b \) resistance coefficient for boundary roughness
- \( C_0 \) maximum possible concentration by volume
- \( C_{1f} \) coefficient of the inertial stress term
- \( C_v \) concentration of fines
- \( C_w \) concentration by weight
- \( d \) flow depth
- \( D_s \) representative grain diameter
- \( g \) gravitational acceleration
G: specific gravity of sediments
k: yield stress
p: internal pressure
r^2: coefficient of determination
R: ratio of inertial stress to viscous stress
R_s: ratio of internal stress to boundary roughness stress
S_b: boundary energy gradient
S_i: internal energy gradient
s: bed slope
t: time
u: velocity
u: mean velocity
x: longitudinal coordinate (positive downstream)
y: upward distance above the channel bed
κ: von Karman constant
θ: angle of the channel with the horizontal
λ: linear concentration
μ_m: dynamic viscosity of the mixture
ρ: density of clear water
ρ_m: density of the fluid mixture
τ: shear stress
τ_b: shear stress from the boundary roughness
τ_D: dispersive stress
τ_i: internal stress
τ_R: turbulence stress

REFERENCES


Dam Breach Erosion Modeling

A BREACH EROSION MODEL FOR EARTHEN DAMS

by D.L. Fread

ABSTRACT. A physically based mathematical model (BREACH) to predict the discharge hydrograph emanating from a breached earthen dam is presented. The earthen dam may be man-made or naturally formed by a landslide. The model is developed by coupling the conservation of mass of the reservoir inflow, spillway outflow, and breach outflow with the sediment transport capacity of the unsteady uniform flow along an erosion-formed breach channel. The bottom slope of the breach channel is assumed to be essentially that of the downstream face of the dam. The growth of the breach channel is dependent on the dam's material properties ($D_{50}$ size, unit weight, friction angle, cohesive strength, and flow resistance factor), and an empirical factor which accounts for the effects of a grass cover. The model considers the possible existence of the following complexities: 1) core material having properties which differ from those of the downstream face of the dam; 2) the necessity of forming an eroded ditch along the downstream face of the dam prior to the actual breach formation by the overtopping water; 3) enlargement of the breach through the mechanism of one or more sudden structural collapses due to the hydrostatic pressure force exceeding the resisting shear and cohesive forces; 4) enlargement of the breach width by slope stability theory; and 5) initiation of the breach via piping with subsequent progression to a free surface breach flow. The outflow hydrograph is obtained through a time-stepping iterative solution that requires only a few seconds for computation on a main-frame computer. The model is not subject to numerical stability or convergence difficulties. The model's predictions are compared with observations of a piping failure in the man-made Teton Dam in Idaho and a breached landslide-formed dam in Peru. Also, the model has been used to predict possible downstream flooding from a potential breach of the landslide blockage of Spirit Lake in the aftermath of the eruption of Mount St. Helens in Washington. Model sensitivity to numerical parameters is minimal; however, it is sensitive to the material cohesion, friction angle, and the empirical grass cover factor when simulating man-made dams and to the cohesion and flow resistance factor when simulating landslide-formed dams.

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Earthen dams are subject to possible failure from either overtopping or piping water which erode a trench (breach) through the dam. The breach formation is gradual with respect to time and its width as measured along the crest of the dam usually encompasses only a portion of the dam's crest length. In many instances, the bottom of the breach progressively erodes downward until it reaches the bottom of the dam; however, in some cases, it may cease its downward progression at some intermediate elevation between the top and bottom of the dam. The size of the breach, as constituted by its depth and its width (which may be a function of the depth), and the rate of the breach formation determine the magnitude and shape of the resulting breach outflow hydrograph which is of vital interest to hydrologists and engineers concerned with real-time forecasting or evacuation planning for floods produced by dam failures.

This paper presents a mathematical model (BREACH) for predicting the breach outflow hydrograph. The model is physically based on the principles of hydraulics, sediment transport, soil mechanics, the geometric and material properties of the dam, and the reservoir properties (storage volume, spillway characteristics, and time dependent reservoir inflow rate). The dam may be either man-made or naturally formed as a consequence of a landslide. In either, the mechanics of breach formation are very similar, the principal difference being one of scale. The landslide-formed dam is often much larger than even the largest of man-made earthen dams as illustrated in Fig. 1. The critical material properties of the dam are the internal friction angle, cohesion strength, and average grain size diameter ($D_{50}$).

The breach erosion model presented herein for synthesizing a dam-breach outflow hydrograph differs from the parametric approach which the author has used in the NWS DAMBRK Model (Fread, 1977, 1981, 1983). The parametric model uses empirical observations of previous dam failures such as the breach width-depth relation, time of breach formation, and depth of breach to develop the outflow hydrograph. The breach erosion model presented herein can provide some advantages over the parametric breach model for application to man-made dams since the critical properties used by the model are measurable or can be estimated within a reasonable range from a qualitative description of the dam materials. However, it should be emphasized that even if the properties can be measured there is a range for their probable value and within this range outflow hydrographs of varying magnitude and shape will be produced by the model. The hydrologist or engineer should investigate the most critical combination of values for the dam's material properties. It is considered essential when predicting breach outflows of landslide dams to utilize a physically based model since observations of such are essentially non-existent, rendering the parametric approach infeasible.

In this paper, the breach erosion model is applied to the piping failure of the man-made Teton Dam in Idaho, the overtopping failure of the Mantaro landslide-formed dam in Peru, and the possible failure of the recently formed landslide blockage of Spirit Lake, near Mount St. Helens in Washington.
Figure 1 - Comparative View of Natural Landslide Dams and Man-Made Dams.
PREVIOUS RESEARCH

Other investigators of dam breach outflows have developed physically based models.

The first was Cristofano (1967) who derived an equation which related the force of the flowing water through the breach to the shear strength of the soil particles on the bottom of the breach and in this manner developed the rate of erosion of the breach channel as a function of the rate of change of water flowing through the breach. He assumed the breach bottom width to be constant with time and always of trapezoidal shape in which the side slopes of the trapezoid were determined by the angle of repose of the breach material, and the bottom slope of the breach channel was equal to the internal friction angle of the breach material. An arbitrary empirical coefficient which was critical to the model's prediction was also utilized.

Harris and Wagner (1967) used the Schoklitsch sediment transport equation and considered the breach to commence its downward progression immediately upon overtopping, and the erosion of the breach was assumed to progress to the bottom of the dam. Brown and Rogers (1977) presented a breach model which was based on the work of Harris and Wagner.

Most recently Ponce and Tsivoglou (1981) presented a rather complex breach erosion model which coupled the Meyer-Peter and Müller sediment transport equation to the one-dimensional differential equations of unsteady flow and sediment conservation. Reservoir storage depletion was included in the upstream boundary equation used in conjunction with the unsteady flow equations. The set of differential equations was solved with a four-point implicit finite difference scheme. Flow resistance was represented through use of the Manning n. Breach width was empirically related to the rate of breach flow. A small rivulet was assumed to be initially present along the flow path. "Outflow at start of the computation is a function of the assumed initial size of the rivulet. Progressive erosion widens and deepens the rivulet, increasing outflow and erosion rate in a self-generating manner. The upper cross-section on the sloping downstream face creeps upstream across the dam top until it reaches the upstream face, whereby rate of flow and erosion increase at a faster rate. If outflow increases enough to lower the reservoir level faster than the channel bed erodes, both outflow and erosion gradually diminish. Of course, outflow will eventually decrease even if the breach bed erodes all the way down to the stream bed. This mode of failure creates the outflow hydrograph in the shape of a sharp but nevertheless gradual flood wave.″ Ponce and Tsivoglou compared the model's predictions with observations of a breached landslide-formed dam on the Mantaro River in Peru. The results were considered good. However they were influenced by the judicious selection of the Manning n, the breach width-flow relation parameter, and a coefficient in the sediment transport equation, although Ponce and Tsivoglou stated that the selected values were within each one's reasonable range of variation. Also, problems of a numerical computational nature were alluded to in connection with solving the implicit finite difference unsteady flow equations. They also implied that further work was needed to improve the breach width-flow relation and
in developing a relation between the Manning n and the hydraulic/sediment characteristics of the breach channel.

The breach erosion model presented in this paper differs substantially from those previously reported. A summation of the important differences will be given after the model has been completely described in the next section.

MODEL DESCRIPTION

General

The breach erosion model (BREACH) simulates the failure of an earthen dam as shown in Fig. 2. The dam may be homogeneous or it may consist of two materials, an outer zone with distinct material properties (φ—friction angle, C—cohesion, D₅₀—average grain size, and γ—unit weight) and an inner core with its φ, C, D₅₀, and γ values. The downstream face of the dam is described by specifying the top of the dam (H₀), the bottom elevation of the dam (Hₑ) or original streambed elevation, and its slope as given by the ratio l (vertical) : 2D (horizontal). Then, the upstream face of the dam is described by specifying its slope as the ratio l (vertical) : 2ZU (horizontal). If the dam is man-made it is further described by specifying a flat crest width (Wₑ) and a spillway rating table of spillway flow vs. water elevation, in which the first elevation represents the spillway crest. Naturally formed landslide dams are assumed to not have a flat crest or, of course, a spillway.

The storage characteristics of the reservoir are described by specifying a table of surface area (Sₐ) in units of acre-ft vs. water elevation, the initial water surface elevation (Hₑ) at the beginning of the simulation, and a table of reservoir inflows (Qᵢ) in cfs vs. the hour of their occurrence (Tᵢ).

If an overtopping failure is simulated, the water level (H) in the reservoir must exceed the top of the dam before any erosion occurs. The first stages of the erosion are only along the downstream face of the dam as denoted by the line A-A in Fig. 2 where, initially, a small rectangular-shaped rivulet is assumed to exist along the face. An erosion channel of depth-dependent width is gradually cut into the downstream face of the dam. The flow into the channel is determined by the broad-crested weir relationship:

\[ Q_b = 3 B_w (H-H_c)^{1.5} \]  

in which \( Q_b \) is the flow into the breach channel, \( B_w \) is the instantaneous width of the initially rectangular-shaped channel, and \( H_c \) is the elevation of the breach bottom. As the breach erodes into the downstream face of the dam, the breach bottom elevation (Hₑ) remains at the top of the dam (H₀).
Figure 2 – Side View of Dam Showing Conceptualized Overtopping Failure Sequence.
and the most upstream point of the breach channel moves across the crest of the dam towards the dam's upstream face. When the bottom of the erosion channel has attained the position of line B-B in Fig. 2, the breach bottom \( B \) starts to erode vertically downward. The breach bottom is allowed to progress downward until it reaches the bottom elevation of the dam \( H_d \) or in unusual circumstances to an elevation \( H_e \) that may be specified as lower than the bottom of the dam.

If a piping breach is simulated, the water level \( H \) in the reservoir must be greater than the assumed center-line elevation \( H_e \) of the Initially rectangular-shaped piping channel before the size of the pipe starts to increase via erosion. The bottom of the pipe is eroded vertically downward while its top erodes at the same rate vertically upwards. The flow into the pipe is controlled by orifice flow, i.e.,

\[
Q_b = 0.9B(2g)^{0.5} A (H - H_p)^{0.5}
\]  

(2)

in which \( Q_b \) is the flow into the pipe, \( g \) is the gravity acceleration constant, \( A \) is the cross-sectional area of the pipe channel, and \( H-H_p \) is the hydrostatic head on the pipe. As the top elevation \( H_p \) of the pipe erodes vertically upward, a point is reached when the flow changes from orifice-control to weir-control. The transition is assumed to occur when the following inequality is satisfied:

\[
H < R_{pu} + 2(H_{pu} - H_p)
\]  

(3)

The weir flow is then governed by Eq. (1) in which \( H_p \) is equivalent to the bottom elevation of the pipe and \( H_{pu} \) is the width of the pipe at the instant of transition. Upon reaching the instant of flow transition from orifice to weir, the remaining material above the top of the pipe and below the top of the dam is assumed to collapse and is transported along the breach channel at the current rate of sediment transport before further erosion occurs. The erosion then proceeds to cut a channel parallel to and along the remaining portion of the downstream face of the dam between the elevation of the bottom of the pipe and the bottom of the dam. The remaining erosion process is quite similar to that described for the overtopping type of failure with the breach channel now in a position similar to line A-A in Fig. 2.

The preceding general description of the erosion process was for a man-made dam. If a landslide dam is simulated the process is identical except, due to the assumption that the landslide dam has no crest width \( W_{cr} \), the erosion initially commences with the breach channel in the position of line B-B in Fig. 2. A failure mode of overtopping or piping may be initiated for a landslide-formed dam.
Breach Width

The method of determining the width of the breach channel is a critical component of any breach model. In this model the width of the breach is dynamically controlled by two mechanisms. The first, assumes the breach has an initial rectangular shape as shown in Fig. 3. The width of the breach \( B_o \) is governed by the following relation

\[
B_o = B_r y
\]

in which \( B_r \) is a factor based on optimum channel hydraulic efficiency and \( y \) is the depth of flow in the breach channel. The parameter \( B_r \) may vary from 2.0 to 2.5 for overtopping failures with the latter recommended on the basis of current testing of the model. For piping failures, \( B_r \) is set to 1.0. The model assumes that \( y \) is the critical depth at the entrance to the breach channel, i.e.,

\[
y = 2/3(H-H_c).
\]

The second mechanism controlling the breach width is derived from the stability of soil slopes (Spangler, 1951). The initial rectangular-shaped channel changes to a trapezoidal channel when the sides of the breach channel collapse, forming an angle (\( \theta \)) with the vertical. The collapse occurs when the depth of the breach cut \( (H') \) reaches the critical depth \( (H^c) \) which is a function of the dam's material properties of internal friction (\( \phi \)), cohesion (\( C \)), and unit weight (\( \gamma \)), i.e.,

\[
H'_k = \frac{\frac{4}{3} C \cos \phi \sin \phi_{k-1}}{\gamma \left(1 + \cos \left(\phi_{k-1} - \phi\right)\right)} \quad k = 1, 2, 3
\]

in which the subscript \( k \) denotes one of three successive collapse conditions as shown in Fig. 3, and \( \theta \) is the angle that the side of the breach channel makes with the horizontal as shown in Fig. 4. Thus, the angle \( \theta \) or \( \phi \) at any time during the breach formation is given as follows:

\[
\theta = \theta_{k-1} \quad H_k < H'_k
\]

\[
\theta = \theta_k' \quad H_k = H'_k
\]

\[
\theta = \theta_k \quad H_k > H'_k
\]

\[
\theta = \theta_{o_k} \quad k = 1
\]

\[
\theta = \theta_{o_{k-1}} \quad k > 1
\]
Figure 3 - Front View of Dam with Breach Formation Sequence.
Figure 4 - Front View of Dam with Breach.
\[ B_{om} = B_r y \quad \text{when } H_t = H'_t \quad (11) \]
\[ a = 0.5\pi - \theta \quad (12) \]

where:
\[ \theta_k' = 0.5\pi \quad (13) \]
\[ \theta_k' = (\theta_{k-1}' + \phi)/2 \quad k = 1, 2, 3 \quad (14) \]
\[ H_k = H'_t - y/3 \quad (15) \]

The subscript \((k)\) is incremented by 1 at the instant when \(H_k > H'_t\). In Eq. (15), the term \((y/3)\) is subtracted from \(H'_t\) to give the actual free-standing depth of breach cut in which the supporting influence of the water on the stability of the sides of the breach is taken into account. Through this mechanism, it is possible for the breach to widen after the peak outflow through the breach has occurred since the flow depth \((y)\) diminishes during the receding flow.

When the sides of the breach channel collapse, the breach bottom does not immediately continue to erode downward until the volume of collapsed material along the breach is removed at the rate of the sediment transport capacity of the breach channel at the instant of collapse. After this characteristically short pause, the breach bottom continues to erode downward.

When landslide dams are simulated, the relatively long breach channel lengths compared to those of man-made dams suggest that the width for the channel be computed apart from the entrance width of the breach. In this case, \(y\) in Eq. (4), (9), (11), and (15) is computed as the normal uniform depth \((y_n)\) in the breach channel rather than the critical depth given by Eq. (5). Equations for computing the normal channel depth are presented in a subsequent section.

Reservoir Level Determination

Conservation of mass is used to compute the change in the reservoir water surface elevation \((H)\) due to the influence of reservoir inflow \((Q_i)\), spillway outflow \((Q_{sp})\), crest overflow \((Q_o)\), breach outflow \((Q_b)\), and the reservoir storage characteristics. The conservation of mass over a time step \((\Delta t)\) in hours is represented by the following:

\[ \bar{Q}_i - (\bar{Q}_b + \bar{Q}_{sp} + \bar{Q}_o) = S_a \Delta H 43560 \quad (16) \]

\[ \Delta t = 3600 \]
in which \( \Delta H \) is the change in water surface elevation during the time interval \( (\Delta t) \), and \( S_a \) is the surface area at elevation \( H \). All flows are expressed in units of cfs and the bar (\( \bar{\cdot} \)) indicates the flow is averaged over the time step. Rearranging Eq. (16) yields the following expression for the change in the reservoir water surface:

\[
\Delta H = 0.0826 \Delta t \left( \bar{Q}_1 - \bar{Q}_b - \bar{Q}_{sp} - \bar{Q}_n \right)
\]  

(17)

The reservoir elevation \( H \) at time \( t \) can easily be obtained from the relation,

\[
H = H' + \Delta H
\]  

(18)

in which \( H' \) is the reservoir elevation at time \( t - \Delta t \).

The reservoir inflow \( (\bar{Q}_1) \) is determined from the specified table of inflows \( (Q_i) \) vs. time \( (T_i) \). The spillway flow \( (\bar{Q}_b) \) is determined from the specified table of spillway flows \( (Q_b) \) vs. reservoir elevation \( H \). The breach flow \( (\bar{Q}_b) \) is computed from Eq. (2) for piping flow. When the breach flow is weir-type, Eq. (1) is used when \( H_c = H_u \); however, when \( H_c < H_u \), the following broad-created weir equation is used:

\[
\bar{Q}_b = 3 B_0 \left( H - H_c \right)^{1.5} + 2 \frac{\tan(\alpha)}{n} \left( H - H_c \right)^{2.5}
\]  

(19)

in which \( B_0 \) is given by Eq. (9) or Eq. (10) and \( \alpha \) is given by Eq. (12). The crest overflow is computed as broad-created weir flow from Eq. (1), where \( B_0 \) is replaced by the crest length of the dam and \( H_c \) is replaced by \( H_u \).

**Breach Channel Hydraulics**

The breach flow is assumed to be adequately described by quasi-steady uniform flow as determined by applying the Manning open channel flow equation at each \( \Delta t \) time step, i.e.,

\[
\bar{Q}_b = 1.49 S^{0.5} A^{1.67} \frac{n}{P^{0.67}}
\]  

(20)

in which \( S = 1/2D \), \( A \) is the channel cross-section area, \( P \) is the wetted perimeter of the channel, and \( n \) is the Manning coefficient. In this model, \( n \) is computed using the Strickler relation which is based on the average grain size of the material forming the breach channel, i.e.,
in which $D_{50}$ represents the average grain size diameter expressed in mm.

The use of quasi-steady uniform flow is considered appropriate because the extremely short reach of breach channel, very steep channel slopes (1/20) for man-made dams, and even in the case of landslide dams where the channel length is greater and the slope is smaller, contribute to produce extremely small variation in flow with distance along the breach channel. The use of quasi-steady uniform flow as opposed to the unsteady flow equations as used by Ponce and Tsivoglou (1981) greatly simplifies the hydraulics and computational algorithm. Such simplification is considered commensurate with the other simplifications inherent in the treatment of the breach development in dams for which precise measurements of material properties are lacking or impossible to obtain and the wide variance which exists in such properties in many dams. The simplified hydraulics eliminates troublesome numerical computation problems and enables the breach model to require only minimal computational resources.

When the breach channel is rectangular, the following relations exist between depth of flow ($y_n$) and discharge ($Q_b$):

$$y_n = \left( \frac{b_n}{1.49 B_o^{0.5}} \right) \left( \frac{Q_b}{0.6} \right)$$

in which $B_o$ is defined by Eqs. (9-11).

When the breach channel is trapezoidal, the following algorithm based on Newton-Raphson iteration is used to compute the depth of flow ($y_n$):

$$y_{n+1} = y_n \frac{f(y_n)}{f'(y_n)}$$

in which

$$f(y_n) = Q_b - 0.67 y_n^{0.67} - 1.49 y_n^{0.5} A^{1.67}$$

in which

$$A = 0.5(B_o + B) y_n^k$$

$$B = B_{om} + y_n \tan(\alpha)$$

$$P = B_{om} + y_n \cos(\alpha)$$
\[ f'(y_n^k) = 0.67 \frac{Q_b}{p^{1/3}} - 1.67 \frac{1.49}{n} 0.5 B A 0.67 \]  

(28)

in which 
\[ F' = 1/cos(\alpha). \]  

(29)

The superscript \((k)\) is an iteration counter; the iteration continues until 
\[ |y_{n+1}^k - y_n^k| < \epsilon \quad \epsilon < 0.01 \]  

(30)

The first estimate for \(y_n\) is obtained from the following: 
\[ y_n^1 = \left( \frac{Q_b n}{1.49 B 0.5} \right)^{0.6} \]  

(31)

where: 
\[ \overline{B} = 0.5(B_m + B') \]  

(32)

in which \(B'\) is the breach channel top width at the water depth \((y_n)\) at \((t-\Delta t)\).

**Sediment Transport**

The rate at which the breach is eroded depends on the capacity of the flowing water to transport the eroded material. For man-made dams the Meyer-Peter and Müller sediment transport relation (Morris and Wiggert, 1972) is used, i.e.,

\[ Q_s = aP(SR - \tau_c)^{1.5} \]  

(33)

where:

- \(Q_s\) = sediment transport rate in cfs;
- \(a = 27.5;\)
- \(P\) = wetted perimeter of the breach channel as given by Eq. 27 for the trapezoidal shaped channel or by \((B_o + 2y_n)\) for the rectangular breach.
- \(R\) = hydraulic radius \((A/P)\);
- \(S\) = slope of the breach channel, \((1/ZD)\) for
Weir channels and piping channels:

\[ \left( \frac{n^2 q_b^2}{(2.21 A^2 R_{1.33})} \right) \]

- \( n \): critical shear stress = 0.0003 \( D_{50} \) \( C_v \); and

\( C_v \): empirical factor to account for additional resistance to sediment transport due to vegetative cover on the downstream face of the dam.

For landslide dams, the duBoys relation (Morris and Wiggert, 1972) is used, i.e.,

\[ Q_s = \frac{b}{D_{50}^{0.75}} PSR(SR - c_c) \]  

where:

\( b = 671 \).

The coefficients \( a \) and \( b \) in Eqs. (33) and (34) are set at the fixed values, 27.5 and 671, respectively. These values were used in all test applications of the breach erosion model (BREACH) presented herein. It was considered inappropriate to vary these coefficients.

**Breach Enlargement By Sudden Collapse**

It is possible for the breach to be enlarged by a rather sudden collapse failure of the upper portions of dam in the vicinity of the breach development. Such a collapse would consist of a wedge-shaped portion of the dam having a vertical dimension \( Y_c \) as shown in Fig. 5. The collapse would be due to the pressure of the water on the upstream face of the dam exceeding the resistive forces due to shear and cohesion which keep the wedge in place. When this occurs the wedge is pushed to the right in Fig. 6 and is then transported by the escaping water through the now enlarged breach. When collapse occurs, the erosion of the breach ceases until the volume of the collapsed wedge is transported through the breach channel at the transport rate of the water escaping through the suddenly enlarged breach. A check for collapse is made at each time step during the simulation. The collapse check consists of assuming an initial value for \( Y_c \) of 10 and then summing the forces acting on the wedge of height, \( Y_c \). The forces are those due to the water pressure \( P_{w} \) and the resisting forces which are the shear force \( P_{sb} \) acting along the bottom of the wedge, the shear force \( P_{ss} \) acting along both sides of the wedge, the force \( P_{cb} \) due to cohesion along the wedge bottom and \( P_{ca} \), the force due to cohesion acting along the sides of the wedge. Thus, collapse occurs if

\[ P_{w} > P_{sb} + P_{ss} + P_{cb} + P_{ca} \]  

(35)
Figure 5 - Side View of Dam Showing the Forces Which Determine the Possible Collapse of the Upper Portion (Yc) of the Dam.
where:

\[ P_w = 0.5 \times 62.4 \times B \left( Y_c + 2 \times h_d \right) \]  
\[ P_{sb} = \tan \phi \left( (Y - 62.4)0.5 \times ZU \times B \times Y_c^2 + \gamma B \times W_{cc} \times Y_c^2 + 0.5 \times 62.4 \times h_d \times W_{cc} \times B + 62.4 \times ZD \times B \times Y_c \right) \]  
\[ P_{ss} = \gamma K \times \tan \phi \left( W_{cc} + (ZU + ZD) \times Y_c \right) \]  
\[ P_{cb} = CB_o \left[ W_{cc} + (ZU + ZD) \times Y_c \right] \]  
\[ P_{cs} = 2C \left[ W_{cc} + (ZU + ZD) \times (B + 2Y_c \cos \alpha) \right] \]

in which

\[ K = \frac{(1 - \sin \phi)}{(1 + \sin \phi)} \]  
\[ \bar{B} = B_o + H_c \times \sin \alpha \]  
\[ ZD' = (1 + ZD^2)^{0.5} \]

and \( Y_c, h_d, ZU, ZD, W_{cc}, Y_n \) are defined in Fig. 5. The top width (B) of the water surface in the breach channel is defined by Eq. (4) or Eq. (26), and \( \alpha \) is defined in Fig. 4 and Eq. (12).

If the inequality of Eq. (35) is not satisfied with the first trial \( Y_c \), then no collapse occurs at this time. If it is satisfied, \( Y_c \) is increased by 2 ft and Eq. (35) is again evaluated. This cycle continues until the inequality is not satisfied. Then the final value for \( Y_c \) is assumed to be \( Y_c - 1 \).

Computational Algorithm

The sequence of computations in the model are iterative since the flow into the breach is dependent on the bottom elevation of the breach and its width while the breach properties are dependent on the sediment transport capacity of the breach flow, and the transport capacity is dependent on the breach size and flow. A simple iterative algorithm is used to account for the mutual dependence of the flow, erosion, and breach properties. An estimated incremental erosion depth (\( \Delta H_c \)) is used at each time step to start the iterative computation. This estimated value can be extrapolated from previously computed incremental erosion depths after the first few time steps. The computational algorithm follows:

1. increment the time: \( t = t' + \Delta t \);
2. compute \( H_c \) using estimated \( \Delta H_c \): \( H_c = H' - \Delta H_c \).
3. compute reservoir elevation: \( H = H' + \Delta H' \), where \( \Delta H' \) is an estimated incremental change in the reservoir elevation as obtained by extrapolation from previous changes and \( H' \) is the reservoir elevation at time \( (t') \):

4. compute \( \bar{Q}_p', \bar{Q}_t', \bar{Q}_o \) associated with elevation \( H' \):

5. compute \( \Delta H \) from Eq. (17) using the previously computed breach flow \( (Q_b) \):

6. compute reservoir elevation: \( H = H' + \Delta H \):

7. compute breach flow \( (Q_b) \) using Eq. (1), Eq. (2), or Eq. (19):

8. correct breach flow for downstream submergence:

\[
Q_b = S_b Q_b, \quad \text{where} \quad S_b = 1.0 - \left( \frac{Y \Delta H}{H - H_c} - 0.67 \right)^3
\]

in which \( Y \) is the tailwater depth due to the total outflow \( (Q_b + \bar{Q}_p + \bar{Q}_o) \), and \( H \) is computed from the Manning equation applied to the tailwater cross-section:

9. compute \( B_o, a, B, P, \) and \( R \) for the breach channel using Eqs. (9-12, 26-27):

10. compute sediment transport rate \( (Q_s) \) from Eq. (33) or Eq. (34):

11. compute \( \Delta H_c \) as follows:

\[
\Delta H_c = 3600 \Delta t \frac{Q_s}{L (1 - P_{or})}
\]

in which \( L \) is the length of the breach channel which may be easily computed from the geometric relations shown in Fig. 2, \( P_{or} \) is the porosity of the breach material, and \( P_o \) is the total perimeter of the breach, \( P_o = B_o + 2(H - H_c) / \cos \alpha \):

12. compute \( \Delta H_c \) with the estimated value \( \Delta H' \):

\[
\text{if } 100(\Delta H' - \Delta H)/\Delta H < E, \quad \text{where} \quad E \text{ is an error tolerance in percent, (an input to the model having a value between 0.1 and 1.0), then the solution for } \Delta H \text{ and the associated outflows } Q_b, Q_s \text{ and } Q'_o \text{ are considered acceptable; If the above inequality is not satisfied step (2) is returned to with the recently computed } \Delta H \text{ replacing } \Delta H' \text{; this cycle is repeated until convergence is attained, usually within 1 or 2 iterations.}
\]

13. check for collapse:

14. extrapolate estimates for \( \Delta H_c \) and \( \Delta H' \)
15. If $t$ is less than the specified duration of the computation ($t_e$), return to step 1; and

16. Plot the outflow hydrograph consisting of the total flow ($Q_b + Q_h + Q_o$) computed at each time step.

Computational Requirements

The basic time step ($\Delta t$) is specified; however when rapid erosion takes place the basic time step is automatically reduced to $\Delta t/20$. The specified value for the basic time step is usually about 0.05 hrs with slightly larger values acceptable for landslide dams. For typical applications, the BREACH model requires less than 10 seconds of CPU time on a Prime 750 computer and less than 2 seconds on an IBM 360/195 computer, both of which are main-frame computers. Although it has not been used on micro-computers, it would be quite amenable to such applications.

The model has displayed a lack of numerical instability or convergence problems. The computations show very little sensitivity to a reasonable variation in basic time step size. Numerical experimentation indicates that as the time step is increased by a factor of 4, the computed peak flow ($Q_p$), time of peak ($T_p$), and final breach dimensions vary by less than 10, 4, and 0.5 percent, respectively.

Comparison With Previous Models

The BREACH model differs from the models of Cristofano (1965) and Harris and Wagner (1967) in the following significant ways:

1) The sediment transport algorithms utilized, 2) the method used for changing the breach shape and width, 3) the delay in breach erosion downward until the downstream face has been sufficiently eroded, 4) the introduction of a possible collapse mechanism for breach enlargement, 5) the accommodation of a piping failure mode, and 6) the consideration of possible tailwater submergence effects on the breach flow. Similarities are their simplicity of the computational algorithm, the use of the $D_{50}$ grain size and internal friction angle ($\phi$), and the assumption of quasi-steady uniform flow hydraulics.

The BREACH model differs from the model reported by Ponce and Tsiogolou (1981) in the following significant ways: 1) items 1, 2, 4, 5, and 6 as stated above, 2) the much simpler computational algorithm used in BREACH, 3) the use of the internal friction angle, 4) the use of the Strickler equation for determining the Manning $n$ and 5) consideration of spillway flows for man-made dams. Similarities between the two models include the use of the Meyer-Peter and Müller transport relation, the gradual development of the breach channel along the downstream face of the dam prior to its erosion vertically through the dam's crest, the use of the Manning $n$ for the breach channel hydraulics, and the way in which the reservoir hydraulics are included in the development of the breach.
The BREACH model was applied to three earthen dams to determine the outflow hydrograph produced by a gradual breach of each. The first was an actual piping failure of the man-made Teton Dam in Idaho, the second was an actual overtopping failure of the landslide-formed dam which blocked the Mantaro River in Peru, and the third was a hypothetical piping failure of the landslide dam which blocks the natural outlet of Spirit Lake near Mount St. Helens in Washington.

Teton Dam

The Teton Dam, a 300 ft high earthen dam with a 3000 ft long crest and 262 ft depth of stored water amounting to about 250,000 acre-ft, failed on June 5, 1976. According to a report by Ray, et. al (1976) the failure started as a piping failure about 10:00 AM and slowly increased the rate of outflow until about 12:00 noon when the portion of the dam above the piping hole collapsed and in the next few minutes (about 12 minutes according to Blanton (1977)) the breach became fully developed allowing an estimated 1.6 to 2.8 million cfs (best estimate of 2.3) peak flow (Brown and Rogers, 1977) to be discharged into the valley below. At the time of peak flow the breach was estimated from photographs to be trapezoidal shape having a top width at the original water surface elevation of about 500 ft and side slopes of about 1 vertical to 0.5 horizontal. After the peak outflow the outflow gradually decreased to a comparatively low flow in about 1.7 hours as the reservoir volume was depleted and the surface elevation receded. The downstream face of the dam had a slope of 1:2 and the upstream face 1:2.5. The crest width was 35 ft and the bulk of the breach material was a D₅₀ size of 0.03 mm. The inflow to the reservoir during failure was insignificant and the reservoir surface area at time of failure was about 1950 acre-ft.

The BREACH model was applied to the piping generated failure of the Teton Dam. The centerline elevation for the piping breach was 180 ft above the bottom of the dam, and an initial width of 1 ft was used for the assumed square-shaped pipe. The material properties of the breach were assumed as follows: \( \phi = 40 \) deg, \( C = 250 \) lb/ft², and \( \gamma = 100 \) lb/ft. The Strickler equation was judged not to be applicable for the extremely fine breach material, and the n value was computed as 0.013 from a Darcy friction factor based on the D₅₀ grain size and the Moody curves (Mortis and Wiggert, 1972). The computed outflow hydrograph is shown in Fig. 6. The timing, shape, and magnitude of the hydrograph compares quite well with the estimated actual values. The computed peak outflow of 2.3 million cfs agrees with the best estimate made by the U.S. Geological Survey and the time of occurrence is also the same. The computed breach width of 470 ft agrees closely with the estimated value of 500 ft at the elevation of the initial reservoir water surface. A larger estimated actual breach width of 650 ft breach width was reported by Brown and Rogers (1977); however this was the final breach width after additional enlargement of the breach occurred. The (BREACH) model produced a final width of 570 ft when the reservoir water elevation has receded to meet the reservoir bottom; the
Figure 6 - Teton Dam: Predicted and Observed Breach Outflow Hydrograph and Breach Properties.
additional widening of the breach during the recession of the outflow is due to the influence of the depth \( y \) in Eq. (15).

Sensitivities of the peak breach outflow \( Q_p \) and the top width (\( W \)) of the trapezoidal-shaped breach to variations in the specified breach material properties consisting of the flow resistance factor (Manning \( n \)), cohesive strength (\( C \)), and internal friction angle (\( \phi \)) are shown in Fig. 7. The dashed lines apply to the Teton simulation. Peak outflow is not affected by the Manning \( n \); however it is sensitive to the \( C \) and \( \phi \) values which control the enlargement of the breach width. Although \( Q_p \) is sensitive to \( C \) and \( \phi \), \( C \) can vary by a factor of 0.2 to 4.0 times the selected value of 250 with less than 35% variation in \( Q_p \). Likewise the \( \phi \) value may vary by \pm 5\) degrees with less than 20% variation in \( Q_p \). The breach width was insensitive to the Manning \( n \), somewhat sensitive to variations in the cohesion (\( C \)), and almost equally sensitive to the \( \phi \) value as was the peak outflow.

Sensitivities of the time of peak outflow \( T_p \) and the time of rise \( T_r \) to variations in \( n \), \( C \), and \( \phi \) as shown by the dashed lines in Fig. 8. \( T_p \) is sensitive to the Manning \( n \) but is not sensitive to variations in the \( C \) and \( \phi \) values. The Manning \( n \) affects the rate of breach development in the early phase of the breaching process during the initial piping formation. This is reflected in the time required for the gradual increase in outflow prior to the rather sudden and dramatic occurrence of the rising limb of the hydrograph in Fig. 5. The time of rise \( T_r \) is somewhat sensitive to variations in \( n \), \( C \), and \( \phi \); however, the apparent variation of up to 25% is not significant when expressed in actual values of less than 0.10 hrs.

**Mantaro Landslide Dam**

A massive landslide occurred in the valley of the Mantaro River in the mountainous area of central Peru on April 25, 1974. The slide, with a volume of approximately \( 5.6 \times 10^{10} \text{ ft}^3 \), dammed the Mantaro River and formed a lake which reached a depth of about 560 ft before overtopping during the period June 6-8, 1974 (Lee and Duncan, 1975). The overtopping flow very gradually eroded a small channel along the approximately 1 mile long downstream face of the slide during the first two days of overtopping. Then a dramatic increase in the breach channel occurred during the next 6-10 hrs resulting in a final trapezoidal-shaped breach channel approximately 350 ft in depth, a top width of some 650-750 ft, and side slopes of about 1:1. The peak flow was estimated at 353,000 cfs as reported by Lee and Duncan (1975), although Ponce and Tsivoglou (1981) reported an estimated value of 484,000 cfs. The breach did not erode down to the original river bed; this caused a rather large lake to remain after the breaching had subsided some 24 hrs after the peak had occurred. The slide material was mostly a mixture of silty sand with some clay resulting in a \( D_{50} \) size of about 11 mm with some material ranging in size up to 3 ft boulders.

The BREACH model was applied to the Mantaro landslide-formed dam using the following parameters: \( Z_0 = 17 \), \( Z_0 = 2.5 \), \( H_s = 560 \text{ ft}, D_{50} = 11 \text{ mm}, \)

\( P_e = 0.5 \), \( S = 1200 \text{ acres}, C = 400 \text{ lb/ft}^2 \), \( \phi = 35 \text{ deg}, \gamma = 100 \text{ lb/ft}^2 \)

\( B_r = 2.5 \), and \( \Delta t = 0.1 \text{ hr} \). The Manning \( n \) was estimated by Eq. (21) as 0.020.
Figure 7 - Sensitivity of Mantaro and Teton Predictions of Peak Outflow (Qp) and Breach Width (W) and Breach Depth (D) to changes in the properties of the Dam: Friction Angle ($\phi$), Cohesion (C), and Manning n.
Figure 8 - Sensitivity of Mantaro and Teton Predictions of Time of Peak Outflow ($T_p$) and Time of Rise of Peak Outflow ($T_r$) to changes in the Properties of the Dam: Friction Angle ($\phi$), Cohesion ($C$), and Manning $n$. 
Figure 9 - Mantaro Landslide Dam: Predicted and Observed Breach Outflow Hydrograph and Breach Properties.
and the initial breach depth was assumed to be 0.5 ft. The computed breach outflow is shown by the solid line in Fig. 9 along with the estimated actual values. The timing of the peak outflow and its magnitude are very similar except for a somewhat more gradual rising limb of 10 hr compared to the estimated actual of 6 hr. The dimensions of the gorge eroded through the dam are similar as shown by the values of D, W, and σ in Fig. 9. The hydrograph denoted by the dashed lines is produced if only the Manning n is increased to 0.0225, a value which for the Montaro slide would be computed by Eq. (21) if the D0 size were replaced by a D0 size. The dashed hydrograph is very similar except it has a peak nearly the same as that reported by Ponce and Tsivoglou (1981). The breach size is somewhat larger as indicated by the D, W, and σ values associated with the dashed hydrograph in Fig. 9. In particular, the depth of breach erosion is greater and nearer the estimated value of 350 ft. The influence of the Manning n on the magnitude of the peak outflow and the breach dimensions is illustrative of the more voluminous landslide dam’s sensitivity to this parameter.

This is further illustrated by the solid lines (Montaro Dam) in Fig. 7 where Qp is seen to be very sensitive to variations in the Manning n while the depth of breach (D) is less sensitive. The peak outflow is also sensitive to the cohesion (C) value, although a change in C of 0.25 to 4.0 times the value used in the simulation produced variations in Qp of less than ± 35%. D is not sensitive to the changes in C. Both Qp and D are not very sensitive to the σ value, variations in each being less than ± 15% for a complete range of physically relevant values of the friction angle (σ).

Sensitivities of Tp and Tr to variations in n, C, and σ are shown by the solid lines (Montaro Dam) in Fig. 8. The time (Tp) at which the peak outflow occurs is very sensitive to the Manning n. As in the man-made Teton Dam, it is the duration of the gradual increase in outflow prior to a rather dramatic development of the rising limb that depends on the value of the Manning n. Also, the time of rise (Tr) is sensitive to the n value; the values of Tr varied from 6 to 16 hr as the n varied from 0.024 to 0.016. Neither Tp nor Tr is sensitive to the internal friction angle (σ).

Spirit Lake Blockage

The violent eruption of Mount St. Helens on May 18, 1980, in Washington, produced a massive debris avalanche which moved down the north side of the volcano depositing about 105 billion ft³ of materials in the upper 17 miles of the North Fork of the Toutle River valley and blocking the former outlet channel of Spirit Lake with deposits of up to 500 ft deep (Swift and Kreusch, 1983). Spirit Lake, itself was drastically changed by the avalanche; the existing lake has a maximum volume of 314,000 acre-ft at the elevation of 3475 ft when breaching of the debris blockage is anticipated. To avoid this the Corps of Engineers have installed temporary pumps to maintain the lake level at about elevation 3462 (275,000 acre-ft) and are expecting to complete in the near future a permanent outlet channel which will bypass the debris dam and maintain safe lake levels.
Greater than normal precipitation, failure of the pumping system, and/or addition of more avalanche material from another eruption of the volcano could cause the lake level to exceed elevation 3475 and possibly cause the debris dam to fail. Such a hypothetical breach was simulated using the BREACH model.

An initial piping failure was assumed to occur at elevation 3448. The following parameters were determined from physical considerations:

- $H_0 = 3475$, $H = 3448$, $Z_0 = 3320$, $Z_{U} = 22$, $D_{50} = 7$, $n = 0.018$ from Eq. (21), $F_p = 0.32$, $r = 100$, $c = 35$, $C = 150$, $B_c = 1.5$, an initial pipe of width 0.25 ft, and $dt = 0.20$ hr. The simulated outflow hydrograph shown in Fig. 10 has a peak of about 550,000 cfs occurring 15 hrs after the start of failure. The time of rise ($T_r$) is about 2 hr. The final breach dimensions are: $D = 155$ ft, $W = 420$ ft, and $\alpha = 50$ deg. Sensitivity tests indicate about a 20% variation in the peak flow may occur with expected variation in the internal friction angle and cohesion values. The predicted outflow hydrograph from Spirit Lake was used in a hazard investigation of possible mud flows along the Toutle and Cowlitz Rivers by Swift and Kresch (1983).

SUMMARY

A breach erosion model (BREACH) based on principles of hydraulics, sediment transport, and soil mechanics is described. The model uses equations of weir or orifice flow to simulate the outflow entering a channel that is gradually eroded through an earthen man-made or landslide-formed dam. Conservation of reservoir inflow, storage volume, and outflow (crest overflow, spillway flow, and breach flow) determines the time-dependent reservoir water elevation which along with the predicted breach bottom elevation determines the head controlling the reservoir outflow. A sediment transport relation, either Meyer-Peter and Müller or Dubois, is used to predict the transport capacity of the breach flow whose depth is determined by a quasi-steady uniform flow relation (the Manning equation applied at each $dt$ time step during the breach simulation). Breach enlargement is governed by the rate of erosion which is a function of the breach bottom slope and depth of flow and by the extent of collapse that occurs to the sides of the breach due to one or more sequential slope failures. The breach material properties (internal friction angle ($\phi$) and cohesive strength ($C$)) are critical in determining the extent of enlargement of the trapezoidal-shaped breach. Another parameter, the Manning $n$, is most critical in determining the rate of breaching of landslide dams but is much less important in the breaching of the much smaller man-made dams. The Manning $n$ may be predicted on the basis of the grain size of the breach material by the Strickler equation or via the Darcy friction factor-grain size-$n$ relation. The dam may consist of two different materials, an outer layer and an inner core. Piping or overtopping failure modes can be simulated as well as sudden collapses of sections of the breach due to excessive hydrostatic pressure. The model has the potential to determine if a breach will develop sufficiently during an overtopping of the dam to cause a catastrophic release of the reservoir's stored water. The BREACH model has a simple iterative computational structure which has well-behaved and
Figure 10 - Spirit Lake Landslide Dam: Predicted Breach Outflow Hydrograph.
efficient numerical properties. A few seconds of computer time is required for a typical application.

The model is tested on a man-made dam (Teton Dam) which failed by an initial piping which progressed to a weir type free surface breach. The predicted outflow hydrograph and breach size and shape compare favorably with estimated actual values. The predictions are somewhat sensitive to the values of $\theta$ and $C$ which were estimated from a grain size and a qualitative description of the dam’s material composition.

The model is also tested on the naturally formed landslide blockage of the Mantaro River in Peru which was overtopped and developed a large gorge which resulted in the gradual release of three-fourths of its stored water. The model predictions compared well with estimated observed values. The Manning $n$ is critical to the prediction of the rate of breaching of massive landslide dams; however, if it is selected on the basis of the breach material’s grain size the results are within a reasonable range of variation.

It is considered that further testing of the model to assess its ability to predict overtopping failures of man-made dams is warranted and that its basic structure is suited to the resources (data and computational) which are commonly available to hydrologists/engineers during a detailed investigation of potential dam-failure flooding.

REFERENCES


UPUNCES


SECTION III - FLOOD HAZARDS
FLOOD HAZARD MEASUREMENT - WHO HAS A RULER?

by L. Douglas James

ECONOMICS TRADEOFFS IN ALLUVIAL FAN DEVELOPMENT

Attractive building sites are causing urban residential development in the Intermountain West to move up alluvial fans towards the apex areas on the alluvial fans where the sediments deposited over geologic time meet the steep mountainsides. One has a panoramic overview below and a mountainside rising a mile higher directly behind. Road access is still easy. Such residential sites command premium market prices.

The purchaser enthralled with the setting, however, must reckon with the possibility of a rushing torrent raging out of the canyon and spreading mud and debris. People vary greatly in the ways that they perceive and adjust to such hazards (James 1975). Some meticulously avoid risk and would avoid any site with the possibility of such exposure. Others are too preoccupied with other activities to become aware of the risk before suffering devastation. Somewhere between these foolish extremes are people who want specifics on the degree of risk.

The individual who learns the characteristics of the flooding that can be expected and the probabilities associated with various severities of those characteristics is in a position to decide whether the amenities gained are worth the potential loss. Furthermore, he is in a position to construct his home for minimal harm when floods occur (U. S. Army Corps of Engineers 1972). The ideal way to select a home site is to quantify the risk and decide whether the amenities gained are worth the potential loss. The ideal way to construct a home is to quantify the characteristics of the flood and build so that minimal harm will occur. The ideal way to construct a street system that will have to convey flood waters is to build a cross section with adequate hydraulic capacity and a surface and subgrade with adequate structural durability. This quantification requires a measurement method; a ruler is needed.

In order to compile the needed information, we need to be able to define the amenities, the harm, and how the amenities are affected by flooding. People select a home site for economic, environmental, and social reasons (Laurent 1971). Flood losses are important in all three dimensions. The primary social concern is danger to life, health, and safety is slight from

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shallow alluvial fan flooding. Mud deposits over landscaped areas are considered an important environmental loss to urban dwellers. The greatest loss is property damage, and a major component of that is to streets and utilities. Local government has been the primary loser in the Utah flooding and could greatly benefit from risk information that can be used to improve the designs.

The economic damage caused by flooding depends primarily on depth, velocity, duration, and sediment content. Depth determines the area of water contact with buildings and contents and the static forces on exterior walls. It is the dominant factor used to estimate urban flood damages (Grigg and Belwag 1975). Velocity adds impact forces (U.S. Army Corps of Engineers 1972), scour away topsoil, and increases the risk of drowning (James and Hall 1984). Duration determines the period of deterioration and the cost of alternative arrangements during lost occupancy. Coarse sediment batters building exteriors and roads, and fines can penetrate the surfaces that they contact to the point where rehabilitation is impossible. Estimates have been made of the effects of these three secondary factors on flood damage (Breaden 1973), but relationships employing them are seldom used in urban areas.

FLOOD HAZARD DEFINITION ALONG THE WASATCH FRONT

The riverine flooding characteristic of humid climates occurs as flows exceed the channel capacity and spill over nearby lowlands. The flood hazard is delineated by selecting a design storm frequency, estimating the peak flow, computing a backwater curve from channel cross sections, and plotting the resulting water surface elevations on a topographic map (Thomas and Lindskov 1983). The economic loss to any property is estimated from the flood depths estimated at the site for events covering the range of frequencies.

The resulting quantification of flood risk is notoriously poor on the alluvial fans below Utah's mountain canyons (James et al. 1980a). There are 10 principal reasons:

1. Damages concentrated in rare events. Perhaps three quarters of the flood damages in the 137 years since the Mormon settlement of Utah, measured in constant dollars, occurred in the three heavy snowmelt years of 1952, 1983, and 1984 (James et al. 1980b, Tempest 1984). Utah mountain soils are generally quite porous; annual precipitation and peak rainfall intensities are both generally small (James et al. 1980b). On many small catchments, many years go by with little or no runoff (or only baseflow spring runoff). Only a rare sequence of several wet years in a row, soil saturating fall rains, heavy snows, and rapid spring snowmelt with intense rains brings flood flows with the critical combination of these conditions varying with catchment characteristics. Flood frequency distributions at the few gaged sites, largely recording flash flood flows, are positively skewed (Hardison and Jennings 1972) with tails containing rare events that are much larger than the normal annual peaks. The physical processes leading to extreme events may not have even occurred over the entire period of record at most sites. The gaged streams are largely selected because of their use for water supply (a factor that increases likelihood of local sponsorship for sharing in the cost) and tend to drain the higher mountain areas that are more humid and are a lesser hazard
with respect to generating sediment laden flood flows. Utah should review this aspect and reconsider state stream gaging priorities.

2. Multiple probability distributions. Utah floods result from two distinct meteorologic situations. Snowmelt and thunderstorm (flash flood) events have different probability distributions; and, furthermore, the two events interact in determining flood flows. Thunderstorms produce more runoff after mountainsides are left wet by melting winter snows, and spring snowmelt runoff is greater after mountain soils are saturated by fall rains. The data base is inadequate to sort out these relationships for the small, steep mountainside catchments where gaged sites are sparse and years go by with little runoff. Yet, runoff from these catchments in a few extreme years causes most of the flood damage in Utah.

3. High sediment production. The rare years with heavy autumn rains followed by deep snows accumulating through a wet winter build pore pressures within the mountain soils, generate subsurface flow along soil rock surfaces, and lead to widespread unstable soil conditions. Mud slides directly into the streams, sediments are washed into the channels by drainage water and surface runoff flowing over areas denuded by the slides, and bank sloughing is accelerated by larger than normal stream flows. The sediment laden water causes much more damage for a given flow. Frequency analysis based on peak water flows alone (characteristically dominated by flash flood events) is a poor index of damage. The sedimentation effect accentuates damages during rare events. The rare wet years bring unstable slopes, high sediment runoff, and major losses; however, very little information is available for quantifying these events.

4. Supercritical velocities. The supercritical flows emerging from mountain canyons interact with the stream beds in sediment transport, erode banks, and do not follow established water courses over flatter areas. The flows are highly erosive and lead to pool and ripple conditions when continued over long periods. However, the short periods of flooding in Utah do not reach anything like this equilibrium state. One is dealing with transient conditions of upstream erosion and downstream scour in patterns that vary from one flood to the next with long intervals between in which other natural processes and human activity can make major changes.

5. Time variable channel capacities. Channel capacities vary over the course of a flood hydrograph as banks slough and sediments are eroded or deposited to fill channels and block culverts. Initial conditions vary from flood to flood according to the sediment content of the flow and channel maintenance activity. Sediment transport rates are not proportional to runoff during the storm hydrograph so that critical conditions may occur during either rising or falling limbs depending largely on the timing of upstream soil slippage. Both flow and sediment hydrographs must be estimated for flood control design (often specified by frequency according to policy) because a system of debris basins and channels can only prevent damages as it both stores the sediment and contains the flows.
6. Long snowmelt flow durations. Where land slopes flatten below the mouths of mountain canyons, the channels aggrade over the years with sediment deposition to become higher than the surrounding land. The volume of overflow and the area required to collect the water for infiltration or return to the stream is determined by the length of time that the flows exceed the channel capacity, and durations are much longer for snowmelt than for flash floods. Longer periods of runoff increase the scouring and deposition processes to more damaging levels, add to the depths and volumes of sediment deposition in the floodplain, and are more costly to convey through streets because of the added scouring of chuck holes and prolonged traffic interruptions.

7. Hydraulically disconnected flows. When the flow exceeds the channel capacity, most of the water that leaves the channel becomes hydraulically disconnected from the natural watercourse. Hazard mapping requires identification and separate analysis of each flow path.

8. Anthropocentric flow patterns. The order of magnitude of the flows emerging from mountain canyons is too small to spread a significant depth of flow over the relative large floodplain areas. Flow depths are of the same general height as street curbs and other urban landscaping features. Consequently, the flow paths of the shallow disconnected flow are largely determined by street patterns and building layouts. Also, irrigation canals intercept flood runoff, convey it for a distance, and are overtopped away from normal floodplains. Such events have caused some of the largest flood damages in Utah (James et al. 1980b).

9. Disposal by infiltration. In nature, much of the flood water goes into aquifer recharge; and much of the recharge is through coarse deposits just below the mouths of the mountain canyons. Urban development reduces the permeability of these areas, and channelization restricts the spreading and quickly conveys the water past the recharge areas. Flood peaks travel much further downstream than they would under natural conditions.

10. Altered damage functions. Conventional stage-damage relationships are based on building damage from short periods of clear water flooding. They severely underestimate damages from high velocity, sediment laden flows that can, for example, fill basements with mud. They are no use at all in estimating damages to urban landscaping, streets, and utilities; yet this is where the bulk of the loss occurs.

A WAY OUT

It is one thing to list these 10 factors and quite another to incorporate them in flood risk mapping, flood damage assessment, and flood control design. Flood hazard studies for Utah have followed (U.S. Army Corps of Engineers 1969, 1974; Hall 1984) or modified (Heefner 1983) the simple riverine assumptions (Thomas and Lindskov 1983) for lack of a better methodology.

Flood hazard delineation is based on empirical measurement of historical events and applications of known theory to hydrologic and hydraulic processes. The high degree of interaction among so many complex factors and the paucity
of data and the difficulty of getting any more because of the rareness of major events forces a study to rely more on conceptual process analysis and less on empirical data. The important consideration at this point is whether we know enough about the processes and have sufficient empirical data to do a meaningful job. The sections below argue that we know enough to begin, provide a basic structure for combining physical and flow data, and stress that only by combining theory with a well-structured data gathering program can we achieve success.

HAZARD DELINEATION BY SIMULATION

The most practical approach is to represent the physical processes and the interactions among them in a parametric model, use that model to simulate catchment response (deterministically or stochastically) to multiple weather sequences, and then simulate resultant downstream geometry, system maintenance, street configurations, land use patterns, and flood-proofing efforts. The modeling has two goals. One is to delineate the hazard on the floodplain, and the other is to quantify design flows for water and sediment control. Presently, we do not know enough about catchment characterization or the interactive processes to construct a refined model, but we can establish a framework and work on its improvement. The effort is now in its initial stages.

The objective is to model flooding by sediment-laden stream waters and not to deal with hazards associated with landslides or debris flows. The modeling of the interactions between runoff and land instability to estimate sediment production rates is not to locate specific sites of unstable land within the catchment but to quantify the aggregate effect of land instability dispersed over a mountain catchment on the hazard from water and sediment coming down the streams to the urban areas below. The goal is to aggregate effects not identify source areas.

PHYSICAL PROCESSES

Parametric modeling begins with definition of the relevant physical processes. Reliable flood hazard delineation must incorporate the 10 complicating factors delineated above and such interactions as those between snow-melt and cloudburst runoff, soil moisture and soil stability, catchment runoff and sediment production, sediment deposition and channel capacity, canyon runoff and storms on the floodplain, floodplain development and the paths of disconnected flow, and overflow volume and area flooded.

The processes being incorporated into the initial model are I) water runoff including a) snowpack accumulation and melt and b) cloudburst runoff; II) sediment production including a) mass erosion or landslides and b) surface and gulley erosion; III) transport processes including a) channel routing of water and sediment flows and b) storage routing of water and sediment flows through pools above culvert entrances and other ponded areas; IV) overflow processes including a) sedimentation filling channels or clogging culverts, b) interception of shallow flood flows by streets, and irrigation systems, and c) simultaneous runoff from precipitation on the flood plain or adjacent areas; and V) dispersion including a) water flow through the street network.
nd b) overland flow across the floodplain lands and its infiltration into quifers.

The availability of working models varies considerably among these processes (James et al. 1982):

I. Water Runoff

Water runoff modeling is well established as an operating hydrologic tool, e.g., the Stanford Watershed Model and its descendants provide basic programming for use as the starting point for modeling runoff from a catchment.

II. Sediment production

Sediment runoff models are being developed for agricultural areas (Foster 1982), but no effort was discovered of someone attempting to address unstable mountain watersheds.

III. Transport processes

Channel and storage routing can be done with well established routines, but considerable uncertainty exists as to how best to handle supercritical flow, the variability of sediment transport during flood hydrographs, and sediment deposition and later scour by subsequent floods.

IV. Overflow processes

Overflow modeling is being expanded to address debris blockages and the division of overflows among alternate flow paths.

V. Dispersion

A great deal of work has been done on modeling urban runoff (McPherson 1977, Overton and Meadows 1976), but very little has been done on quantifying the dispersal and infiltration of shallow flood waters over urban areas.

Modeling of the II processes in the five groups is presented below, in part to outline the physical principles used, but more to set forth the framework to be initiated to describe the interactions among the processes that are to be included. Representation of the interaction requires depiction of how the processes interrelate, how conditions vary over time, and how one should represent spatial variability over the catchment. The modeling will draw from the literature in depicting the processes, and thus the spatial relationships will be emphasized in the following presentations.

I. Water Runoff

a. Snowmelt runoff

Mountain snowpacks accumulate through the winter and melt over a few weeks in the spring. Accumulation and melt rates vary considerably with
elevation and aspect. Colder temperatures cause more snow and slower melt at higher elevations. The more direct exposure to solar radiation and longer hours of sunshine speed melt on south-facing and, to a lesser degree, west-facing slopes. Elevation and aspect variabilities can be modeled by dividing the total catchment into elevation zones spanning 2000 feet and aspect zones separating north from south facing slopes (Figure 1). The effect of elevation is represented by a lapse rate, generally between 2.5 and 5.0 degrees Fahrenheit per 1000 feet, with the amount varying between day and night and between clear and storm periods. The effect of aspect is introduced through relationships between potential solar radiation and day of the year, latitude, and slope and between hours of sunlight and the same three variables (Frank and Lee 1966). A snow accumulation and melt model adds to the snowpack when precipitation occurs at temperatures below freezing and melts snow on the ground when temperatures exceed freezing. Anderson and Crawford (1964) developed the first operational snowmelt model and later refined it for the river forecast system of the National Weather Service (Anderson 1973).

b. Storm runoff

Rain falls in the warmer months with the greatest intensities normally occurring in the late summer. Precipitation amounts and intensities generally increase with elevation, but maximum values are generally reached short of the highest elevations (James et al. 1984). Rare high rainfall rates can exceed the soil infiltration capacity (an amount that declines with wetter soil), cause overland flow, and erode the soil surface. A number of watershed models quantify the above processes (Renard et al. 1982). Beginning with the Stanford Watershed Model (Crawford and Linsley 1966), a series of digital models have been based on water balance accounting incorporating such processes as interception, depression storage, infiltration, overland flow, evapotranspiration, deep seepage, and interflow. The computations for this study will be applied separately by elevation and aspect zones.

The heart of runoff modeling is the separation between runoff and infiltration. The method depicted on Figure 2 assumes that infiltration rates vary over a zone from zero up to a maximum that depends on soil characteristics (LZC = the soil moisture storage capacity and BMP = a basic infiltration rate) and the moisture content of the soil (LZS = current soil moisture storage). If one assumes that infiltration rates vary linearly over the basin, infiltration and runoff volumes can be estimated from areas on the figure. Areas with larger infiltration rates would be expected to have greater soil moisture and slide potential. Consideration will have to be given to the advantages and disadvantages of representing this variability by dividing the zones into subzones according to infiltration rate.

II. Sediment Production

a. Landslides or mass erosion

Infiltration soaks into the ground and, in large amounts, builds pore pressures and contributes to soil instability. After the soil gives way,
Figure 1. Runoff source areas.

Figure 2. Disposition of precipitation.
continuing drainage of soil water may carry the loosened material downstream. Landslides are associated with three principal subsurface conditions: 1) a deep soil where the failure lies along the weakest cylindrical surface and goes deep below the ground, 2) a shallow soil situation where the wetter, heavier soil slides down a lubricated rock surface, and 3) a fissured rock formation where differential water pressures can build between adjacent cracks. The deep and shallow soil cases are the primary considerations and can be modeled from the diagrams shown in Figure 3.

Subsurface conditions in mountain areas are heterogeneous and so time variant that the dynamics of the change may be an important factor contributing to instability. Flows are often unsaturated and sometimes by vapor transport, particularly during snowmelt periods. Water seeps along many paths and collects above interfaces with underlying less pervious strata. The greater pore pressures and reduced internal friction within the soil lead to soil failures. In this situation, one must look at multiphase, unsaturated water flow through an extremely complex geological structure to model the changing soil moisture content through time and space.

For parametric modeling on larger areas one must try to quantify relationships for estimating mountainside instabilities from 1) structural, conveyance, and water storage properties of soil and rocks and 2) the flow paths, flow rates and soil water content at a given time (Freeze and Cherry 1979).

For forecasting failures in deep soils (Figure a), the hillsides are sectioned into vertical slices for comparing sliding and resisting moments. Bishop (1955) expanded the analysis to take into account the stresses along the boundaries between the slices. Pore pressures carry some of the weight of the soil, reducing internal friction, and also alter the cohesion and angle of friction for the soil.

In shallow soil failures, mountainsides have a thin layer of soil covering the bedrock (Figure 3b). Water infiltrates into the soil, is trapped above the rock, adds weight to the layer, creates pore pressures that reduce the friction along the rock surface, and adds seepage forces by moving along the interface (Patton and Deere 1971). Once the forces favoring sliding exceed the resisting forces, the soil begins to move, carrying mud down the mountainside and exposing a loose surface to later erosion.

Sliding was modeled by 1) incorporating the simulated values of soil moisture content estimated by the hydrologic models into equations estimating the pushing and the resisting forces and 2) using hillside length and slope to represent the propensity to slide. The tie to soil moisture content involved a) adding the weight of the infiltrated water to that of the soil, b) adding seepage forces proportional to flow along the soil-rock interface, and c) using infiltrated water to estimate pore pressure and reduction in sliding friction.

The primary parameters used to estimate instability are thus the structural properties of the soil (cohesion, angle of internal friction, porosity,
a. slip circle in a homogeneous material

b. failure along a slippery rock surface

Figure 3. Landslide representations.
and unit weight), the resisting forces along the soil-rock interface (friction and cohesion), and the depth, slope, and length of the soil layers. All of these factors vary over the catchment, even at a microlevel. As an initial pass, however, catchment instability was represented by uniform soil and interface characteristics with the variability associated with the depth, slope, and length parameters.

For short slopes, the soil is largely held in place by the passive pressure at the toe. For longer slopes, more of the resistance is from friction and cohesion along the soil-rock interface. The critical slope length varies with soil moisture content and with seepage rates. The distribution of the catchment area in slopes of different lengths is described as shown in Figure 4. The curves are plotted from topographic maps. These curves were separately plotted by catchment zone for a mountainous area where the low elevations are somewhat steeper than the higher elevations.

For each hour of the year, the hydrologic model estimates the soilwater content. The forces can then be estimated and balanced for each intersection point along the vertical axis on Figure 4. The point with the largest ratio of the pushing to the resisting forces is the most prone to slide. If the pushing forces at that point exceed the resisting forces, sliding is noted within the model. The program then moves to the right along the curve and recalculates forces until the slope has flattened enough for the resisting forces to be larger than the pushing forces. A portion of the material is estimated as sliding directly into the stream, and the area affected is treated as some multiple of the area shown in Figure 4 to account for disturbed land downslope from the slide.

b. Surface erosion

The surfaces of natural soils are eroded by intense rainstorms; slides loosen large masses of material and increase erosion rates until vegetation is restored; and gulleys are formed below points of emergence of flowing water. Erosion begins with the loosening of the soil surface by raindrop energy; and the material is moved downstream by sheet, rill, and gully erosion at rates limited by the transport capacity of the flowing water. Sediment loading is augmented by material sliding into the stream from landslides above or from bank undercutting below. The subprocesses used are loading from A1) sheet and rill erosion, A2) landslide material directly entering the stream, and A3) gully erosion and B) transport in the rills and gulleys. If the loading rate exceeds the transport capacity, C) the material accumulates in the channel and gradually attaches itself to the bed over time. These deposits then become a source of sediment for floods that occur later.

(A1). The universal soil loss equation is the most widely used relationship to estimate sheet-rill erosion. Williams (1975) presented a modification for estimating the erosion associated with a single storm. Erosion from landslide and nonlandslide areas can be distinguished by using different factors for vegetative cover. When a landslide is simulated, the cover factor is increased to unity and then gradually reduced to its customary value for
Figure 4. Curves characterizing slope-length relationships by catchment zones.
that soil type over the period required for the vegetation to be reestablished.

(A2). Direct sliding into flowing streams depends on the distance of the head of the slide above the stream, the steepness of the slope, and the flow characteristics of the moving material.

(A3). Gulleys form when a significant flow continues over a prolonged period. Whereas landslides tend to occur in the steepest portions of the catchment, gulleys tend to begin in the concave section toward the base of a steep slope where the anisotropy of a layered soil concentrates the flow (Zaslavsky and Sinai 1981).

(B). The sediment entering the flow can be estimated from the sum of the loadings by subprocesses A1, A2, and A3. The sediment transport capacity of the stream tells how much of the material can be carried away (Meyer and Wischmeier 1969). The rate of sediment movement is the smaller of the loading rate and the transport capacity.

(C). Material is deposited on the bed and around the banks of the channel during periods when the loading rate exceeds the transport capacity. Later, the reverse situation may cause movement of material deposited previously. However, older deposits may become bonded to the native material and resist erosion. The hydrologic model simulates water contents and flows hourly throughout the year. When sediment is washed into the stream faster than it can be carried away, the amounts are accumulated in a storage variable. The accumulated total is periodically reduced by a bonding factor. The runoff carries sediment until the storage variable drops to zero.

III. Transport

a. Channel routing

The hourly rates of water runoff and sediment movement from the various zones of the catchment need to be routed through the channel system to the mouth of the canyon represented as shown in Figure 5. Kinematic routing generally provides the best performance for hydrologic models (Lumb and James 1976). Bren and Turner (1978) found the kinematic wave to fit shock waves in a steep rough channel in a field laboratory. Kellerhals (1970) found the kinematic wave to approximate tumbling flow in British Columbia streams with slopes up to 10 percent.

Sediment transport equations distinguish between suspended load carried within the water and bed load rolled along the channel bottom. Smart (1984) provides a bedload sediment transport formula for stream channels of slopes exceeding 3 percent. For a first approximation of the quantitatively larger bed load movement (Hall 1984), the Meyer-Peter Muller formula, as examined (Sheppard 1960) and adopted by the U.S. Bureau of Reclamation (1977), was applied. The application raises questions as to the validity of using this steady state formula for short time increments in rapidly varied flow and whether the same representative particle diameters should be varied over the course of a flood hydrograph. Both issues deserve further study.
A - G - channel reaches
O points where runoff is added to stream
1 one square mile
L local runoff added for routing
D local runoff added at canyon mouth

b. schematic diagram

Figure 5. Basin discretization of inflow points and channel reaches.
b. Storage routing

Debris basins are commonly built at the mouths of the mountain canyons to trap the sediment. In other cases, clogged or undersized culverts form retention reservoirs. These ponds discharge water down the channel, but water often overtops the embankment and flows onto the street. Most street flooding in Utah begins at such points. Reservoir water routing combines an inflow hydrograph with stage-storage and stage-discharge relationships to solve for outflows. Reservoir sediment routing requires estimation of trap efficiency, provides for the reservoir being filled, and changes the routing relationships over time (Ward et al. 1977, Simons and Senturk 1976). The amount of sediment, as well as the amount of water stored in the basin, is tracked during the modeling. Basin cleaning can be simulated by specifying removal of the stored sediment volume.

IV. Overflow processes

a. Sedimentation

Streams overflow when the flow exceeds the channel capacity, and channel capacity is reduced by sedimentation. With channel banks higher than the nearby lands, the water flows overland. Rates, however, are small compared with the size of the floodplain and the directions of flow are largely guided by the street pattern. Simultaneous rainfall on areas tributary to the street add to the flow.

Sediment transport depends on the availability of material and the transport capacity of the flow. Sediment enters a channel reach by transport from upstream or by bank sloughing. The sum of these two amounts is then compared with the transport capacity. When the stream cannot carry the load, sedimentation aggravates the channel. This volume can be translated to a depth and used to determine the rate at which the channel bed is aggrading.

The blocking of a culvert entrance is a positive feedback process in which sediment deposition increases the required head, reduces the flow velocity, and add to the degree of blockage. Clogging generally means orifice control at the culvert entrance. Bodhaine (1968) related the orifice coefficient to flow type and culvert geometry. Yen and Pansic (1980) formulated an expression for partially clogged entrances. James et al. (1984) incorporated a head loss coefficient previously proposed by Hoffman (1977) for application to the trashracks.

b. Interception of shallow flows

The typical channel overflow passes over a curb as the street crosses or parallels the stream. The process can be represented hydraulically as flow crossing a broad-crested weir. Sometimes, irrigation canals are purposefully used to intercept the flood waters and convey them to water users or to safe disposal. Major flood losses have occurred where the intercepted water exceeded canal capacity.
c. Local storm runoff

Rain may fall in an urban area even as a flood hydrograph emerges from the mouth of a mountain canyon. At other times, a local cloudburst may cause street flooding in its own right. Assessment of the combined probabilities should consider rain occurring during snowmelt periods and the spatial pattern of thunderstorms up the mountainside in the summer. The areal and time distributions of the precipitation are both important. The same hydrologic model used for the upper catchment can be separately calibrated for the urban area. Full runoff may be assumed from the rain falling on paved areas connected directly to the streets.

V. Dispersion

a. Street flow

Wasatch Front communities have streets laid out in square grids with the north-south streets generally following the contours and intercepting the runoff on their uphill sides and the east-west streets having steep slopes and conveying the water down the hillsides. The streets are typically quite wide and are often specifically designed to carry storm water. Flows often divide at street intersections and at the entrances to storm drains (Chow and Yen 1976, Hall 1984). The division at intersections is handled by solving simultaneously the equations of continuity, equating specific energy for the outflow directions, and employing the depth-flow relationship used in the kinematic routing (James et al. 1984). Kinematic routing is used for the flow in the streets.

b. Overland flow and infiltration

If the required flow area exceeds the cross section of the street flowing curb full, the water flows onto adjacent yards, which are normally landscaped to drain toward the street. If the depth of flow should cause water to flow through the yard, the entire block would be designated as flood plain. The flow is routed through the development on the block, and combined with the street flow on the other side. Some of the water infiltrates in transient so that flows are less at the downstream side of the block (unless rainfall is occurring and making up the difference). The model has not been refined to incorporate these concepts, partly because events this large are rare and data are minimal, even with the wet conditions in 1983 and 1984.

FREQUENCY ANALYSIS

The model as outlined above can be used directly to estimate the sediment deposition and area flooded by any event specified by values of the model parameters. It can readily be extended to estimate economic loss. The more general need, however, is to provide information on risk through frequency analysis. This can be done by running the model with an extended period of meteorological record to determine how the catchment and floodplain would respond. A frequency analysis could then be based on a long period of synthesized events.
One problem that would have to be overcome would be replication of thunderstorm patterns in ungauged mountain areas. Two possible approaches are through modeling the thunderstorm formation process (Sorman and Wallace 1972) or using radar data to interpolate to infer patterns and then accumulate the measurements over time in a way that will build a base for future modeling (Krajewski and Georgakakos 1984).

A frequency analysis based on this data could be used directly for flood hazard mapping and structural and nonstructural design applications. For purposes of system operation and updated risk assessment, one must deal with the conditional probabilities of known current snowpack depths, soil moisture conditions, areas of landslides, and volumes of loosened sediment within the catchment. One can specify these estimated quantities as initial conditions and run the model with alternative meteorological sequences from that starting point much as has been done for the Great Salt Lake (James et al. 1984). Spring conditional probabilities will vary a great deal with snowpack depth. General catchment moisture and sedimentation conditions are likely to have effects that will continue over extended periods.

**PRELIMINARY RESULTS**

Data have already been collected and used in model formulation and validity checking. For example, Heefner (1983) applied a model (Dispersed Urban Flow Simulation Model, DUFSM) combining processes IVb, IVc, and Va to the Mill Creek drainage basin in Bountiful, Utah, to map the 100-year floodplain with the results shown in Figure 6. The strong influence of the urban street pattern in determining the inundated area is characteristic of the Wasatch Front. In this case, the flow leaves the stream at culverts and flows down a slight gradient on a north-south street until reaching a clear downhill shot on an east-west street. Below, the flow is detained behind a freeway fill until it can pass under the highway on its way to the Great Salt Lake.

Hall (1984) applied the model expanded to incorporate process IIIb to Stone Creek in Bountiful. The flooding recorded on June 1, 1983, and the floodplain shown in the Flood Insurance Rate Map for the community (Committee on a Levee Policy 1982) are shown in Figure 7. The results with DUFSM and as improved with the incorporation of culvert clogging are also shown. The results are much improved by incorporating the blockage by sediment of the culvert on 900 North Street. The above modeling provides a starting point for estimating the depths and locations of flooding and sediment deposition on alluvial fans and other lands at the base of mountain ranges for both snowmelt and cloudburst flooding.

**CONCLUSION**

The definitions of hazard areas used for floodplain management programs on alluvial fans have often been developed from minimal information by faulty methods. James et al. (1980b) reviewed Utah’s flood history and found the two primary dangers to be from 1) flows of high velocity and high sediment content bursting out of the mouths of mountain canyons onto the apex areas of alluvial fans and 2) channel overflow at blocked culverts or
Figure 6. Approximate area affected by the 100-year flood.
Figure 7. Comparative mapping of a portion of the Stone Creek Floodplain, Bountiful, Utah.
other clogged locations. Whereas the methods used to map flood hazards on riverine flood plains in humid areas work poorly in both situations, the simulation modeling now underway promises a framework for quantifying the flows and resulting hazards both for long term planning and conditionally as based on current conditions. Local information can be used to simulate channel sedimentation and flood depths by location. This information can then be translated into economic losses, dangers to personal safety, and environmental impacts and used to encourage an optimal balance between flood risk and land use.

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System Operation for Flood Damage Mitigation

FLOOD PREVENTION ON THE STRAWBERRY RIVER, UTAH

by Franklin E. Dimick

Abstract:

Much of the attention of the news media during the spring floods of 1983 was focused along the Wasatch Front. Other areas such as the Strawberry River received very little attention. This was due not only to the remoteness of the area but also because flood damage was held to a minimum with the use of storage reservoirs on the river system.

By coordinating the operation of four separate dams controlled by three different entities, the flood flow of the Strawberry River through the city of Duchesne, Utah, was held to a maximum of 1900 cfs. The uncontrolled flow would have been in excess of 4,000 cfs.

This effort required early planning, lifting of restrictions on Soldier Creek Dam, and 24 hour-a-day operation of Currant Creek Dam. The result was not only beneficial to the city of Duchesne but had an effect on the entire river system below Strawberry Dam, including the Colorado River.

OVERVIEW

During the spring runoff of 1983, almost the entire State of Utah was affected by flooding. Most of the media attention was focused on the area of the Wasatch Front from Brigham City to Nephi. One area that experienced flooding but did not receive extensive media coverage was the Strawberry River. This river is located in Wasatch and Duchesne Counties, having its headwaters just above Strawberry Reservoir and ending as it joins the Duchesne River just below the town of Duchesne, Utah.

There appears to be two main reasons why flooding along this river was not highly publicized. First, the area is sparsely populated for the most part with the town of Duchesne being the only significant population center along the entire length of the Strawberry River. Therefore, it was not a newsworthy event since flooding along the Strawberry did not affect as many people nor cause as much property damage as that occurring along the Wasatch Front.

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The second and more important reason was that major flooding of the town of Duchesne and other areas along the river was prevented by the use of storage reservoirs on the Strawberry River and one of its tributaries. These reservoirs were used to store and regulate the flows in the Strawberry River, thereby reducing the peak flow through the town of Duchesne by as much as 50% and peak flows on other parts of the river by up to 99%. This judicious use of reservoir storage prevented approximately $97,000 worth of flood damage along the river. The achieving of this significant reduction in peak flow and the resulting prevention of damage required extensive early planning, coordination of effort, and continual monitoring of conditions to make necessary changes in operating plans.

BACKGROUND

To fully comprehend the magnitude of effort required to use all of these reservoirs in a coordinated flood control program it is necessary to have an understanding of each reservoir and its operating limits. There are three storage reservoirs on the main stem of the Strawberry River and one small reservoir on Current Creek, a tributary of the Strawberry. (See Figure 1.)

Strawberry Reservoir is located near the headwaters of the Strawberry River in Wasatch County. The Strawberry Dam was completed in 1912 as a storage dam for irrigation purposes only. It captures all water from Strawberry River above the dam as well as several other streams including Indian Creek and Trail Hollow. The water is then released as needed through a 3.6 mile long tunnel on the west side of the reservoir through the mountains into the Diamond Fork River. This river runs into the Spanish Fork River from which water is diverted to irrigate lands in the south end of Utah Valley in Utah County. Water is only released from the reservoir when the flow of the Spanish Fork River drops below the irrigation demand. Since average irrigation demands are equal to the average inflow to Strawberry Reservoir, there is usually no excess water to release downstream into the Strawberry River as the reservoir seldom fills to full capacity. Because of this the outlet gates in the dam have never been operated and are considered inoperable. The few times that the reservoir has filled beyond its full capacity of 270,000 acre-feet, the excess water has flowed over the spillway into the river. Prior to 1983, the reservoir had spilled only two times. Strawberry Dam is operated by the Strawberry Water Users Association.

The next reservoir on the Strawberry River is Soldier Creek Dam. It is located in Wasatch County about seven miles downstream from Strawberry Dam. Soldier Creek Dam was designed as a large storage reservoir to replace and enlarge the existing Strawberry Reservoir and Dam. When completed and filled, the water backed up by the dam will inundate the old Strawberry Dam and will increase the storage capacity of the reservoir from the present 270,000 acre-feet to 1.1 million acre-feet. Major releases will still be through a tunnel on the west side of the reservoir into the Diamond Fork River and thus into the Spanish Fork River. Small releases of approximately 26 cubic feet per second will be made into the Strawberry River to maintain fish habitat. During the final phases of construction at Soldier Creek Dam, the reservoir level was restricted to elevation 7500.0 feet above sea level. This backed water to the downstream toe of the old Strawberry Dam. Upon completion of
construction, the two reservoirs will be equalized and Strawberry Dam will be breached. Until the two reservoirs are equalized, they must be operated as two separate reservoirs. Since the dam is still in the construction phase it is operated by the U.S. Bureau of Reclamation.

The third reservoir on the main stem of the Strawberry River is Starvation Reservoir. The dam is located in Duchesne County about three miles upstream from the town of Duchesne and about 5 miles upstream from its junction with the Duchesne River. Starvation Dam was completed in 1970, as a multi-purpose reservoir to store excess flows of the Strawberry and Duchesne Rivers. Excess flows of the Duchesne River are brought into Starvation through a diversion dam and tunnel system. Starvation has an active storage capacity of 152,330 acre-feet and is operated by the Central Utah Water Conservancy District. Water released from Starvation Dam flows through the town of Duchesne. Releases or spills in excess of 1,500 cubic feet per second are considered to be damaging to the town.

Currant Creek Dam is located in Wasatch County on Currant Creek, a tributary of Strawberry River, about 13 miles North of U.S. Highway 40. The reservoir has a total capacity of 15,670 acre-feet. It was built to provide regulation of flow in the Strawberry Aqueduct and will divert excess flows of Currant Creek into the aqueduct. The dam is in the final stages of construction and was in the initial filling phase during the 1983 runoff. Initial filling criteria limited the filling rate to .5 feet per day and to a maximum of 27 feet for the year. The dam is operated by the U.S. Bureau of Reclamation.

Developing a system wide flood control program requires that operating limits of each facility in the system be considered and accounted for.

THE PROBLEM

The first indication that a problem might be present occurred in the fall of 1982. Late fall rains had kept Soldier Creek, Strawberry, and Starvation Reservoirs at very high levels going into the winter season. It was apparent that if a normal amount of precipitation were to fall during the winter months that Strawberry Reservoir would fill and spill in the spring of 1983. All the water spilled would flow into Soldier Creek Reservoir and the restricted elevation of 7500.0 would be exceeded unless the reservoir were drawn down. Water could not be released from Strawberry Reservoir through the tunnel to Diamond Fork as that water would end up in Utah Lake which was already fuller than normal. Since the gates at Strawberry Dam are inoperable, water could not be released downstream to Strawberry River or Soldier Creek Reservoir. To further complicate the problem, any water released from Soldier Creek Dam must be passed on through Starvation Reservoir. This would be in addition to what must be released from Starvation in order to prepare for the spring runoff.

FIRST SOLUTION TO PROBLEM

The Bureau of Reclamation began releasing water from Soldier Creek Dam on October 21, 1982, in order to prevent the reservoir from filling beyond the 7500.0 elevation restriction when the spring runoff occurred. The intent was
to draw the reservoir down far enough that it could accommodate natural runoff as well as the spill from Strawberry Reservoir. The Bureau also examined ways of minimizing the anticipated spill from Strawberry Dam. Two solutions were finally adopted. First, under agreement with the Strawberry Water Users Association, Indian Creek was diverted from Strawberry Reservoir directly into Soldier Creek Reservoir. Indian Creek supplies about 1/3 of the total water supply for Strawberry. By diverting this water directly to Soldier Creek, it would reduce the potential spill from Strawberry by 1/3. The actual diversion took place in the middle of winter and required the use of explosives to excavate the frozen ground. This diverted water was then passed on through Soldier Creek and Starvation Reservoirs.

The second solution to help reduce the amount of uncontrolled spill from Strawberry was to remove a wooden portion of the spillway, thus the elevation of the spillway was reduced by 1.5 feet. This was done with approval of the Strawberry Water Users Association with the understanding that if the reservoir did not spill, the Bureau of Reclamation would replace any water lost. This replacement would be through the use of the Strawberry Aqueduct, which was also shut off to reduce flows into Strawberry Reservoir. Reducing the level of the spillway allowed the reservoir to spill early and reduced the potential spill by approximately 7,000 acre-feet.

COMPLICATIONS

Reducing the storage of Soldier Creek Reservoir was going as planned until New Years Eve, December 31, 1982. On that day, the Strawberry River froze and the resulting ice jam, just above Starvation Reservoir, caused flooding of several homes and a county road. The high flows being released from Soldier Creek aggravated the flooding situation. The releases from Soldier Creek were drastically reduced to alleviate some of the flooding and to help county and state personnel who were trying to clear the ice jam. Due to the extremely cold weather that settled into the Uintah Basin that year, the ice jam could not be broken enough to allow increased releases out of Soldier Creek until the weather moderated in February. This put the drawdown of Soldier Creek Reservoir behind schedule.

The next complication in the planned operation of the system was the Thistle landslide. This slide occurred on the Spanish Fork River and formed a natural dam, which held back the high spring flood flows of the Spanish Fork River. This prevented flooding on the lower Spanish Fork River but meant disaster for the operation of the Strawberry system. By catching the high flows and forming a natural reservoir, the slide provided a natural irrigation reservoir. As the reservoir behind the slide was drained through the late summer and early fall, it provided almost all the irrigation water needed by the Strawberry Water Users Association. Therefore only 9,327 acre-feet of water was taken from Strawberry Reservoir for irrigation purposes during 1983. Approximately 60,000 acre-feet are released during an average year. This meant that an additional 51,000 acre-feet of water would now be spilled into Soldier Creek rather than being released to the Spanish Fork River.

The final complication that occurred which required modification of the operating plan was the extremely wet and cold spring that was followed by a
very warm spell in late May and early June. This caused the actual runoff of water to far exceed the forecasted amount. This condition is almost impossible to plan for because there is not enough time to provide sufficient storage space when the problem arises in just a few weeks as it did in May 1983.

MODIFICATION OF OPERATING PLAN

When the weather changed from cold and wet to very warm it was obvious that record flows in the Strawberry River and its tributaries would occur. This meant that a change was needed in order to hold to a minimum the flooding that would occur. The first modification was to work closely with the Central Utah Water Conservancy District to increase releases out of Starvation Reservoir in an attempt not to exceed the 1,500 cfs safe channel capacity. Constant radio and telephone contact was maintained with the operators of Soldier Creek, Currant Creek, and Starvation Dams to coordinate all releases.

The inflow to Soldier Creek Reservoir was fast approaching 4,000 acre-feet per day, thereby raising the water surface at a rate in excess of three feet a day. It was apparent that the restricted elevation of 7500.0 feet would be attained long before the peak of the runoff was reached. If this occurred, releases out of Soldier Creek would have to be increased to over 2,000 cfs. The Utah Projects Office, Bureau of Reclamation, petitioned its Regional Office and the Bureau's Engineering and Research Center to lift the restriction. Due to the unusual conditions that had transpired to create this situation it was considered an "act of God" and the restriction was lifted. One of the conditions for lifting the restriction was that the dam be attended on a twenty-four hour basis. A Bureau of Reclamation employee was immediately moved to the site in a trailer for full time attendance. With the restriction lifted, releases from Soldier Creek were reduced to less than 50 cfs.

Currant Creek Dam, being operated under the restriction of .5 foot rise in reservoir level per day, was used to take the peak off of the daily flows on Currant Creek. Since the peak flow on this stream occurred during the middle of the night, this action required twenty-four hour a day operation. Although this reservoir is relatively small it did reduce the peak flow on Currant Creek by an estimated 15% and on the Strawberry by approximately 5%.

RESULTS

Mainly through the lifting of the restriction on Soldier Creek Reservoir elevation but also through all of the efforts mentioned, record flows on the Strawberry River were handled with only minimal damage. Although the safe channel capacity of 1,500 cfs through the town of Duchesne was exceeded by about 400 cfs for a brief period, causing some minor damage, a disaster was averted. If the system of reservoirs on the Strawberry had not been operated as they were, flows exceeding 4,000 cfs would have run through the town of Duchesne, causing extensive and severe damage.

Over 75,000 acre-feet of additional water was stored in Soldier Creek Reservoir in the month following the lifting of the filling restriction. This represents an average flow exceeding 1200 cfs that was removed from the Strawberry River for that same period.
CONCLUSIONS

Operation of storage reservoirs as a system for flood control can result in a significant reduction of flood potential and the problems relating to the operation of such a system can be resolved for the mutual benefit of all parties involved.

Figure 1.
"THE OPERATION OF MAJOR DAMS ON THE COLORADO RIVER SYSTEM DURING THE FLOOD OF 1983"

by John D. Newman

INTRODUCTION

As the gates at Glen Canyon Dam were closed on March 13, 1963, it was the opinion of many that the huge empty reservoir behind the dam would never fill with water. Even so, huge spillways were constructed in each abutment so that if the lake filled, the probable maximum flood of the Colorado River could be controlled. As the average annual flow of the Colorado River steadily declined over the next 17 years the prospect of ever using them remained remote. In June of 1980, however, as Lake Powell filled for the first time the spillways were used to control the rise of Lake Powell. Just 3 years later, however, Lake Powell filled for the second time, but with much more significance. Early releases through the spillways in 1983 caused cavitation damage to the concrete lining so severe that their use became restricted before peak inflows had occurred. Without the full use of these spillways, operation of Glen Canyon Dam through the remainder of the 1983 runoff was a major problem. Limiting the flow of water through the spillways to avoid further damages plus controlling the magnitude of flooding downstream were the two major objectives throughout the runoff period of 1983.

This paper will document the decision making which took place in the Upper Colorado Region and the subsequent impacts in the Lower Colorado Region during the 1983 flooding on the Colorado River and will discuss some rationale as to why such decisions were needed. Also, a chronology of events will be presented with explanations of the "overnight" structural modifications that became necessary.

GENERAL OPERATIONAL PHILOSOPHY - COLORADO RIVER STORAGE PROJECT

The system of reservoirs that comprise the majority of regulatory storage in the Upper Colorado River Basin are Fontenelle and Flaming Gorge Reservoirs on the Upper Green River in Wyoming and Northern Utah, Blue Mesa, Morrow Point, and Crystal Reservoirs on the Gunnison River near Montrose, Colorado, Navajo Reservoir on the San Juan River in New Mexico, and Lake Powell and Lake Mead...

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on the mainstem of the Colorado River in southern Utah, Arizona, and Nevada. Figure 1 is a general map of the Upper Colorado River basin which shows the general location of each of these features. Table 1 indicates the various capacities and elevations of these reservoirs.

An annual operating plan for the major reservoirs on the Colorado River system is required by law and is initially prepared at the start of each water year which is October 1. At that time, operation studies are prepared using several different levels of possible inflow for the upcoming year. Since forecast data is not available for the next runoff season on October 1, a statistical analysis of historical water years is used to develop different inflow scenarios. These inflows are then used in reservoir operation studies to determine the operating limits and release schedules for each inflow level. The actual reservoir releases for October, November, and December, of each water year, then, are planned so as to achieve a prudent target level on January 1. The annual operating plan is updated at the first of each month to reflect current reservoir levels and changing hydrologic conditions.

Forecasts of reservoir inflow for the spring runoff are made beginning on January 1 and are updated at the beginning and middle of each month thereafter through the end of July. Reservoir release schedules for January through July, then, are primarily a function of the runoff forecast. If these forecasts are high, then release schedules are planned so as to develop sufficient reservoir space to contain the most probable runoff level without spills. If the forecast is low, then release schedules are usually planned to conserve the maximum amount of conservation storage while optimizing the generation of hydropower to meet firm energy requirements. In the case of Glen Canyon, an objective minimum annual release volume exists which prevents releases from dropping below certain monthly levels. A monthly breakdown of this minimum annual release at Glen Canyon is usually scheduled to achieve an optimum balance between generated and purchased energy.

The amount of reservoir space in Lake Powell achieved on January 1 is usually dictated by the amount of runoff in the preceding water year. If the runoff was sufficiently low, then the January 1 reservoir space becomes whatever is produced by meeting the minimum annual release requirements or energy needs. If the runoff was high then the January 1 space level is planned to limit the risk of reservoir spills for the upcoming runoff to an acceptable level. Certain Lake Powell space levels on January 1 can be directly associated with risks of spilling later in the year. These risks of spilling are balanced against risks to water conservation in planning January 1 reservoir levels.

WATER YEAR 1983 OPERATING PLAN - COLORADO RIVER STORAGE PROJECT

The annual operating plan for water year 1983 considered five different levels of projected inflow. These inflows had exceedance percentages of 10 percent, 25 percent, 50 percent, 75 percent, and 90 percent with the 50 percent level being average. The reservoir system comprising the Colorado River Storage Project was then modeled with each of these inflow levels to determine if certain reservoirs would fill and what release levels should be achieved to reduce the risk of spilling.
COLORADO RIVER STORAGE PROJECT
AND PARTICIPATING PROJECTS

UNITED STATES
DEPARTMENT OF THE INTERIOR
James G. Watt, Secretary
BUREAU OF RECLAMATION
Robert N. Bradburd, Commissioner
UPPER COLORADO REGION
Salt Lake City, Utah
1984-85

LEGEND
* CRSP STORAGE UNITS
* Participating Projects
Cities and Towns
<table>
<thead>
<tr>
<th>Reservoir</th>
<th>Dead Storage</th>
<th>Minimum Power Pool</th>
<th>Rated Powerplant Head</th>
<th>Storage units: acre-feet</th>
<th>Normal Maximum Storage Without Surcharge</th>
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<td></td>
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With the reservoir levels that existed on September 30, 1982, it was apparent that an average inflow for water year 1983 could possibly fill the majority of reservoirs in the Upper Colorado River system. Inflows higher than average would require higher release schedules to prevent spills. The inflow level with an annual exceedance of 10 percent indicated that maximum powerplant releases would be necessary for the remainder of the water year to avoid spilling at most reservoirs. With these facts in mind, a January 1 target level for Lake Powell of 22.6 million acre-feet was planned to reduce the risk of spilling to an acceptable level.

The forecast of runoff into Lake Powell for the April through July period was near normal on January 1, 1983, and as a result the plan of reservoir releases was not modified. Subsequent forecasts were actually less than the January forecast until about April 1, 1983, which reinforced the decision to moderate releases. However, the actual April through July 1983 unregulated runoff into Lake Powell was 14.5 million acre-feet (MAF) which is about 210 percent of average. This is the highest April through July runoff volume since 1917 and is the second highest ever recorded. Since Lake Powell and the other major Upper Basin reservoirs were lowered to a level in expectation of a near average April through July runoff volume, this unexpected event quickly filled these reservoirs and, as a result, significant amounts of water were spilled.

Many flow control structures were operated at levels never experienced before and Glen Canyon's spillways were severely damaged. It is of interest and major importance to note that these damages occurred at flow levels well below those for which the spillways were originally designed.

CHRONOLOGY OF EVENTS - RUNOFF OF 1983

The following is a chronological accounting of the events which took place and decisions that were made at some of the major impoundments in the Upper Colorado River Basin during the 1983 runoff.

Fontenelle Dam and Reservoir

During the 1982 runoff Fontenelle Reservoir remained near full from about June through September or 4 months. Seepage problems were noted on the left abutment of Fontenelle Dam which became a real concern during the fall of 1982. It was noted that seepage dramatically increased at lake elevations above 6,495 feet, therefore, plans were made to fill Fontenelle no higher than that level during the 1983 runoff while investigations were underway. The reservoir reached elevation 6,495 feet in early June 1983 and releases were increased to hold the lake at that level. Near the end of June new information revealed that the dam was potentially unsafe at lake elevations above 6,482 feet. Therefore, on July 1 releases were steadily increased to bring Fontenelle Reservoir down to a safer level. This dramatic increase in releases occurred at a very critical time with respect to Flaming Gorge Dam which is about 125 miles downstream of Fontenelle Dam on the Green River. Fontenelle Reservoir has remained at or near elevation 6,482 feet since early July 1983.
Flaming Gorge Dam and Reservoir

Flaming Gorge Reservoir was lowered to about elevation 6,025 feet in preparation for the forecasted runoff of 1983. The normal maximum elevation is 6,040 feet; however, it was planned to fill no higher than 6,035 feet to help preclude the use of the spillway. This provided almost 400,000 acre-feet of vacant reservoir space to contain a projected April through July runoff of 1,105,000 acre-feet. This forecast was made on April 1, 1983, and was about 96 percent of normal. Subtracting these two figures leaves about 700,000 acre-feet to be released during the April through July period. This was easily within the powerplant capability at Flaming Gorge of about 250,000 acre-feet per month. The actual April 1983 unregulated inflow was 224,000 acre-feet which was just slightly above the forecast for April of 195,000 acre-feet. The May forecast projected that 315,000 acre-feet of unregulated runoff would occur. The actual May unregulated volume was 445,000 acre-feet which was significantly higher than projected. Near the first of June the forecasted inflow was 525,000 acre-feet; however, the actual June inflow was 973,000 acre-feet. On June 3, 1983, it became apparent that the lake would fill to elevation 6,040 feet and the river outlet works were opened to bypass the additional water. The river outlets plus the powerplant discharged a total of 8,000 ft³/s until the lake filled. On June 22, the spillway was opened and the total release was boosted to 10,000 ft³/s. The lake still continued to rise. On July 1, the elevation was more than 2.5 feet above the normal maximum level of 6,040 feet. Inflows at that time showed signs of receding; however, the increased releases from Fontenelle caused Flaming Gorge to rise even further. Spillway flows were increased until a total of nearly 13,000 ft³/s was being released. The lake peaked at elevation 6,043.78 on July 14 nearly 4 feet above its normal maximum level. The 5 feet of reservoir space originally planned as a buffer to preclude the use of the spillway plus the 4 feet of reservoir surcharge had to be used to handle the actual runoff.

These circumstances provoked several key decisions concerning the spillway tunnel at Flaming Gorge. Repair work to the concrete lining inside the tunnel had not yet been completed so there was considerable concern about using the spillway. An aeration slot to eliminate cavitation damage had been constructed in the inclined section of the tunnel during the summer of 1982, but it had never been operationally tested. For these reasons it was decided that temporary wooden flashboards should be installed on the two vertical slide gates which control flow through the spillway. This would allow lake elevations to rise above 6,040 feet without using the spillway. Model studies and initial calculations indicated that the spillway could be used at a flow of 5,000 ft³/s without a significant risk of further damaging the concrete lining. It was decided that this option was preferable to storing water against the wooden flashboards. Also, maximum discharge from the dam without the spillway was 8,000 ft³/s and lake inflows were more than 15,000 ft³/s when it filled. It was evident that additional outlet capacity would be needed to stop the rise of the water surface. Spillway flows were initiated on June 22, 1983. A discharge of 5,000 ft³/s was established in the spillway while the flow through the river outlets was reduced to 1,000 ft³/s. This configuration when combined with powerplant
releases of about 4,000 ft³/s brought the total release to 10,000 ft³/s. This release level was held until inflows increased as a result of the rapid drawdown of Fontenelle Reservoir. At that time the river outlets were increased to their maximum of 4,000 ft³/s which brought the total release to near 13,000 ft³/s. The reservoir continued to rise to a peak elevation of 6,043.78 feet on July 14, 1983.

One additional structural modification that was unanticipated involved the emergency air intakes for the powerplant penstocks. These air intakes are located on the upstream face of the dam above elevation 6,040 feet and are intended for use during an emergency closure of the penstock fixed-wheel gates. If the turbine wicket gates do not close as the fixed-wheel gates close during an emergency shut down, the penstocks would be evacuated of water so rapidly that the steel penstock lining could theoretically collapse. These air intakes would bring air into the penstock to stop this from occurring. These intakes became inundated as the reservoir peaked and it became necessary to "snorkle" the intakes to the atmosphere. This was accomplished by constructing a metal cage around the face of the intakes which exposed them to the atmosphere at reservoir elevations above the normal maximum of 6,040 feet. It is interesting to note that this work was accomplished while the intakes were under water.

Blue Mesa Dam and Reservoir

Blue Mesa Reservoir was lowered to elevation 7471.6 feet by April 1, 1983, in anticipation of a forecasted April through July runoff of about 630,000 acre-feet which is 62 percent of normal. The reservoir continued to decline during April 1983 as the actual inflow was less than anticipated. The May forecast of April through July runoff was increased to 680,000 acre-feet or 88 percent of normal. The actual May inflow was again less than anticipated. On June 1, 1983, the forecast of April through July runoff was decreased to 640,000 acre-feet of which 275,000 acre-feet was projected for June. Heavy rain brought the actual June volume to 399,000 acre-feet. This unexpected water quickly filled Blue Mesa Reservoir and on June 23, 1983, the river outlets were opened to stop the rise of the water surface. The actual April through July inflow to Blue Mesa was 840,000 acre-feet which is 188 percent of normal and 210,000 acre-feet more than forecasted on April 1.

The spillway at Blue Mesa is essentially the same design as the spillways at Flaming Gorge and Glen Canyon. Since it was known that the concrete lining in these types of spillways has a relatively short life without aeration slot modification, it was decided to avoid using the spillway at Blue Mesa if possible. To do this required maximum use of the river outlets.

The two powerplant turbines and the two 84-inch diameter hollow-jet valves on the river outlets at Blue Mesa are served by a single 16-foot diameter penstock. As the river outlets are opened, water flows past the bifurcation to the river outlets which begins to decrease the discharge of the powerplant. As the river outlets approach 5,000 ft³/s, the powerplant discharge is reduced from a maximum of 3,000 ft³/s to near 2,000 ft³/s because of the reduced pressure at the bifurcation. At the time of their use in June 1983, a
restriction of 6,000 ft³/s total discharge had been placed on the penstock. This restriction was invoked because it was felt that at discharges above 6,000 ft³/s pressure at the bifurcation would be so low that potential damage could occur to the turbines. During June of 1983, the inflows to Blue Mesa when the lake filled were about 7,000 ft³/s. This in turn required a release of 7,000 ft³/s to stop the rise of the water surface. It was decided to increase the total penstock discharge to its design maximum of 7,000 ft³/s and closely monitor the pressure at the bifurcation. As this was accomplished the pressures declined; however, they remained at safe levels. As a result of this operation the restriction on the total penstock discharge was eventually removed. It is interesting to note that this penstock discharge "test" occurred at a very critical time with respect to filling the reservoir. If the test had produced excessively low pressures at the bifurcation the spillway would have been opened. Spillway flows would have eventually caused cavitation damage to the concrete lining and severe operational problems as use of the spillway became restricted. The discharge of 7,000 ft³/s stopped the water surface at elevation 7,516.65 feet which is 0.8 feet below Blue Mesa's normal maximum elevation of 7,519.4 feet.

Glen Canyon Dam and Lake Powell

The January forecast of runoff into Lake Powell for the April through July period was 7.8 MAF or 112 percent of normal. On February 1, the forecast decreased to 7.1 MAF or 102 percent of average, and on March 1 it decreased to 6.7 MAF or 96 percent of average. Reservoir releases during this time were moderately low because the price of purchased energy was such that interchanges of energy were attractive and the runoff forecast did not warrant higher releases to control anticipated runoff. In April and May, as the forecast increased slightly to 7.9 MAF on April 1 and 8.13 MAF May 1, 1983, reservoir releases were increased to contain the anticipated runoff. It is interesting to note that only a slight increase in the forecast required increases in releases. By mid-May, Glen Canyon Powerplant was operating at its maximum, discharging 28,000 ft³/s in response to an increased mid-May forecast of 8.33 MAF. The remaining forecasts of runoff increased dramatically as the inflow to Lake Powell continued to climb. The June 1 forecast was 9.1 MAF. The June 14 forecast increased to 11.3 MAF, and on June 18 it increased again to 13.3 MAF. Finally on June 22 the forecast was 14.6 MAF. Table 2 shows this forecast data in tabular form. The numbers shown in parenthesis are the actual recorded inflow figures.

On June 2, as it became readily apparent that Lake Powell would fill, the left spillway gates were opened and a discharge of 10,000 ft³/s was established. Lake Powell elevation at that time was 3696.8 feet. The flow was increased to 20,000 ft³/s by that weekend as the hydrograph of inflow continued to climb. This hydrograph is shown in Figure 2. On Sunday, June 5, 1983, it was reported that loud rumbling noises could be heard and material was observed in the spillway discharge plume. The left spillway was immediately shut off and the right spillway was opened to a discharge of 8,000 ft³/s. The four 96-inch diameter hollow jet valves which comprise the river outlet works at Glen Canyon were also opened to a discharge of 15,000 ft³/s. The left spillway tunnel was inspected on Monday, June 6, 1983, and significant cavitation
# TABLE 2

1983 RUNOFF FORECASTS - LAKE POWELL

**UNITS:** Million Acre-Feet (MAF)

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<th>Feb 1st</th>
<th>Mar 1st</th>
<th>Apr 1st</th>
<th>May 1st</th>
<th>June 1st</th>
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<td>March</td>
<td>.68</td>
<td>.66</td>
<td>.66</td>
<td>(1.05)</td>
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<td>(1.05)</td>
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<td>(1.12)</td>
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<td>(3.31)</td>
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<td>7.2</td>
<td>6.7</td>
<td>7.9</td>
<td>8.1</td>
<td>9.1</td>
<td>14.6</td>
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</table>
UNREGULATED RUNOFF INTO LAKE POWELL

Figure 2
damage was observed near the lowest portion of the elbow section where the inclined section of the tunnel intersects the concrete diversion tunnel plug near the horizontal section. It was apparent at that time that some method had to be devised to limit the flow of water through both spillways while the reservoir was above 3,700 feet or in the surcharge pool. Routines of the June forecast of inflow indicated that Lake Powell should not rise above elevation 3,702.5 feet if a total discharge of 48,000 ft³/s could be maintained from Glen Canyon Dam. It was decided at that time that 4-foot high flashboards consisting of plywood sheets supported by angle iron braces welded to the top of the gates would provide sufficient protection.

Also, as the river outlets were opened to their maximum discharge of 17,000 ft³/s several of the penstock couplings began to leak excessively. Plugged drains caused this leakage to eventually fill the inspection chamber and pressurize the manhole covers as water spurted out of small openings. However, as the drains were opened the pressure was reduced and as the couplings were eventually tightened the leakage was no longer a problem.

On June 7, it became obvious that the runoff would exceed the June forecast as the hydrograph of inflow to Lake Powell had not significantly receded. Spillway design experts in Reclamation’s Engineering and Research Center recommended that spillway flows be limited to 4,000 ft³/s on the right side and 2,000 ft³/s on the left side to limit further damage. It was also recommended that use of the right spillway be delayed as long as possible should the need arise to use it later on for higher discharges. Lake Powell reached elevation 3,700 feet on June 8, 1983.

By June 8, the 4-foot high plywood flashboards were completed on the left spillway gates and it was determined that similar extensions should be constructed on the right spillway. This would permit reduced spillway discharges in both tunnels while the reservoir was above 3,700 feet. Total discharge remained near 49,000 ft³/s with 28,000 ft³/s from the powerplant, 17,000 ft³/s from the river outlets, and 4,000 ft³/s from the right spillway. On June 13, the runoff forecast was increased to 11.3 MAF. With releases of 49,000 ft³/s, this new volume of projected inflow would cause Lake Powell to rise to a peak elevation of 3,704.8 feet. It was then decided to increase releases to 54,000 ft³/s which would limit the peak elevation to 3,703.7 feet. This was necessary so that the spillway gates could be closed as the lake peaked without overtopping the 4-foot plywood flashboards.

On June 15, 1983, the total discharge was increased to 58,000 ft³/s. This was accomplished by shutting off the right spillway and using the left spillway at 13,000 ft³/s. It was our opinion that the left spillway tunnel should be used as long as safely possible to preserve the right spillway for any subsequent increases. On June 19 the hydraulic jump in the left spillway collapsed indicating further cavitation damage to the tunnel lining. The left spillway discharge was increased to 17,000 ft³/s which reestablished the sweep or hydraulic jump. Total discharge was then about 62,000 ft³/s. By June 22, the left spillway again lost its sweep and the discharge was increased to 20,000 ft³/s. This produced a total discharge of 65,000 ft³/s from Glen Canyon Dam. By this time the forecast of inflow to Lake Powell had reached 14.6 MAF which
promoted an increase in total discharge to 70,000 ft³/s. This was accomplished by using the right spillway at 4,000 ft³/s and the left spillway at 21,000 ft³/s. On June 23, however, the left spillway again lost its sweep. Flow in the left spillway was immediately reduced to 10,000 ft³/s and the right spillway discharge was increased to 15,000 ft³/s to maintain a total release of 70,000 ft³/s. Flood routings of the latest projected inflow to Lake Powell indicated that releases of more than 70,000 ft³/s would be needed to keep Lake Powell below elevation 3,708 feet. Again, it was decided to use the left spillway at higher discharges to preserve the right side. Therefore, on June 27, the right spillway was decreased to 10,000 ft³/s and the left spillway was increased to 25,000 ft³/s. This produced a total discharge of 80,000 ft³/s. On June 28, however, loud noises were heard in the dam galleries near the left spillway. It was decided to try to stabilize the left spillway plume by increasing left spillway flows to 32,000 ft³/s. The right spillway was also increased to 15,000 ft³/s which produced a total release of 92,000 ft³/s. Very soon, however, the noise in the left side became worse and the sweep collapsed as material from the tunnel colored the plume. Flow in the left spillway was reduced to 20,000 ft³/s and the right side was increased to 27,000 ft³/s to maintain the total release of 92,000 ft³/s. Inflow to Lake Powell on June 28 was over 115,000 ft³/s with a lake elevation of 3,706.7 feet.

On June 29 the flow in the left spillway was reduced to 15,000 ft³/s as inflows to Lake Powell declined. The elevation of Lake Powell at that time was over 3,707 feet. In order to limit spillway flows with the lake over 7 feet in surcharge it was decided that 8 foot high reinforced steel extensions should replace the plywood flashboards. Releases were maintained at 85,000 ft³/s with 20,000 ft³/s in each spillway. By July 2 the right spillway began losing its sweep so its discharge was increased to 24,000 ft³/s. At that time, drainage holes in adits of the dam just above the right spillway tunnel began surging air and water indicating a connection between joints in the abutment rock and the drain holes. Flows in the right spillway were immediately reduced to 20,000 ft³/s.

During the July 4 weekend Guy F. Atkinson Company was working at the dam installing the 8-foot high reinforced steel extensions on each spillway gate. Work was also underway to snorkle the penstock air intakes. This problem was identical to that at Flaming Gorge. In fact, it was the discovery of the problem with the air intakes at Glen Canyon that prompted the extension of the intakes at Flaming Gorge. By July 8 the steel extensions were completed on both spillways. On July 14, 1983, Lake Powell peaked at elevation 3,708.34 feet. Releases from the dam of 55,000 ft³/s were maintained by discharging 5,000 ft³/s from each spillway until the reservoir reached elevation 3,707.8 feet. On July 23, both spillways were shut off with total discharge from the dam at 41,000 ft³/s.

Inspection of the spillway tunnels was accomplished by first boating up the horizontal sections to the elbows. This revealed that major portions of the concrete lining on both spillways was missing. Huge sections of reinforcing steel had been torn away and left in massive twisted balls. The tunnel floors of both spillways above the elbow showed a series of holes progressing up the
inclined section getting smaller towards the top. It was speculated that the largest hole in the left spillway which was at that time under water was much larger than any of the holes above the elbow section. Pumping began immediately. Inspection of the left tunnel after the water was pumped out revealed devastation far beyond what was expected. A huge hole about 35 feet deep and several hundred feet long had been eroded from the spillway tunnel floor on the left side. Also, a huge boulder about the size of a dumptruck had been plucked out of this hole and moved several hundred feet downstream by the force of the moving water. The right spillway had not been damaged nearly as much since it had been reserved for even higher discharges.

LOWER COLORADO RIVER

For the 17 years while Lake Powell filled a significant amount of building had occurred below Parker Dam on the Lower Colorado River in what is called the Parker strip. Releases from Parker Dam during this 17 years never exceeded about 19,000 ft³/s which is near Parker Powerplant’s maximum discharge. Encroachment in the flood plain had progressed to a point where any flow above about 20,000 ft³/s would cause flood damages to permanent structures. Hoover Dam and Lake Mead began feeling the impact of Glen Canyon’s large releases almost immediately. By about June 1 Hoover Dam was releasing 28,000 ft³/s and by the end of June releases were at 39,000 ft³/s. Releases of this magnitude from Hoover Dam were successively released from Davis and Parker Dams within a short time.

Flows from Hoover Dam of 40,000 ft³/s were causing severe flood damages below Parker Dam and in other areas where encroachment within the flood plain had occurred. It was inevitable that the spillway at Hoover would eventually be used since the reservoir was rising rapidly and releases were being held at 40,000 ft³/s to limit flooding downstream. As the Hoover spillways began discharging on July 2, the powerplant releases were correspondingly reduced to keep total Hoover Dam releases near 40,000 ft³/s. Once Hoover Dam began spilling, it was evident that the water which was stored in Lake Powell’s surcharge pool (above 3,700 feet) should be held back as long as possible. By reducing Glen Canyon’s releases as quickly as the inflows to Lake Powell allowed, the magnitude of spill at Hoover could be minimized and the flood damages could also be reduced. About 1.4 MAF was stored in Lake Powell above elevation 3,700 feet when it peaked. As inflows to Lake Powell gradually subsided the releases from Glen Canyon were correspondingly reduced. Thus, the water in the surcharge pool at Lake Powell was evacuated very slowly which helped alleviate flooding on the Lower Colorado River below Hoover Dam and caused Hoover Dam spills to stop several weeks early. This slow withdrawal was possible due to the 8-foot extensions on each spillway gate.

CONCLUSION

It is interesting to hypothesize what decisions would have been made had Glen Canyon’s spillways operated as designed. The spillways at Glen Canyon were designed to discharge a maximum of 135,000 ft³/s each. At flows of 30,000 ft³/s, however, damages to the concrete lining restricted their use to the point that structural modification was necessary to limit damages. If the
spillways at Glen Canyon had operated as designed, over 100,000 ft$^3$/s would have been spilled as the spillway gates remained open to empty the surcharge pool. This in turn would have caused a 70,000 ft$^3$/s spill at Hoover Dam. Flood damages and loss of life would have been devastating. It is not known if extensions would have been constructed on Glen Canyon's spillway gates solely to limit spills and flooding below Hoover Dam. As it happened, the extensions served several important functions other than protection of Glen Canyon's spillways, including lesser spills at Hoover Dam, less flooding on the Lower Colorado River below Hoover Dam, and protection of Hoover's spillways. Structural modification which was originally intended to compensate for spillway design deficiencies had equally significant impacts in terms of flood control protection downstream.

Several important lessons were learned as a result of the incidents that took place. First, structures which are seldomly used should be operationally tested at or near their design limits; if economically and physically prudent, on a regular basis. This would ensure that when the time comes for their use that they will function as designed. This would have eliminated problems with the penstock restriction at Blue Mesa and leakage with the river outlet works at Glen Canyon. Of course, regular operation of spillways at or near their design limits is not prudent in terms of foregone power revenue and resulting high river flows.

Second, it may not be always prudent to operate a large multi-purpose reservoir system at high storage levels to protect water conservation interests alone. Possibly, some level of buffer space should be reserved for unforeseen events by attempting to balance water conservation against flood control, hydropower, and river control for fish and wildlife and recreation. Since, on the Colorado River, the law specifies the priority of these conflicting interests the actual application of such a philosophy is difficult. As a result, management and operation of the major dams on the Colorado will continue to be a sensitive and sometimes volatile issue as these competing interests interact.
Flood problems and flood awareness in Utah has increased as record setting precipitation and runoff has persisted since the fall of 1982. Serious flooding occurred in northern Utah in September 1982 as a result of runoff from rainfall. Snowmelt runoff in both 1983 and 1984 resulted in widespread flood problems and associated problems of unstable soil conditions, landslides, debris flow, erosion, and deposition of sediment in valley channel reaches. From the flood experience many things have been learned to cope with floods and to reduce associated hazards. These include: preserving critical flood plains through zoning and regulating development; avoiding encroachments and restrictions to channels; recognizing that channel improvements and storage do not provide complete protection against flooding; developing contingency flood control plans in advance of possible floods; reducing damages through the use of available resources, personnel and volunteer assistance; and utilizing intergovernmental expertise and programs.

Responsibility for flood control is vested with the individual property owner affected but for extensive flood problems may best be resolved on a community, a county, a state, multiple state or a Federal basis. Solution of flood problems should begin at the lowest level of responsibility and normally includes problem identification, evaluation of potential structural and nonstructural management measures, and finally development and adoption of a flood control plan. Public involvement and intergovernmental coordination is an essential element focusing on involvement of all affected entities to avoid transferring the problem to another location and to effect implementation of improvements consistent with the needs or overall area of influence.

Efforts are continuing to mitigate flood damages, to restore public facilities, and to alleviate critical flood related problems. Effective flood control requires continued effort and support at all levels of government to insure implementing of practical and affordable protective and corrective management measures, development of workable contingency plans, and adoption of enforceable, nonstructural measures in areas where protection cannot be economically justified.

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FLOOD PROBLEMS

Flood problems and flood awareness have increased in Utah since 1982. The 1981-82 water year was a record year of precipitation at the Salt Lake City Airport. The 12 month total precipitation was 25.15 inches, or about 165 percent of normal. September 1982 was a record month in which 7.04 inches of precipitation was recorded. Over 3 inches of precipitation accumulated during a two day period, September 27-28, and caused severe flooding in Salt Lake County and other northern Utah locations. The heavy precipitation in 1982 saturated the soil mantle and was followed by heavy accumulations of snow during the winter and late spring in 1983 and a sudden warming trend.

Rapid snowmelt during the spring of 1983 caused flows along the Wasatch Front in Utah to exceed 100-year flood levels. The runoff was heavily laden with sediment; and a variety of flood related problems occurred, including landslides, mud flows, stream erosion, reduced stream and conveyance capacities, overflow of streams, and a dam break.

Flood damages totaled about $478 million, affecting about 120 communities and displacing about 4,500 people. Traffic, businesses, industries, tourism, and basically all economic activity were impacted.

During 1983, 22 of Utah's 29 counties qualified for Federal disaster assistance. Federal, State, and local resources have been used in a continuing effort to mitigate flood damages and to restore the watersheds, channel capacities, and public facilities to preproject conditions. Despite implementation of restoration and mitigation measures during late 1983 and early 1984, flooding and damages were also extensive in 1984. Statewide damages have not been estimated but 9 counties were again declared disaster areas. Figure 1 shows the 1983 and 1984 disaster areas.

The recent flooding provides opportunity to review, in retrospect, lessons that may be learned to minimize the causes and effects of those aspects of flood problems attributable or controllable by intergovernmental decisions and actions.

LESSONS LEARNED

Many things can be learned from the recent flood experiences to avoid some of the situations that contributed to the flood and water related problems. These are illustrated by slides (shown at the specialty conference but not reproduced herewith).

1. Continued emphasis needs to be given to the identification of flood prone areas and enforcement of zoning and regulation of lands in the flood plain fringe areas. New developments should be sited to eliminate flood damages up to the 100-year flood level, a flood which has a 1 percent chance of occurrence in any given year. A primary floodway needs to be identified in which no encroachment or restriction would be permitted. Primary floodways are being identified under the Federal Emergency Management Agency, FEMA, flood mapping program; however, many areas remain to be mapped.
COUNTIES DECLARED DISASTER AREAS BY THE PRESIDENT

Figure 1.
Priority for mapping should continue to be coordinated through county, State and Federal channels.

2. Bridges and utility crossings should be designed and sited to avoid flow restrictions. Piers should be avoided, if possible, and the bridge or crossing should be at sufficient elevation to permit passage of debris. Coordination between flood control and highway/utilities interests is required to insure that overall objectives are met.

3. Storm drains and outlets to streams should be constructed to avoid backwater effects and should be consistent with the capability of the receiving stream.

4. Upstream debris, erosion, and slide areas should be treated on site, if possible, or other provisions made to insure that downstream systems can effectively function under critical conditions.

5. Flood control improvements and storage facilities provide protection; however the protection is often limited - local interests need to be aware of limitations and develop contingency plans.

6. Jurisdictional responsibilities for periodically cleaning streams should be clearly established and provisions made for financing maintenance and emergency measures.

7. Flood forecasting, warning procedures, and operating criteria should periodically be reviewed and updated.

8. Flood control plans should be developed and publicized for intergovernmental coordination purposes and for public information.

9. The flood experiences have shown that human resources can effectively be used to reduce flood effects and damages under many circumstances. The experiences gained can be applied in other areas which were not seriously affected by the recent floods but may be subject to future problems.

10. The flood experience also provided an opportunity to evaluate intergovernmental resources and programs and to determine ways to more effectively utilize available expertise and resources.

**Flood Control**

Flood control involves consideration of all options or alternatives giving consideration to preventive and corrective actions. Effective flood control must be planned and implemented through coordinated efforts of property owners, various local government entities, and as appropriate through State and Federal governments. The extent of intergovernmental involvement depends upon the magnitude of the problem. Primary responsibility is vested with the individual property owner, but it must be recognized that action on an individual basis can impact on adjacent owners and even on
an entire river system. For this reason, and the fact that flooding generally affects a number of private ownerships and public facilities, local government becomes the second order of responsibility. To the extent that the problem is within a county and within the capability of local interests to resolve, without impacting on other counties, the problem should be resolved locally but should be coordinated with appropriate State and Federal agencies. The State assists the counties in intercounty problems and in providing for certain flood control measures that are beyond county capability. The Federal Government similarly assist the State and local governments in providing for flood control consistent with the Federal interest and vested authorities. Primary Federal flood control is available through the Federal Emergency Management Agency (FEMA) and the Corps of Engineers. FEMA programs include flood plain mapping, hazard evaluation and mitigation assistance, and emergency flood restoration measures under the Disaster Relief Act of 1974 (P.L. 93-288). Public assistance, through FEMA, following the 1983 flood amounted to over $46 million. Other Federal assistance was provided through the Small Business Administration, Farmers Home Administration and other agencies having authority to participate in individual and business grants and loans. The Federal Highway Administration was involved in restoring and mitigating damages to Federally supported highways. The Soil Conservation Service was involved in emergency measures to protect irrigation diversion structures and appurtenant irrigation facilities. The Forest Service did much to keep roads and facilities protected and functioning within their areas of jurisdiction. Many other agencies including the National Weather Service, Geological Survey, Environmental Protection Agency, and others participated in providing data and expertise in flood fight and restoration activities. The Corps of Engineers participated in flood fight activities and provided expertise and assistance in responding to technical assistance needs. In addition to the emergency authorities, several Federal agencies have programs to assist local government in mitigating potential flood related problems associated with their jurisdictional areas of responsibility.

The Corps of Engineers, as an example, in addition to emergency authorities has an array of flood control and water resource development authorities that are available to assist local interests in terms of providing technical assistance, conducting investigations, and seeking Federal funding of projects. Flood control assistance through the Corps of Engineers is requested through established lines of assistance, county-State-Federal.

Flood control evaluations by the Corps are made in cooperation with state and local governments and includes provisions for public involvement. The procedure includes a systematic analysis including the following major steps:

1. Problem identification
2. Identification of potential flood control measures
3. Development of alternative plans
4. Evaluation of alternative plans
5. Coordination with local interests

6. Selection of a plan

7. Implementation

The investigation and implementation varies with the scope, complexity, and type of problem. As an example, for many reoccurring problems throughout the country, the Corps has been given discretionary authority by the Congress within the limits of funds provided. These authorities include the following:

Section 14, 1946 Flood Control Act, as amended, authorizes Corps assistance in providing emergency streambank protection at critical locations to protect public facilities. Maximum Federal participation is $250,000 for each location.

Section 208, 1954 Flood Control Act, as amended, authorizes Corps assistance in removing accumulated snags and other debris, and clearing and straightening channels in the interest of flood control. The maximum allowable is $250,000 for any single tributary in any one year.

Section 205, 1948 Flood Control Act as amended authorizes Corps assistance in the construction of small flood control projects. The maximum Federal cost is $4 million.

Investigation and approval of projects under these authorities vary, but normally require about 18 to 36 months for implementation. Local interests are required to provide a portion of the cost and are required to maintain and operate the completed project.

The Flood Control Act of 1938 authorized the Corps of Engineers to make flood control studies on streams in the Great Basin of Utah and Nevada and within the Colorado River drainage above Lees Ferry, Arizona. Preliminary evaluations were made within the study area in 1939 and in the 1940's. Investigations in the Jordan River basin are currently in progress; however, studies in the Bear, Weber, and Sevier River Basins are currently in an inactive status. Under this general authority comprehensive flood control investigations can be made basically for any location in Utah upon request from local interests. Investigations, however, require Congressional funding, authorization of the project, and construction funding. Accordingly, these projects take years to complete, but provide a means for local interests to obtain Federal assistance in making long-term flood control improvements.

CONCLUSIONS

Efforts are continuing at all levels of government to alleviate damages associated with the recent flooding. Restoration and mitigation measures, made possible by Federal, State, and local funding, have done much to restore public facilities, to remove immediate threats and to alleviate future flood
damage potential. However, there are residual problems for which continued support and concerted efforts at all levels of government are required.

State and local governments are proceeding with local mitigation plans and considering the means of financing future projects through established potential funding sources. Sacramento District, Corps of Engineers are engaged in a special Wasatch Front and Central Utah Flood Control Study to evaluate problems and potential solutions and to identify areas of possible Federal involvement. The study is scheduled to be completed about October 1, 1984 and will be made available to local interests following agency clearance.

Effective flood control will require continued effort and support at all levels of government. All problems cannot be resolved by structural measures. Flood control is expected to include implementing practical and affordable protective and corrective management measures to resolve the most serious problems and developing nonstructural floodproofing, flood insurance or other nonstructural measures in areas where protection cannot be economically justified.
Flood Frequency Analysis in Utah

FREQUENCY AND TEMPERATURE ANALYSIS OF THE 1983 WASATCH FRONT FLOODS

Randall P. Julander, Richard H. Hawkins

Abstract

The flooding along Utah's Wasatch Front is considered for 14 stations. An unusual combination of antecedent conditions, weather, and geology provoked particularly destructive late season snowmelt flooding and debris movement. Standard log Pearson III, lognormal, and extreme value type I methods were applied to one day peaks on existing and extrapolated data for a common time base. These methods were also applied to snowpack water equivalent and average fall flow. Based on the analysis, several of the flood peaks were estimated to be of a return period on the order of several thousands of years.

Temperature sequences were analyzed for arbitrarily selected flood years and years that had a high snowpack yet failed to produce flooding. Two temperature stations were used, one at high, the other at low elevation. The watershed temperature was determined by the lapse rate and by weighting each 1000 ft elevational bond by its proportion of the total area. The lapse rate was determined by the difference in temperature of the high and low elevation stations. These watershed temperatures were analyzed for mass balance, difference from normal, and the slope of the temperature sequence as it approaches a flood or nonflood event. Based on these analyses, April temperatures seem to be a key in differentiating flood from nonflood events.

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STUDY AREA

The study area consisted of 6 Wasatch Front streams in the Salt Lake County area, and 7 streams in Davis County, Utah. The Salt Lake County streams are; Little and Big Cottonwood, Hill, Parleys, Emigration, Red Butte and City Creeks. The Davis County streams are; Hill, Stone, Centerville, Parrish, Ricks, Farmington and Holmes Creeks. All of the streams trend east-west across the Wasatch Front. They range in size from 2 to 50 square miles. Little and Big Cottonwood, Hill, Parleys, Emigration, City and Farmington Creeks are comparatively large, with well-developed and deeply incised watersheds. These mature basins have gentle longitudinal slopes, with extremely steep transverse slopes. The remaining 7 watersheds, primarily in Davis County are relatively small. They have much steeper longitudinal slopes, often twice as steep as the previously mentioned watersheds.

Vegetation tends to be similar in composition for a given aspect on all watersheds. Much of the upper portions of the watersheds are forested with aspen, spruce and fir. The soils vary markedly from site to site on a given watershed and between watersheds. They are derived from a complex series of igneous, metamorphic and sedimentary rocks. The soils are in large part residual but are modified by colluvial effects (Olson, 1949).

FREQUENCY ANALYSIS

Three probability density functions were used in the frequency analysis. They were the log Pearson Type III (U.S.G.S., 1982), lognormal (Haan, 1977), and the extreme-value type I (Cumbel, 1958). These distributions were applied to instantaneous peak flow, maximum average daily flow, average fall flow and the maximum snowpack water equivalent. The variables of peak flow, maximum average daily flow and snowpack water equivalent are self-explanatory. Average fall flow is the average flow in cfs for the preceding months of October, November and December. It is a relative index of the storage status of the soil and ground water reservoirs and to some extent, climatic conditions of the period. The contribution from ground water to the flow of a stream is proportional to the slope of the water table $q = k \frac{dh}{dx}$. As the upland ground water reservoir fills, this gradient steepens resulting in greater stream flow. Recharge to the upper levels of shallow mountainous aquifer, approaches zero once a continuous snowpack has been established in the late fall. Past this time, the major portion of stream flow is proportional to the level of ground water. The relative frequency of the average fall flow is an index to the frequency of the level of the ground water table. Saturated surface and subsurface soils along with high ground water levels can be associated with mud and debris flows and are frequently suspected as major contributing factors. This was certainly the case in 1983 along the Wasatch Front. Thus the frequency of average fall flow may be interpreted as the frequency of one of the factors of mud and debris flows. It must be stressed that this is not the frequency of the mud and debris flows, only an estimate of the frequency of one of the conditions conducive to such events.

Flow estimates were obtained from the U.S.G.S., Salt Lake City, the
Flood Frequency Analysis in Utah

Forest Service, and by regression analysis. Flow estimates for the Davis County streams were estimated using regression analysis and flow from Red Butte creek as well as flow estimates provided by the Forest Service.

Temperature Analysis

Temperature was assumed to have played a major role in causing the 1983 floods. The spring of 1983 was unusually cool and wet. Snowpacks were augmented well into May. During the last week of May temperatures rose dramatically, most likely causing the floods.

For this analysis, two sets of data were used, "flood" and "non-flood". A flood year is defined as a year that had flow in the upper 20% of all recorded data on at least 50% of all watersheds. A flood potential year (hereafter designated a "non-flood" year) is one that had a snow pack water equivalent in the upper 20% of recorded data on 70% of the stations. Figure 1 showing Big Cottonwood Creek flow versus Silver Lake Brighton snowpack segments the comparative samples of flood years on the top and non-flood years on the right. The remainder of the data were not floods and had little potential to become such. It is a common perception that the difference between the flood and non-flood events is a function of temperature. Average flood, non-flood and normal temperature sequences are compared by mass balance and the slope of the temperature sequence. Individual watershed temperatures calculated by the lapse rate method are also compared.

Results

The return periods of the 1983 maximum average daily flow and instantaneous peak flow are given in Table 1. In Table 1, the Davis County return periods often have 2 values. Each Davis County watershed has 2 estimates of flow, one by normal stream gaging, the other by linear regression.

Using the most conservative estimate of return period, several extreme events are evident. The maximum average daily flow for City Creek is approximated as a 900 year event. This could be realistic in view of the fact that the 1983 event was 7.9 standard deviations from the Mean. Red Butte Creek was 6.2 standard deviations from the Mean. Big and Little Cottonwood Creeks did not have extreme events. Their return periods are at the 20 to 30 year level. Parleys and Emigration Creeks appear to have had 100 year events. With the exception of Red Butte Creek, there were 65 years of data used for the above creeks.

The Davis County watersheds most conservative estimates of return periods range from 200 to 400 years. These data are far less rigorous than the Salt Lake County data. The Davis County data were obtained from instantaneous flow estimates by the Forest Service and by linear regression. A small error in measurement may produce a tremendous error in return period estimates.

The return periods of the 1982 average fall flow are given in Table 2. Upon examination of these return periods, again using the most conservative
FIGURE 1.

BIG COTTONWOOD CREEK

FLOOD YEARS

LOW FLOOD POTENTIAL YEARS

NON-FLOOD YEARS

SILVER LAKE SNOW WATER EQUIVALENCY IN.
<table>
<thead>
<tr>
<th>Watershed</th>
<th>LPIII</th>
<th>Log N</th>
<th>EVTI</th>
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<tbody>
<tr>
<td>Little Cottonwood</td>
<td>95</td>
<td>95</td>
<td>20</td>
</tr>
<tr>
<td>Big Cottonwood</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mill Creek</td>
<td>100</td>
<td>95</td>
<td>30</td>
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<tr>
<td>Parleys Creek</td>
<td>&gt;10,000</td>
<td>100</td>
<td>200</td>
</tr>
<tr>
<td>Emigration</td>
<td>&gt;10,000</td>
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<td>900</td>
</tr>
<tr>
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<td>&gt;10,000</td>
<td>98</td>
<td>300</td>
</tr>
<tr>
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<td>&gt;10,000</td>
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<td>700</td>
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<td>98-800</td>
<td>90-3000</td>
</tr>
</tbody>
</table>
Table 2. Return period of 1982 Average Fall Flow; years.

<table>
<thead>
<tr>
<th>Watershed</th>
<th>LP III</th>
<th>Log Normal</th>
<th>EVT I</th>
</tr>
</thead>
<tbody>
<tr>
<td>Little Cottonwood</td>
<td>&gt;10,000</td>
<td>&gt;10,000</td>
<td>&gt;10,000</td>
</tr>
<tr>
<td>Big Cottonwood</td>
<td>&gt;10,000</td>
<td>&gt;10,000</td>
<td>&gt;10,000</td>
</tr>
<tr>
<td>Mill Creek</td>
<td>100</td>
<td>&gt;10,000</td>
<td>10,000</td>
</tr>
<tr>
<td>Parleys Creek</td>
<td>&gt;10,000</td>
<td>900</td>
<td>800</td>
</tr>
<tr>
<td>Emigration</td>
<td>90</td>
<td>90</td>
<td>10</td>
</tr>
<tr>
<td>City Creek</td>
<td>100</td>
<td>98</td>
<td>40</td>
</tr>
<tr>
<td>Red Butte Creek</td>
<td>100</td>
<td>98</td>
<td>45</td>
</tr>
<tr>
<td>Centerville Creek</td>
<td>200</td>
<td>200</td>
<td>70</td>
</tr>
<tr>
<td>Farmington Creek</td>
<td>98</td>
<td>100</td>
<td>60</td>
</tr>
<tr>
<td>Holmes Creek</td>
<td>&gt;10,000</td>
<td>&gt;10,000</td>
<td>2000</td>
</tr>
<tr>
<td>Parrish Creek</td>
<td>200</td>
<td>1000</td>
<td>350</td>
</tr>
<tr>
<td>Ricks Creek</td>
<td>10,000</td>
<td>4000</td>
<td>700</td>
</tr>
<tr>
<td>Stone Creek</td>
<td>600</td>
<td>800</td>
<td>300</td>
</tr>
<tr>
<td>Mill Creek M.P.</td>
<td>98</td>
<td>200</td>
<td>70</td>
</tr>
</tbody>
</table>

Table 3. Return period of 1983 maximum snow water equivalent, years.

<table>
<thead>
<tr>
<th>Watershed</th>
<th>LP III</th>
<th>Log Normal</th>
<th>EVT I</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silver Lake Brighton</td>
<td>2000</td>
<td>95</td>
<td>25</td>
</tr>
<tr>
<td>Mill D</td>
<td>87</td>
<td>89</td>
<td>8</td>
</tr>
<tr>
<td>Parleys Canyon Summit</td>
<td>65</td>
<td>70</td>
<td>4</td>
</tr>
<tr>
<td>Mill Creek</td>
<td>80</td>
<td>80</td>
<td>5</td>
</tr>
<tr>
<td>Lambs Canyon II</td>
<td>70</td>
<td>80</td>
<td>5</td>
</tr>
<tr>
<td>Park City Summit</td>
<td>65</td>
<td>70</td>
<td>2</td>
</tr>
<tr>
<td>Farmington Lower</td>
<td>93</td>
<td>94</td>
<td>15</td>
</tr>
<tr>
<td>Farmington Upper</td>
<td>100</td>
<td>100</td>
<td>35</td>
</tr>
</tbody>
</table>
distribution in each case, several extreme events are apparent. Both Big and Little Cottonwood Creeks experienced rare events that were 6.7 and 8.3 standard deviations from their means. Several of the Davis County watersheds might also have had rare events. Holmes and Ricks Creeks had predicted 1982 events that were 3.4 and 4.6 standard deviations from their means.

The return periods of the 1983 maximum snow pack water equivalent are given in Table 3. The maximum snowpack water equivalent return periods show that the 1983 snowpack was not a rare event. Using the most conservative estimate, which in this case are all EVT values, the return periods range from 2 to 35 years. The log Pearson III and the log normal values are essentially equal ranging from 45 to 100 years.

The average flood, non-flood and normal temperatures for the Salt Lake City Airport station are shown in Figure 2. They represent average daily values from March 1 to June 30 and were smoothed using a 5 point moving average. Each daily point of the flood curve represents the average of 12 flood years, whereas each daily point of the non-flood curve is the average of 6 years. The normal line was derived from the 30 year normal temperatures given by Brough et al. (1983). Between days 90 and 120 which is the month of April, the average temperature of non-flood years is hotter than normal.

The average flood, non-flood and normal temperatures for Silver Lake Brighton are shown in Figure 3. Again the flood line daily values represent the average of 12 years, and the non-flood values represent 6 years. The non-flood curve again shows that April is a key month in separating flood from non-flood events.

A mass balance was performed for these temperature stations. This was done by subtracting the daily normal temperature value from the actual daily value and then summing over discrete time increments for each flood and non-flood year. The only portion of the non-flood curve that was significantly different than the flood curve at the alpha .05 level was the month of April. Under the condition of above normal snowpack, hot Aprils tend to produce non-flooding conditions in May and June.

Most flooding along the Wasatch Front occurs in late May and early June, thus an evaluation of the temperature sequences immediately preceding flood and non-flood events is warranted. Average watershed temperatures were used in the following analyses. Mass balances were performed for the 5 weeks preceding each flood and non-flood event. Only 2 creeks, Parleys and Centerville, had significant differences between flood and non-flood temperature sequences at the alpha .05 level. They both had non-flood temperature sequences during the total 5 week period preceding an event that were warmer than the flood events. Figures 4 and 5 show the average flood, non-flood and normal temperature sequences preceding an event for Parleys and City Creeks. Because hot temperatures are associated with rapid snowmelt and flooding, the results of these 6 week comparisons seem intuitively reversed. Therefore, this 5 week period was broken into 1 week segments and again tested for significance. In only 2 cases, Mill Creek and City Creek were the flood temperatures significantly hotter than the non-flood temperatures. These both
FLOOD FREQUENCY ANALYSIS IN UTAH

Diagram: Silver Lake Temperature vs. Average Temperature (°F)
Figure 4:

Parley's Temperature

Legend:

- - - FLOOD
- - - NON-FLOOD
- - - FLOOD NORMAL
- - - NON-FLOOD NORMAL

Average Temperature (°F)

Days Preceding Flood

Days Preceding Flood

35 30 25 20 15 10 5 0

35 30 25 20 15 10 5 0
occurred in the week immediately preceding the flood event. Farnells and Centerville Creeks again showed that non-flood events were warmer than flood events early in the 5 week segments. They showed no difference in the week preceding an event. No other watersheds showed significant differences.

One of the assumed contributing factors of the 1983 floods was the fact that temperatures rose dramatically. This can be seen in figures 6 and 7. Figure 6 shows the slope of the 1983 temperature line along with the slope of the average flood year temperature line for Big Cottonwood Creek. A normal temperature line was omitted because it virtually coincided with the average flood year temperature line. Figure 7 shows the same curves for City Creek.

A comparison of the temperature line slopes between flood and non-flood years for all watersheds yielded only one, City Creek, where flood years got warmer significantly faster than non-flood years.

**DISCUSSION AND CONCLUSIONS**

Utilizing standard statistical techniques of frequency analysis, many of the 1983 flood return periods were extrapolated far beyond the current data base. We are quite aware of the limitations of these mathematical techniques and that these return periods may in reality be in gross error. Still, they serve to show that the 1983 Wasatch Front Floods were indeed extreme events. It is significant to note that one of the factors associated with the floods, snowpack water equivalent, was not a rare occurrence based on the return periods derived from the same techniques. Another factor associated with snowmelt flooding as well as mud and debris flows is the moisture status of the soil or groundwater. The groundwater status was indexed in this study by average fall flow. The average fall flow preceding the 1983 floods was also shown to have been a rare event.

Temperature is a factor associated with a snowmelt flooding. The single most critical time period in differentiating flood year from non-flood year temperatures appears to be the month of April and the week immediately preceding an event. Warm or hot Aprils tend to produce non-flood events given an above normal snowpack. Cool or normal Aprils are associated with flood events. Conceptually, a warm April will melt a major portion of the snowpack at lower elevations as well as that pack most proximal to the stream.

During a cool or normal April, this snowmelt occurs simultaneously with melt at the modal watershed elevation. It is interesting to note that the difference between the range of flood and non-flood years over a 2 week period of critical flow represents an average 3 percent of the snow pack. This suggests that the flood source areas are very proximal to the stream channel and are most likely the modal watershed elevation. In general, the temperatures of flood years do not become hotter faster than non-flood years based on the analysis of temperature sequences of the 5 weeks preceding an event.
FIGURE 6.
BIG COTTONWOOD CREEK
TEMP. SLOPE 1983, AND NORMAL FLOOD

FIGURE 7.
CITY CREEK
TEMP. SLOPE 1983, AND NORMAL FLOOD
ACKNOWLEDGEMENTS

This study was funded by a grant from the United States Forest Service. Portions of this paper have been used in a MS thesis at Utah State University by R.P. Julander.

LITERATURE CITED


PROBLEMS WITH STATISTICAL FLOOD-FREQUENCY ANALYSES OF STREAMS IN UTAH

by Blakemore E. Thomas

ABSTRACT

The commonly used statistical flood-frequency analyses of gaging-station records do not adequately describe the flood characteristics of many streams in Utah. Thunderstorms cause most of the large-magnitude floods in Utah; and at a particular site, the frequency of an intense thunderstorm usually is extremely small. Thus, the samples of thunderstorm floods included in gaging-station records may not be representative of the flood populations, because most of the records in Utah are fairly short. Statistical flood-frequency analyses are inadequate when this sampling problem occurs. Another problem is that the annual peak flows for many streams can result from snowmelt or rainfall, and a statistical flood-frequency relation determined for an array of annual peaks from a mixed population usually does not adequately fit the larger peak flows.

Analysis of the morphology and stratigraphy of stream channels and the establishment of a network of long-term partial-record gaging stations are two independent means for increasing the sample size of the flood population. A method that has been successfully applied to the mixed-population problem is: determine the rainfall and snowmelt peaks for each year, do a separate flood-frequency analysis for each array of peaks, and then combine relations in a composite or joint flood-frequency relation. The flood-frequency analysis of ungaged streams can be improved by obtaining estimates of the areal extent of intense thunderstorms, which in turn can be used in the development and calibration of flood-information transfer methods and rainfall-runoff models.

INTRODUCTION

Flood-frequency relations are needed for the design of bridges, culverts, dams, embankments, and for use in flood-plain studies. In Utah, floods can result from rainfall, snowmelt, rain on snow, or dam breaks. Conventional flood-frequency analyses generally are not adequate for estimating the magnitude of infrequent large floods that are caused by rainfall, and additional flood-frequency studies are needed.

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For a site with a streamflow-gaging station, the records of the station usually are analyzed by fitting a theoretical probability distribution to the records. At ungaged sites, flood-frequency relations usually are estimated using flood-information-transfer methods or rainfall-runoff models. The flood-frequency analysis of gaging-station records is the primary subject of this paper, but pertinent limitations of methods used at ungaged sites will be mentioned.

The U.S. Water Resources Council (1981, p. 3) recommends using the method of moments with the log-Pearson Type-III probability distribution to define flood-frequency relations for gaging-station records. The Pearson Type-III distribution is defined by three moments: mean, standard deviation, and coefficient of skewness. The moments are computed for the sample data (gaging-station records), and they are assumed to be equivalent to the moments of the theoretical distribution and the population.

When a statistical method is used for estimating flood-frequency relations, it is important to recognize the assumptions inherent in the method. The assumptions used when the method of moments is used to analyze annual peak flows are that the sample (record of the gaging station) is representative of the population, and the annual peak flows are independent, homogeneous, and occur randomly.

The population of annual peak flows is described and used for predictions. The population is defined as the whole class of possible occurrences of annual peak flows in the past, present, and future. To describe and make inferences about this population, a sample is used that is an observed part of this population.

The annual peak flows generally are independent and random; however, homogeneity may be a problem and it needs to be examined carefully. The factors that affect the annual peak flows must remain relatively constant in order to provide a homogeneous sample or population. Thus, watershed conditions must be constant during the period of record for the sample and for the period of time for which flood frequency is to be estimated. Cover conditions of the watershed such as vegetation, soil, and extent of urban areas should remain fairly constant. The streamflow regime should not change significantly as a result of activities of man such as urbanization, channelization, and the construction of reservoirs, diversions, and levees. Variation in the cause of floods can cause a lack of homogeneity in a flood record, and the importance of this factor is discussed later in this report.

If all the assumptions are met, the gaging-station records (sample) can be used to predict the future magnitude and frequency of floods. The relations of magnitude and frequency for past floods are assumed to be true for future floods.
PROBLEMS IN FLOOD-FREQUENCY ANALYSIS

The method of moments, using the log-Pearson Type III probability distribution, was used by Thomas and Lindskov (1983) to define flood-frequency relations for streams at more than 300 gaging stations in Utah and adjoining States. They found that most of the problems concerning the flood-frequency analysis of streams in Utah are related to floods caused by thunderstorms. Woolley (1946) and Butler and Marsell (1972) studied the location and frequency of floods caused by thunderstorms; and they noted that they occur anywhere in Utah, including the densely populated areas along the Wasatch Front in north-central Utah. Thus, the potential for thunderstorm floods near population centers is quite large, and emphasis needs to be placed on obtaining the best possible estimates of flood characteristics in those areas. The major problems in flood-frequency analysis are discussed in the following three subsections.

Short Annual Peak-Flow Records

The first problem in flood-frequency analysis relates to the small sample size of most of the gaging-station records in Utah. At a particular site, the frequency of an intense thunderstorm usually is extremely small; thus, the sample of floods included in a gaging-station record may not include a large thunderstorm flood, although such a flood probably will occur at that station sometime in 100 years. The station record, therefore, may not be a representative sample of the flood population. This violates one of the basic assumptions of a statistical flood-frequency analysis of gaging-station records. In 1981, of 241 stations on unregulated streams in Utah, 187 stations had records of between 10 and 25 years, 39 stations had records of between 26 and 50 years, and 15 stations had records of more than 50 years.

The samples of annual peak flows probably are representative for stations in the eastern and southern parts of the State below altitudes of about 8,000 feet because thunderstorm floods are fairly frequent in these areas. Many thunderstorm floods in and near the mountains in northern Utah and in the desert basins of the Great Basin in western Utah have been documented in newspapers and in reports by Woolley (1946) and Butler and Marsell (1972), but these floods have not been measured.

Gaging-Station Records with a Mixed Population

A second problem in flood-frequency analysis concerns streams with a mixed population of floods. A mixed population is defined as an aggregation of floods that are caused by two or more distinct and generally independent hydrometeorologic conditions, such as snowmelt and rainfall. Snowmelt floods usually are in late March through June, whereas the destructive rainfall floods that result from intense thunderstorms usually are in the summer or early autumn. Records with some annual peak flows caused by snowmelt and some by rainfall represent a mixed population of floods, and a flood-frequency analysis using the method of moments on such records usually does not adequately fit the larger peak flows.
The U.S. Geological Survey recently has completed investigations in Colorado and Idaho on the flood-frequency analysis of streams with a mixed population of floods (Jarrett and Costa, 1982, and Kjelstrom and Moffatt, 1981). In most of Colorado, the transition zone where streams had a mixed population was found between altitudes of 6,500 and 7,500 feet. In the Arkansas River Basin (southeastern Colorado), the transition zone was found between 8,000 and 9,000 feet (McCain and Jarrett, 1976, p. 31). In Idaho, the transition zone where streams had a mixed population of floods ranged between 3,000 and 6,000 feet. Both studies concluded that a conventional flood-frequency analysis of records in the transition zone did not adequately fit the larger peak flows.

In Utah, streams with mixed populations of floods are in zones of altitude that vary throughout the State. Woolley (1946, p. 50) stated that most of the thunderstorm floods in Utah originate in areas below 8,000 feet. Thomas and Lindskov (1983) found that the upper altitudinal limit for floods caused by intense thunderstorms increased from north to south in the State. A preliminary estimate of this upper altitude limit is 7,500 feet north of latitude 40°, 9,000 feet between latitudes 39° and 40°, and 9,000 feet south of latitude 39°. Above these limits for altitude, flood-frequency relations mostly are controlled by snowmelt, except for recurrence intervals greater than 100 years, when thunderstorms may contribute to the flood potential. The mixed population of floods is found from the upper limits for altitude to lower limits of about 6,000 feet in the north and 7,000 feet in the south part of the State. Below the lower limits for altitude, thunderstorms dominate the flood characteristics.

**Flood-Frequency Relations at Ungaged Sites**

Similar problems to those for gaging-station records exist with methods for estimating flood-frequency relations at ungaged sites. Two methods commonly are used to determine flood-frequency relations at ungaged sites in Utah. One is an information-transfer method wherein flood-frequency relations determined at gaged sites are transferred to ungaged sites using multiple regression or an index-flow relation (Riggs, 1973). The second method involves using a rainfall intensity of a specific probability and determining the runoff characteristics by a deterministic model of rainfall-runoff relations (Crawford and Linley, 1966, and Leaveney and others, 1983) or an empirical relation such as the rational method (Chow, 1964, p. 14-6 - 14-8). It is beyond the scope of this paper to discuss the advantages and limitations of these methods and the only subject that will be addressed here is how these methods relate to the estimation of frequency of thunderstorm floods.
It has been shown that some uncertainty exists in the analysis of records for gaged streams on which there are thunderstorm floods. This uncertainty and errors that may result from the analysis are transferred to the estimates of flood-frequency relations at ungaged sites when using the transfer methods of multiple regression or index-flow relations. However, some of the time-sampling error due to short gaging-station records will be decreased, because each estimate from the transfer method will average the samples of floods of many gaging stations over a longer time than most single samples. Thus, the short-record problem probably is helped to some degree by using the transfer methods. The problem with mixed populations, however, would be transferred with the transfer methods.

A limitation in estimating flood-frequency relations for thunderstorm floods for large basins, which is common to both the transfer methods and rainfall-runoff models, is that these methods use contributing drainage area in their equations. A flood resulting from an intense thunderstorm usually comes from only part of the drainage area upstream from the site of interest. Thus, only the contributing drainage area needs to be considered for a thunderstorm flood, and this area may be much smaller than the total area of the basin. Both methods for ungaged sites can be adapted, with varying degrees of difficulty, to this problem.

SOLUTIONS TO PROBLEMS IN FLOOD-FREQUENCY ANALYSIS

Solutions to the Problem of Short Records for Annual Peak Flow

One solution to the problem of short records in thunderstorm areas is to establish a network of continuous-record or partial-record gaging stations for which annual peak flows can be determined. These records will provide a valuable sample of the population of floods, however, the sample always will be relatively small for estimating rare, large-magnitude floods. Another method that has been used successfully during the past 20 years entails the combination of the frequency analysis of gaging-station records with data obtained from studies of the morphology and stratigraphy of stream channels. Reasonable estimates of the magnitude and frequency of rare floods can be made by using methods developed for the analysis of Holocene stratigraphy. Some of these methods are: botanical studies of flood-plain vegetation (Sigafoos, 1964), analysis of soil development on flood plains (Cain and Beaty, 1968), study of flood bars and overbank deposits and dating of organic materials entrapped in the deposits (Costa, 1974), dating of landforms truncated by floodflows (Costa, 1978), and analysis of sediments accumulated in slack-water sites (Rochel and Baker, 1982). These stratigraphic methods may facilitate the frequency analysis of large-magnitude floods with recurrence intervals of hundreds or thousands of years.

A method based on channel morphology can be useful for estimating flood-frequency relations at ungaged sites or at gaged sites where the gaging-station sample is not considered to be representative of the population. Fields (1975) and Hedman and Oesterkamp (1982) developed empirical equations for estimating the magnitude and frequency of floods using the measurement of the width of the stream channel.
An estimate of the maximum potential flood at a gaged or ungaged site can be obtained using envelope curves provided by Crippen and Bue (1977, p. 14-15). The envelope curves are drawn over the maximum peak discharges observed through 1974 at sites in Utah and surrounding States.

A Solution to the Mixed-Population Problem

A solution to the poor fit of a flood-frequency analysis for a gaging-station record with a mixed population is: determine the rainfall and snowmelt peaks for each year, do a separate flood-frequency analysis for each array of peaks, and then combine relations using the following formula for the union of independent annual peak flows:

\[
P(\text{Composite}) = P(\text{Snowmelt}) + P(\text{Rainfall}) - [P(\text{Snowmelt}) \times P(\text{Rainfall})]
\]

where \( P \) is the probability that the annual peak flow from the indicated population will exceed a stated magnitude (Crippen, 1978).

This approach was used successfully in the studies in Colorado and Idaho (Jarrett and Costa, 1982, p. 566, and Kjelstrom and Moffatt, 1981, p. 16-17). The composite relation fits the larger peaks better than an analysis done on a mixed-population sample. This approach also is theoretically more correct than using the mixed-population sample, because the flood-frequency analysis using a probability distribution is based on the assumption that all the flood peaks are from the same population. The moments of a rainfall-peak distribution usually are quite different from the moments of a snowmelt-peak distribution.

Flood-frequency relations were computed using the method of moments for several streams in Utah based on: the array of annual maximum-peak flows (mixed population), the array of annual rainfall-peak flows for the same period of record, the array of annual snowmelt-peak flows for the same period of record, and the composite relation. The coefficient of skew for the station record was used for all the flood-frequency relations. From a comparison of the four different relations, streams can be placed in four groups based on the altitude and drainage area of the site. This classification can be used to determine if the composite flood-frequency analysis might be needed. The four groups are:

<table>
<thead>
<tr>
<th>Group</th>
<th>Altitude</th>
<th>Drainage area</th>
<th>Flood population</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>High</td>
<td>All</td>
<td>Mostly snowmelt</td>
</tr>
<tr>
<td>2</td>
<td>Middle</td>
<td>Small</td>
<td>Mixed</td>
</tr>
<tr>
<td>3</td>
<td>Middle</td>
<td>Large</td>
<td>Mixed</td>
</tr>
<tr>
<td>4</td>
<td>Low</td>
<td>All</td>
<td>Mostly rainfall</td>
</tr>
</tbody>
</table>

These four groups were selected on the basis of preliminary studies, and they need to be studied further to obtain a more precise estimate of the criteria for altitude and drainage area. Estimates for the altitude limits for these groups were given earlier in this report, and a preliminary estimate for a small versus a large drainage area is about 100 square miles.
At altitudes above the mixed-population zone, snowmelt floods control most of the flood-frequency relation. Thunderstorms occur in these areas, however, large thunderstorm floods are rare; and at altitudes above 10,000 feet recurrence intervals for intense thunderstorms that cause significant flooding may be on the order of 500 years. Flood-frequency relations for a gaged site on the South Fork of Rock Creek near Hanna, Utah, a stream which probably is typical of group 1 are shown in figure 1. This site has 24 years of record, a drainage area of 15.7 square miles, and an altitude of 7,860 feet. The magnitude of the snowmelt peaks is greater than the rainfall peaks until a recurrence interval of between 100 and 200 years (exceedance probability of 0.01 and 0.005), where the curves cross. Based on these preliminary studies and work by Randall Peterson (U.S. Bureau of Reclamation, written commun., 1984), the intersection point of the rainfall- and snowmelt-frequency relation for high-altitude streams probably varies from near the 100-year flood to rare recurrence intervals on the order of 1,000 years. The relation computed using the array of annual maximum peaks (mixed population) is adequate until this intersection point is reached, and the rainfall-peak relation or composite relation needs to be used for recurrence intervals greater than the intersection point.

Flood-frequency relations for a gaged site on Vernon Creek near Vernon, Utah, which has 19 years of record, a drainage area of 25 square miles, and an altitude of 6,200 feet are shown in figure 2. This site fits in group 2, and the flood-frequency relations are fairly typical for sites with a small drainage area in the mixed-population zone. The composite relation is a more accurate depiction of the flood characteristics, and it needs to be used for all recurrence intervals. Rainfall peaks predominate for the infrequent floods, and these peaks need to be used to define the upper end of the frequency curve.

Flood-frequency relations for a gaged site on Cottonwood Creek near Orangeville, Utah, which has 40 years of record, a drainage area of 208 square miles, and an altitude of 6,050 feet are shown in figure 3. This site (group 3) has a mixed population of floods, however, the rainfall- and snowmelt-frequency curves do not cross, and the relation based on the annual maximum peaks is adequate for the commonly used range of recurrence interval of 10 to 500 years. Comparison of figures 2 and 3 shows that drainage area needs to be considered as well as altitude. The composite relation may not be needed for any site with a drainage area that exceeds 100 square miles, regardless of altitude. This is a preliminary conclusion, and the relation of drainage area and altitude to the rainfall- and snowmelt-frequency relations needs further study. Also, predictions of the rare recurrence intervals of greater than 200 years may need the composite relation at any site that has a mixed population. A reason for this relation of drainage area to flood-frequency relations is that thunderstorms usually occur in a small area. In small basins, therefore, the probability of an intense thunderstorm occurring over the basin in a given year is small, whereas, in a large basin, some thunderstorms (and floods) occur most every year, either near the measurement site or far away.
Figure 1.—Flood frequency curves for the South Fork of Rock Creek near Hanna, Utah
Figure 2—Flood frequency curves for Vernon Creek near Vernon, Utah
Figure 3.—Flood frequency curves for Cottonwood Creek near Orangeville, Utah
Flood-frequency relations for a site on a typical stream in the low-altitude zone (group 4), where floods caused by rainfall dominate the flood characteristics, are shown in figure 4. This site is on Mill Creek near Moab, Utah, and it has 28 years of record, a drainage area of 74.9 square miles, and an altitude of 4,240 feet. The rainfall relation is always greater than the snowmelt relation, and the composite method is not needed for this type of stream.

Improvements to Flood-Frequency Analyses at Ungaged Sites

The flood-information-transfer methods and rainfall-runoff models can be used to determine flood-frequency relations for streams with the potential for thunderstorm floods. The development and calibration of the equations used in these methods would need to focus on the contributing drainage area of thunderstorm floods used in the frequency analyses. Geomorphic, sedimentologic, and botanical methods have been successfully applied to identify contributing drainage areas after intense rainstorms (Jarrett and Costa, 1982). In addition to flood-frequency estimates obtained from a transfer method or rainfall-runoff model, the stratigraphic methods described earlier can be used to obtain an independent estimate of the magnitude and frequency of rare floods at ungaged sites.

The flood-information-transfer method, which uses multiple regression, probably is more economical and practical than rainfall-runoff modeling for a regional study. New flood-frequency relations at gaged sites, which are improved using the composite analysis and stratigraphic methods, would improve the accuracy of the dependent variable in the regression equation. Independent variables that might be significant in a multiple-regression analysis are the commonly used variables that can be measured from maps. Some of these variables are drainage area, mean basin altitude, altitude of the site, and channel slope. Other onsite characteristics that may be significant are channel morphology, vegetation on channel sides and sideslopes near the channel, and deposits in and near the stream channel.

NEEDED STUDIES AND INFORMATION

The problem of estimating rare, large magnitude floods with a small sample can be partly solved in several ways. Keeping continuous-record gaging stations operating as long as possible and creating a network of long-term partial-record stations would increase the sample size that can be used in a statistical flood-frequency analysis. Studies of the morphology and stratigraphy of stream channels are needed for independent estimates of flood-frequency relations. Gaging stations and stratigraphic analyses are needed in the Colorado River Basin above 7,500 feet, and in the Great Basin at all altitudes, particularly near the densely populated areas of the Wasatch Front.
Figure 4.—Flood frequency curves for Mill Creek near Moab, Utah.
FLOOD FREQUENCY ANALYSIS IN UTAH

The zone of altitude for streams with a mixed population of floods needs to be defined. This is important because the separation of peaks for the composite method is a time-consuming process; and if rainfall or snowmelt peaks predominate, then it is not necessary to do the separate analysis. The zone of altitude can be estimated with existing data using flood-frequency relations—means of samples, precipitation data, altitudes of gaging stations, and the season of annual peak flows from existing gaging-station records. Studies of the morphology and stratigraphy of stream channels can help to determine the relation of altitude to intense thunderstorms. This would help to define the zone of altitude for streams with a mixed population of floods.

The composite flood-frequency analysis needs to be done on existing gaging-station records in the mixed-population zone. To do this analysis, regional skew coefficients are needed for peak flows resulting from snowmelt and rainfall. The regional skew map for Utah published by the U.S. Water Resources Council (1981, p. 12) is primarily for snowmelt floods; therefore, a regional skew map is needed for floods caused by rainfall. Concurrently with the composite analysis, the effect of the size of drainage area on the snowmelt- and rainfall-frequency curves needs to be studied. An upper limit to drainage area may exist where the composite analysis is not needed.

Additional data would improve the flood-frequency analysis of streams with a mixed population of floods. The existing gaging-station network is sparse in the areas of mixed-population floods, and more stations are needed. Precipitation data are sparse in these areas and recording gages would be useful. Also, studies of stratigraphy can improve flood-frequency relations for streams with a mixed population by improving the estimates of frequency for rare, large-magnitude floods.

The improvement of flood-frequency relations at gaged sites will improve the analysis at ungaged sites. To transfer the flood characteristics from gaged to ungaged sites, the multiple-regression method probably is the most practical for a regional study. Needed data for this method are estimates of contributing drainage area for large thunderstorm floods used in the frequency analysis and measurements of the morphology and stratigraphy of stream channels.

SUMMARY

The commonly used statistical flood-frequency analyses of gaging-station records do not adequately describe the flood characteristics of many streams in Utah. Thunderstorms cause most of the large-magnitude floods in Utah, and at a particular site, the frequency of an intense thunderstorm usually is extremely small. This creates a problem because most of the gaging-station records in Utah are fairly short, and many of the records probably are not representative of the flood populations. The analysis of Holocene stratigraphy is an independent method for estimating flood-frequency relations. When combined with a statistical analysis, these methods can decrease some of the uncertainty in estimating the recurrence interval of rare, large-magnitude floods.
The flood-frequency analysis of gaging-station records with a mixed population of floods can be improved by determining the rainfall and snowmelt peaks for each year of the record, doing a frequency analysis on each array of peaks, and then combining relations in a composite or joint relation using a formula for the union of independent annual peak flows. Streams in Utah at altitudes between 6,000 and 9,000 feet probably have a mixed population of floods.

The frequency analysis of thunderstorm floods for ungaged streams can be improved by obtaining estimates of the areal extent of intense thunderstorms, which in turn can be used in the development and calibration of flood-information-transfer methods and rainfall-runoff models. Contributing drainage areas after intense rainstorms can be identified using geomorphic, sedimentologic, and botanical techniques. Multiple-regression analysis, using improved flood-frequency relations at gaged sites along with contributing drainage area and onsite stream-channel characteristics, is a promising method for developing more accurate regional equations for estimating flood-frequency relations.

Some additional data and studies are needed to improve the flood-frequency analysis of streams in Utah. More streamflow and precipitation data are needed in the areas of mixed-population floods. A network of long-term partial-record gaging stations is needed throughout the State to increase the sample size of floods. Studies are needed to determine the relation of altitude and drainage area to intense thunderstorm floods and streams with a mixed population of floods. Geomorphic and stratigraphic studies of stream channels may be useful for independent estimates of frequency relations for large-magnitude floods.
REFERENCES CITED


Costa, J. E., 1974, Stratigraphic, morphologic, and pedologic evidence of large floods in humid environments: Geology, v. 2, no. 6, p. 301-303.


ABSTRACT: Most flood mitigating efforts center around choosing a return period level of protection, determining the flood magnitude associated with this return period, and constructing or zoning to accommodate this flood flow. Of these three steps, the most nebulous is the determination of the flood magnitude of specific frequency floods, particularly for rare events. There are diversities of opinion in determining these extreme frequency flood flows chiefly due to data sparsity and interpretation. From a regionalized analysis of flood runoff, greater insight can be gained regarding the flood potential of a drainage area. The difficulties of predicting extreme frequency flood magnitudes is discussed and a procedure is proposed for a regional relationship for estimating snowmelt and thunderstorm flood magnitudes for specific drainages.

INTRODUCTION

At one time or another, nearly every flood hydrologist has agonized over the calculation of a peak flow for a specific return period. Am I too conservative, thus adding to the cost of an already expensive water resources project? Are my calculated flood peaks too low, thus endangering the public safety? These questions stem from a common problem in the western states, little or no hydrologic data for a specific watershed.

The hydrologist is often asked by private firms and governmental agencies to estimate these specific frequency flood peaks to allow the construction of water regulating projects. These projects may include dams, canals, detention basins, flumes, and storm sewers. An implicit level of protection is determined for each project; for example, a canal in an uninhabited area may be designed for cross-drainage inflows of a 25-year return period while a detention basin that lies above a highly populated area may be designed for inflows of a 100-year return period. The additional cost associated with the longer return period design is usually deemed acceptable in order to protect the expensive property lying just below the project.

The problem arises in determining specific frequency peak flows. If annual peak data for that particular watershed are available, a statistical analysis of the annual peaks will produce the desired number. However, due to

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the limited number of years of data for most Utah stream gages, the calculated flow is of questionable accuracy, especially for rarer frequency events. Unfortunately, most drainages have no gage record at all and the confidence band of our estimate becomes even larger.

Purpose

A solution common to both of these problems, uncertain calculations and a lack of data, requires the use of all the available hydrologic data. This approach was taken when Thomas and Lindskov (1983) used discharge records from 254 Utah stream gages to estimate specific frequency flood peaks for both gaged and ungaged watersheds. Their equations give frequency flood peaks as a function of drainage area, mean basin elevation, and hydrologic subarea but do not differentiate between snowmelt and rainflood peaks.

From an examination of historical flood records, it is apparent that for some drainages in northern Utah, there are two distinct types of flood events, snowmelt and rainfall. By segregating these event types in the historical records, greater understanding can be gained of the nature of flood events in general and the estimate of specific frequency events in particular. This paper presents the results of such a segregation of the stream gages in northern Utah.

This study focused on: (1) the analysis of the historical streamflow records, (2) the observation of general trends for drainages with bivariate frequency distributions, (3) the application of historical data to results of precipitation-runoff modeling, and (4) the need for further data analysis. By using flood-type segregation in a mixed population gage record, a more accurate estimate of extreme flood events can be made.

ANALYSIS OF HISTORICAL STREAMFLOW RECORDS

The annual streamflow records of 93 gages in northern Utah were analyzed to determine the characteristics of mixed population watersheds. Appendix I lists these stream gages. These gages were predominately unregulated by diversion or storage and had a period of record of 10 years or longer. An effort was made to obtain a sample from a hydrologically homogeneous area, thus, gage records were omitted from the southern portions of the state and from low lying desert regions where the snowmelt contribution to the annual hydrograph was minimal.

Using a direct data access link with the United States Geological Survey (USGS) computer in Reston, Virginia, the annual primary and secondary runoff peaks were tabulated. In many cases, a base discharge flow was established by the USGS in the identification of annual secondary peaks. Unless secondary snowmelt or rainflood peaks were above the base discharge, they were not tabulated.

In those cases where the missing secondary peaks were snowmelt, the annual maximum mean daily discharge was multiplied by the average historical gage ratio of the maximum annual instantaneous peak to the corresponding mean
daily peak. This produced a complete set of annual snowmelt peaks for each gage. If the missing secondary peaks were of the rainflood type, they were not able to be quantified and an incomplete set of rainflood peaks resulted. Time constraints prevented an analysis of the original gage chart records to obtain a complete frequency curve of the rainflood peaks.

Each complete set of snowmelt peaks were statistically evaluated using the methods described in the U.S. Water Resources Council Bulletin #17B, Guidelines For Determining Flood Flow Frequency (1982). A log-Pearson Type III distribution was fit to a Weibull plot of the annual peaks and the curves were adjusted for low outliers. The generalized skew coefficients normally used in this type of analysis were replaced by the calculated station skew since the generalized coefficients were determined from a mixed population analysis.

On each of these snowmelt frequency plots, the annual rainflood peaks were also plotted using the same Weibull plotting position used for the snowmelt peaks. For example, if the gage record had only one rainflood peak above the base discharge, that peak was plotted using the plotting position of the highest snowmelt peak. If the gage record had 20 rainflood peaks above the base discharge, these were plotted using the highest 20 plotting positions of the snowmelt record. Since this method accounts for only part of the rainflood frequency curve, it was not possible to fit a log-Pearson curve to the plot of the rainflood peaks. An approximate curve was fit to these data points by hand, recognizing the limitations of such an approach. Figure 1 is an example of a frequency plot showing both the snowmelt and rainflood curves.

![Figure 1. Frequency distribution curves for Tie Fork near Soldier Summit, Utah.](attachment:image.png)
Many of the gage records had few or no rainflood data points making a determination of the rainflood frequency curve impossible. Twenty-five of the gages, however, exhibited both snowmelt and rainflood curves that were well defined and important conclusions can be drawn from these instances.

In the remainder of this paper, reference is made to "common" and "rare" frequency events. For the purpose of comparing frequency distributions, a "common" frequency is defined as having a return period less than 10 years while a "rare" frequency is defined as having a return period longer than 100 years.

OBSERVATION OF GENERAL TRENDS

As mentioned previously, the periods of record for many of the gages in Utah are unfortunately short and the time sampling error for such a record may be quite large. Other hydrologic factors combine to cause varying frequency distributions for drainages that appear to be identical and make exact correlations difficult. These problems are best overcome by using as many gage records as possible and by noting broad similarities between the frequency distributions. These similarities can be grouped in several main topics.

Snowmelt Frequency Distributions

The log-Pearson Type III curves of the snowmelt peaks were very similar for the majority of the gage records. A "typical" snowmelt frequency distribution would asymptotically approach a limiting value for the rare frequency events. The standard deviation of the snowmelt distributions had an average value of 0.24, and for 90% of the gages, varied between 0.10 and 0.35. On a regional basis, there appears to be specific topographic reasons for variations in the snowmelt standard deviation. For example, those distributions of high elevation drainages and the drainages in Cache Valley and southern Idaho generally had standard deviations less than the majority, while drainages on the southern end of the Wasatch Range and on the southeastern end of the Uinta Mountains had standard deviations much larger than the majority. Factors such as groundwater aquifers, basin aspect, and varying or constant annual snowfall depths may play a large role in determining the snowmelt standard deviation.

Although the limited periods of gage record constrained the accuracy of the station skew coefficients, there appears to be a significant tendency toward a negative skew coefficient for most drainages. The calculated log-Pearson skew coefficients ranged from -1.60 to +1.13 and had an average value of -0.39. The majority of these station skewes had a lesser value than the generalized skew map of Bulletin #17B, reflecting the absence of the rare frequency rainflood events included in that mixed population analysis.

Rainflood Frequency Distributions

Each of the 25 rainflood frequency distributions of the gages listed in Appendix 2 had standard deviations greater than the gage's snowmelt
distribution standard deviation. The rainflood standard deviations ranged from 0.32 to 1.2 and had an average value of 0.61. Figure 2 plots a histogram of both the snowmelt and rainflood standard deviations for all the gages and clearly shows the greater variance of the rainflood peaks.

Figure 2. Histogram of standard deviations for 93 Northern Utah stream gages.

Only two of these rainflood distributions showed discernable skewing of the frequency curve. Some curvature similar to the snowmelt curves would be expected at the extreme frequencies but with the limited available data, the rainflood curves appear to be steeply-sloped straight lines.

Intersection Points of Bivariate Distributions

Most people recognize from common experience the repetitive nature of the annual snowmelt peaks and the intermittent nature of severe rainflood events. These impressions are supported by the historical record and directly affect the estimate of extreme frequency flood peaks. These two frequency curves have different slopes and must therefore intersect. The return period at which this intersection occurs is a function of the following drainage basin morphology:

1- Basins with a high mean elevation or a large drainage area typically have a rare frequency intersection point. Examples of this category are the Uinta River near Neola, Whiterocks River near Whiterocks, Ashley Creek near Vernal, White River near Soldier Summit, and the Wolf Creek near Hanna gages. These gages have been identified by a sufficient number of rainflood peaks to
support this rare frequency intersection point. In addition to these gages, many of the 93 gages examined had high mean basin elevations and had no recorded rainflood peaks, also indicating a rare frequency intersection point of the two curves.

2- Basins with a low mean elevation typically have a common frequency intersection point. Most often the characteristic of a small drainage area is also associated with this category. Examples are the Tie Fork near Soldier Summit, Salt Creek at Nephi, Dairy Fork near Thistle, and many of the lower elevation gages in the Uinta Basin.

The result of these gage analyses is the verification of bivariate frequency distributions for unregulated drainages. It is believed that those gages without recorded rain peaks have intersection points at extremely rare frequencies and lack extreme rainflood data due to a time sampling error or the limited gage record. A complete analysis of the rainflood frequency curve should provide information to substantiate this belief.

APPLYING THE RESULTS OF FLOOD TYPE SEGREGATION

Segregating historical flood types benefits hydrologic analysis in two important areas: (1) determining flood peaks estimates for common frequency events, and (2) predicting the shape of the distribution curve at extreme frequencies for use in risk analysis evaluations.

Flood Peak Estimates for Common Frequencies

If a common frequency flood peak is required for a gaged watershed, it is a straightforward procedure to perform a log Pearson analysis and pick a value from the frequency curve. However, ungaged watersheds cause much more confusion and difficulty. Usually, an empirically derived equation such as the Rational Method or the Unit Hydrograph Method is used with specific frequency rainfall values to calculate the desired flood peak. Although these methods are widely accepted, this approach often produces serious errors.

A theoretical example will illustrate this problem. The snowmelt runoff of 1983 caused great property damage along the Wasatch Range in Davis County. If the cities involved in this flooding desired to protect their residents from the threat of future flooding, they might construct some type of flood control facility such as detention basins, dams, or flumes. They would first decide on a return period level of protection. A commonly used return period for this situation is 50 or 100 years. Using the Stone Creek drainage, the unit hydrograph approach, the 50-year rainfall values from the NOAA Precipitation-Frequency Atlas for Utah, and typical infiltration rates, a calculated peak of 850 cfs seems reasonable. However, the historical gage records tell quite a different story. Six stream gages recorded streamflow data within a 10 mile radius of Stone Creek and had a total of 134 gage-years of record, an average period of record of 22 years. Each of the drainages are quite similar to that of Stone Creek with regard to basin morphology. None of the gages experienced a flow of even one-half of the calculated peak. In
fact, none of the gages experienced a severe rainflood. If we were to rely on frequency curves of the historical data, the 50-year peak for Stone Creek would be about 125 cfs and it would be a snowmelt peak.

The recommended approach for the situation of an ungaged watershed would be to locate gaged drainages nearby with similar characteristics. Using the historical record, determine the magnitude of the flood peak for the specific frequency that is desired. Transfer this information to the desired drainage, taking into account factors such as drainage area, mean basin elevation, and channel topography. If a snowmelt event controls the peak, transfer the data on a cfs/square mile basis. If a rainflood event controls the peak, calibrate the unitgraph method on the gaged drainage and transfer the calibrated variables to the ungaged drainage. Use the relationships explained in the previous section, OBSERVATION OF GENERAL TRENDS, as a guide in determining the controlling event type. In many cases, an uncalibrated empirical or theoretical model bears no resemblance to reality.

At the common frequencies, segregating a bivariate distribution would not significantly improve the accuracy of the plotting of the mixed population distribution. It would, however, provide information about the type of event and the time of occurrence during the year, important information when considering the life and death nature of flood control.

Shape of the Distribution Curve at Extreme Frequencies

The use of frequency distribution curves is especially important in analyzing the risk costs associated with any flood control project. Major projects are often designed for the probable maximum flood (PMF), usually a flood of immense proportions. To analyze the risk of designing for some lesser degree of protection, the shape of the frequency distribution must be assumed. Most engineers would hesitate to estimate the 1,000-year or 5,000-year flood, yet to determine the frequency at which the least risk cost occurs, these frequencies must be given a value. One approach is to connect the 100-year flood peak value and the PMF value with a straight line; however, this approach is based on supposition rather than historical data and suffers accordingly. It is here that flood type segregation provides valuable assistance.

If the drainage is small or at a low elevation, there is a high probability that the intersection point of the two distributions occurs at a common frequency. The combined frequency curve would then head at a steep slope from the intersection point to the rarer frequencies. Figure 1 is an example of this type of distribution. The rare frequency events would have quite a large peak compared to the magnitude of the common frequency events.

If the drainage has a high mean elevation or a large drainage area, the intersection point of the two distributions would probably occur at a rare frequency. For very large drainages, the snowmelt distribution might entirely control the combined distribution, even at very rare frequencies. In these cases, the rare events of 100-, 500-, and 1,000-years return period would
probably be an extension of the snowmelt curve and would not be influenced by the steeper slope of the rainflood curve.

The calculated peaks of these rare frequency events would, of course, have a large confidence interval due to the limited amount of streamgage data. However, understanding the probable shape of the frequency curve allows more intelligent assumptions about rare frequency events and therefore, a better assessment of the risk costs involved in any project.

FURTHER DATA ANALYSIS

As mentioned previously, the complete rainflood distribution of many of the study gages could not be determined. An analysis of the original gage chart records would provide the required information and would allow a better analysis of the rainflood distribution standard deviation and skew. The author intends to make these calculations and publish the findings in the near future.

SUMMARY

A segregation of historical streamgage records into snowmelt and rainflood events was made to discover possible relationships between basin morphology and flood type. A substantial number of basins in northern Utah were found to have both distributions and similarities were noted among the drainages with regard to the standard deviation and the intersection point of the two distributions. When applied to specific drainages, these relationships can provide valuable information regarding specific flood peak estimates for common frequency events and the shape of the combined distribution for rare frequency events.


### APPENDIX 1

**LIST OF STREAM GAGES**

<table>
<thead>
<tr>
<th>Station</th>
<th>Gage Name</th>
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<tr>
<td>10148200</td>
<td>Tie Fork near Soldier Summit</td>
</tr>
<tr>
<td>10147000</td>
<td>Summit Creek near Santaquin</td>
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<td>Salt Creek at Nephi</td>
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<td>Spanish Fork near Thistle</td>
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<td>Hobble Creek near Springville</td>
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<td>Weber River near Oakley</td>
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<td>09287000</td>
<td>Currant Creek below Red Ledge Hollow</td>
</tr>
<tr>
<td>09268500</td>
<td>North Fork Dry Fork near Dry Fork</td>
</tr>
<tr>
<td>09268900</td>
<td>Brownie Canyon above Sinks near Dry Fork</td>
</tr>
<tr>
<td>09266500</td>
<td>Ashley Creek near Vernal</td>
</tr>
<tr>
<td>09262000</td>
<td>Big Brush Creek near Vernal</td>
</tr>
<tr>
<td>09280400</td>
<td>Hobble Creek at DanielIs Summit</td>
</tr>
<tr>
<td>10137500</td>
<td>South Fk Ogden River or Huntsville</td>
</tr>
</tbody>
</table>
## APPENDIX 2
### GAGES WITH RAINFLOOD FREQUENCY CURVES

<table>
<thead>
<tr>
<th>Station</th>
<th>Gage Name</th>
<th>Rainflood Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>10148200</td>
<td>Tie Fk nr Soldier Summit</td>
<td>1.13</td>
</tr>
<tr>
<td>10146000</td>
<td>Salt Ck at Nephi</td>
<td>0.48</td>
</tr>
<tr>
<td>10148500</td>
<td>Spanish Fk nr Thistle</td>
<td>0.33</td>
</tr>
<tr>
<td>10113500</td>
<td>Blacksmith Fk ab UP&amp;L Dam nr Hyrum</td>
<td>0.90</td>
</tr>
<tr>
<td>10131000</td>
<td>Chalk Ck nr Coalville</td>
<td>0.50</td>
</tr>
<tr>
<td>10139300</td>
<td>Wheeler Ck nr Huntsville</td>
<td>1.10</td>
</tr>
<tr>
<td>10148300</td>
<td>Dairy Fk nr Thistle</td>
<td>0.63</td>
</tr>
<tr>
<td>10146900</td>
<td>Utah Lake trib. nr Elberta</td>
<td>1.00</td>
</tr>
<tr>
<td>10137680</td>
<td>N.F. Ogden R. nr Eden</td>
<td>0.39</td>
</tr>
<tr>
<td>10104700</td>
<td>Little Bear R. nr Avon</td>
<td>--</td>
</tr>
<tr>
<td>09288000</td>
<td>Currant Ck nr Fruitland</td>
<td>0.46</td>
</tr>
<tr>
<td>09297000</td>
<td>Uinta R. nr Neola</td>
<td>0.37</td>
</tr>
<tr>
<td>09288150</td>
<td>W.F. Avintaquin Ck</td>
<td>0.80</td>
</tr>
<tr>
<td>09288100</td>
<td>Red Ck below Currant Ck</td>
<td>0.32</td>
</tr>
<tr>
<td>09287500</td>
<td>Water Hollow nr Fruitland</td>
<td>0.59</td>
</tr>
<tr>
<td>09288900</td>
<td>Sowers Ck nr Duchesne</td>
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</tr>
<tr>
<td>09326500</td>
<td>Ferron Ck nr Ferron</td>
<td>0.40</td>
</tr>
<tr>
<td>09299500</td>
<td>Whiterocks R. nr Whiterocks</td>
<td>0.53</td>
</tr>
<tr>
<td>09276000</td>
<td>Wolf Ck nr Hanna</td>
<td>0.36</td>
</tr>
<tr>
<td>09307500</td>
<td>Willow Ck ab div. nr Ouray</td>
<td>0.51</td>
</tr>
<tr>
<td>09308500</td>
<td>Minnie Maud Ck nr Myton</td>
<td>0.87</td>
</tr>
<tr>
<td>09312500</td>
<td>White R. nr Soldier Summit</td>
<td>0.35</td>
</tr>
<tr>
<td>09312800</td>
<td>Willow Ck nr Castle Gate</td>
<td>0.41</td>
</tr>
<tr>
<td>09262000</td>
<td>Big Brush Ck nr Vernal</td>
<td>0.37</td>
</tr>
<tr>
<td>09266500</td>
<td>Ashley Ck nr Vernal</td>
<td>--</td>
</tr>
</tbody>
</table>
Special Problems in Flood Hazard Delineation

FLASH FLOOD DAMAGES ON
THE SANTA CRUZ RIVER

Dr. H.S. Oey¹, Othon Medina²,
and Samuel Carreon³

INTRODUCTION

The upper Santa Cruz River Basin near Tucson drains, 2,222 square miles of basin area in Southern Arizona and Northern Sonora, Mexico. The river flows north through Tucson and discharges into the Gila River. The Santa Cruz Basin is characterized by isolated mountains separated by broad alluvial-filled valleys. The altitude of the valleys ranges from 2,100 to 4,700 feet above sea level, and the higher mountains reach beyond 9,000 feet. Streams in the upper Santa Cruz River Basin are mostly ephemeral with groundwater sustaining low flows in only a few locations. The basin and the streambed are very permeable. Although the amount of stream flow in the basin is normally very small, the area is subject to flash floods generated by thunderstorms of high intensity precipitation that covers some areas in the basin. These are characterized by sharp peaked hydrographs. The ten-year and the hundred-year peak flows at the Tucson gage are 11,000 cfs and 25,000 cfs respectively. A 25-year flood occurred in December 1967 and a 100-year flood was observed in October 1977 followed by another one in the fall of 1983. The 1977 flood altered the river alignment so much and exposed an underground pipe-line crossing. The damage done by the 1977 flood rendered the underground pipeline crossing undesirable and a pipeline bridge was built to replace it.

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2. Pipeline System Engineer, El Paso Natural Gas Company
3. Senior Engineer, El Paso Natural Gas Company
BEHAVIOR OF THE RIVER

Historical Record

Topographic maps available include USGS quads developed from 1966, photos, cross-sections at the pipeline crossing for 1947 and 1967, and spot elevations at bridges and the crossing for 1947, 1955, and 1966. The areal photographs used to develop the topographic maps for January 1976 and October 1977 were available. A single areal photograph of part of the study reach taken in 1964 was also available. A USGS gaging station downstream from the study reach (at Tucson) has been operational since 1905. Peak flows over 1,700 cfs were studied. Eight scour gages consisting of a column of red granular 3/4 inch material were installed in February 1976. Following the October 1977 flood, the gages were excavated to determine the depth of maximum scour at these locations.

River Meandering

Natural channels occur in three general forms: straight, braided, and meandering. Important factors influencing the form that a channel might assume are bed and bank material and properties, slope, flow, and sediment load. Slope is considered one of the most critical factors. Meandering channels are usually found in valleys of moderate slope. In the case of the Santa Cruz River, the slope alone does not account for the observed channel forms. The slope of the river is about 0.004 in a 10-mile reach including the study reach.

The sinuosity over this reach is 1.15 which is considered a straight reach. The sinuosity of the study reach is 1.6 which is greater than 1.5, the value used as the definition of a meandering channel. The meanders in the study reach apparently originated from local disturbances. This could have been caused by the presence of local gravel removal operations and a hill upstream of this reach. Once meanders are formed, equilibrium requirements will resist any change to a straight channel. Areal photographs taken in 1964 show the existence of some meanders of considerably longer length and narrower width than those present today.

Meandering Pattern of the River in the Study Reach

Prior to 1964 bends A and B curve around Martinez hill (Figure 1).
Figure 1. The Study Reach showing the bends, I-19 Highway bridge, and the pipeline crossing. Banks eroded by the October 1983 flood are shown as the cross-hatched area.
In 1964, after the completion of the I-19 and Mission Road Bridges, the flow was constricted under the bridges. This caused some downstream migration of banks. Attempts were made to protect the freeway by stabilizing bend D with wire fences, jacks, and a rock crib fence. Bend E was lightly protected.

From 1964 to 1976 gravel extraction at the gravel pit downstream resulted in the lowering of the bed at a rapid rate. Flow around bent C was controlled by the bridge embankments and bank erosion occurred at the downstream edge of the bend. This resulted in the eventual bypassing of the stabilizing structures at bend D. Severe scour occurred just downstream of bend E on the west bank. Strengthening the protection at bend E was deemed necessary. Concrete walls along both banks upstream of the crossing (between E and the crossing) were erected. These measures resulted in a sharp deflection of flow at bend E and a constriction of flow at the crossing which in turn resulted in severe bed downcutting. Bends F and G eroded rapidly in response.

The major flood of October 1977 created a chute cut off between bends A and B. The banks at C were quite stable, but erosion was greatly increased at the downstream end of the bend and the flow cut into the bank at bend D. The flow then attacked the stabilizers at bend E and caused a portion of it to fail. Thus the flow got behind the stabilizing structure (Figure 3) and caused rapid erosion of the banks at bend E. The above events resulted in a downstream migration of the bends. As a result, the pipeline at the crossing was exposed some 150 feet on the west bank side. (Figures 3 and 4).

The above sequence of events show how partial protection of banks could alter the meandering behaviour of a river. Because some banks were left to erode, the angle of attack of flow into downstream protected banks increased until the protections failed. In 1964, the width of the meanders was approximately 1000 feet. In 1977, the meanders width ranged from 1250 feet to 1800 feet. If we use the 25,000 cfs peak flow of the 1977 flood and a coefficient of $C_v = 14$, the meander with is $w_m = 14 \times 25,000 = 2214$ feet. Based on the same Q of 25,000 cfs, the regime width should be $B = 2.67 \times 25000 = 422$ feet and $W_m = 6B = 2532$ feet. Meander length is $L_m = 2 W_m = 5000$ feet. The meander length in our study reach is less than 3000 feet.

**PIPELINE CROSSING**

After the October 1977 flood that exposed some 150 feet of the buried pipe on the west bank, studies were made to determine the most practical and economical means of crossing the Santa Cruz River in that vicinity. It was decided that a pipeline bridge should be built to replace the underground crossing. The original alignment was kept taking advantage of the right of way already secured, little additional pipeline, and minimum service interruption. Allowing the river to establish a natural meander pattern was considered the most advantageous. This avoids the high cost of extensive bank protection. There remains to be an important question to be answered. How far back into the west bank is the natural meander going to extend? Study of meander behavior shows that there seems to be a point beyond which meanders will not grow.
Figure 2. The exposed pipes after the October 1977 flood eroded the west bank. The bottom left corner of the picture shows part of the foundation structure of an electric transmission-line tower.

Figure 3. Protective structure located at the old river bank has been rendered nonfunctional by the flood. The new band is now way behind this structure.
Figure 4. The old exposed (formerly buried) pipeline is shown with the temporary support structure. The new pipeline bridge was to replace the old buried crossing.

Figure 5. Another look at the eroded west bank and the undermined foundation of the tower.
Figure 6. Protective structure under the I-19 bridge.
in width. When this point is reached, the flow cuts the meanders off and no longer attacks the outside of the bend.

Bridge piers were constructed using the "slurry assisted drilled shaft" method. They reached to a depth of 75 feet below the river bed which was necessary to provide lateral strength to withstand the horizontal force produced by the stream flow during floods. The possible scour depth of 17 feet observed from the 1977 flood was also taken into consideration in determining this depth.

It was decided to allow the river to follow its natural meandering pattern which will allow the west bank to move farther west until regime condition is acquired. Based on the maximum meanders width estimated in our study, bridge piers were built on dry land beyond the west bank over a distance of some 700 feet. These buried piers will be exposed one by one as the bank moves west.

As predicted, a small flood in early 1979 cut the west bank some 70 feet. Subsequent floods moved the bank farther west until the land owner adjacent to the west bank immediately upstream from the crossing built rock rip-rap bank protection. This altered the natural meandering pattern. Flow was diverted and directed toward the east bank of bend E.

Prior to the construction of this rock riprap the bend was actively migrating westwardly and northwardly. Piers 9 and 10 were exposed on the bridge. When our bridge was constructed in 1978, piers 9 through 15 were buried due to anticipation of bend E migrating westwardly. This occurred during the winter 1978-79 flood. Prevention of future migration at bend E by the rock riprap and considering the truncation of the bend, the acceleration of erosion on the east bank shear was inevitable due to the difference in bank shear stresses on both sides of the river at the crossing. In October 1983, a flood exceeding the record flood of 26,000 cfs occurred. The U.S. Geological Survey estimated the peak flow at approximately 45,000 cfs. At the onset of the flood, active river bend erosion occurred on all unprotected banks, especially on the outside of the bends. Since migration was prevented at bend E, the river had no recourse but to erode towards the east bank of the crossing. Had bend E not been protected for the intervening years prior to the October 1983 flood, its apex would have been in a more westwardly direction such that the transition between bends D and E would have been smoother. This would have lessened the erosion on the inside of bend E.
Figure 7. Detail of the pipeline bridge plan built in July 1978. The west bank had eroded to pier 10 by January 1979 after a small flood, before the construction of the rock riprap.
Figure 8. An aerial view of the pipeline bridge after the October 1983 flood showing the eroded east bank (left end of the bridge). Upper left is east and lower right is west.
Figure 9. The exposed pipe on the east bank after the 1983 flood.
Figure 10. The collapsed span of the I-19 bridge between bends B and C after the 1983 flood.
PERIODIC SURGES IN EPHEMERAL FLOODS

by Richard J. Heggen

ABSTRACT

Periodic surges, also known as roll, slug, or intermittent waves, have been noted in ephemeral discharge. Such surges may arise from a rapid increase in discharge, or may develop in response to fluid instabilities in steep channels at a steady mean discharge. Periodic surges initiate as small undulations; with growth they become traveling hydraulic jumps.

Periodic surges in flash floods may cause brief but significant spikes in the discharge hydrograph. Associated with the surges are impulses which may substantially exceed the forces associated with steady mean discharge. Surge impulses may affect debris flow, channel stability, and the stability of hydraulic structures.

The analytic fundamentals of surge hydraulics are identified. Using a case study, criteria for surge formation are identified. Surge form and translation are evaluated. The hydraulic consequences of surges in an ephemeral channel are discussed.

Richard J. Heggen is with Department of Civil Engineering, The University of Mexico.
SECTION IV - HAZARD MITIGATION MEASURES
HAZARD MITIGATION THROUGH COMPREHENSIVE EMERGENCY MANAGEMENT

by Wesley G. Dewsnup

Hazard mitigation is as complex as the physical systems that produce the hazards. It is a marriage of the political system and the technical community to achieve activities, programs and policies that will reduce the potential loss of life and property damage. It must be accomplished at all levels of government, with the greatest responsibility at the local level where regulatory authority rests. State and federal governments are less effective in the implementation of hazard mitigation activities than local government and should be technical and financial resources for the local government.

In undertaking mitigation, there are two approaches, that of removing or mitigating the hazard or removing or mitigating the risks. Each approach must be considered and the most appropriate selected. More often than not a combination of the two approaches is most appropriate. One cannot remove the earthquake threat by physically moving the source of the earthquake, however, one can reduce the risks faced by the population by implementing some mitigation techniques such as anchoring heavy non-structural items in the house, or identifying and practicing personal earthquake response activities.

The term "comprehensive" implies a holistic approach in which the functional relationships between hazards, risks and their mitigation are considered. In Farmington, Utah, following the debris flow in Rudd Canyon the city fathers had a decision to make in dealing with predictions of future flows. In this emergency environment the decision was made to construct a debris basin costing in excess of one million dollars. They did not have the luxury of reviewing a variety of alternatives or considering the potential of other hazards when the decision was made. Their decision may not have changed, however, it would have had a much better foundation if they had been able to take a comprehensive look at their problems. Salt Lake City had to deal with the floodplain on the lower Jordan River and the pressure to develop the area. In a non-emergency environment they were able to evaluate several options outlined by a consulting firm and put together a private/local/state/federal cooperative project for diking the lower Jordan River that has been a very successful solution to a natural hazard problem.

Wesley G. Dewsnup is Utah Multi Hazard Project Manager, Utah Division of Comprehensive Emergency Management.
Emergency management in the past was limited to post-disaster response and recovery efforts. This type of effort, while providing a valuable service, did very little to prevent future problems of the same or greater magnitude. Under the current emergency management scheme, there is support for post-disaster mitigation. Following a Presidentially Declared Disaster the state is required under Section 406 of Public Law 93-288 to prepare a hazard mitigation plan that will significantly reduce the state's susceptibility to that hazard in the future. If the plan is not prepared and/or implemented, the level of post-disaster relief from the federal government may be significantly reduced.

Comprehensive emergency management is the current program in the State of Utah. Both the preparation of plans and the response and recovery to disasters are treated in a comprehensive manner. The program is based on the availability of information supplied by investigation and research. With that information the Division is able to estimate the kinds of hazards and risks that the various areas of the State are susceptible to and provide preparedness plans and a scheme of operations to effectively deal with the response to and recovery from the variety of hazards.

The State of Utah has undertaken a federally funded pilot project to deal with pre-disaster hazard mitigation to reduce the potential for loss of life and property damage. The impetus behind the project is the desire to protect lives and property as well as reduce the amount of money being spent on response and recovery. The project is founded on the principle that an ounce of prevention is worth a pound of cure. The project deals with four identified hazards: earthquakes; landslides; floods; and dam failures. It has defined a process that outlines for local government the steps necessary to achieve hazard mitigation. The steps of the process are outlined in Figure I. The first step is the establishment of a body of local officials called the Administrative Review Committee. Their function is to define the goals and objectives of the community and make sure that hazard mitigation can work within that set of goals and objectives. The Committee helps establish the database upon which the project is built. It includes the identification of the critical facilities in the community, and identification of all the study materials that help define the nature and extent of the hazards in the area. Next maps of the information are prepared and used in the education and awareness of the other public officials and the general public. Provision must be made to keep the database current as the information being prepared by the research and scientific community. Probability and consequence scenarios are developed and appropriate hazard mitigation alternatives that are politically, socially and economically feasible are selected and prepared for implementation. The Committee then prepares an implementation strategy that includes funding options and presents the plan to the local legislative body for adoption. It is important that the implementation of the plan is "canonized" by legislative action to take it out of the political realm and assure long term responsibility for hazard mitigation.

Of primary importance in hazard mitigation is an understanding of where and how big the hazard is and when it is likely to occur. Given this
information the local government is able to act. The information is being generated by the scientific community, however, it is often presented in a manner that is confusing and sometimes contradictory in the eyes of the local decision maker. Translated information both for and by the user is critical to hazard mitigation.

Given the availability of information, the local government officials must be able to "market" the need for mitigation of the identified hazards. This involves a strong education and public awareness effort. The public officials must be made aware of the hazards and educated in hazard mitigation benefits and techniques. The professional community must go through the same process, and finally the general public must be made aware and educated, thus building a constituency for hazard mitigation.

Finally there needs to be an incentive for local governments to undertake hazard mitigation. The first incentive is the legal liability that a local government might be assuming should they choose to ignore the potential for the hazard or fail to act as a reasonable person would when made aware of
the potential for hazard. The courts are starting to hold local governments liable for damages to private property incurred as a result of a disaster situation that the community had the capability to foresee. Another incentive is the opportunity for local government to maintain control of hazard mitigation at their level. All too often programs are presented to the local government in a predigested state and the minimum standards become the maximum design criteria. The local levels of control must be maintained for hazard mitigation programs to be successful. Third is the financial incentive. Currently the federal government is covering 75 percent of the costs of the impact of a major disaster on the public facilities. However, the funding available for pre-disaster mitigation programs is virtually non-existent. The signal this is sending to state and local governments is that it is cheaper to wait for a disaster to occur and use the federal funds to clean up than it is to spend their own scarce budget money on hazard mitigation programs that may not come to fruition during their term of office. The federal government needs to rethink its posture for pre-disaster hazard mitigation funding.
Nonstructural Measures

SOME THOUGHTS ON DECISION-ANALYSIS
IN EVALUATING AND MITIGATING NATURAL HAZARDS
by Brent D. Taylor, Ph.D., P.E.¹

Abstract: Many federal, state, and local governmental agencies, along with private companies, and individuals continually make decisions involving human safety, economic liability and environmental concerns, in the face of uncertainties as to future natural events such as landslides and floods. In these decisions, with economic limitations on each of the different levels of government and the large cost of many mitigation measures, it is important that costs and benefits for alternative courses of action be evaluated as realistically as possible. There are three basic approaches for decisionmaking on these kinds of problems. One of these approaches includes a risk-based evaluation. Where this approach can be used it offers significant advantages over the two more traditional approaches. The three basic decision-making approaches are identified and discussed with examples illustrating their interrelationships and applications.

INTRODUCTION

Federal, state and local governmental agencies and private companies continually make decisions involving human safety, economic liability, and environmental concerns in the face of uncertainties regarding the probability of occurrence and magnitude of future natural events such as tornadoes, floods, drought, landslides, earthquakes, dam failure, etc.

In these decisions the decision maker(s) generally seeks to identify the course of action that will provide the desired level of human safety and environmental safeguards at minimum economic cost. Alternative courses of action for mitigation: 1) reduce the probability of the natural event (e.g. landslide stabilization); 2) mitigate the event or probable damages (e.g. deflection walls for landslides, insurance); and/or 3) reduce hazards to property and human life by removing them from potential damage areas (e.g. landslide hazard or flood plain zoning, emergency warning systems).

There are three basic approaches available for decision-analysis on natural hazards, and these approaches can lead to quite different results. A complete

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risk-based analysis yields the best informational basis for decision making. But under some conditions such an analysis is not possible, and a less accurate approach must be used.

The purpose of this paper is to clarify the three basic approaches to decision-analysis, identifying their individual advantages, disadvantages, and applicability under different conditions. Also, some comments are offered on problematic biases that can affect decision making.

Because of the author's experience with the Bureau of Reclamation, examples are drawn primarily from the field of dam safety. However, the concepts and conclusions also apply to decision-analysis on other natural hazards.

**BASIC DECISION TYPES**

There are three basic approaches to decision making with regard to mitigation of natural hazards. These three approaches can be categorized as follows:

<table>
<thead>
<tr>
<th>Decision Approach</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>With this approach it is implicitly concluded that the hazardous event won't occur during the period of interest (i.e., project life), or if the event does occur, from a practical standpoint, nothing can be done to significantly mitigate it. This might be called the &quot;won't happen&quot; or &quot;can't do anything about it&quot; approach. This approach leads automatically to a no-action decision.</td>
</tr>
<tr>
<td>2</td>
<td>With this approach it is, in effect, concluded that the hazardous event will occur sometime during the period of interest or that potential damages are &quot;unacceptable,&quot; and action is taken to prevent the event or to reduce damages to an acceptable level and prevent loss of life. This might be called the &quot;will happen&quot; approach and is characterized by a maximum-action decision. When a decision is made by a government agency to always design for the probable maximum flood, maximum credible earthquake, or other fixed hazard level, the decision approach is of this type.</td>
</tr>
</tbody>
</table>

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2 Also, an index-based hazard assessment, like the federal government's Safety Evaluation of Existing Dams (SEED) program, is a form of decision approaches 1 and 2.
With the third approach, there is an attempt to quantify hazardous event probabilities and risk costs as a technical basis for decision making. The decision then is based on a comparison of risk costs in view of pertinent project values, e.g., public safety, economic constraints, etc. This might be called the "may happen" approach and can lead to a no-action, maximum-action, or more likely, an intermediate-level-action decision.

It should be noted that a priori objections to the use of a risk-based approach due to imprecise definitions of probabilities are ill-founded. Contrary to the reasoning often given for the inadequacy or inappropriateness of risk analysis methods, uncertainty is a primary reason for the use of this approach, rather than an argument against it. As De Finetti (1962) states:

"The choice of a particular action among a sufficiently wide set of permitted possibilities is equivalent to an evaluation of the probabilities concerned."

Thus, with approaches 1 or 2 imprecise estimates of probabilities play as large a part as they do with approach 3. But, with the risk-based approach uncertainties and probabilities are dealt with explicitly. Whereas with approaches 1 and 2 the imprecisions in the implicit either/or (0 or 1) probability estimates are generally hidden, but not inconsequential. For example, consider a dam with a 100-year project life. If this dam has an annual probability of overtopping failure of $10^{-4}$ per year, this would give a probability of dam failure during the project life of about $10^{-2}$. If, as a matter of agency policy, the decision is made to upgrade the dam to contain the PMF, this decision is equivalent to assuming that the hazardous event will happen, thus yielding an effective imprecision of 0.99 in the implicitly assumed event probability, with the type 2 approach.

The consequence of this approach-2 imprecision is that if in fact the hazardous event does not take place during the project life there were economic costs without any economic benefits. Where the cost of modification to improve safety is fairly large and the probability of the hazardous event is small, the economic cost with "false positive" conditions (i.e., event does not occur during the project life) can be large, and, there is a high probability that a false-positive condition will obtain. To illustrate this, assume an agency has 100 dams that will not accommodate the PMF without overtopping, and that it would cost $1 million per dam for modification to provide for containment of the PMF. With a $10^{-4}$ annual probability of hydrologic dam failure, it would be expected

While risk-based analysis can be used to objectively evaluate hazard mitigation with different alternative courses of action, it may not lead to a single solution. If there are multiple objectives such as "minimum economic cost" and "minimum threat to human life," results from a risk-based analysis may suggest that one alternative minimizes economic costs, while a different alternative minimizes the threat to human life. In such cases the decision maker must make the necessary tradeoff evaluation based on project values, to identify the preferred alternative.
that 1 of the 100 dams would fail hydrologically during a 100-year project life. Then if the economic damages with failure were less than $100 million for each dam the probable false-positive cost for safety modification of the 100 dams would be larger than the economic expected benefits. In cases such as this when approach-2 is adopted, the saving grace with probable large false-positive costs, as identified by Petak (1983), is that even though this approach does not provide the best decision economically, it is a "safe" decision because if the hazardous event occurs during the project life the decision maker is praised, and if the event does not occur, usually no one notices or complains, especially if the project life is several decades!

With a risk-based approach, potential false positive (and false negative) costs are implicitly considered in the calculation of risk costs and expected benefits.

INFORMATIONAL BASIS FOR DECISION-MAKING

In different natural hazard situations the degree of precision possible in a quantitative evaluation will vary. This variation based on state-of-the-art analytical capabilities, and available data and information can range from a weak qualitative evaluation to a complete quantitative risk-based assessment. With some natural hazards the magnitude-frequency relations may be well defined by historical data, at least for intermediate level events such as 10-year to 100-year floods, and their corresponding economic and human safety liabilities can be accurately estimated. Whereas for other hazards, e.g. tornadoes in Salt Lake City, magnitude-frequency relations may be non-existent and potential event damages are sketchy, at best. From a practical standpoint, three general levels of informational conditions might be identified as follows:

<table>
<thead>
<tr>
<th>Informational Conditions</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Level A:</strong></td>
<td>With existing data, information, and state-of-the-art analyses a complete risk-based evaluation of alternative courses of action is possible.</td>
</tr>
<tr>
<td><strong>Level B:</strong></td>
<td>While there is sufficient data for a partial quantitative comparison of alternative courses of action, there is not a sufficient informational base to adequately quantify all risk costs.</td>
</tr>
<tr>
<td><strong>Level C:</strong></td>
<td>There is insufficient data for any useful quantitative comparison of alternatives. Basically only qualitative-type information is available.</td>
</tr>
</tbody>
</table>

If the hazardous event magnitudes, return periods, damages, etc., can be estimated such that useful comparisons of risk costs are possible for all viable alternative courses of action, Level A informational conditions would obtain.

However, uncertainties in key parameters may be too large to permit comparisons of all pertinent risk costs, and thus a complete risk-based evaluation may not
be possible. In such cases some useful differential evaluation of alternative courses of action might still be possible. For example, flood insurance may be available such that property in a potential inundation area can be insured against flood damages resulting from possible dam failure for X dollars per year, and the developed areas may be located such that for minimal cost a system could be installed which would provide adequate public warning in the event of hazardous flooding. Then if the minimum cost for modification to the dam to reduce it’s possibility of failure would be Y dollars per year, and X<<Y, there could be a useful economic comparison of these two alternatives even though perhaps other factors might not be estimated precisely enough for a full comparison of viable alternatives. This would be an example of Level B conditions.

Level C conditions would obtain in this example if no useful quantitative comparisons were possible, and essentially, only qualitative information were available for decision making. Under these conditions, “weak” qualitative evidence of a hazard should lead to little or no precautionary action (approach-1), and “strong” evidence of a hazard should lead to a “strong” precautionary action (approach-2).  

Figure 1 is a schematic diagram which illustrates how the three types of informational conditions just discussed rationally lead to the three decision making approaches identified earlier. When Level A conditions obtain, the best approach to decision making is a value-oriented tradeoff (approach-3) based on pertinent risk-costs (economic, public safety, etc.). With Level B conditions key elements in the problem may be quantified with sufficient precision to enable an approach-3 decision. But if there are over-riding uncertainties in spite of whatever risk-cost comparisons are possible, the decision must revert to a 1 or 2 approach. The choice of approach 1 or 2 will depend on the specific information available, and project values, i.e. acceptable level of risk, which will together indicate either “strong” or “weak” evidence of a hazard.

As illustrated in Figure 1 and explained earlier, Level C informational conditions allow only for a type 1 or 2 decision approach.

If a Level A - Approach 1 analysis and decision are possible but instead Level B or C - Approach 1 or 2 are taken, the chosen course of action will likely be sub-optimal in providing for the desired public safety and minimizing economic costs, etc. The reason for this probable sub-optimality is that with the Level B or C - 1 or 2 approaches the costs and benefits of alternative courses of action will be less well defined for the decision evaluation than with a Level A - Approach 1 decision-analysis.

__Notes__


2. It should be noted that a calculation of public safety risk costs does not imply that human life is arbitrarily assigned a dollar value. Public safety risk costs are typically quantified as annual probability of loss of life, or probable event casualties.
Figure 1 - Rational Connection between Informational Conditions and Decision-Making Approaches

Assessment capabilities based on available information, data and state-of-the-art analyses

INFORMATIONAL

Conditions

Case A
(Risk-Based Evaluation)

Case B
(Partial Risk-Based Evaluation and Qualitative Information)

Case C
(Qualitative Information)

3
(value-oriented decision based on risk costs)

2
(maximum-action decision)

1
(no-action decision)

DECISION MAKING APPROACH
ADDITIONAL COMMENTS ON THE RISK BASED APPROACH

To further identify the potential value of a risk-based decision evaluation, again consider the situation with a dam. A dam system may be thought of as being composed of three sub-elements as shown in Figure 2. The three elements include inputs to the dam and reservoir in terms of flood inflows, seismic events, etc., the dam response to these inputs, and the hazardous downstream effects resulting from the dam response.

As indicated in Figure 2, the flow of events is downstream and thus possible errors or uncertainties in element II - DAM RESPONSE can produce errors and uncertainties in estimations of "DOWNSTREAM EFFECTS," and errors and uncertainties in element I - INPUTS can produce errors and uncertainties in both DAM RESPONSE and DOWNSTREAM EFFECTS. With regard to this propagation, errors and uncertainties in INPUTS and DAM RESPONSE may lead to even larger errors and uncertainties in DOWNSTREAM EFFECTS, depending upon local conditions. As an example of amplification of upstream errors and uncertainties in the DOWNSTREAM EFFECTS consider the situation where an error of +10 percent in the estimated probable maximum flood (PMF) would produce an overtopping condition with consequent dam failure (DAM RESPONSE), widespread flood damage and a significant threat to human life (DOWNSTREAM EFFECTS). In this case, a 10 percent error in the estimated PMF (INPUT) would lead to "much-larger-than-10 percent" error in the predicted DAM RESPONSE and DOWNSTREAM EFFECTS.

Conversely, with another dam perhaps an error of +50 percent in the PMF would produce a reservoir inflow that could still be contained by the dam, and reservoir outflow with the larger PMF would be about the same as that which would obtain with the lesser (actual) PMF. In this case, the errors in the estimation of DOWNSTREAM EFFECTS due to the error in the PMF would be minimal.

The importance, then, of a parametric error or uncertainty can vary significantly depending on its location in the dam system and also specific characteristics of the system. Because of this it is not possible to generalize with regard to errors and uncertainties, and it is important to quantify the effects on study results of possible parametric errors and uncertainties for each natural hazard system. This type of (sensitivity) analysis is an integral part of a risk-based assessment, but is not included with approaches 1 or 2.

In addition to providing the most advantageous method for evaluating alternative courses of action with regard to an individual natural hazard, risk analysis can also provide a rigorous basis for prioritizing multiple hazards, and thus help identify the best strategy for applying limited resources to multiple hazard mitigation. It should also be noted that in cases where informational conditions preclude a type 3 approach to adequately evaluate alternative courses of action for a given hazard, a type 3 approach may still be possible for comparing (prioritizing) this hazard and other similar hazards.

For example, consider two hypothetical dams. If the first dam failed it would threaten the lives of 10,000 people and do 100 million dollars worth of property damage. Whereas if the second dam on a neighboring drainage failed it would threaten 100 human lives and do 1 million dollars worth of damage. Assume that in each case it is a PMF event that would cause dam failure and it is the uncertainty in the probability of the PMF event that leads to the indecisiveness of an individual risk analysis on either dam. Since the dams are on neighboring drainages, the probabilities would be approximately the same, regardless of
Figure 2 - Sequential Elements in a Dam System

I. **INPUTS** to Dam and Reservoir

- Hydrologic Events
- Seismic Events
- Other Conditions

II. **RESPONSE OF DAM** to Inputs

- Threat to Human life
- Economic Damages
- Environmental Impacts

III. Hazardous **DOWNSTREAM EFFECTS** resulting from Inputs and Dam/Reservoir Response
whether the specific value was $10^{-4}$ or $10^{-6}$ (assuming this is the range of uncertainty in the probabilities of the PMF's). Thus, regardless of what the actual probability of the PMF is, the risk costs (economic and public safety) are approximately 100 times greater for the first dam than they would be for the second dam.

BIASES IN DECISION-ANALYSIS

With regard to decision-analysis on natural hazards, the following caveat might be offered. Baecher et al. (1980) have argued that "the civil engineering profession has traditionally viewed itself as responsible for guaranteeing safety. Large civil works must be designed so that they will not fail. This is a societal duty many civil engineers implicitly accept. In fulfilling this responsibility, designers of dams often incur very high expenditures in design and construction to protect structures from unknown conditions,... The persuasiveness of this view in civil engineering is apparent by comparing the low risk of death due to failures of civil works with the much higher risk of deaths due to other engineering activities such as automotive transportation or industrial accidents."

There are several factors which could contribute to a bias toward additional safety on large civil works. First, for the same societal probability of loss of life, if the scale of a hazard is small the public will generally accept a higher risk than they will for a large hazard of the same type. For example, the probability of drowning while swimming or boating in a river below a dam is significantly larger than the probability of drowning due to dam failure. The first type of drowning occurs relatively frequently but generally involves only one or two people per event. Whereas, if the dam were to fail many people could lose their lives in a single event. The public is more accepting of the first kind of hazard even though considerably more lives are lost each year this way than as a result of dam failures. Thus the scale of the event as well as the individual annual probability of loss of life influence public preference with regard to acceptable risks.

Second, in any group of people there will be differences in the acceptable level of personal risk, both economically and with regard to personal safety. One person will skydive while his neighbor may not even be willing to ride a bicycle. Therefore, because the threat to a person's life due to dam failure is somewhat involuntary, i.e. a person is not always free to move from a flood plain below a dam, or he may be inadvertently unaware of his personal hazard in living in such an area, it can be argued that with large civil works which may affect a heterogeneous group of people, the acceptable risk level would be the risk the more cautious members of the group are willing to accept, or minimum societally acceptable risk rather than average acceptable risk which the "automotive transportation and industrial accident" risk levels are more a reflection of.

Third, engineers working on the design of large civil works generally share a strong ethical concern for public safety, and a professional incentive to not have "my structure" fail. Both factors bias the engineer's design decisions towards additional safety features. In public agencies this bias is not balanced by acute economic considerations since additional economic costs for added
safety are not "his money" or "his company's money" and also he is not preparing a design that will be evaluated competitively as is usually the case with private industry.

Fourth, there are significant differences between the decision making on safety matters in automotive transportation and the decision making with large civil works. In the case of automotive safety there is market competition such that the public can constantly reflect its (safety) choices in the cars they buy. For example, a few years ago automobile manufacturers gave some consideration to offering "air bags" which would greatly reduce the probability of loss of life and personal injury in automotive transportation. These air-bags, however, would have added several hundred dollars to the cost of each vehicle, and based on marketing surveys it was determined that the public was generally unwilling to pay for this added safety. So, as a reflection of this consumer preference air-bags were not installed at that time. With large civil works the public does not have this kind of opportunity to reject additional safety features which are often included in the alternatives offered by a governmental agency, and usually accepts one of the alternatives without asking for (or being aware of) a wider range of choices.

Finally, with most public works the economic cost is subsidized by higher levels of government. Thus the local constituency do not have to pay the entire project cost, and in their evaluation of an agency's proposed alternatives they may be biased in favor of additional safety at additional economic expense, because they will only have to pay a small part of the additional cost.

Among the factors, then, that can bias technical analysis and public decision making toward safety with large civil works, some are legitimate and other are somewhat spurious. Consequently, engineers and decision makers dealing with these kinds of projects which include natural hazard mitigation, have an especial responsibility to identify and interpret situational biases to fairly represent "those who pay" as well as "those who benefit," in identifying acceptable levels of public safety, since subtle biases can lead to significant increases in mitigation costs.

SUMMARY AND CONCLUSIONS

There are basically three approaches to decision-making in evaluating alternative courses of action for natural hazard mitigation. With the two traditional approaches it is effectively assumed that the hazardous event in question will occur during the project life or else that it will not occur. The more recently introduced risk-based approach incorporates uncertainties in event occurrences in the analysis, and risk costs are compared for viable alternative courses of action. A risk-based evaluation however, does not place a dollar-value on human life, or environmental factors.

When a risk-based assessment is possible it offers the following advantages over the traditional approaches:

1. Explicit quantitative treatment of all factors (costs and benefits) related to an evaluation of alternative courses of action for dealing with a potential hazard.
2. Quantitative evaluation of possible errors and uncertainties in the analysis and identification of their potential effects on study results (sensitivity analysis).

3. Quantitative identification of the best alternative for each value objective—public safety, economic cost, environmental factors, etc.

4. Rigorous comparison and prioritization of multiple hazards in terms of their respective probabilities of occurrence and risk costs.

Because of the explicit nature of risk-based analysis it also helps engineers and decision-makers focus on important but inadequately defined parameters, e.g. 1,000-year flood magnitude, and thereby assists in guiding (research) efforts to improve the basis for future hazard evaluations.

Risk-based analysis does not provide a perfect tool for decision making. There are both conceptual problems, e.g. objective identification of intangibles such as environmental aesthetics, and problems in application, e.g. determination of specific seismic or hydrologic event probabilities. However, when it can be applied it provides for the most rigorous evaluation of natural hazards, and thus provides the most reliable basis for decision making.

When there is not sufficient data and information for an adequate quantification of risk costs, one must resort to the traditional approaches. In this case the particular approach adopted should depend on whether there is "strong" or "weak" qualitative evidence of a hazardous event during the period of interest.
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EFFECTIVENESS OF WARNING SYSTEMS WITH DAM FAILURES IN UTAH

by Wayne Graham

ABSTRACT

Dam failures have claimed the lives of about 300 people in the United States during the last 20 years. In Utah, two people have died from dam failures during this same period. The effectiveness of a dam failure warning system is dependent upon the success of the various warning system components. The components of a dam failure warning system can be defined as follows: (1) means for predicting or detecting dam failure, (2) criteria for deciding to warn the population at risk, (3) dissemination of dam failure flood warnings, and (4) appropriate response among the population at risk. Large losses of life can occur when there are deficiencies in any of the first three components. The most tragic failures, however, occur when the first component is unsuccessful, i.e., when the failure is not predicted or goes undetected. Adequate surveillance or monitoring of dams appears to be the key element in avoiding large human losses from dam failure.

The number of dam failure fatalities that have been recorded in Utah is remarkably small. Part of this can be attributed to the fact that there was inadvertent early detection of dam failure and adequate downstream warning associated with many of the failure events. In the absence of more reliable detection and warning systems, there may be future dam failures in Utah where the outcome in terms of loss of life may be considerably less favorable.

MAGNITUDE OF THE DAM SAFETY PROBLEM IN THE UNITED STATES AND UTAH

The Corps of Engineers completed a nationwide dam inventory and inspection program in 1981. The Corps' inventory contained about 68,000 dams in the United States meeting predefined size requirements. (A dam was a structure in excess of 6 feet in height and with a maximum impounding capacity of at least 50 acre-feet of water, or at least 25 feet in height and with a maximum impounding capacity in excess of 15 acre-feet of water.) The Corps' inventory listed more than 400 dams in Utah.

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About 8,800 high-hazard dams were inspected nationwide under the Corps' program from 1977 through 1981. The term high-hazard does not refer to a dam's condition, but to the magnitude of potential losses resulting from a dam failure. About 2,900 or one out of three of the dams did not meet safety standards set up for this program and were, therefore, categorized as unsafe. Inadequate spillway capacity was the reason for four out of every five unsafe ratings. In Utah, about 60 percent of the 134 dams inspected were categorized as unsafe. The Corps of Engineers' data indicate that there is a significant dam safety problem in Utah and the United States.

An alternate way to determine the magnitude of the dam safety risk in the United States is by measuring and tabulating the human and property losses that result from dam failures. A complete set of data on dam failures is currently not available. However, the available records indicate that the likelihood of a dam, selected at random from the United States dam inventory, failing in any given single year is very small. It is perhaps in the order of 1 chance in 10,000. Dam failures in the United States cause about $50,000,000 in damages on an average annual basis. More importantly, and the main topic of this discussion, is the loss of life associated with dam failures. During the last 25 years, an average of 15 deaths has occurred annually in the United States as a result of dam failures. There were no fatalities in many years, but as many as 160 in 1972 and more than 80 in 1977. The last United States dam failure which claimed many lives was that of Kelly Barnes Dam. This dam, located in Georgia, failed in November 1977 killing 39 people. Since 1977, fewer than two people have died per year as a result of dam failures in the United States. By way of comparison, 1- and 2-family house fires kill more than 4,000 people annually and nearly 6,000 people drown in the United States every year. The low number of dam failure fatalities since 1977 may be attributable to the randomness of dam failures or due to risk reduction actions taken by owners and regulatory bodies following the issuance of the Corps' inspection reports.

The potential for human losses caused by dam failures in Utah is high. For example, the U.S. Geological Survey completed a study in 1976 and determined that the failure of the Bureau of Reclamation's Pineview Dam would cause 8,000 deaths if the dam failed in a "catastrophic manner...thereby not allowing downstream evacuation."

The actual number of fatalities that would occur from any dam failure is directly related to the adequacy of preparedness planning by the dam owner, local governments, and individual flood plain occupants. An effective warning system must also be in operation to greatly reduce or actually eliminate fatalities caused by dam failure.

WHAT IS A DAM FAILURE WARNING SYSTEM?

Before discussing the effectiveness of warning systems, it would be useful to provide a definition of what a dam failure warning system consists of. Warning is a process having certain components and stages which can be categorized as follows:
1. Means for predicting or detecting dam failure.
2. Criteria for deciding to warn the population at risk.
3. Dissemination of dam failure flood warnings.
4. Appropriate response among the population at risk.

Emergency preparedness plans or emergency action plans can be written or developed by a dam owner. The plans can describe necessary actions needed to prevent dam failure and actions to take when dam failure is inevitable. The plans can incorporate relevant components of a warning system. Preparedness planning or evacuation planning by communities will improve the chances of having a successful warning system.

Judged by the millions of people who permanently reside in flood-prone areas, it appears that our society is not willing to leave flood plains permanently unoccupied. However, society generally appears willing to temporarily leave or evacuate when threatened by short-term natural or manmade hazards. Thus, dam failure does not need to result in loss of human life.

**DAM FAILURE WARNING EXPERIENCES IN UTAH**

Detailed information is available for some of the Utah dam failures. This information was analyzed for the purpose of evaluating the various components of an effective warning system. A summary of these data follows:

Utah has been fortunate in that very few people have died as a result of dam failures. The publication "Cloudburst Floods in Utah, 1850-1938," documents more than 30 dam failures during this period of time. It does not indicate that any fatalities occurred as a result of these failures. Undoubtedly, other dam failures occurred during this time period but were not included in the publication. The Hatchtown and Mammoth Dam failures, described in the next section, were two failures that the publication omitted.

**Hatchtown Dam**

Hatchtown Dam failed at about 8:00 p.m. on May 25, 1914. The dam, located in Garfield County, near Panguitch, was about 60 feet high. About 12,000 acre-feet of water was released during the failure. The maximum outflow reached during the failure has been estimated to be as much as 247,000 ft³/s. From the information available, it appears that a very effective warning process was initiated resulting in no known dam failure fatalities. The watchman observed the dam during the forenoon of the day of failure and "did not see anything out of the ordinary, normal conditions seeming to prevail at that time" (Utah State Engineer 1915). At 2:00 p.m. or about 6 hours before the dam failure, a person who had been staying with the watchman discovered muddy water coming out through the downstream face of the dam. As soon as the watchman learned that the dam was in danger of failing, "warning was telephoned to the farmers and small communities scattered along the valley between the Hatchtown and Piute Reservoirs, and the inhabitants, comprising about 1,000 people, promptly moved their families and most of their livestock and household belongings to higher ground" (Sterling 1914). Couriers were also sent out to warn the people at risk.
Mammoth Dam

Mammoth Dam failed in June of 1917. The dam, located northwest of Price, was about 70 feet high. Failure resulted in the release of about 11,000 acre-feet of water. The available data indicate that the warning process was very effective. On a Sunday afternoon, the watchman returned to the dam after having a meal and discovered water flowing through an earth dike at the dam. The watchman's wife telephoned the news to a "central" telephone operator who in turn passed the word to various officials. Word was then communicated to all persons who might be endangered by the flood. The main break occurred more than 24 hours after the initial smaller break. The flood crest passed Price about 11:00 p.m. Monday night. Reports on this failure indicate that "a very heavy loss of life" was avoided due to the "ample warning of the impending disaster after the initial break" (Kleinschmidt 1917). One fatality was associated with this dam failure. A woman, who had traveled to the river's bank to view the flood as it passed through Price, drowned when the automobile she was riding in backed into the flood. A peak discharge of 7,300 ft³/s was estimated by the U.S. Geological Survey for a site a few miles upstream from Price.

Little Deer Creek Dam

Little Deer Creek Dam failed on Sunday, June 16, 1963. The dam, located northwest of Duchesne, was about 70 feet high and contained about 1,000 acre-feet of water at the time of failure. The U.S. Geological Survey estimated that the maximum outflow reached during the dam failure was about 47,000 ft³/s. The dam failure went undetected until the flood wave crashed through a National Forest campground located about 7 miles downstream from the dam. Flooding at the campground, which peaked at an unprecedented 39,000 ft³/s, resulted in a roar that some people described as sounding like an earthquake or an airplane. These observations alerted many of the campers to danger, thus allowing them to safely escape the rapidly rising floodwaters. A 4-year-old boy, camped in a tent with other young brothers, drowned when the water crashed through his campsite. No other fatalities were reported as a result of this failure. Dissemination of warnings in downstream areas was initiated when the deceased boy's father drove a mile and a half downstream to summon help at a ranch. A summer forest guide drove from the ranch warning campers of the approaching rampaging waters.

Box Reservoir Dam

Box Reservoir Dam failed on Monday, May 21, 1973, at about 9:00 p.m. The dam, located about 13 miles upstream from Payson, impounded a reservoir containing about 200 acre-feet of water. People had been monitoring Payson Creek for more than 10 days due to increased snowmelt runoff caused by hot weather. People in the canyon, including a resident Utah Highway Patrol Trooper, telephoned and radioed a warning that the earth dam was breaking. A city councilman then immediately set off the fire siren. Residents and merchants sandbagged their properties in order to reduce flood damages. Floodwaters reached Payson after midnight with the peak occurring around 1:30 a.m. Law enforcement officials had difficulty with spectators who had lined
the streets of the business district to view the flood. Highway patrol and sheriff bullhorns were used to disperse the approximately 1,000 spectators.

D.M.A.D. Dam

D.M.A.D. Dam was named using the first letters of four private irrigation companies. The dam failed on Thursday, June 23, 1983, at about 1:00 p.m. The dam, located northeast of Delta, contained about 16,000 acre-feet of water, with a maximum depth of 30 feet, at the time of failure. During the days preceding the failure, construction crews were at the dam trying to prevent spillway failure due to headcutting in the downstream channel which was moving toward the dam. The dam was being monitored (attended) 24 hours a day. There was an awareness among residents in the area that the dam might break.

Before the failure, residents were directed to listen to the Delta radio station. The station remained on the air 24 hours a day prior to and immediately after the failure. The local public safety officials had planned on channeling all relevant information concerning the safety or integrity of the dam through the station. A sheriff was in the radio studio for the purpose of alerting the residents of the imminent danger of a possible failure when word was received by the sheriff that the dam had actually failed. Within 60 seconds of the failure, the sheriff was reporting live on-the-air. A list of people living in the flood plain and their telephone numbers had been prepared prior to the dam failure. These people were called by Millard County dispatchers and secretaries. Sheriff deputies also went door to door notifying residents that they were in immediate danger and advising that they leave. Dam failure inundation maps were not available prior to the failure. The Millard County Administrator indicated that the telephone list was prepared based upon an educated guess of which areas were most flood prone. Additionally, prior dam failures in the area proved useful in determining which areas were at risk. Newspaper reports indicated that about 500 people evacuated. The reports also stated that the evacuation "was almost like a rehearsal," and that "people were calm and methodical." "Most people were gone within two hours" and "the majority went to the homes of relatives and friends" (Madsen and Van Leer 1983).

Gunnison Bend Dam, located about 8 miles downstream from D.M.A.D. Dam, was intentionally breached in order to prevent a catastrophic or uncontrolled failure of that structure. This work was carried out under a plan made days before the D.M.A.D. Dam failure. The leading edge of the flood moved downstream from D.M.A.D. Dam at about 2 miles per hour reaching the communities of Oasis and Deseret during hours of darkness, some 7 to 12 hours after D.M.A.D. Dam failed.

A man, thought to be a transient, died after an unsuccessful attempt to cross the swollen Sevier River in the community of Deseret by clinging to a cable strung across the river. The bridge in this community had been washed out.
NOTABLE DAM FAILURE WARNING EXPERIENCES
IN THE UNITED STATES

Additional insight regarding the value of an adequate warning process can be gained by looking at the warning experience associated with dam failures outside of Utah. Four failures will be discussed: Teton Dam, Idaho; Buffalo Creek Coal Waste Dam, West Virginia; Kelly Barnes Dam, Georgia; and Lawn Lake Dam, Colorado. Each failure has an important story associated with it.

Teton Dam

Teton Dam was located northeast of Idaho Falls, Idaho. The dam failed at 11:57 a.m., Saturday, June 5, 1976, during its initial filling. The reservoir contained about 252,000 acre-feet of water at the time of failure, and the reservoir water surface elevation was about 275 feet above the original valley floor.

On Thursday, 2 days before the failure, construction personnel found two small seeps at the downstream toe of the dam. The water was clear and totaled about 100 gallons per minute. On Friday, 1 day before the failure, a smaller seep was observed in a different location. Clear water was also flowing out of this seep and it was not considered a problem. A Bureau employee remained at the damsite until 12:30 a.m. on June 5th, the day of the failure. The dam was then unattended until about 7:00 a.m. Between 7:00 and 8:00 a.m. of the day of the failure, a survey crew discovered slightly turbid leakage. As early as 9:30 a.m., the Project Construction Engineer considered alerting area residents but he decided that an emergency situation was not yet imminent. There was concern about causing panic although numerous researchers have found that panic almost never occurs on a large scale in disaster situations (Quarantelli and Dynes 1972). At about 10:00 a.m., a larger leak, flowing turbid water, was discovered. Between 10:30 and 10:45 a.m., less than 1-1/2 hours before failure, the Project Construction Engineer notified the sheriff’s offices in Madison and Fremont Counties to begin evacuation of downstream areas. Dam failure inundation maps were not available at the time of dam failure. The evidence indicates that authorities were able to access and determine which areas were at risk. Every known means of communication was used to transmit the warning and evacuation messages. People living in areas within 20 miles of Teton Dam learned of the failure as follows (Idaho Falls Chamber of Commerce 1977):

- Radio - 44 percent
- Neighbor - 28 percent
- Telephone - 7 percent
- Police - 7 percent
- Other - 14 percent

The failure resulted in damages that were estimated to be about $1/2 billion. More than 35,000 people were evacuated from their homes. The overall effectiveness of the warning and evacuation can be judged by what happened at Sugar City and Rexburg, two of the communities closest to the
Sugar City, a community of about 600 people, is located about 12 miles downstream from Teton Dam. Floodwaters reached this community about 1 hour after Teton Dam failed with depths of up to 15 feet being recorded. No fatalities were recorded in Sugar City. About 80 percent of Rexburg, with a population of 10,000 people, was inundated to depths of 6 to 8 feet. Floodwaters reached this community, located about 15 miles downstream from Teton Dam, about 1 hour 40 minutes after Teton Dam failed. Two nondrowning fatalities were recorded in Rexburg.

Despite the warnings that were issued, there were a total of 11 fatalities attributable to the failure of Teton Dam. A 21-year-old person fishing just below the dam drowned. He was the only deceased person who had not received a dam failure warning message from some source. All other people who died had varying beliefs or degrees of knowledge concerning the dam failure and associated danger. An elderly couple failed to evacuate after receiving an in-person warning from their 23-year-old grandson. The couple drowned. A group of three men drove from an area of safety into an area that was soon to be flooded in order to help remove household items from the home of a relative of one of the three. The homeowner was quoted as saying: "I told (the three men) there would be a lot of water coming down the canyon, but they seemed to think they had time to put another load in that pickup (truck). I was awfully scared, I thought I would have a heart attack. I had to leave." The three men remained at the home and drowned. A 94-year-old woman died after being evacuated from the flood area. Another person died of an accidental gunshot wound sustained while removing a gun from his vehicle. Two people died from heart attacks. A man living outside of the flood-damaged area committed suicide 5 days after the dam failure.

Buffalo Creek Coal Waste Dam, West Virginia

Buffalo Creek, West Virginia, is far from Utah but it is relevant to any discussion of warnings associated with dam failure. At the time of failure, the water was about 46 feet above the streambed and the impoundment contained about 400 acre-feet of water. The dam failed at 8:00 a.m. Saturday, February 26, 1972, during a flood with a 2-year frequency of recurrence.

Representatives (the dam was owned by a subsidiary of Pittson Coal Company) were at the dam the morning of the failure. These individuals realized that there were some problems and were taking some action to prevent dam failure. The warning process failed because company officials did not make the decision to warn the public at risk. In fact, the senior dam owner representative official on the site dismissed two deputy sheriffs who had been sent to the area to aid in the evacuation. Response to the meager warnings disseminated was inadequate because false alarms had occurred on at least four earlier occasions. The leading edge of the floodwater traveled the 15-mile length of the Buffalo Creek Valley in about 3 hours, killing 125 people. About 500 homes were destroyed and about 4,000 people were left homeless.
Kelly Barnes Dam, near Toccoa Falls, Georgia

At the time of failure, Kelly Barnes Dam impounded about 630 acre-feet of water and the water was about 34 feet deep. The dam failed at approximately 1:30 a.m., November 6, 1977, during a flood that the U.S. Geological Survey estimated to have a 10-year frequency of recurrence. The warning process failed because the dam was not being monitored. Thus, the dam failure went undetected until floodwaters crashed through the campus of Toccoa Falls College, drowning 39 people all of whom, at the time of the failure, were located within 1 mile of the dam and 500 feet of Toccoa Creek. This was a very large number, considering that only a few hundred people were at risk. This points out that large numbers of people have perished, and will perish in the future, when dam failures are not predicted or detected.

Lawn Lake Dam, Rocky Mountain National Park, Colorado

Lawn Lake Dam was 26 feet high. Less than 700 acre-feet of water was released when the dam failed at about 5:30 a.m. on Thursday, July 15, 1982. Lawn Lake Dam was located in Rocky Mountain National Park, 4 air miles from the nearest road or telephone. There were as many as 25 hikers camped in the roadless area immediately downstream from the dam. The campers received no official warning but environmental warnings, such as seeing trees breaking and hearing the roar of the river, described by many as like continuous thunder, enabled most of these campers to escape. Nonetheless, one camper in this area was swept to his death.

A garbage collector, while making pickups on the road nearest the dam, thought a jet was crashing and then saw mud and debris on the road. He then drove a short distance and used a National Park Service Emergency telephone to report this observation to the National Park Service Dispatch Center. This telephone call set off a series of actions which resulted in thousands of people evacuating from flood-threatened areas. The National Park Service warned and evacuated people occupying the most threatened portions of the Aspenglen Campground where 275 people were camped. Law enforcement and other Government officials warned and evacuated people in Larimer County and Estes Park. The only radio station in Estes Park learned about the dam failure on a police scanner and the station was instrumental in spreading the message of the dam failure. Two fatalities occurred at Aspenglen Campground when campers left an area of safety and tried to walk into inland campsites. Both deceased campers had been informed of the impending flood (but not of a dam failure) by other campers.

CONCLUSIONS

Increased emphasis and expenditure on dam safety will likely reduce the number of future dam failures. It is, however, unlikely that dam failures will ever be completely eliminated as evidenced by the following two statements: "I (the Colorado State Engineer) could have had all six inspectors standing on the [Lawn Lake] dam the day before and not detected the failure"
Effective warning systems can reduce greatly the risk associated with operation and failure of dams. Warning systems need not be excessively costly. Serious consideration should be given to installing warning systems at dams that pose unacceptable risks to society.

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HAZARD MITIGATION - UPGRADING THE "STANDARD OF CARE"

by Michael F. Richman

ABSTRACT

1. STANDARD OF CARE - DEFINITION
   a. Same Locality
   b. Similar Locality

2. GOVERNMENTS ROLE IN ESTABLISHING "STANDARD OF CARE"
   a. Reporting Requirements
   b. Code Control
   c. Enforcement
   d. Requisite Expertise

3. SOILS PROFESSIONALS RESPONSIBILITY IN ESTABLISHING "STANDARD OF CARE"
   a. Compliance with Governmental Structures
   b. Utilization of "State of Art" Techniques
   c. Policing the Profession

4. PROPOSALS FOR UPGRADING THE STANDARD
   a. Statewide Adoption of UBC § 70
   b. Statewide Adoption of CDMG NOTE #44
   c. Professionalizing Governmental Authority
   d. Registration of Soils Professionals

5. ANTICIPATED BENEFITS
   a. Reduction in Disaster Related Damage
   b. Reduction in Cost to Taxpayers for Natural Hazards

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MITIGATING DAMAGE FROM GEOLOGIC HAZARDS: A LEGAL PERSPECTIVE

By Jennifer Falk

Scientific methods for predicting and at times preventing geologic hazards are constantly being developed and improved. As a consequence, when property damage or loss of life results from a geologic hazard, people are no longer content to consider it an act of God, but are looking to the parties they feel are responsible to help pay for the loss. Because of this, geologic hazards are a source of interest to the legal community, which is often involved because of the controversy and litigation over who should pay.

However, this is not the only way that law can be involved in geologic hazards. In addition to being involved allocating the resulting loss, in recent years many states and local governments are turning to law as a way of organizing the community and of preventing some of the damage caused by geologic hazards. Through providing mechanisms to alert individuals to the presence of geologic hazards or to restrict certain activities which compound the danger of destruction, many governments are mitigating the cost of the damage caused by geologic hazards. California has passed several bills concerned with lessening the damage caused by earthquakes. One example is the Earthquake Protection Act, requiring the construction and design of buildings to meet building code regulations to resist stresses produced by lateral forces. In 1974, the Colorado Legislature passed a statute empowering and encouraging local governments, through the help of the soil and water conservation boards and the State geologist, to identify areas with known geologic hazards and to designate them as areas of "state interest." Once so designated, the local government is able to regulate the area more stringently than before, consistent with the purposes of the state, to minimize significant hazards to public health and safety or property.

At this point in time, Utah has not taken advantage of the opportunity law provides as a means of mitigating damage caused by geologic hazards. This paper will examine various methods which Utah might implement at either the state or local level, and will discuss advantages and disadvantages of each, including, where possible, information on how similar methods have worked in other states. To facilitate discussion, the various methods available will be grouped into three general areas: (1) disclosure, (2) regulation by governmental entities, (3) tort liability.


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1. DISCLOSURE

Disclosure, as a prevention technique, means making an individual aware of the existence of known geologic hazards on his property or in the general area. The theory is that if the individual is informed of these known geologic hazards he will take the initiative and seek ways to minimize the risk. He might do this through soliciting the advice of experts, through employing different construction techniques, or through insurance. He might also choose to avoid the hazard altogether.

California has used disclosure on a limited basis through the Alquist-Priolo Special Studies Act. Under this statute, areas which encompass potentially or recently active faults are mapped by the State geologist and placed into "special study zones." In these areas, ordinarily a quarter mile wide, the seller must disclose to any prospective buyer that the property is located within a special study zone.* Disclosure of this type is specific, since it is concerned with only one particular hazard. Specific disclosure has also been used on a limited basis in Utah. In the town of Alta, located near the top of Little Cottonwood Canyon on the Wasatch Front, a prospective buyer of property is made aware of the high avalanche risk in the area and the potential threat to property and life. Other local governments, facing one hazard in particular, could adopt similar measures. For example, if a prospective buyer chooses to buy a house in an area in which landslides are a major problem and he has been made aware of this, he could seek advice about how best to protect his house. He might weight the toe of the slide for instance, or minimize the amount of water seepage.

One problem with specific disclosure is that in focusing on one hazard only, other more potentially dangerous geologic hazards on the same property or in the general area may be overlooked. This can be avoided by adopting a mechanism by which all known geologic hazards are disclosed. This would provide the most information to the public. To date, however, no state has enacted such a measure. The probable reason for this is the expense involved. In order to be useful to the property owner, the information disclosed must be on a scale small enough to give precise information to a property owner concerning his particular acreage. Maps must be drawn so that laymen as well as trained geologists can understand them.

4 Id at §2621.9.
5 This is explained in the Alta "Hold Harmless Agreement" that every purchaser signs which states: "The town of Alta has determined and observed that the property is located in an area frequented by avalanches and that the hazards and dangers arising therefrom pose serious threats of destruction, injury, and harm to property located in an area within said area or individuals residing or visiting said area."
A more practical method of disclosure, and an approach which Utah has the resources to implement right now, would be to simply make available all existing information on known geologic hazards in a given area. Under this method, a property owner would be made aware of the existence of geologic hazards in the general area and whether or not more specific information is available. It is then up to him to pursue the matter further. The biggest question here is deciding how the information should be disclosed to the property owner. One method which Utah could consider is affixing a clause to the title search report stating whether or not geologic information is available on the property. Another method is requiring developers and condominium owners to include in the public offering statement a general statement concerning the known geologic hazards in the area and whether or not more specific information exists.

A drawback to any of these proposed methods is opposition the state will face from any group made responsible for disclosing the information. In January of 1984, legislation went before the Utah House of Representatives which would have alleviated this problem. The bill was known as "House Bill 28." Rather than requiring any one group to be responsible for disclosing information, the bill provided that the information be made accessible to owners who could obtain it on their own. The first section of the proposed statute defined known geologic hazards. It listed information which the Utah Geological and Mineral Survey (UGMS) was required to provide by January of 1987. This information included a series of maps, interpretive pamphlets, and indexes of other geologic information available concerning a given area. The UGMS was also responsible for making sure that the information was in easily duplicated form, such as microfiche, and given to various governmental entities, including city and county engineers, county recorders, and city and county planning commissions. The mapping was to be scrutinized and monitored by the UGMS before being indexed to assure its quality and reliability.

Of the three types of disclosures mentioned, this seems the most feasible plan for Utah. Although there would be an initial cost to assemble the necessary information and disseminate it to local governments, it would allow Utah residents concerned with geologic hazards to be able to obtain information. As of now, any information of this sort is difficult for a layman to obtain without hiring a private geologist. Until such an act is finally passed, however, local governments should be encouraged to provide information on their own.

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7 Interview, May 29, 1984, Genevieve Atwood, Director of the Utah Geological and Mineral Survey
II. REGULATION BY GOVERNMENTAL ENTITIES

A second way in which law may be used as a source of prevention is through regulations created, adopted and enforced by state and local governments. This area, the one in which most preventive actions fall, is highly diverse. Methods range from actual restrictive laws to the creation of means by which other governments or individuals are able to regulate geologic hazard areas.

In some states, legislation regulating geologic hazards exists on a state level through statutes passed by the state legislatures. For example, Colorado has a provision passed by the Colorado General Assembly in 1973 which requires all proposed developments of subdivided land in unincorporated areas to be accompanied by a report on geologic characteristics that might significantly affect the proposed land use. However, for the most part, legislation passed on the state level is enabling legislation. It provides local governments, and sometimes individuals, with the power to oversee the use of land, taking geologic hazards into account. Colorado has two such provisions: The Local Government Land Use Control Enabling Act, which specifically authorizes local governments to regulate for geologic hazards; and The Areas and Activities of State Interest Act, which encourages local governments to designate areas of geologic hazards as areas of state interest and regulate them. Utah could easily adopt similar legislation.

California has statutes similar to that of Colorado, which empower municipal corporations to regulate land and to provide for geologic hazards. In addition, California has passed legislation on the state level which allows a district to be created by individuals wishing to regulate a geologic hazard. These districts, called Geologic Hazard Abatement Districts, are formulated in much the same manner as a special district. A petition signed by not less than ten percent of the landowners must be presented to the legislature along with a plan of control. A public hearing must then be held, and if more than fifty percent of the property owners in the proposed district object, proceedings are abandoned. Once created, the district does not answer to any local government but is a political subdivision of the state. While the district's authority includes acquiring, disposing of, and annexing land, it is not considered to have the full powers of a special district.

Utah could consider passing such a measure on the state level. One advantage of forming such a district is that it can include non-contiguous as well as contiguous land, and can include territory located within the boundaries of more than one local agency. This is particularly useful, since a geologic hazard rarely falls squarely within the boundaries of a single government.

Utah could also amend existing legislation to provide similar authority to counties. The "Utah Special Service District Act" already gives a county or municipality the authority to establish a service district for flood control, a particular geologic hazard. The act could be amended so that it would grant authority to create districts for other types of geologic hazards as well. This act also enables the district to cross the boundaries of counties, municipalities, and improvement districts. In addition, it has the power to issue bonds and notes to procure necessary funding.

The above-mentioned forms of regulation available to Utah all require action by the legislature. However, it is probable that municipalities and counties in Utah already have enough power to regulate for geologic hazards through the enabling laws. There are two relevant enabling laws, one for municipalities, and one for counties. In the latter part of the nineteenth century and the early part of the twentieth century, courts tended to use Dillon's Rule in construing statutes which delegated power to local governments. The rule, first enunciated in 1868, provided for strict construction of the delegated power. The statute had to either grant the power with express words, or the power had to be "necessarily or fairly implied in or incident to the expressed power, or essential to the purpose of the statute." Today, however, this is no longer the rule in Utah. In State of Utah v. Hutchinson, a 1980 case before the Utah Supreme Court, Justice Stewart, in writing for the majority, stated that "The Dillon rule of strict construction is antithetical to effective and efficient local and state government. If at one time it served a valid purpose, it does no longer." In its place, the Court held that:

14 Utah Code Ann. §10-8-84 (Supp 1983).
15 Utah Code Ann. §17-5-77 (1953), as amended.
17 624 P.2d 1116 (Utah 1980).
18 Id at 1126.
When the State has granted general welfare power to local governments, those governments have independent authority apart from, and in addition to, specific grants of authority, to pass ordinances which are reasonably and appropriately related to the objectives of that power, i.e., providing for the public safety, health, morals and welfare. And the Courts will not interfere with the legislative choice of the means selected unless it is arbitrary, or it is directly prohibited by, or is inconsistent with the policy of, the state of federal laws or the Constitution of this State or of the United States.19

Under this and subsequent rulings,20 it appears possible for a county to require that geologic hazards be taken into consideration when preparing a master plan. It can regulate for geologic hazards because of its authority to promote the health and safety of its inhabitants.

III. TORT LIABILITY

The third area in which law can be used to mitigate damage caused by geologic hazards is the area of tort liability -- placing responsibility on persons, including corporations, developers, and governmental entities, to pay for a loss caused by their actions. While this method is currently used by individuals as a way to recoup losses, it can also serve as preventive measure. Once held liable for damage caused, one is likely to take steps that will lessen the likelihood of such incidents occurring again.

In the past few years, more and more courts are holding developers and contractors liable for defective construction, not only of the building, but also on filled or otherwise unstable land.21 The most common theory used in such situations is one of negligence. In a negligence action, the plaintiff must show that the defendant "has failed to exercise the degree of care which a reasonable person would have exercised under the same circumstances, whether by acting or by failing to act."22 The Wyoming court in ABC Builders Inc. v. Phillips,23 used this theory in finding a defendant contractor negligent for building a home on the toe of

19 Id supra n. 18.
a landslide. The defendant was held liable for the resulting damage to the home and the cost of moving it to a more stable location. In Kansas the court has held that a builder-vendor could be held liable for damages in a negligence action for failing to test the level of the water table before building. In Eeri Inc. v. Salishan Properties, Inc., the Oregon Supreme Court cited Kansas decision and a ruling from Michigan in which building contractors were held liable for failing to conduct soil bearing tests before building a home on plaintiff's lot. The Oregon court, after referring to the cases, stated:

If builders can be held liable for their negligence in constructing a building without first making reasonable tests to determine the quality of the underlying soil, we see no reason why a land developer -- one who chooses land and lays it into lots which are sold for the specific and limited purpose of building a dwelling thereon -- may not be held responsible for losses to purchasers caused by his failure to take reasonable precautions to determine whether the lots he offers are fit for the purpose.

In addition to allowing the original property owners to bring an action against the builder or developer, courts are doing away with the privity requirement and allowing subsequent purchasers to state a claim against a builder on the theory of negligence as long as the statute of limitations has not run. In Utah, the time limit for bringing such an action is seven years, which is longer than that of California. However, California considers each act as a separate occurrence. For example, in a suit for damages resulting from slope failure, it is considered a separate action each time the slope fails.

In addition to recovering on the more traditional theory of negligence, plaintiffs in California have also recovered from developers and builders on a breach of warranty theory, the builders being held to have breached the warranty of a safe product. The California courts have also held mass developers strictly liable for damage to homes from geologic hazards. To maintain a strict liability theory, the plaintiff must prove that the damage was caused by a defect in the product which existed both before the defendant sold it and at the time the plaintiff purchased it.

26 Id at 177.
and that the resulting injury was reasonably foreseeable to the defendant. In Avner v. Longridge Estates, the court ruled in favor of the plaintiff in a suit for damages against the developer of hillside property in the Santa Monica mountains. The court held that the developer was strictly liable for the slope failure and subsidence on the property which resulted from improper filling and grading.

Another area in which tort liability can be an effective method to prevent damage caused by geologic hazards is in suits against governmental entities. For many years this method was unavailable because of the doctrine of governmental immunity. Under this doctrine, the governments are not responsible for the wrongful acts or omissions of their employees. Although an employee is usually capable of being held liable, he is usually too poor to enable recovery of any sizeable judgment, and hence is not worth suing. Over the years, this doctrine has been severely criticized in legal circles. As a result, there has been a gradual erosion of the doctrine, either through judicial decisions or statutory reform. Some courts have approached the problem by drawing a distinction between whether an act causing the injury was proprietary or governmental function. If proprietary, the government is treated as a corporation from the private sector would be, and is exposed to common law tort liability. In Utah, the court has made this distinction.

In addition to court decisions, the Governmental Immunity Act is regulated by statute. However, the present statute is less stringent than its predecessor. Although entitled the "Governmental Immunity Act," rather than providing blanket immunity, the statute is little more than a list of specific exceptions for which immunity is not waived. Examples of exceptions are actions which (a) arise out of the exercise or performance or the failure to exercise or perform a discretionary function, whether or not the discretion is abused, or (b) arise out of a failure to make an inspection, or by reason of making an inadequate or negligent inspection of any property. Although making more room for recovery than before, these exceptions still make it very difficult for plaintiffs to recover for something the government failed to do, such as providing information or enforcing building regulations in accordance with minimizing risk from

33 Id at §63-30-10.
geologic hazards. However, a plaintiff is not totally foreclosed from recovery just because it cannot claim a tort.

In other situations, a plaintiff can employ a different legal theory which the Governmental Immunity Act does not address. For example, in some states plaintiffs foreclosed from suing a governmental entity by an immunity act have been allowed to recover on a nuisance theory. 34

An even better theory under which plaintiffs have been able to recover is one of inverse condemnation. This theory is not a tort but a constitutional theory, based on the compensation clause of the Fifth Amendment which prohibits the taking of private property by the government without just compensation. The Utah Constitution has a similar provision. 35 For an action to be maintained under this theory, the plaintiff must show that the defendant substantially participated in some activity for public use or benefit, that the activity or failure to act as planned was the proximate cause or a concurring substantial cause of the damage, and that the plaintiff filed a claim which was denied. An example of this is Albers v. Los Angeles, 36 where considerable physical damage occurred to the plaintiff's home as a result of a landslide caused by the county constructing a road. The trial court held the defendant county not liable on any tort theory and therefore plaintiff appealed. The Supreme Court, while not finding any tort liability, nevertheless concluded that the plaintiff could recover under the compensation clause of the California Constitution. 37

IV. CONCLUSION

Any of the methods discussed above, either singly or in combination with others, could aid in the prevention of damage caused by geologic hazards. Those who draft hazards legislation must of course be aware of the political realities such bills must face. In Utah, there was considerable support for House Bill 28 when it went before the legislature in January of 1984. It failed to pass but showed that such a bill has a chance of becoming law. As floods and mudslides continue along the Wasatch Front, public awareness of geologic hazards seem to be growing. If introduced, a bill which proposes a feasible plan for disclosure at a manageable expense, as House Bill 28 did, would be politically realistic and would offer perhaps the most effective means of mitigating hazards damage.

35 Constitution of Utah, Art. 1, §22.
36 62 C. 2d 250, 42 Cal. Rptr. 89.
37 See also Holtz v. Superior Court, 475 P.2d 441 (Cal. 1970); Bacich v. Board of Control, 144 P.2d 419 (Cal. 1943).
ABSTRACT

In recent years the City of Provo, located on the Wasatch Front, has experienced increasing pressure to allow development of attractive "view lots" on the high elevation benches and hillsides of its "East Bench." Although the area is desirable and picturesque, large parts of the East Bench consist of soil and rock in a naturally fragile state of stability underlain by multiple branches of the Wasatch Fault. In the past, development was planned and designed utilizing inadequate guidelines for foothill development in a geologically hazardous environment.

The wet springs of 1983 and 1984 activated a variety of instability problems including landslides and subsidence that damaged homes, apartments, and other buildings, broke utility lines, and closed streets. The previous hillside ordinance regulations did not adequately consider the implications of geological hazards such as shallow water table, soil conditions, landslide areas, faults, or alluvial fans.

The purpose of this study was to produce geological hazard maps of the East Bench of Provo which would be useful as a basis for revising the existing subdivision regulations on hillside development.

The geological hazards were divided into the following map categories:

A. No Known Hazard

B. Shallow Water Table
Expansive Soils

Collapsible Soils
Potential Landslide
Secondary Fault

Alluvial Fan (potential debris flow or flash flood areas)
Active Landslide
Primary Wasatch Fault

Flash flood zones were not part of this study.

The geological hazard categories were ranked in groups from A to D in order of increased potential risk to lives and property and the corresponding difficulty and expense in mitigating that risk.

Revisions to the existing City of Provo subdivision ordinance were proposed to reduce or avoid geological hazards. The revisions were based on new regulations requiring specific geotechnical investigations by qualified professionals for each hazard category. The investigations are to be performed by a qualified engineering geologist, geotechnical engineer, or civil engineer. These investigators must certify that the design of the development incorporates their conclusions and recommendations.

It was recommended that human dwellings should not be constructed over the trace of a secondary fault or within a 50-foot setback from a primary Wasatch Fault Zone. Essential or critical structures, such as high-rise buildings, hospitals, and schools may be subject to a wider setback at the discretion of the City of Provo.

INTRODUCTION

Provo is located in north-central Utah about 40 miles south of Salt Lake City. The city is situated near where the Provo River enters Utah Lake between the lake and the precipitous western front of the Wasatch Range (Figure 1).

The land surface is characterized by a series of river and lake terrace sediments which formed during the Pleistocene Epoch in response to the varying elevations of Lake Bonneville. The terraces include the present Utah Lake level (about 4,190 feet), the Provo level (4,800 feet), the Alpine level (5,100 feet), and the Bonneville level (5,135 feet). The terraces form prominent levels along the Wasatch Front.

The Wasatch Range has been uplifted above Utah Valley along the Wasatch Fault, a normal fault zone which is a major structural feature in the area. Thousands of feet of clay, silt, sand, and gravel valley fill exists beneath the Pleistocene terraces. The valley fill and terrace sediments are thin in the vicinity of the Wasatch Fault. On the lower mountain slopes, Precambrian and Paleozoic-aged quartzites, sandstones, shales, and limestones are exposed. These rocks generally form stable slopes with the exception of the Manning Canyon shale. At the surface, the Manning Canyon is characteristically a deeply-weathered soft, black shale which is weak and
LOCATIONS OF GEOLOGIC HAZARDS MAPPING

FIGURE 1
slightly expansive. Where it is sheared by branches of the Wasatch Fault, black clayey gouge is formed.

In the past three abnormally wet years Provo, like other Wasatch Front cities, has been experiencing a variety of ground stability problems. The problems have concentrated in the higher elevation foothill subdivision areas where landslides and subsidence have damaged buildings, broken utility lines, and closed streets.

The hillsides are covered with unconsolidated, fine-grained Lake Bonneville sediments which are susceptible to rapid erosion, subsidence upon loading or saturation (hydrocompaction), and landslides. Large areas are underlain by old landslide deposits which formed in the Lake Bonneville sediments or colluvium. Many old landslides moved along the naturally weak contact between the Lake Bonneville sediments and colluvium and the underlying highly weathered Manning Canyon Shale. In addition, the foothill area is crossed by multiple branches of the Wasatch Fault.

The City of Provo recognized the inadequacy of the slope-density ordinance approach to deal effectively with the complex geologic conditions in hillside development. The purpose of this study for the City was to produce simple, useable geologic hazard maps to overlay Provo's 1" = 100' scale quarter-section base maps and to recommend revisions to the existing subdivision regulations on hillside development based on these maps. Each of the quarter-sections mapped are shown on Figure 1. The objective was to identify and map the geologic hazards of the foothills and then determine what detailed geotechnical investigations and reports should be required in those areas. Determination of flood hazard zones was not included in the study.

METHODS

The geologic maps were drawn after reviewing existing literature in the Provo area, studying standard black and white air photographs, and conducting reconnaissance geologic mapping. Some of the pertinent literature included geologic maps (Baker, 1964 and Hintze, 1978), guidebooks (Blissell, 1966, and Rigby and Hintze, 1968), a study of the southern portion of the Wasatch Fault (Cluff and others, 1973), and a soil hazards survey (Rollins, Brown, and Gunnell, 1977). Ideas for constructing the geologic hazards maps and basing the hillslope ordinances on the maps were modified from a paper on relative ground stability (Hoexter and others, 1978) in the San Francisco Bay area, a comprehensive foothill development study done for the Davis County Planning Commission, Utah (Davis County Planning Commission, 1980), and Chapter 4, Geologic Provisions, of the Subdivision Codes for Santa Clara County, California. Hillslope ordinances were reviewed for a number of other cities and counties troubled by similar natural hazards in Utah, Idaho, Colorado and California.

Traditionally, ordinances regulating foothill development follow one of three approaches: (2) Slope Density, (2) Soil Overlay, or (3) Guiding Principles. Some are modified by combining parts of other approaches.
The slope density approach defines the amount of land which can be developed based on the average slope. As average slopes increase, either lot sizes increase or buildable areas decrease. The advantage of this approach is its simplicity which results in low administration costs. A disadvantage is that all areas are considered the same regardless of varying geologic hazards.

The soil overlay approach identifies hillside areas which are subject to erosion, soil failure, and drainage problems. The advantage of this approach is that precise development standards can be set for local areas and administration is easy. Disadvantages include high initial costs and the fact that other geologic hazards, such as faults, debris flow zones, and flood zones are not considered.

In the guiding principles approach, no precise development standards are set. Development is allowed as long as public health, safety, and welfare is preserved. The guiding principles approach is often modified by standards for grading and erosion controls. The main disadvantage is that administration is time-consuming and requires a lot of expert evaluation.

The geotechnical approach, presented here, combines soil overlay with other natural geologic hazards. Geologic hazard maps are produced which delineate categories of potential hazards. Recommendations are made for specific geotechnical investigations by qualified professional engineering geologists and engineers for each potential hazard category. The advantages of this approach are that all types of geologic hazards are considered, precise standards can be set, and administration should be relatively simple. High initial cost should be offset, in the long run, by savings from reduced maintenance in geotechnically sound developments.

GEOLOGIC HAZARD MAP CATEGORIES

After careful assessment of the geologic conditions in the foothills, the geologic hazards were divided into nine map categories. The categories were then qualitatively ranked in order of increasing hazard from areas with no known hazard to primary Wasatch Fault zones (Table 1).

<table>
<thead>
<tr>
<th>Map Designation</th>
<th>Map Category</th>
<th>Relative Hazard Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>No Known Hazard</td>
<td>A (Least hazardous)</td>
</tr>
<tr>
<td>Bw</td>
<td>Shallow Water Table</td>
<td>B</td>
</tr>
<tr>
<td>Be</td>
<td>Expansive Soils</td>
<td>C</td>
</tr>
<tr>
<td>Cc</td>
<td>Collapsible Soils</td>
<td>C</td>
</tr>
<tr>
<td>CI</td>
<td>Potential Landslide Area</td>
<td>C</td>
</tr>
<tr>
<td>Cf</td>
<td>Secondary Fault</td>
<td>D</td>
</tr>
<tr>
<td>Da</td>
<td>Alluvial Fan</td>
<td>D</td>
</tr>
<tr>
<td>D1</td>
<td>Active Landslide</td>
<td>D</td>
</tr>
<tr>
<td>Df</td>
<td>Primary Wasatch Fault Zone</td>
<td>D (Most hazardous)</td>
</tr>
</tbody>
</table>

TABLE 1 - MAP CATEGORIES AND RELATIVE HAZARD GROUPS
The geologic hazard map categories were placed in relative groups designated A to D in order of increasing potential risk to lives and property, and corresponding difficulty and expense in mitigating that risk. Within each group, assignment of relative hazards was somewhat arbitrary. The relative hazard groups are defined as follows:

**Group A** - Areas of relatively stable soil and rock representing the least hazard received this designation.

**Group B** - Shallow water table and expansive soils were ranked as B, assuming that risk to lives is minimal and that relatively simple engineering solutions will mitigate the problems.

**Group C** - Collapsible soils, potential landslide areas, and secondary faults are included. This hazard group represents a greater risk to the safety of people and property than Group B. Engineering solutions to mitigate the risks are more difficult and expensive.

**Group D** - Map categories within the most hazardous group are generally unsuitable for development. Included are alluvial fan areas (debris flow-flash flood zones), active landslide areas, and primary Wasatch Fault zones. In these areas risks to the safety of people and property are highest and engineering solutions to mitigate these hazards are complicated, expensive, and may be impractical. Areas within 50 feet of primary Wasatch Fault zones should be avoided.

An example of an area mapped for geologic hazards is shown on Figure 2. The quarter-section map, P-17, covers an area at the north end of Oak Hills and the mouth of Rock Canyon. On Figure 3 can be seen the geologic hazard map overlay with the map categories designated.

**DISCUSSION OF MAP CATEGORIES**

The following is a brief description of the mapped geologic hazards and the recommended geotechnical investigations for each hazard. The recommended geotechnical investigations are summarized in Table 2.

**No Known Hazard (A)**

Areas mapped as (A) generally include stable granular soils and colluvium on gentle slopes.

Although no geologic hazard was observed in these areas, it was recommended that specific sites should be field inspected and a letter report submitted by an engineering geologist or a geotechnical engineer before building permits are issued. This is felt necessary due to the reconnaissance nature of the geologic hazards mapping and the fact that hazards, such as landslides, can suddenly develop in new areas.
A GEOLOGIC HAZARD MAPPED AREA

FIGURE 2
QUARTER-SECTION MAP P-17

FIGURE 3
### Table 2 - Recommended Geotechnical Investigations

<table>
<thead>
<tr>
<th>Potential Hazards Scale</th>
<th>Recommended Work</th>
<th>Work Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Least</td>
<td>Field Inspection (G or GE)</td>
<td>Development of specific sites should be field inspected.</td>
</tr>
<tr>
<td></td>
<td>Evaluate hydrogeology and liquefaction potential (GGE)</td>
<td>Subsurface exploration of ground water, laboratory testing of soil samples, liquefaction analysis, and foundation engineering.</td>
</tr>
<tr>
<td></td>
<td>Characterize potential for soil expansion (G)</td>
<td>Shallow subsurface exploration of foundation soils, laboratory testing to determine expected expansion, and, if necessary, a plan for mitigation.</td>
</tr>
<tr>
<td></td>
<td>Engineering analysis of geology, slope stability, and susceptibility to erosion (GGE+GE)</td>
<td>Subsurface exploration and laboratory testing of soils to define allowable loads, predict stable fills and cuts on slopes, and recommend erosion control procedures.</td>
</tr>
<tr>
<td></td>
<td>Engineering geologic investigation and slope stability analysis (GGE)</td>
<td>Subsurface exploration and laboratory testing of soils to define potential landslide materials and insure slope stability by predicting stable cut slopes and fills.</td>
</tr>
<tr>
<td></td>
<td>Engineering geologic investigation (G)</td>
<td>Subsurface exploration to accurately locate and characterize the fault.</td>
</tr>
<tr>
<td></td>
<td>Flooding and debris flow study (GCE)</td>
<td>The extent of flooding and debris flow potential should be determined and a plan of avoidance or mitigation developed.</td>
</tr>
<tr>
<td></td>
<td>Engineering geologic investigation and slope stability analysis (GGE)</td>
<td>Subsurface exploration and laboratory testing to characterize the depth, areal extent, and type of unstable materials including a plan to stabilize the slide and prevent future re-activation.</td>
</tr>
<tr>
<td></td>
<td>Engineering geologic investigation (G)</td>
<td>Subsurface exploration, laboratory testing, and detailed geologic mapping to accurately locate and characterize the fault.</td>
</tr>
</tbody>
</table>

**Notes:**

Geotechnical investigation reports should be reviewed by the City of Provo and placed on open file.

1. Work shall be performed by (G) Certified Engineering Geologist; (GE) Certified Geotechnical Engineer; or (CE) Registered Civil Engineer.

2. Subsurface exploration may include drilling, trenching, or geophysical techniques. Laboratory testing may include Atterberg limits, gradation, hydrometer, relative density, specific gravity, swelling, and load tests.

3. Riser wells should not be built on the trace of a secondary fault or within a minimum of 50 feet from a primary Wasatch Fault Zone. Essential or critical structures such as hospitals, buildings, schools, etc. may be subject to a wider setback at the discretion of the City of Provo.
Some areas, designated A*, include distal portions of alluvial fans and natural channels where there is potential for flash flooding. Other areas were designated, A-Cl where thick alluvial sands and gravels on moderate slopes may become unstable in overly steep cuts.

Shallow Water Table (Bw)

The water table is at or near the surface in scattered, small springs and wet zones adjacent to branches of the Wasatch Fault and in two areas where sizeable sag ponds have formed by differential settlement or secondary faulting on the down-dropped side of the Wasatch Fault. This settlement or faulting has disturbed the continuity of the surface or subsurface drainage.

The major problem in shallow water table areas is dewatering during construction. If the soil contains appreciable amounts of silt or clay, settlement may occur. Normally, conventional foundation engineering should be adequate.

For developments in shallow water table areas, the hydrogeology should be investigated by an engineering geologist and the potential for liquefaction should be determined by a geotechnical engineer. This will involve subsurface investigation and laboratory testing.

Expansive Soils (Be)

Areas overlying the Manning Canyon Shale and a severely weathered clayey breccia which appears to have formed over the shale may have expansive properties. In the Soil Hazards Survey by Rollins, Brown and Gunnell, Inc., 1977, clays in Sherwood Hills are said to have a slight tendency towards expansion. Expansive soils can cause troublesome buckling and heaving of foundations for streets and buildings upon wetting and drying.

Soil at foundation grade should be sampled by a geotechnical engineer and laboratory tested to characterize the potential for expansion and to determine the requirements for stable foundations.

A variety of mitigating methods may be employed depending on the potential for expansion and the thickness of the expansive soils. For instance, if the soils are thin, they may be removed and replaced with compacted granular fill. Buildings may be constructed elevated a few inches above the ground by caissons driven through the soil to a depth that is below the zone of seasonal volume change. Or, the soil may be compacted at a moisture content several percent above optimum followed by building on a slab on grade.

Collapsible Soils (Cc)

Areas included in this category are underlain by fine-grained lake sediments, mainly silt (ML) and silty sand (SM). According to the Soil Hazards Survey, this soil is susceptible to excessive differential settlement when it becomes saturated. Landslides may also develop where these collapsible
soils exist on steep slopes; consequently, this hazard category is often combined with the potential landslide (Cl) category. In addition, where steep, unvegetated slopes are subjected to surface water runoff, rapid erosion results.

The main problem with structures on collapsible soils is differential settlement. The result of such settlement is cracked foundations and cracks and collapsed areas in streets.

The stability of collapsible soil foundations should be evaluated by an engineering geologist and a geotechnical engineer utilizing subsurface exploration and laboratory testing. On the gently sloping terraces, allowable bearing capacity should be designated to minimize settlement. Consolidation tests may be made to determine expected settlement. In steeper areas a slope stability analysis is needed to assure stable cuts and fills. Landscape irrigation should be minimized to avoid collapse and excessive erosion. As the fine-grained lake sediments are easily eroded, a civil engineer should develop a coordinated plan for controlling drainage and erosion.

Mitigating the hazard would depend on the thickness and bearing capacity of the collapsible soils and on the type of structure involved. For one or two story frame structures simply increasing the footing wall width may suffice. Heavier buildings may require precompaction or caissons. Cuts and fills on slopes should be carefully engineered. Attention to adequate drainage is essential.

Potential Landslide (Cl)

This category includes a variety of loose sediments, generally greater than five feet thick, which are situated on moderate to steep slopes. As evidenced by the many new slides in old landslide materials, old landslide deposits appear to have the highest potential for landslides. For this reason, hummocky terrain that appears to have been modified by old landslides has been indicated with hachure lines on the hazards maps. As previously discussed, areas of collapsible soils (Cc) on steep slopes are considered to be potential landslide areas. A landslide hazard may also exist where the slightly expansive soils (Be) occur on moderate to steep slopes. Finally, areas underlain by thick, loose soils of sand and gravel or colluvium on moderate to steep slopes are included in this category. The present stability of these slopes can easily be disturbed by cuts and fills for streets, utilities, and building lots.

Potential landslide areas should be investigated by an engineering geologist and a geotechnical engineer before development of the property. Investigation may include subsurface exploration and laboratory testing. The objective of the study is to define the thickness, extent, and type of potential landslide materials. A slope stability analysis should be performed to predict stable cut and fill slopes and, if feasible, to recommend a plan for safe development in the area.
Safe development in a potential landslide area would require careful adherence to engineered slopes and cuts, a very good drainage plan, and the possible use of caissons to carry foundation loads to stable bedrock.

Secondary Fault (Cf)

Secondary faults have developed as a result of previous movement on the main trace of the Wasatch Fault. These faults offset unconsolidated sediments which overlie the down-thrown block west of the trace of the primary Wasatch Fault. Evidence that they are active include geomorphically young features such as small topographic scarps visible in low sun-angle aerial photographs and sag areas with poor drainage.

A subsurface investigation should be conducted by an engineering geologist to establish the existence and accurate location of any faults.

Structures for human occupancy should not be permitted on the trace of a secondary fault.

Alluvial Fan (Da)

Alluvial fan areas exist at the base of several large canyons and steep drainages including Rock Canyon, Slide Canyon, and Slate Canyon. The portion of the alluvial fan deposits closest to the canyon mouth was designated as a hazard area. The outline of these areas was inferred from the presence of coarse fan deposits of sand, gravel, and boulders deposited by flash floods.

Flood boundaries are generally depicted on FEMA's flood insurance rate maps. However, before specific areas are developed, a professional civil engineer experienced in hydraulic engineering should conduct a flash flood study in the alluvial fan hazard area.

The alluvial fans are also potential debris flow areas. Assessment of this hazard will require a thorough geologic study of the stratigraphy and geomorphology of the fan sediments and conditions near the heads of the canyons.

Generally, it is not practicable to engineer safe development at the head of an alluvial fan area. These areas are subject to sudden flash flooding and possible debris (mud) flows.

Active Landslide (D1)

Many small to large, active landslide slumps were observed in loose, wet soils on gentle to steep slopes. In many cases the slides have been activated subsequent to cutting or filling on slopes in developed areas. Most active landslides are re-activated old landslide deposits or have developed on moderate to steep slopes in collapsible soils (Cc) or in loose colluvium.
Development in an active landslide area should be preceded by a thorough investigation utilizing subsurface exploration and laboratory testing to characterize the depth, extent, and type of landslide materials. The study should be done by an engineering geologist and a geotechnical engineer. A plan must be developed to stabilize the landslide and assure that development would not lead to future re-activation.

Construction of buildings and streets in the stabilized landslide area would require strict adherence to a drainage plan and engineered cuts and fills. Building loads should be transferred to stable ground or rock through caissons.

Primary Wasatch Fault Zone (DF)

The primary Wasatch Fault is a normal fault which dips west and is known to offset bedrock in a major structural zone along the Wasatch Front. Micro-earthquakes occur along the fault zone several times a year. However, no earthquakes associated with surface faulting are known to have occurred since settlement of the area began in 1847.

In many areas of the foothills several subparallel branches exist. The primary faults are not simple fault planes, but are characterized by irregular zones observed to be at least as wide as 100 feet consisting of black, clayey gouge, closely fractured rock, and springs. For this reason, branches of the Wasatch Fault were mapped as a minimum of 100-foot wide zones. Structures for human occupancy should not be permitted within 50 feet from a Wasatch Fault zone.

In areas where development is planned within 50 feet of a Wasatch Fault zone, an engineering geologist should conduct an investigation involving subsurface exploration, laboratory testing, and additional mapping to characterize the nature of the gouge zone and the location of the fault. In some areas, where branches of the Wasatch Fault are approximately located or projected, the investigation should prove or disprove the assumed position of the fault.

Types of geologic hazard induced by earthquakes include landsliding, rock falls, liquefaction, lateral spreading, and differential settlement. The hazard of rock falls, rock slides, or rock avalanches was not mapped because they could occur at any place along the hillside area below the steep rock slopes of the Wasatch Range. Rock falls of individual boulders or disrupted masses of rock that descend slopes by bounding, rolling, or free-fall are the most abundant earthquake-induced landslide. Rock falls were much in evidence in Challis, Idaho, during the Borah Peak earthquake in October, 1983.

HILLSIDE DEVELOPMENT ORDINANCE

The City of Provo currently has a hillside development ordinance in use. However, it makes little or no mention of geologic hazards. It was suggested to the City of Provo that the ordinance be amended to include the requirement that geologic hazards be considered prior to the development of any hillside property.
The suggested amendments included changing two ordinances. One was a chapter in the Hillside Development Standards and the other was the Sensitive Lands Development Ordinance. The purpose of these changes was to establish minimum requirements for the geologic evaluation of land based on the proposed land use and the Geologic Hazard Overlay Maps. A procedure was outlined where by an owner, developer or contractor is required to determine if the property to be developed is in a geologic hazards area. In relative Hazard Group A, a geologic letter is usually considered sufficient. However, if the geologic investigation turned up previously unknown problems, a more in-depth geologic report is required. The minimum requirements for geologic mapping, exploration, inter-related effects of the development and geologic hazards and mitigating measures are set forth in the ordinance.

Building restrictions related to the primary Wasatch Fault zone were determined to be that no structure for human occupancy will be placed across the trace of a secondary or a primary Wasatch Fault. Furthermore, the area within a minimum of fifty (50) feet of a primary Wasatch Fault zone, which was mapped 100 feet wide, shall be restricted of all buildings for human habitation. The City of Provo may also increase the set back for certain essential or critical structures such as hospitals and schools.

Two final suggestions were made in the ordinance which related to disclosure and acknowledgment statements of potential geological hazards. The purpose of the disclosure statement is to inform any prospective purchaser of the fact that the property is located within a geologic hazard area. The acknowledgment statement indicates the knowledge regarding the geologic conditions stated in the geologic report and that the owner, developer or any contractor is prepared to mitigate the hazards insofar as it is feasible and accepts any risk which remains.

SUMMARY

The City of Provo has experienced the effects of geologic hazards within the hillside area for the city over the years. In recent years, development in the hillside areas and the record precipitation during 1982-83 and 1983-84 has resulted in damaged homes, streets and utilities. To improve the hillside ordinance for the protection of the property owners as well as the city, a geologic mapping program was initiated. The suggested hillside development ordinance amendments requiring the property owner or developer to determine, acknowledge and mitigate the geologic hazards is a major step forward for the City of Provo. At the 1983 Utah Governor’s conference on geologic hazards it was generally agreed that the most useful information concerning geologic hazards for decision-makers and citizens would be multi-hazard maps depicting all known geologic hazards.

The City of Provo has become the first municipality along the Wasatch Front to institute the use of geologic hazard maps for hillside development.
REFERENCES


THE GEOLOGIC-HAZARD OVERLAY ZONE: A POTENTIALLY USEFUL TOOL FOR LAND-USE MANAGEMENT IN HAZARDOUS AREAS

Dr. James McCalpin

ABSTRACT

Land use management in private lands subject to potentially devastating geologic hazards is fraught with problems. State-level government agencies can delineate areas of high risk, describe the potential hazards, and suggest mitigating measures, but are not empowered with land-use control. Local governments (county, city) do have land use controls, but do not have the scientific expertise necessary to identify and manage hazard areas. A solution to this dilemma has been successfully implemented for about eight years in a county in the Colorado Front Range Urban Corridor, which has many characteristics similar to counties along the Wasatch Front. In the Jefferson County, Colorado, system, a special Geologic-Hazard (G-H) Overlay Zone was created which overlies pre-existing zone districts in areas identified to be potentially hazardous. The restrictions of the Overlay Zone are in addition to those of the underlying zone district. Prohibited uses within the G-H Zone include buildings intended for human occupancy, and any land use which significantly increases the danger from the geologic hazard. Certain provisional uses may be authorized by county technical personnel, such as roads, fills, utilities, or structures for livestock or storage. Zone boundaries are determined based on published geologic mapping and expert witness testimony, and are adopted within a County-Initiated rezoning process. Full disclosure is obtained because the zoning category appears on all property deeds and on maps on file in the respective Planning Departments. The zone may be altered or removed during an application to rezone if the applicant can supply positive verification that: 1) the hazard as mapped does not actually exist, or 2) the hazard has been successfully mitigated by some remedial measure. The advantages of the G-H Overlay Zone system are: 1) It can utilize Federal and State level geologic expertise while maintaining actual land-use regulation within local governments, and 2) zones may be modified once approved, but the burden of proof is placed on potential developers to prove the absence of hazard.

HISTORY OF LAND-USE LEGISLATION IN COLORADO

Background

A. Following the Second World War, several socio-economic trends radically

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altered the pre-war land-use patterns in Colorado. Suburban developments spread outward on the fringes of Front Range cities, reaching east to the plains and west into the adjacent foothills. Forest land in nearby mountain ranges and parks was subdivided for recreational homosites in such huge quantities that by the 1970's there were enough building lots for approximately three times the existing population. The "environmental movement" of the 1970's formed partly in response to poor land-use schemes rushed by developers through compliant local governments. By 1971, the public resistance against the large-scale, indiscriminate development of Colorado's precious scenery climaxed in a public referendum rejecting the 1972 Winter Olympics, the long-term effects of which were thought to be environmentally unacceptable.

Legislative Response of the State

8. Public pressure on the state legislative to enact stronger land-use laws resulted in several key pieces of legislation in 1973 and 1974. Senate Bill 35 required that numerous reports describing site conditions and development impact be prepared for all new land-subdivision involving lots smaller than 35 acres. House Bill 1041 (CRS 106-7-101 et seq) charged local governments to identify and selectively manage "areas of state interest", which included areas containing geologic hazards or commercial mineral resources. The exact wording stated "(a)III: In geologic hazard areas all developments shall be engineered and administered in a manner that will minimize significant hazards to public health and safety or to property due to a geologic hazard. The Colorado geological survey shall promulgate a model geologic hazard area control regulation no later than September 30, 1974.

"(b) After promulgation of guidelines for land use in natural hazard areas by the Colorado water conservation board, the Colorado soil conservation board through the soil conservation districts, the Colorado state forest service, and the Colorado geological survey, natural hazard areas shall be administered by local government in a manner which is consistent with the guidelines for land use in each of the natural hazard areas." (Colorado Revised Statutes, 1963), Section 106-7-202, (2)(a)III to (2)(b))NOTE: underlining added by author.

Importantly, HB 1041 did not specify how such areas must be delineated, nor exactly how they should be administered. Those choices were left to the discretion of the local governments, acting within their existing framework of land-use controls.

Because most local governments lack the technical expertise to identify hazardous areas, the Colorado Geological Survey in 1974 published a 146-page publication entitled "Guidelines and Criteria for Identification and Land-Use Controls of Geologic Hazard and Mineral Reserve Areas" (Rogers et al, 1974). This report described in detail the kind of methodology one should use in a geologic hazard ordinance in an appendix, so local governments could have some guidance in implementation. The Colorado Geological Survey also assisted in compilation and interpretation of existing geologic hazard mapping for any interested local government.
Local response to State Legislation

Local governments responded in different ways and at different speeds to the HB 1041 directive. Counties with severe development pressure, abundant geologic hazards and available large-scale geologic map coverage (like Jefferson County) made early and strict land-use controls. Other governments responded more informally or not at all, because, while HB 1041 set a compliance deadline for Nov. 1974 for having a management plan in place, it set no specific penalties for non-compliance.

Jefferson County was fortunate in being included within the Front Range Urban Corridor Project (FRUCP) of the U.S. Geological Survey. FRUCE products included not only the traditional 1:24,000-scale geologic maps, but also included surficial geologic maps and interpretive maps showing various geologic hazards. For example, the map folios for the Golden Quadrangle (Maps I-761-A through E) and Morrison Quadrangle (Maps I-790-A through C) contained five and seven 1:24,000-scale maps respectively (Table 1). The importance of such detailed, topical mapping cannot be overemphasized. Without it, it is unlikely that a strong functioning geo-hazard regulation would have the necessary detail to be of use at the individual lot scale, or the credibility to survive attacks by hostile developers, homeowners, or their lawyers.

TABLE 1: Examples of interpretive U.S.G.S. geologic maps produced by the FRUCP

<table>
<thead>
<tr>
<th>Map Number</th>
<th>Map Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>I-761-A</td>
<td>Surficial and Bedrock Geologic Map of the Golden Quadrangle</td>
</tr>
<tr>
<td>B</td>
<td>Map Showing Landslides in...</td>
</tr>
<tr>
<td>C</td>
<td>Map Showing Areas of Potential Rockfall In...</td>
</tr>
<tr>
<td>D</td>
<td>Map Showing Earth Materials That May Compact and Cause Settlement in...</td>
</tr>
<tr>
<td>E</td>
<td>Map Showing Man-Modified Land and Man-Made Deposits in...</td>
</tr>
</tbody>
</table>

The response of Jefferson County to the directive of HB 1041 was three-fold. First, the county in 1976, created a new zoning district, the Geological-Hazard (G-H) Overlay District. Second, the Board of Adjustment (BOA) was granted specific but limited powers to allow "specially excepted uses" within the G-H Overlay Zone that would ordinarily be prohibited. Third, the BOA adopted a more detailed Procedural Guide specifying exactly what kind of additional geologic data must be presented to support a request for a special exception use within a G-H Zone. In the following text, each of these three responses will be described in more detail.

THE GEOLOGIC-HAZARD OVERLAY DISTRICT

Natural hazard areas within Jefferson County are managed within three districts: Flood Plain Overlay District, Geologic Hazard Overlay District, or Wildfire Hazard Overlay District. Below, the text of the G-H District will appear in italics, with a brief commentary on each section following.
A.  INTENT AND PURPOSE

This district is intended to promote the public health, safety and general welfare of the citizens of Jefferson County; minimize the risk of loss of life and property; encourage and regulate prudent land use; permit only such uses that will minimize the danger to the public health, safety, welfare and property; reduce the demands for public expenditures for relief and protection of structures and facilities permitted in this district and regulate buildings and structures so as to minimize the hazard to the public health or property. (orig. 1-20-76; am. 6-15-76; am. 9-7-82)

Notably, the justification for the district mentions saving public funds needed for disaster cleanup as well as the more obvious preventative goals of protecting life and property.

B.  GENERAL PROVISIONS AND RESTRICTIONS

1.  Geologic Hazard District Overlays Other Zone Districts

The Geologic Hazard (G-H) Overlay Zone District shall overlay that portion of any other zone district, including Planned Development (P-D) Zone District, located in the geologic hazard area. The regulations of this district shall be construed as being supplementary to the regulations imposed on the same lands by any underlying zone district or other overlay district. When the regulations of this district conflict with any provision of the underlying zone district, the provisions of the Geologic Hazard Overlay District shall control; otherwise, the provisions of any underlying district shall remain in full force and effect. (orig. 1-20-76; am. 9-7-82)

The overlay concept is convenient in case all or part of the geologic-hazard designation is later removed from a parcel based on new geologic information. The zoning would then automatically revert to the underlying zoning wherever it is "uncovered." The G-H Zone can thus be seen as a temporary "cover" superimposed on underlying zones, a cover which may possibly be removed or altered in shape. The provisions of the G-H Zone, being almost always stricter than those of any underlying zone, must prevail in any conflict.

2.  Boundaries

This district shall encompass those general areas depicted on Geologic Hazard Overlay District Zoning Maps, more particularly defined by legal descriptions appearing in the County Commissioner Resolutions rezoning property to Geologic Hazard (G-H) Overlay Zone District. The boundaries of the Geologic Hazard (G-H) Overlay Zone District may be amended through the County's rezoning process when appropriate, based on site-specific geologic information. (orig. 1-20-76; am. 9-7-82)

The original G-H Zone boundaries were transferred directly from U.S. Geological Survey 1:24,000 interpretive maps (such as maps T-761 B. C, in Table 1) to the official larger-scale Zoning Department orthophotographs. The width
of boundary lines on original 1:24,000 geological maps is approximately 0.15 mm, which represents 3.6 m on the ground surface. Considering the uncertainty in positioning the line, plus the fact that residential lots are occasionally less than 15 m wide, it is apparent that original data sources of smaller scale than 1:24,000 cannot be realistically used.

Interpretive geologic maps such as I-761-C usually identify areas subjected to several degrees of relative hazard (high, moderate, low, very low), or, if mapping landslides, distinguish active from inactive landslides. The decision on which areas to G-H Zone must be realistic. Jefferson County decided to G-H Zone only the highest hazard areas on multi-category maps, and only known landslides (as opposed to probable or possible) on landslide maps.

Areas with lower hazard ratings may pose some risk but because that risk is harder to quantify in terms of boundaries or recurrence time, they are best left out of regulations. The legal descriptions referred to were generated by digitizing the hazard boundaries and fitting straight-line segments to them which could then be written in traditional metes and bounds terms. Inaccuracies occurring during this transformation compound initial uncertainties in line placement on the original source map.

Provisions must be made for modifying zone boundaries on the basis of new geologic or engineering data. This procedure of zone modification is discussed further in the general planning context in McCaZpin (in press). The burden of proof for zone modification rests with the applicant. If an applicant fails to present sufficient data for zone removal from his parcel, he can only develop: 1)subject to G-H Zone restrictions, or 2)pursuant to a special exception (variance) granted by the Board of Adjustment (discussed in a later section).

3. Hazard Description

Properties shall be classified according to 4 types of hazards:

A. Slope Failure Complex:

A geologic hazard which means a combination of more than one of the following geologic hazards: (orig. 9-7-82)

1) Landslide
2) Rockfall
3) Mudflow
4) Creep (orig. 9-7-82)

B. Landslide area

A geologic hazard which means a mass movement where there is a distinct surface rupture or zone of weakness which separates the slide material from more stable underlying material. (orig. 9-7-82)
C. Rockfall area

A geologic hazard which means the rapid free-falling, bounding, sliding, or rolling of large masses of rock or individual rocks. (orig. 9-7-82)

D. Subsidence area

A process characterized by downward displacement of surface material caused by natural phenomena such as removal of underground fluids, natural consolidation, or dissolution of underground minerals, or by man-made phenomena such as underground mining. (orig. 9-7-82)

Hazard zonation could conceivably include only one broad category, but subdivision of the G-H Zone into several different types of hazard areas has the advantage that: 1) it identifies to the public the specific hazard present, and 2) it automatically suggests certain remedial measures. The subdivision followed here results from those processes which are locally most important. Regions with a different suite of processes might adopt another classification.

4. Restrictions

Unless authorized under the provisions of Subsections C. or D. below, the following activities or uses are prohibited within the Geologic Hazard (G-H) Overlay Zone District. (orig. 9-7-82)

a. Permanent or temporary structures and buildings, including mobile homes and trailers but not including signs, fences, corrals or other open facilities for the containment of livestock. (orig. 9-7-82)

b. Physical improvements or modifications, such as roads, bridges, bike-ways, excavation or fills, solid or liquid waste disposal, utilities, or underground bulk storage of fuels. (orig. 9-7-82)

c. Other land use activities that significantly increase the danger from the geologic hazard. (orig. 1-20-76; am. 9-7-82; am. 12-28-82)

d. Restrictions a. through c. shall not apply to legal mining operations or accessory activities. (orig. 9-7-82)

The enforcement strategy in the G-H Zone was to initially prohibit the erection of almost any conceivable structure or improvement within the zone with specific language, and then to allow certain provisional uses later subject to specific constraints. Thus, sections a and b prohibit all structures and improvements and c provides a catch-all clause to cover uses not specifically mentioned previously. Section d exempts active mining operations which may be temporarily working near unstable slopes or subsidence zones, which would presumably be reclaimed pursuant to the state mined land reclamation standards.
C. PERMITTED USES AND ACTIVITIES

1. The following uses and activities are permitted:

   A. All land uses permitted by an underlying zone district, so long as the
      same are not in conflict with the use limitations set forth in subparagraph
      D.i. above (orig. 1-20-76; am. 9-7-82)

   B. Any land use activity permitted in an underlying zone when authorized
      by a plat approved by the Board of County Commissioners subsequent to the
      inclusion of said parcel within the Geologic Hazard (G-H) Overlay Zone
      District. (orig. 9-7-82)

The only uses permitted in the G-H Zone are those which are not prohibited
by the previous section, and which are allowed by the underlying zone district.
The only exception to this rule is that uses authorized by a plat approved
after the inclusion of said parcel into the G-H Zone are permitted. Basically,
this section says that most uses are prohibited by statute, but if you can get
the County Commissioners to approve any use on a plat, then it will be allowed.

D. PROVISIONAL USES

1. The County Engineer and County Geologist may authorize, in writing, certain
uses, which are permitted in the underlying zone district, and specified below,
providing that plans and design criteria have been approved by both the County
Engineer and County Geologist as having reasonably mitigated the potential
danger to persons and property of the geologic hazard, and that necessary
permits are obtained from the Engineering Division, the Planning and Zoning
and/or Building Departments prior to starting any earthwork, construction or
installation. (orig. 9-7-82)

   a. Roads, bridges, bikeways and similar improvements. (orig. 9-7-82)
   b. Excavations or fills (orig. 9-7-82)
   c. Utilities, above or below ground. (orig. 9-7-82)
   d. Energy collection devices, such as windmills or solar collectors. 
      (orig. 9-7-82)
   e. Structures exclusively for livestock. (orig. 9-7-82)
   f. Structures exclusively for bulk storage, such as silos. (orig. 9-7-82)
   g. Park or recreational uses without completed structures or buildings.
      (orig. 9-7-82)
   h. Accessory out buildings and garages. (orig. 9-7-82)
   i. Underground bulk storage of fuels. (orig. 9-7-82)

2. Under certain conditions as contained in Section 8C, the Board of Adjustment
may permit by special exception these uses allowed in underlying zone districts,
but prohibited by the provisions of Paragraph B.1. above and not provided for
in Paragraph D.1. above. (orig. 1-20-76; am. 9-7-82)

This entire section was added six years after the original ordinance was
adopted. It was felt there should be a provision made for uses which would
not expose the public at large to risk, yet which should undergo some review. For example, in a landslide area some small excavations or fills (section b) in certain areas might not cause renewed movement, whereas in other locations they might be expected to. Some provisional uses reflected special cases, such as i, which was written to exempt a public utility’s use of an abandoned underground coal mine for storage of natural gas (within a subsidence hazard zone).

E. BUILDING AND LOT STANDARDS

Building and lot standards, including height, minimum area, and setback requirements, shall conform to those of the underlying zone district. (Orig. 1-20-76; am. 3-7-82)

F. WARNING AND DISCLAIMER

Geologic Hazard (G-H) Overlay Zone Districts represent only those hazardous areas known to the County at the present time, and should not be construed to include all possible potential hazard areas. The provisions of this District do not in any way assure or imply that areas outside its boundaries, or land uses permitted within its boundaries, will be free from the possible adverse effects of geologic hazards. (Orig. 1-20-76; am. 3-7-82)

Section F reflects the incomplete knowledge of hazards by the local government, and makes it clear that there may be hazardous areas outside of those G-H zoned. Nor does the government guarantee that even uses permitted within a G-H Zone (such as signs, fences, Section B4a) will be free from damage.

THE BOARD OF ADJUSTMENT

As mentioned in Section D2 of the G-H regulations, the Board of Adjustment may grant special exceptions for the erection of structures within a G-H Zone. Section Li-C-10 of the Zoning Ordinance lists the powers and restrictions of the BOA in granting such variances. Briefly, it is the applicant’s responsibility to submit both a site plan and report describing the area, and a separate report detailing the proposed abatement program and its anticipated impacts.

To further guide the BOA (a non-technical body) in evaluating requests for variance, a Procedural Guide was drawn up by the Planning Staff enumerating exactly what kind of data should be required of an applicant to support a variance request. The nine-page Guide begins with a consideration of boundary interpretations. Applicants who dispute the location of a G-H boundary line must submit a map (scale 1” equals no more than 500 feet) showing the location of: 1) the existing zone boundary, 2) the property in question, and 3) a property survey by a registered surveyor. The County Geologist has 24 days in which to determine the actual boundary in the disputed area.

Application requirements for variances in the four types of G-H Zones vary, but generally include: 1) a map no smaller scale than 1” equals 100 feet, showing past occurrences of the hazardous process, 2) type of process, 3) rates
of movement, 4) location of buildings, 5) surface and subsurface drainage, 6) pertinent on-site boring and test data, and 7) a report describing remedial measures to be undertaken.

All items submitted must be prepared by a professional geologist experienced in the field of engineering geology. Because Colorado has no registration of geologists, local governments use the definition of geologist made in H.B. 1536 which requires a bachelor's degree in geology. However, because not all geologists are competent to analyze geologic hazards with state-of-the-art methods, some stipulation concerning engineering geologic experience is necessary. Finally, the county may require a performance bond to be deposited to assure that public funds do not have to be expended to remedy the effects of development within a G-H Zone.

ALTERNATIVE APPROACHES TO G-H ADMINISTRATION

Many local governments do not have the high-quality data base necessary to support a rigid zoning-type approach to G-H management. Clearly, the comprehensiveness and the strictness of any management plan should be appropriate to the detail and confidence of the geologic hazards mapping, as well as to the severity of the hazard. A continuum of approaches exist, ranging from purely voluntary participation by landowners, to the zoning system described in this paper. Table 2 shows some alternatives that might be considered in areas without good 1:24,000 scale geologic hazards mapping.

TABLE 2: Alternatives in G-H area management

<table>
<thead>
<tr>
<th>I. In undeveloped areas: control by zoning</th>
</tr>
</thead>
<tbody>
<tr>
<td>Have developer read geologic hazards publications.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>II. In undeveloped rezoned areas: control by building permit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Have homebuilder read geologic hazards publications.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>III. In developed areas: control by non-conforming use</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uses existing when G-H Overlay is superimposed automatically become &quot;non-conforming&quot; uses, subject to special restrictions.</td>
</tr>
</tbody>
</table>

INCREASING PREVENTION OF DAMAGE

INCREASING POLITICAL RESISTANCE
ACKNOWLEDGEMENTS

The author is indebted to the Jefferson County Planning and Zoning Staff for their advice and encouragement while he was County Geologist in 1981.

REFERENCES


The recent increase in attention given to geologic and natural hazards to development along the Wasatch Front is more than welcome. It is welcome by scientists and engineers who have been mapping and describing such environmental phenomena, it is welcome by planners buffeted by developers desiring to maximize profits from the land, and it seems to be welcome by a segment of the public whose curiosity, if not direct losses, have raised a new level of concern about individual protection and the common good.

THE WASATCH FRONT

Natural hazards to human occupancy occur to some degree in most parts of the world. Few places in the world, however, can match the Wasatch Front for a concentration of risks to human habitat. In a narrow belt a few miles wide and a hundred miles long, there are frequent reminders of the sensitivity and instability of the local landscape. Hazards ranging from drought to rising lake levels, to avalanches are well known in the region. This paper will emphasize the common and extensive terrestrial hazards listed in Table I. The items are arranged basically in descending order of frequency of occurrence, but generally in increasing impact per event.

Although there is much we don’t know about precise locations and degrees of risk, much of what we do know remains essentially inaccessible to planners and decision makers, for three reasons: (1) the information is scattered in the scientific and technical literature, (2) it is rarely presented in a usable language and format, and (3) it is typically isolated and un-integrated, failing to show the functional interrelationship of the several hazards.

Natural systems are interfunctional, and so are the hazards they impose. We can never accomplish multi-hazard mitigation until we establish a framework for integrated analysis of the several geotechnical and biophysical systems operating in the particular environment. Furthermore, man’s activities at and near the earth’s surface often alter the direction and/or rate of those interrelated events. Thus, human structures and alterations of the

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TABLE I Common terrestrial hazards along the Wasatch Front.

EROSION AND DEPOSITION
   Especially from disturbed slopes

GROUND WATER ALTERATION
   a. interception (especially from perched water tables)
   b. alteration of recharge, quantity and quality

SOIL INSTABILITY
   a. expansive soils
   b. collapsible soils

FLOODING (Snowmelt and Cloudburst)
   a. from natural slopes and channels
   b. induced by development

SLOPE FAILURE (Mass Movement)
   a. rock fall
   b. surficial sliding
   c. bedrock failure

SEISMICITY
   a. ground shaking
   b. surface rupture
   c. slope failure

There is, in addition, a set of biologically related losses resulting from human activity along the Front:

DETERIORATION OF WILDLIFE AND FISH HABITAT

DETERIORATION OF VISUAL QUALITY

INCREASE IN FIRE HAZARD

The landscape must be a part of hazard analysis and mitigation. The objective of this brief paper is to suggest a mechanism that can help to overcome the three difficulties listed above. Specifically, a case is made for relating the several natural hazards to units of land, where the interrelated processes and patterns can be mapped and analyzed.

COMPREHENSIVE TERRAIN UNITS

Inasmuch as the landscape is perceived, bought and sold, zoned, and built upon as units of land, and inasmuch as carefully identified terrain units can be expected to behave in particular ways under environmental
stress, a carefully chosen set of terrain units is a logical foundation for both the technical investigations and for the planning and management process. Such land units as are recommended are geomorphic "process-form" units, so called to emphasize their dynamic origin and on-going behavior.

The basic premise is that geomorphology is the integrating science of the earth's surface and development. It invokes the processes and patterns of tectonics and of external forces of gradation and mass movement, acting upon various earth surface materials and structures and terrain configurations. The on-going processes of geologic, hydrologic, pedogenic, biotic, and climatic forces that created a given landform feature continues to shape it. The science of geomorphology encompasses these interrelated processes in defining landform units in genetic terms. For a given geomorphic terrain unit, the materials and properties may be described and mapped. Keaton (1984) elaborates on a procedure for identifying such properties for geotechnical purposes.

Since the materials were deposited or placed in a certain manner, the conditions of runoff, infiltration, plant rooting, engineering properties, and performance under stress can be more or less stated. By identifying the distinctive landform units in genetic terms, such as alluvial fans, beaches, channels, and colluvial slopes, we may predict, to some extent, their behavior under certain conditions. We may model their performance given certain scenarios. Once the terrain units are identified and mapped, they may therefore become the foundation for decision making and even ordinance preparation. They may also become the foundation for further geotechnical research. Although the terrain units are originally defined using the principles of geomorphology, they eventually become instruments of multidisciplinary expression, and therefore may be called something like comprehensive terrain units (CTU's).

Boundaries of CTU's may be adjusted as new information is obtained, or they may be subdivided, representing subunits of particular character. But the principle remains intact that an undivided land unit is treated as essentially homogeneous. If a fault scarp is found to lie along the boundary of a unit, a subunit zone may be drawn along the edge, if terrain behavior and management performance would seem to justify it. If perched water is found in various places through a unit, the unit may remain intact, with an appropriate warning note. Thus, the maps and reports describing them become a "living document," an evolving and enlarging instrument for analysis and management.

Since man can alter landscape conditions, both natural and man-induced hazards must be identified. Both can be identified on a CTU basis. Mitigating measures may also be best defined within the context of those homogeneous landform features. Ordinances may also be tied to the terrain unit. The CTU, then, may become the basic structure for an integrated multi-hazard mapping, assessment, mitigation, and management program.

Maps and data from the procedure are readily entered into a geographic information system (GIS) for data management and planning. Within the
computer, areas (CTU's), lines, and points may be analyzed and accessed for decision making. Updating is facilitated on the computer base.

A PILOT PROJECT

A pilot project employing this framework was applied recently in Davis County (Ridd et al. 1980). By way of brief illustration, a few highlights are presented here. Figure 1 is a schematic diagram of the typical Davis County landscape, with some terrain units identified. Figure 2 is a CTU classification hierarchy of the terrain. Figure 3 shows the intersected research efforts of the 18-member technical team through five phases (A through E) of physical environmental analysis. Findings by each team group were shared at each phase before proceeding to the next phase. Preliminary terrain units (PTU's) were prepared in phase A through geomorphic interpretation of aerial photos and field reconnaissance, and shared subsequently by all investigators. They served as an analysis and communication medium throughout. Finally, they were adjusted and renamed CTU's and then served as the basis for subsequent work on the project. Figure 4 shows the development capability or risk assessment rating sheet. Note the division of risk types: on the left, those risks imposed by nature on a human structure or development, and on the right, those risks introduced or aggravated by man himself. The rating scale 0-5 is relative, and nonquantitative. The four-volume Davis County report describes the "nature of impact" and the "rating criteria" for each. Figure 5 is an outline of the model ordinance prepared by the project.

In summary, the CTU is presented not as a panacea, but as one of many possible steps to assist in the collection, analysis, and presentation of hazard information. The main value is that it is site specific and graphic. It facilitates integration of knowledge of natural processes and technical information. It provides a continual "mailing address" for new information. It facilitates hierarchical subdivision as new technical detail is discovered at specific sites. It serves as a communication bridge between technical specialists, planners, elected officials, and the public.

CITATIONS


FIGURE 1 Graphic representation of selected terrain unit types.
*C Channels

1* All Class 1 channels are distinguished as Terrain Units. These are deeply entrenched in the upper bench and delta areas with sufficient relief and length to require separate management.

2* Major Class 2 channels are distinguished as Terrain Units. These are entrenched in the upper bench and delta areas with sufficient relief and length to require separate management.

3 Major Class 3 channels are distinguished as Terrain Units. Most of these are in the Weber delta area, and are abandoned former Class 1 channels, now detached from mountain drainage.

*In the mountain area, an "m" follows the C1 or C2 channel Terrain Units to distinguish the units from the channel Terrain Units in the bench area.

FIGURE 2 Terrain unit classification.
FIGURE 3
**Figure 4: Summary development capability matrix.**

<table>
<thead>
<tr>
<th>Terrain Elements</th>
<th>Impact Elements</th>
<th>Potential Impact on Development</th>
<th>Potential Impact on Development</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flooding</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Faulting (Ground Rupture)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock Fall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slope Failure</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compaction/Consolidation</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Liquifaction</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ground Water Interception</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Earthquake Hazards</td>
<td>Single Family Dwellings</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Critical Facilities</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Change in Ground Water Recharge</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Induce Flooding</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Induce Soil Erosion</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reduce Wildlife/Fisheries</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Increase Fire Risk</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reduce Visual Quality</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reduce Mountain Access</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loss of Sand/Gravel Resource</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
100. PURPOSE

200. DEFINITIONS
201. General Terms
202. Impact Elements

300. SCOPE AND APPLICATION
301. Applicability
302. Terrain Units

400. DEVELOPMENT REGULATIONS
401. Buildable Areas
402. Impact Elements and Ratings
403. Hydrologic and Geologic Standards and Requirements
   403.1 Flood hydrology
   403.2 Surface drainage
   403.3 Erosion control
   403.4 Geologic conditions
   403.5 Soil characteristics
   403.6 Slope stability
   403.7 Rockfall
404. Natural Resource Protection Standards and Requirements
   404.1 Wildlife corridors
   404.2 Fire hazard
   404.3 Mountain access
   405.4 Visual quality
   405.5 Sand and gravel
405. Impact Rating Table

500. REVIEW AND APPROVAL PROCESS
501. Pre-Application Conference
502. Conceptual Review and Approval
503. Preliminary Review and Approval
504. Final Review and Approval

600. ADDITIONAL PROVISIONS
ABSTRACT

This paper reports the results of one phase of a comprehensive study of flood hydraulics and flood hazard mitigation on alluvial fans. A conceptual model of an urban development on an alluvial fan was constructed and subjected to various levels of flooding. Among the factors studied were the design of streets to act as flood drainage channels, design of protective flood dikes, construction of elevated buildings on piers, incorporation of drop structures among individual house lots, efficiency of flood interceptor channels, and concentration of buildings and/or streets. Conclusions are drawn regarding the relative effectiveness of different flood damage mitigation methods, design for certain flood damage prevention structures, safety factors for floodproofing structures, and recommendations for proper urban landscaping to minimize flood damage.

INTRODUCTION

With the increasing pace of urban development, such flood-prone areas as alluvial fans become human habitats and quickly develop into cities and urban communities. Periodic severe flooding in such areas can cause extensive damage and loss of life. Urban communities which are planned for flood-prone areas can be designed to allow the "safe" passage of floods up to a certain size. Safe passage in this context means that flood damage will be confined to the streets and the exterior of the city, thus preventing the loss of life and property damage likely to occur in the interior of buildings.

The purpose of this study was to evaluate the performance of selected urban landscaping measures with respect to their flood mitigation efficiencies, and to provide preliminary design criteria for the development of new areas for residential or other purposes.
THE MODEL AND THE TEST CONDITIONS

An undistorted 1:30 scale model of a conceptual city arrangement was constructed and tested. The test variables included depressed streets and elevated lots, vertical drops between lots, protective dikes upstream of the lots, buildings elevated on piers, different street side-slope protection materials, and interceptor channels upstream of the development. The tests were conducted for a maximum estimated 100-year flood. To simulate the location of the city on an alluvial fan, two flow entrance conditions were tested. Figure 1 shows the model arrangement which assumes the city is down-fan and the flow attacks the city in several small channels. In an alternate arrangement, the concrete flow channels were combined into one larger channel to simulate the effect of the city being up-fan and the flood water being confined to one channel after leaving the watershed. The houses were wooden boxes filled with sand to reproduce a realistic house weight. The streets had concrete bottom linings and different side-slope protection materials. The bed material for the fan surface and the lots was a fine sand with a median size of 0.57 mm. The same material was fed into the flow at the concrete upstream channels in pre-measured quantities. The model had a general slope of 3.5 percent.

The procedures and conditions studied are shown in Table 1 in summary form. Runs No. 1, 2, 4, and 5 were conducted to study relative effects of some mitigation measures. Run No. 3 shows the effect of the same runoff volume as Run No. 4 if it can be routed at a smaller discharge and longer time base. Runs No. 6 and 7 show the effect of the interceptor channel. Runs No. 8, 9, and 10 show relative effectiveness of additional mitigation measures and sediment load effect. Runs No. 11 and 12 show the effect of lot/house density. Run No. 13 represents a 100-year flood for a city located up-fan.

DETERMINATION OF UPSTREAM DIKE HEIGHT AND STREET FLOW CAPACITY

To study the effect of the upstream protective dikes and provide design guidelines for future development areas, it is necessary to formulate proper equations based on existing hydraulic principles. Figure 2 shows a control volume for a general street orientation case. The processes occurring within the control volume are quite complex and can be formulated only by using some simplified assumptions. The validity of the assumptions is checked by utilizing the collected data.

The law of conservation of linear momentum is applicable to the control volume of Figure 2. Experience shows that at flow rates high enough to overtop the protective wall, sediment quickly fills in upstream of the wall and eliminates its effective height. Bearing this in mind, the momentum equation in the direction parallel to the streets becomes:

$$\frac{d}{dt} \left( I \omega \square^2_o - I \omega \square^2_s - I \omega \square^2_L \right) = \rho \left( o_b \omega_s + o_l \omega_l - o_o \omega_o \cos \theta \right)$$

where the variables are defined in Figure 2. It is assumed that the approach conditions and the geometry variables are all known. This leaves four
FIGURE 1 Model dimensions and orientation
TABLE 1  Summary of the tests.

<table>
<thead>
<tr>
<th>Run No.</th>
<th>Prototype's Hydrograph $Q_{max}$ (cfs)</th>
<th>Sediment Feeding</th>
<th>Test Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2,400</td>
<td>Yes</td>
<td>Multiple channel, no protection for street berms</td>
</tr>
<tr>
<td>2</td>
<td>2,400</td>
<td>Yes</td>
<td>Multiple channel, some houses placed on piers</td>
</tr>
<tr>
<td>3</td>
<td>1,200 (5 1/2 hr)</td>
<td>Yes</td>
<td>Multiple channel, dike protection upstream</td>
</tr>
<tr>
<td>4</td>
<td>2,400 (2 3/4 hr)</td>
<td>Yes</td>
<td>Multiple channel, dike protection upstream</td>
</tr>
<tr>
<td>5</td>
<td>2,400 (2 3/4 hr)</td>
<td>Yes</td>
<td>Upstream dikes removed</td>
</tr>
<tr>
<td>6</td>
<td>1,200 (5 1/2 hr)</td>
<td>Yes</td>
<td>Interceptor channel upstream</td>
</tr>
<tr>
<td>7</td>
<td>3,800</td>
<td>Yes</td>
<td>Same as No. 6</td>
</tr>
<tr>
<td>8</td>
<td>1,500</td>
<td>Yes</td>
<td>Street berms protected by concrete; no upstream dike protection</td>
</tr>
<tr>
<td>9</td>
<td>3,000 (street capacity)</td>
<td>Yes</td>
<td>Concrete berms and 2 upstream V-shaped dikes</td>
</tr>
<tr>
<td>10</td>
<td>2,400</td>
<td>Excessive</td>
<td>Same as No. 9</td>
</tr>
<tr>
<td>11</td>
<td>1,200</td>
<td>Yes</td>
<td>Middle street closed. Only 2 streets for flow to pass</td>
</tr>
<tr>
<td>12</td>
<td>1,200</td>
<td>Yes</td>
<td>One row (2 houses) added in the middle block</td>
</tr>
<tr>
<td>13</td>
<td>4,000 (100-year flood)</td>
<td>Yes</td>
<td>Single channel, other conditions same as Run 2</td>
</tr>
</tbody>
</table>

unknowns in Equation 1, namely, $V_S$, $D_S$, $V_L$ and $D_L$. Knowing these, $Q_S$ and $Q_L$ can be calculated. To solve for the four unknowns, three other equations are needed. It is therefore assumed that:
1. Chezy's equation for flow in the streets applies; assuming $R_S = D_S$:

$$V_S = C_S D_S S \cos \theta$$  \hspace{1cm} (2)

where $C_S$ is Chezy's coefficient.

2. Continuity equation holds for the control volume, letting

$$Q_o - Q_r = W_D V_o \cos \theta; Q_S = W_S V_S D_S;$$ and $Q_L = W_L V_L D_L$; then

$$W_L V_L D_L = W_o D_o V_o \cos \theta - W_S V_S D_S$$  \hspace{1cm} (3)

3. Flow on top of the protective wall is critical, leading to
Combining Equations 3 and 4 and rearranging,

\[ V_L = \left( \frac{(W_0 D_0 V_0 \cos \theta - W_s V_s D_s) g}{W_L} \right)^{1/3} \]  

Substituting Equation 5 in Equation 4 and rearranging,

\[ \frac{D}{L} = \left( \frac{W_0 D_0 V_0 \cos \theta - W_s V_s D_s}{W_L \sqrt{g}} \right)^{2/3} \]  

Substituting Equations 2, 5, and 6 into Equation 1 and noting that \( \gamma = \rho g \) and simplifying

\[ \frac{g}{2} \left[ \frac{W_0 D_0^2}{2} - W_s D_s^2 - W_L \left( W_0 D_0 V_0 \cos \theta - W_s C_s \sqrt{s} D_s^3 \right)^{4/3} \left( \frac{W_L}{\sqrt{g}} \right)^{-4/3} \right] = W_s C_s^2 \left( \frac{D}{s} \right)^2 + \left( W_0 D_0 V_0 \cos \theta - W_s C_s \sqrt{s} D_s^3 \right)^{4/3} \left( \frac{g}{W_L} \right)^{1/3} \]

Equation 7 can be solved for \( D_s \) only if flow overtops the protection wall and floods the lots. Before Equation 7 is used, a criterion is necessary to determine if the flow is large enough to flood the lots. This is determined using the total energy at the junction between the street and the protection wall for the straight model,

\[ \frac{V_0^2}{2g} \frac{V_s^2}{2g} < \Delta h \]

where \( \Delta h \) is the height of the protective wall above the street level. The validity of Equation 8 is verified in Table 2.

Table 3 shows the summary of calculations based on Equation 7. The agreement between measured and computed \( Q_S \) and \( Q_L \) is reasonably good, indicating the validity of Equation 7.

There is one extra case to consider. In the case of V-shaped protective dikes, the tests show that the criterion of Equation 8 will not apply. In this case, using Manning's equation for flow alongside the dike and assuming \( W_r = W_s \) in Figure 2, the flow depth can be found from:
TABLE 2 Verification of the effectiveness of protective wall.

<table>
<thead>
<tr>
<th>Run No.</th>
<th>Measured</th>
<th>Avg. $V_s$ (fps)</th>
<th>$\frac{V_s^2}{s+2g}$ (ft)</th>
<th>$\Delta h$ (ft)</th>
<th>Wall Overtopped?</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.021</td>
<td>1.6</td>
<td>0.061</td>
<td>0.167</td>
<td>No</td>
</tr>
<tr>
<td>4</td>
<td>0.045</td>
<td>2.8</td>
<td>0.167</td>
<td>0.167</td>
<td>Almost</td>
</tr>
<tr>
<td>5</td>
<td>0.070</td>
<td>2.6</td>
<td>0.174</td>
<td>0.065</td>
<td>Yes</td>
</tr>
<tr>
<td>8</td>
<td>0.042</td>
<td>2.1</td>
<td>0.110</td>
<td>0.065</td>
<td>Slightly</td>
</tr>
<tr>
<td>10</td>
<td>0.052</td>
<td>2.7</td>
<td>0.162</td>
<td>0.167</td>
<td>No</td>
</tr>
<tr>
<td>11</td>
<td>0.044</td>
<td>2.2</td>
<td>0.122</td>
<td>0.065</td>
<td>Yes</td>
</tr>
<tr>
<td>13</td>
<td>0.062</td>
<td>2.8</td>
<td>0.184</td>
<td>0.167</td>
<td>Yes</td>
</tr>
</tbody>
</table>

TABLE 3 Calculations of flow distribution using Equation 7.

<table>
<thead>
<tr>
<th>Run No.</th>
<th>Measured</th>
<th>Results Obtained from Equation 7</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.54</td>
<td>0.053 2.58 0.41 0.023 0.851 0.23</td>
</tr>
<tr>
<td>8</td>
<td>0.26</td>
<td>0.038 2.19 0.25 0.008 0.513 0.05</td>
</tr>
<tr>
<td>11</td>
<td>0.20</td>
<td>0.048 2.46 0.24 0.477 0.007 0.04</td>
</tr>
<tr>
<td>13</td>
<td>0.22</td>
<td>0.069 2.95 0.20 0.034 1.050 0.21</td>
</tr>
</tbody>
</table>

$D_r = \left( \frac{Q_o^n}{1.49 \frac{0.7}{S \sin \theta}} \right)^{3/5}$

The value of Manning's $n$ in Equation 9 would be that of the fan surface since flow in this case occurs on the fan. The overtopping criterion will be Equation 8, with $D_r$ substituted for $D_S$.

Since the purpose of this discussion is to provide some design criteria, it should be considered that Equation 7 only provides a tool for determining the average distribution of flow between the streets and the lots. Tests show that the flow in one street may be as high as double that of another street. Therefore, if streets are to be designed to act as drainage routes for flood water, a factor of safety of 2 is recommended for them.
For newly developing areas, heights of protective dikes or raised lots can be determined using Equation 8 together with proper factors of safety, so that expected floods would be confined to the streets. If the protective heights become excessive, the same criteria and methodology explained above can be used to determine the size of the flood that can safely pass through the city.

Local dikes built upstream of the lots could be very efficient means of preventing floods from inundating the lots. A series of V-shaped dikes seems to be more efficient in preventing damages than straight dikes. The direct attack of the water on the straight dike causes deposition of sediment upstream of the dike and quick overtopping of water. Combined with other factors, such as depressed streets with concrete (or other nonerodible) side slopes, a series of V-shaped dikes may be sufficient in guiding an entire 100-year flood through the streets.

LOCAL SCOUR

The flow entering a lot soon concentrates into narrower sections and erodes a channel, flowing at higher depths and velocities. Such a flow can quickly erode around houses and expose foundations, causing severe damage. Test results show that the local scour around the corners of houses can be as high as seven to eight feet in the prototype. For severe floods it can be assumed that flow over the lots concentrates between houses and the resulting channel soon assumes the general slope of the region. The depth of local scour around a sharp-edged building can be found from the equation developed by Karaki et al. (1974) as

\[ D_{ls} = 2.15 D_l \left( \frac{a}{D_l} \right)^{0.40} N_F^{0.33} \]  

where \( D_{ls} \) is the depth of local scour in ft; \( D_l \) is the depth of flow between the houses; \( a \) is the width of an embankment, in this case half the dimension of the building in perpendicular direction to flow; and \( N_F \) is the Froude number of the flow. The local scour around the piers of elevated houses can be found from

\[ D_{ls} = 2.0 D_l \left( \frac{a}{D_l} \right)^{0.65} N_F^{0.43} \]  

where \( a \) is the diameter of a cylindrical pier, and other terms are as defined before. In this case the flow should be considered as occurring on the entire width of the lot. Equations for other pier shapes are available in the same or other references. Equations 10 and 11 can be utilized to determine damage due to local scour.

The material which is used in construction of the streets is a factor to consider among mitigation measures. Test results show that if street side-slopes are made of concrete, the carrying capacity of the streets can be
improved considerably, and even in the absence of upstream protection (Run No. 8), floods that would otherwise inundate the lots can be safely carried through the streets. Figure 3 shows this case, with slight flooding of one lot. When protective dikes are added to this case (Run No. 9), much higher flow rates could still be passed through the streets without damage to the lots. Figure 4 shows the case of unprotected street side-slopes with upstream dikes. The flow rate in Figure 4 is 0.24 cfs, compared to 0.30 cfs in Figure 3.

**FIGURE 3** Concrete street side slopes, Run No. 8

**PERFORMANCE OF MITIGATION MEASURES**

**Performance of Drop Structures**

Drop structures tested in the model were nonerodible walls built between every two lots or between a lot and a transverse street where the lot level has to drop. The top of the drop structure was level with the higher lot. Such a drop structure cannot prevent flooding of the lot, but it can reduce the damage because the flow on the lot cannot establish a channel and cannot attain any slope. Furthermore, presence of the drop structures protects the
lot downstream, across the transverse street. This is mainly due to a reduction in water velocity as it passes over the drop and enters the transverse street rather than attacking the downstream lot. Such drop structures are aesthetically inconspicuous and inexpensive and could be effectively used to reduce flood damage. The effectiveness of this mitigation measure is further discussed later.

Performance of Interceptor Channels and Local Dikes

An interceptor channel, the centerline of which is shown in Figure 2, was constructed and tested in Runs No. 6 and 7. Soon after the beginning of the flood, the channel almost completely filled with sediment and failed. This is inevitable, since the slope of the interceptor channel tested in the model was smaller than the slope of the approaching stream, and the sediment carrying capacity of the interceptor channel was less than the incoming sediment. If an interceptor channel is not adequately designed to carry flood water and sediment and to resist erosion, little success could be expected other than delaying the time of arrival of a flood onto the downstream city.

Performance of Elevated Structures

When a lot is inundated, houses built on piers provide a much better passage for water such that the flow is shallower underneath, causes less local erosion, and does not inundate the house. Figure 5 shows a 100-year flood case. Much of the severe erosion under the elevated house is caused by the street flow eroding the entire lot. The flow over the lot itself causes little local erosion around the piers. If the street side-slopes had been...
FIGURE 5 Erosion of lot underneath elevated buildings.

adequately protected, much of the severe erosion underneath the house would have been prevented in this case.

RELATIVE EFFECTIVENESS OF MITIGATION MEASURES

The purpose of this section is to present some comparative effects of the different measures. In order to understand the efficiency of each protective measure, it is useful to understand the types and extent of damages and flood behavior that can occur on a flooded city.

Figure 6 shows deposition upstream of a straight protective wall. In this case the discharge has not been large enough to overtop the wall. The amount of deposition is significant, but due to transverse flow towards the streets the deposition region ends somewhere upstream of the wall and the structure protects the adjacent lot. Figure 7 shows deposition upstream of a V-shaped protective wall, without overtopping. The deposition has covered almost half of the original height of the wall and a higher discharge would have caused overtopping. However, the specific shape of this structure is such that high velocities are established along the wall, washing off some of the possible deposition and making the structure safe against overtopping at higher discharges than a straight wall can withstand. Equation 8 applies in this case, with $D_s$ replaced by $D_r$ calculated from Equation 9. Another advantage of the V-shaped dikes, if used in a series, is that they tend to equalize the discharge among the different streets. Thus, each street could
be designed with smaller capacity and excessive flow accumulation on one portion of the city could be avoided.
Along with the upstream protective dikes is the requirement that the streets should have the capacity to carry the flood water through the city to some type of drainage downstream of the city. Damages include deposition of sediment in the streets, damage to or destruction of cars, and damage to the landscape or any other structure that may exist in the streets. However, it can be safely assumed that if a flood is confined to the streets, the amount of damage it causes is going to be several time less than if the entire city were flooded. It is also important to realize that a combination of several of the tested measures is necessary to produce effective and reliable flood mitigation. This fact has been clearly observed through the conducted tests. Table 4 shows the comparison of four of the tests.

**TABLE 4 Comparison between Runs No. 3, 8, 9, and 10.**

<table>
<thead>
<tr>
<th>Run No.</th>
<th>Berm Protection</th>
<th>Upstream wall Protection*</th>
<th>Sediment Supply</th>
<th>Max. Prototype Flow without Flooding the Lots</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>no</td>
<td>yes (combination)</td>
<td>normal</td>
<td>&lt;1200 cfs</td>
</tr>
<tr>
<td>8</td>
<td>yes</td>
<td>no</td>
<td>normal</td>
<td>1500 cfs</td>
</tr>
<tr>
<td>9</td>
<td>yes</td>
<td>yes (2 V's)</td>
<td>normal</td>
<td>3000 cfs</td>
</tr>
<tr>
<td>10</td>
<td>yes</td>
<td>yes (2 V's)</td>
<td>excessive</td>
<td>2400 cfs</td>
</tr>
</tbody>
</table>

*Straight and V-shaped walls have been used for different tests.*

Since flooding of the lots does not necessarily start from upstream, but can also start from the street (see Figure 4), protection of the street berms increases the size of flood that can pass through the city without causing damage to the lots. Effectiveness of the measures greatly increases when they are combined, such as in Run No. 9. Finally, since sediment supply is a major cause of flooding of the lots, its effect is shown in Run No. 10. Figure 8 shows values of scour and deposition after Run No. 5.

When the lots cannot be protected from flooding, other mitigation measures become necessary. Elevated houses and drop structures are the two measures tested for this case. Figure 9 shows the transverse street and a comparative effect of drop structures. The lot in front has no drop structure, so the flow entrenches the lot, attains high velocities, and then flows across the transverse street and attacks the downstream lot. The lot in the back of Figure 9 has a drop structure, so flow cannot entrench the lot into a defined stream. At the same time, water flowing over the drop structure does not have enough momentum to attack the downstream lot. As mentioned before, the drop structures are economically and aesthetically efficient structures for reducing flood damages.
Elevated houses are considerably more expensive to build, yet do not guarantee safety against severe floods. In the event of a relatively small flood, the elevated house would be safe against inundation. But in case of a large flood, if flow channelizes between the piers or if the lot is eroded due to insufficient berm protection, the piers may be undermined, causing very severe damages. Therefore, in this case also, a combination, such as elevated buildings and drop structures, would prevent channelization of flow between the piers. Berm protection would also be useful in preventing lot erosion and exposure of pier foundations.

Interceptor channels were among the mitigation measures tested. Of course, whenever feasible, it is desirable to control floods on a whole-fan level and divert them away from cities. It has been shown that if not adequately designed to transport the flood water and sediment and able to resist erosion, interceptor channels cannot provide adequate protection for a city. The presence of interceptor channels may be quite effective for containing relatively small flows that would otherwise flood the city. If used for this

FIGURE 8 Erosion and deposition pattern in Run No. 5
FIGURE 9 The transverse street after a 100-year flood

purpose, the channel must be constantly maintained and the deposited sediment must be dredged out of the channel immediately after each flood.

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

In this study, a conceptual model of a city was constructed and tested, with different street orientations and lot/house concentrations. The effect of several flood mitigation measures and the relative effectiveness of each measure were tested. Highlights of the conclusions are listed below.

1. When whole-fan measures, such as debris basins or levee/channel systems, fail to protect developed areas from flooding, effective local measures can be utilized to mitigate flood damage. Such measures essentially involve carrying the flood water through the city streets in order to minimize or eliminate lot and house inundation. Raised lots and depressed streets with well-protected berms are very effective in carrying flood water through the city. Alignment of the streets in the direction of incoming flow and addition of upstream protective dikes to this system considerably improve the flood-carrying capacity of streets, and
presence of flow-delaying structures further assists in "safe" flood passage through the city.

2. When lot and house inundation cannot be prevented, other measures can be utilized to minimize damages. Elevated buildings and drop structures between adjacent lots are among such measures, and both are effective in reducing damages within certain limits.

3. Flood mitigation measures on alluvial fans should be carried out in a comprehensive program of flood awareness and flood protection. An active program of maintenance of the flood mitigation structures should be an essential part of any protection system. These factors should be incorporated into the development of any new alluvial fan area for residential or other purposes.

ACKNOWLEDGEMENTS

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REFERENCES


INTRODUCTION

A range of mitigation measures for landslide and debris flow hazards is presented herein. Hazard mitigation for landslide and debris flow is very site specific and depends on the soil conditions, topography, hydraulic conditions, and nature of manmade activities. Without question, the most effective mitigation measure for these hazards is avoidance. The tool of avoidance is local land use controls such as zoning restrictions, building codes, and grading requirements. Hazard mitigation measures can be placed into five general categories: 1) relocation, 2) regulations, 3) insurance, 4) warning, and 5) remedial measures.

Hazard mitigation can be applied in two general cases; for sparsely developed areas and for well developed areas. Unfortunately the only measure mentioned above that is geared towards directly reducing damage to existing development is remedial measures. Since this paper emphasizes hazard mitigation for developed areas, it will emphasize the use of remedial measures. Hazard mitigation for developed areas includes protection for both buildings and roads.

The first section of this paper discusses selection factors for hazard mitigation measures. The second and fourth sections go over available landslide and debris flow hazard mitigation measures, respectively. Section 3 presents a case study of a landslide study performed by the U.S. Army Corps of Engineers. Section 5 discusses the use of public education to inform the local property owners of hazard mitigation.

It is very important to emphasize that all landslide and debris flow remedial measures must be designed by qualified professional geologists and engineers.

SECTION 1: SELECTION FACTORS FOR HAZARD MITIGATION MEASURES

There are many factors to take into consideration when developing measures to mitigate landslide and debris flow hazards. They can be broken...
down into five general categories; 1) Engineering feasibility, 2) Economic feasibility, 3) Legal/regulatory conformity, 4) Social acceptability, and 5) Environmental acceptability.

**Engineering Feasibility**

The most important factor should be the physical effectiveness of the measure. A thorough geologic and hydraulic analysis of the site conditions should be made and measures that can withstand the forces generated by the hazard be developed. Furthermore, the designs should insure that a mitigation measure does not divert the problem elsewhere and thereby make a bad situation worse.

**Economic Feasibility**

Before implementation, an assessment should be made as to the benefits of the plan versus the costs of implementing it. The elimination or reduction of future damages should exceed the costs and annual maintenance of the mitigation measure. Intangible benefits such as physical well-being, human suffering, etc. may also need to be considered.

**Legal/Regulatory Conformity**

The legal implications of implementing mitigation measures in terms of liability must be considered. If a property owner takes an action that damages adjacent property he may be liable. Also the local zoning laws must be checked to determine if the measure can conform to local building and safety codes.

**Social Acceptability**

A mitigation measure should be acceptable to the community and adjacent neighbors. Aesthetically, it should not be an eyesore. For example if an earthen ring dike was placed around a home, it could be unacceptable to neighboring property owners.

**Environmental Acceptability**

The proposed action should not cause adverse environmental impact.

**SECTION II: LANDSLIDE MITIGATION MEASURES FOR DEVELOPED AREAS**

The only category of mitigation measure that can reduce damages to existing development (building, roads, rail lines, etc.) is remedial measures. It is very important to recognize that all landslide remedial measures should be designed and implemented by qualified professional engineers and geologists with the approval of appropriate local agencies. This chapter will focus on remedial measures. Measures which are used for the prevention and control of landslides depend upon the type, magnitude, and frequency of soil movement, topography, hydraulic conditions, and the nature of manmade
installations or activities. Landslide mitigation measures can be classified into four major categories: 1) Avoidance, 2) Water Control, 3) Excavation/Regrading, 4) Retaining Structures.

1. Avoidance. Complete avoidance of unstable terrain is the best solution to dealing with extremely hazardous slopes. This method has the advantage of being the safest alternative, especially when adjacent stable areas can be used instead. Obviously, this method has a very limited application to existing development and roads. Nevertheless, avoidance of some remaining undeveloped area is an alternative to preclude future slide problems.

An alternative to complete avoidance of unstable terrain is regrading under an approved grading and building code. The Department of Building and Safety for the City of Los Angeles has adopted General Grading Requirements. The regulations require submission of plans and specifications, posting surety bonds, inspections during grading operations, and conformance to adopted design standards. Development of future sites within this code is a suitable method to permit development and to minimize landslide problems.

2. Water control. Water plays such a significant role in many different types of landslides that techniques for its control and removal from the earth materials are the most universally applied methods to prevent landslides and stabilize hillsides. The basic principles of landslide control are to reduce the stress within the system and to increase the shear resistance of the earth materials. It is water pressure which increases stress within the system. It is also water which reduces the shear resistance of the earth materials.

Surface water controls. Uncontrolled surface water runoff is the single cause of heavy erosion, undercutting of drainage structures and potentially dangerous landslides. Surficial water is prevented from entering the landslide area or removed from the ground by techniques such as the following: divert water from streams, springs, and sheet wash through pipes or lined open trenches; drain water that collects in depressions; regrade slopes to allow more uniform drainage into lined ditches or water disposal systems; and fill cracks and other openings with grouting or sealant to prevent water loss or penetration. Sealants include such impermeable materials as clay, concrete, or bitumens. Paving an area to enhance runoff and prevent infiltration is also a suitable technique.

Vegetation. Control of surface water runoff alone is not sufficient to prevent erosion and stabilize slopes. Proper vegetative cover plays an extremely important role in the process of slope stabilization. The plants utilized should meet three selection criteria: fast growth, fire retardant, and deep-rooted. Of the three, the most important is the last. Deeply seated root systems create a stable superstructure, capable of sustaining heavy erosive processes. Excessive watering of these plants reduces deep-root system growth since there is no need to penetrate deeply for nutrients.

Subsurface water control. Subsurface water control techniques are used to dewater earth materials and to lower subsurface water levels; thereby
increasing the shear resistance of the upper layers and stabilizing the slope. Horizontal drains and tiles are effective in reaching subsurface water within 5 to 10 feet of the surface. Longitudinal trench drains are constructed by placing a slotted or perforated pipe near the bottom of a trench filled with appropriate pervious filter aggregate. Horizontal drains are especially effective in soils that are granular, uncemented, and very permeable. Vertical drains, holes, and wells take the form of various types of borings that become sumps or collector systems for water removal commonly below the water table.

3. Excavation. These methods are some of the first to be used where slopes have already failed and imminent danger threatens. The remedial measures involved are deliberate cuts and modifications of the slope's geometry and gradient. The more severe modifications often mean the loss of property, necessitating right-of-way cuts or property acquisitions. At many sites, the use of excavation methods to control landslides is limited due to the space constraints.

Removal of slide. This method is the most drastic and can be used only for small slides or those commensurate with a benefit-to-cost ratio. The advantages are the elimination of the hazard and the guaranteed safety for developments. The disadvantages are that only small slides are capable of being removed, and the method is often the most costly.

Unloading head of the slide. The use of this method depends upon accessibility. It may be one of the least expensive of the excavation remedial measures. When feasible, the material removed from the head may be used in the regrading process and placed at the foot where it may act as a buttress.

Regrading and slope reduction. Knowledge of stable angle of repose is all-important in regrading. The type of slide and the soil mechanics determine the angle and, in turn, the angle is critical for calculating right-of-way costs. Another aspect of slope regrading is to eliminate irregularities and depressions that might serve as collectors for surface water, which would add pore-water pressure and increase the potential for landslides.

Hillside benches. Construction of berms or terraces relieve stress on steep and long slopes and when the toe of the slide is severed. Whether the benches are designed to be horizontal, to slope toward or away from the slope, or if there is a need for several berms depends upon the nature of the earth materials, and the slope length and steepness. The Department of Building and Safety for the City of Los Angeles requires benches on cuts and fills. Standards adopted by the Department for interceptor terraces and benches require reverse slope berms and concrete-lined drainage ditches.

4. Retaining structures. Retaining structures are generally restricted to small areas and at the toe of a predictably unstable zone. Their purpose is preventive rather than corrective, and their construction is generally tied into the original design of the slope cut. Their character is dependent upon the local situation as determined by cost analysis, and their
composition may include timber, concrete and grout, stone, metal ribbing, solid fencing, and gabions. They are also used as a last resort when the developed site is so near a landslide that something must be done. To be effective, such structures should be used with other preventive methods and should be constructed generally with materials that are easily drained, because such structures invariably lead to water table changes that increase seepage pressure. The various types of retaining structures are discussed in Figures 1 through 6.

For a more detailed or technical discussion of landslide remedial measures there are several texts and papers that deal with the subject. See references 17, 19, 28, and 30.

SECTION III: BALDWIN HILLS - A CASE STUDY

In July 1979, the U.S. House of Representatives adopted a resolution requesting the Corps of Engineers to perform a study of the landslide problems at Baldwin Hills, a community in the city of Los Angeles, CA. The resolution requested the Corps' Los Angeles District to determine what measures are feasible in the Federal interest to alleviate the landslide problems in the Baldwin Hills area. This is probably the only time the Corps has performed a landslide study under an authorization for flood control.

Landslide Factors

The results of the landslide study showed that there were many factors that contributed to the slope instability. The primary factors were the soil properties, steep gradients, excessive precipitation and erosion, and manmade activity that caused increased steepness of the slope. The major reason for slope failure in the Baldwin Hills area was the steep hillside gradients; many of them the result of construction activity. Analysis of slope damage charts indicate that over 70% of all slopes steeper than 1 vertical on 1.5 horizontal have sustained damage whereas less than 1 percent of all slopes flatter than 1 vertical and 4 horizontal have sustained damage.

Repair Method

The methods chosen for slope repairs were terracing and/or the use of retaining walls, depending on site characteristics. Terracing has been an effective method for repairing slopes in the area in the past; it is most effective on slopes with a horizontal distance of 25 feet or more (see Figure 7). Retaining walls can be used effectively on slopes of shorter horizontal length and on steeper slopes unsuitable for other methods. A combination of retaining walls and terracing can also be employed. The repair methods chosen include provisions for drainage devices. Other drainage from around the house should be directed away from slopes and into storm drains.

Study Results

In order for the Corps of Engineers to undertake a project such as the repair of slope problems at Baldwin Hills, the benefits of the project must
Gravity Walls

FIGURE 1a Gravity walls consist of unreinforced concrete. The mass of concrete is what provides the support and limits their use to low walls.

Semi Gravity Walls

Reinforcing Steel

FIGURE 1b Semi-gravity walls incorporate reinforcing steel for tensile strength. The addition of the steel reduces the mass of concrete required and expands the use of semi-gravity walls to moderately high walls.

Cantilever Walls

Stem

Base Slab

FIGURE 1c Cantilever walls resemble an inverted T, with each projecting portion acting as a cantilever. The walls are generally made of reinforced concrete. This type of wall is economical for walls of low to moderate height (about 20 to 25 feet).
Counterfort Walls

FIGURE 2a Counterfort walls are suitable for high retaining walls (greater than 20 feet). Additional support is gained from brackets joined between the face of the wall and the base slab.

Buttressed Walls

FIGURE 2b Buttressed walls are similar to the counterfort walls except the brackets (buttresses) are exposed and the backfill is on the opposite side. This type of design is not commonly used because of the exposed buttresses.
Gabions

FIGURE 3a Low to medium high walls are constructed of gabions - wire baskets filled with rock. The baskets are used as building blocks and are tied together for continuity of the structure. Gabions allow significant drainage without major loss of soils, can be covered with vegetation, and are easy to install. They are relatively inexpensive and may be implanted by hand.

Sheet Piles

FIGURE 3b For higher backfills mechanically driven mechanically driven into existing soil to form the retaining structure. Stability is derived from ground penetration by the piles and, in the case of higher walls, by means of tie rods attached to suitable anchors. Vibrations caused during placement may weaken existing soil structure. Sheet piles are relatively expensive, require heavy equipment for installation, and are esthetically unpleasant.
Earth Buttresses

FIGURE 4a Earth buttresses are composed of rock or earthfill material that is added to the toe of the slide to provide extra weight to increase the shear strength of the original materials. Placement of the added burden is done only after soil mechanics studies have shown that the weight will not increase the driving force of the slide. When properly designed, however, the added support feeds back into the system and gives extra resistance to materials above. They are effective for slip-out types of movement and are commonly used in embankments. Most failures occur because the buttress did not extend sufficiently deep or was not coupled to subsurface drainage structures. Earth buttresses require large amounts of space at the toe of the slide and are not suitable in restricted areas.

Shear Keys

FIGURE 4b Shear keys are essentially prisms of compacted fill placed to support only certain sections of the slide. This is a variation of the buttress, but the excavation and insertion of new materials occur at different levels of the slide, such as near the head where shear keys seem most effective.
Batter Boards

FIGURE 5a Batter boards are metal or wooden lathes supported by vertical posts. The posts are driven deep, spaced frequently and of adequate size to provide the counter support needed. Drainage is sloped away from the batter board face toward a lateral drain.

Bin or Crib Walls

FIGURE 5b Bin or crib walls are formed by timber, precast concrete or prefabricated steel members and filled with granular soil. Crib walls are suitable for walls of low to moderate height (maximum of approximately 20 feet) which are subjected to moderate earth pressures. No surcharge load except the earthfill should be placed directly above the crib wall.
Reinforced Earth

A vertical wall is formed by precast concrete modules supported by friction held bands compacted into the earth. The reinforced earth itself is often imported backfill which provides easy drainage. Reinforced earth is suitable for high walls, and restoration of lost real estate (yards, roads, etc.). The wall is generally esthetically pleasing, and relatively inexpensive for capacity.

Welded Wire Walls

Welded-wire walls are similar to reinforced earth structures except a patented wire mesh system is used. Good drainage is essential and an advantage is that it may be installed by hand in restricted areas.
TERRACING REPAIR METHOD

FIGURE 7 Terracing repair method. This method includes brick, block or board (batter board) walls. A critical feature of this method is the depth of the footing and the surface area of the support below slope. Otherwise the tipping or bending movement will exceed static strength and the wall will fail. (This is a common occurrence at Baldwin Hills, especially with batter boards.)
outweigh the costs. An analysis of the benefits and costs showed that the approximate $11,710,000 cost of performing the slope corrections would be far greater than the resulting benefits; reduction of potential damages of about $8,652,000. Therefore, no Federal action can be justified.

Public Information Brochure

As a part of the study the Corps printed a brochure that describes the reason landslide problems exist, and the kind of landslide problems Baldwin Hills has (figure 8). A general terracing repair method and a general retaining wall repair method was described (see Figure 9). The brochure emphasized that slope repair measures should be designed and carried out by qualified professionals such as engineers and geologists with the approval of appropriate local agencies. Many property owners in Baldwin Hills have joined together and hired a professional engineering firm to design remedial measures to mitigate the landslide hazard.

SECTION IV: DEBRIS FLOW HAZARD MITIGATION MEASURES

It is important that a distinction be made between a debris flow and a debris flood. Debris flow is a form of wet landslide where the flow has sufficient viscosity to include large boulders and rocks as well as soil materials (also known as mudflow). Its consistency is similar to wet concrete. Debris floods contain soil particles with a greater proportion of water and a lesser quantity of large boulders and rocks (also known as mudflow). A debris flow can become a debris flood if the proportion of water increases relative to the debris and sediment particles. Debris flows generally stop flowing upon reaching the canyon mouth while debris floods will continue past the canyon mouth.

In the mitigation of debris flow-flood hazards, there is no standard solution for all problems because every situation is different. Local geologic and manmade features have a greater effect on debris flow dynamics than on clear water flooding. There may be no right or perfect solution because the perfect solution is usually too expensive. For debris flow-floods in developed areas mitigation measures for structures depend on their location and orientation near the canyon mouth or alluvial fan. Discussion of mitigation measures for developed areas will include the following measures; 1) debris basins, 2) debris fences, 3) rail and timber barrier, 4) building deflection devices, and 5) street orientation.

Debris Basins

Debris basins are intended to trap sediment from debris flows-floods at the canyon mouth while releasing less destructive (less viscous) flows downstream. Debris basins are designed with enough capacity to contain the debris production from a major storm event or from a partly-detached landslide with a quantifiable volume of debris. They can be the preferred solution if the estimated debris production is too great to be handled by other mitigation tools and if the area below the canyon mouth is highly
FIGURE 8 Factors contributing to slope failures.
FIGURE 9a General terracing repair method used for cost analysis. This method includes brick, block or board (batter board) walls. A critical feature of this method is the depth of the footing and the surface area of the support below slope. Otherwise the tipping or bending movement will exceed static strength and the wall will fail. (This is a common occurrence at Baldwin Hills, especially with batter boards.)

FIGURE 9b General retaining wall repair method used for cost analysis. The above diagrams show general repair measures only. Before undertaking any slope repairs, you should employ the services of qualified professionals and have the work performed in accordance with all local regulations.
developed. But debris basins can be expensive and should only be used if there is enough development below the canyon mouth to justify its cost.

Debris Fences

These structures are intended to mitigate debris flow hazards by trapping large rocks and debris while permitting water and sediment to pass through. These structures can be helpful in converting debris flows into debris floods. These structures will reduce impact damages to buildings and reduce debris deposition on yards and streets. But if the debris flows are too great the fences may be destroyed or buried (see Figure 10).

Rail and Timber Barriers

These structures can be used to protect smaller storage structures along well defined watercourses. They have been used by the Los Angeles County Flood Control District.

Building Deflection Devices

These devices are used when debris floods are anticipated and will not be as effective against debris flows. Deflection devices consist of wood or metal shields for windows and doors and wooden/masonry deflection walls to protect the uphill side of buildings. These devices should be used in conjunction with a proper street diversion and arming system and must be part of an overall street orientation system as discussed in the next paragraph.

Street Orientation

Depending on how the existing subdivision is laid out street orientation or diversion may be effective. Streets can be aligned to carry the debris floods downslope. This would require deflection or diversion of flows toward the street. Heavy arming of street sides may also be required. There are many factors that influence the effectiveness of this measure. The slope and carrying capacity of the street must be adequate to handle the debris flood. Existing street orientation or alignment is important in determining the effectiveness of street diversion. Care must be taken to provide street arming to prevent scour and erosion. A combination of protection/deflection devices, some kind of elevation of buildings on piles and street diversion can be combined for maximum effectiveness (see Figure 11).

There are many good discussions on mitigation measures for debris flows-floods. FEMA lists the various measures and their use and effectiveness against certain factors (reference 1). References 11, 14, and 29 also discuss various mitigation measures for debris flow-flood hazards.

SECTION V: PUBLIC EDUCATION

It is critical that the general public be made aware of the hazards of landslide and debris flow areas. The primary consideration should be to
Fence 6-8 ft (1.8-2.4 m) high with vertical bars 18 in. (50 cm) apart

Reinforced Buttress  Steel Beams  Slab

Debris-Storage Reservoir

Cut Area

6-ft (1.8 m) high Buttress

Spread footing designed for overturning moment

irrigation Ditch

Effect: Large boulders and debris are stopped; smaller material and mud is washed through and over structure and continues as a debris flood rather than as a debris flow.

FIGURE 10 Structural defense for arresting and separating debris flows (diagrammatic sketch).
Source: Reference 14.
FIGURE II Homes protected from major damage. Source: Los Angeles County Flood Control District.
publicize the potential dangers involved to prospective homeowners, real estate interests, developers, and local zoning agencies. Ideally, local zoning and building codes should prohibit or restrict construction on identified hazardous areas. However, in many cases, development has already occurred and it is too late for local landuse regulations to reduce damage to existing development. In this case, it is important to inform existing property owners of the nature and extent of the landslide or debris flow hazard. The homeowner should be made aware of the various options available to him to mitigate the hazard. Measures ranging from relocation to remedial measures should be discussed.

A good way of educating the property owner as to what mitigation measures are available to protect him against slope instability and debris hazards is to distribute public information brochures. These brochures should discuss the nature of the hazards, the causes and the range of potential mitigation measures. It is imperative that a disclaimer or warning be made that all mitigation work be designed by a professional engineer and/or geologist, and that local regulations and building codes be conformed to. The brochure should discuss the relative effectiveness and cost of available measures in sufficient detail for the homeowner to make some intelligent decisions. Examples of three good brochures can be found in those issued by the Corps of Engineers (Ref. 26), SCS (Ref. 22), and L.A. County Flood Control District (Ref. 11).

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PROTECTIVE DEVICES FOR MITIGATING DEBRIS-FLOW DAMAGE

by Robert A. Hollingsworth

ABSTRACT

Debris flows generated by soil slumps present a hazard to developments in hillside areas. Where structures are located below natural slopes, protective devices should be incorporated into the design.

Determination of the type and location of protective devices for a particular site should consider the site constraints, including the topography, depth to bedrock, location and direction of concentrated flow, and expected volume of debris. Where adequate area is available to contain the expected debris, retaining devices such as debris basins, retaining walls with freeboard, and slough walls may be utilized. Other situations require the use of self-cleaning protective devices such as deflection walls to direct debris around structures.

Sidehill drains and debris fences can be used in conjunction with other devices. Sidehill drains are placed upslope of other devices to intercept and channel sheet flow runoff and erosion debris. Debris fences are commonly used in canyons to retard the rate of the debris flow, retain a portion of the debris and break up the flowing mass, thereby allowing the escape of any air trapped under the flow.

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DRAINING HILLSLOPES AS A PREVENTIVE MEASURE FOR LOCAL LANDSLIDES

by S. A. Jenab and C. G. Clyde

ABSTRACT

Landslides are rare but hazardous events of nature. Each year they cause much damage, and force many people from their homes throughout the world. Recent slides in Ecuador, South America, and Thistle, Utah, are examples of such events.

One type of landslide may occur where layered subsurface strata near steep hillside topography causes a perched water table to develop from accumulation of groundwater above layers of low permeability. In recent years in Cache Valley, Utah, landslide events of this type have occurred which have endangered canals, highways, farmland, etc.

Water affects the hillslope landslide problem in several different ways:

a. By increasing the weight of the overlying material and thus increasing the driving force.

b. By decreasing the cohesion and the shearing strength of the material.

c. By eroding of hillside material by the outflowing groundwater above the low permeability layer.

Excessive groundwater seems to be a main cause of many slides occurring in Utah and other locations. If excess water could be drained from the critical seepage area of these hillslopes, the driving force would be decreased, the shearing strength of the soil mass would be increased, the hillside erosion would be reduced and often the slides would be prevented.

In this paper, some theoretical drainage approaches will be made to determine the extent of the necessary dewatering zone, the rates of discharge, and the size and arrangement of drain pipes for effective hillside drainage. Suggestions will be made for the measures needed to prevent such localized hillslope landslides.

S. A. Jenab and C. G. Clyde are with Utah Water Research Laboratory, Utah State University.
ALT E R N A T E DRAINAGE AND RECHARGE OF SURFACE DEPRESSIONS
AND UNDERGROUND AQUIFERS IN UPPER BASINS: AN
ALTERNATIVE IN WATER RESOURCES MANAGEMENT AND LEVEL
CONTROL FOR TERMINAL LAKES

by S. A. Jenab, J. M. Bagley, J. C. Batty,
C. G. Clyde, and J. P. Riley

ABSTRACT

Terminal lakes are repositories for all the residual runoff and salts
from the hydrologic drainage basins for which they become a sink. During wet
cycles, inflow rates to terminal lakes increase. Consequently, lake water
levels rise and inundate the near-shore areas. Conversely, in dry years,
lake water levels decline and shorelines recede. Municipalities and indus-
tries around a lake are especially vulnerable to extreme fluctuations of lake
water levels.

Alternating drainage and recharge of wetlands and groundwater aquifers
in the upper basin helps control the fluctuation of terminal lake water
levels, reduces flood hazards to cities, industries, and agricultural lands,
and conserves water for municipal and irrigation purposes. Thus, draining
wetlands and shallow groundwater aquifers provides:

a. water for irrigation,
b. underground storage space for recharging in wet years,
c. water for preventing extreme declines of terminal lake water levels
   in dry years,
d. additional hydropower as the water flows downstream.

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Recharging the surface depressions and shallow aquifers provides:

a. replenishment of the depleted underground water storage,
b. control of rapid rise of terminal lake levels and the associated inundation of the near-shore areas,
c. moderation of flood damages along rivers.

Recorded water levels in the Great Salt Lake in Utah have fluctuated between extremes of 4211.50 above mean sea level in 1875 and 4191.35 feet in 1961. These levels represent 13.6 million acre-feet of water more and 8.0 million acre-feet less than the historical average lake volume at a level of 4201.1 feet. If the lake level is to be controlled between 4202.0 and 4195.0, a total of 12.6 million acre-feet of water would have to be stored elsewhere during a high runoff series of years such as occurred during the 1870s. In the drier cycles such as that of the early 1960s, 2.4 million acre-feet of water would need to be released to the lake to hold the level at 4195.0 feet.

In the Upper Bear River basin, many aquifers offer promising storage-recharge possibilities. In Cache Valley, the aquifers have an estimated storage capacity of nearly 40 million acre-feet and reduction of 100 feet in the groundwater elevation would provide 2 million acre-feet of storage. In the Upper Bear River basin, the upper 100 feet of saturated deposits in the valley groundwater reservoirs contain an estimated 12 million acre-feet of water which could be withdrawn by wells and drains.

The alternate draining and recharging of wetlands, surface reservoirs and groundwater reservoirs offers a long-term alternative for controlling terminal lake levels which has much merit.

INTRODUCTION

Fluctuation of the surface level of terminal lakes is a serious problem for those who live and work along the shores. Terminal lakes are at the very bottom of the hydrologic system and accumulate residual waters and salts from their drainage basins. Upstream tributary diversions, depletions, and return effluents all affect the quantity, quality, and timing of water ultimately entering the lake. All inflow must ultimately be evaporated. During wet cycles, the lake enlarges. Consequently, the water level rises and the near-shore areas are inundated. In dry periods, water levels decline. Municipalities and industries around a terminal lake are especially vulnerable to extreme fluctuations of surface water levels.

The Great Salt Lake in northern Utah is the principal terminal lake which is a remnant of ancient Lake Bonneville. Historically, the surface elevation of Great Salt Lake has fluctuated extensively, and both high and low extremes cause a variety of economic losses. High water levels also cause major environmental damage to waterfowl feeding and nesting areas. In the 1870s the water level rose to its highest recorded point of 4211.5 feet above sea level, and in the 1960s it dropped to its lowest elevation of 4191.35 feet. This fluctuation in water surface level of more than 20 feet
corresponds to a change in lake surface area of over 960,000 acres (from 580,000 to 1,540,000 acres), a difference that almost equals the entire lake surface area at 4200 feet.

Variation of the lake water level is a serious problem for businesses or recreation developments that require location near the shore of the lake. Figure 1 shows the elevations which directly affect various uses of the shoreline area of Great Salt Lake.

In the last 3 years the water level of the Great Salt Lake has risen rapidly and in 1983 rose to an elevation of 4205. During the summer of 1983 the water level dropped only 0.4 feet and then went up again. As shown in Figure 2, the Great Salt Lake level reached 4209.4 in July 1984. The elevation-damage curve of Figure 3 indicates more than 350 million dollars damage to the facilities adjacent to the Great Salt Lake in the water year 1983-1984.

If the precipitation trend in 1984-85 is similar to 1983-84, the Great Salt Lake water level may approach 4212 which would be the highest elevation ever recorded.

ALTERNATIVES FOR TERMINAL LAKE LEVEL CONTROL

There are only two long-time permanent solutions to the problem of rising water levels in terminal lakes, such as the Great Salt Lake, namely, (1) increase the consumptive use (evaporation and/or transpiration) of water in the basin, and (2) export excess water from the basin (transbasin diversions). Some short term relief from damages can be obtained by temporary storage of excess water above or below the ground either at upstream or near-shore sites or by diking threatened areas. Permanent relief to the high water problem, however, comes only by disposing of the water by evaporation or exportation.

Schemes to increase evaporation can take the form of pumping excess water out of the lake into adjacent reservoirs, thus increasing the area from which evaporation can occur. An alternative way to increase water loss by evaporation and transpiration is to capture the water before it reaches the lake, store it for a time if necessary, and transport it to lands needing irrigation to raise crops. Here there are benefits both from lowering the lake level and from the value of crops grown. However, such a system, once established, requires that the needed water supplies be available both in wet years and dry years. Thus, excess waters must be stored at added expense in wet years for release to irrigation use in dry years. Underground storage as well as surface storage should be considered.

Transbasin diversion of good quality water only in wet years also can reduce maximum terminal lake levels. However, those downstream in the receiving basins might already have adequate supplies during wet years. Thus, storage might be required to allow release of waters at times welcomed by the downstream basins.
Figure 1: Lake elevations which directly affect possible uses of Great Salt
Figure 3. Showing the direct and indirect damages to facilities adjacent to Great Salt Lake due to high and low elevations (Utah DNR 1983).
In line with these general options for lake level control, some specific proposed control alternatives are outlined as follows:

1. Pumping from the lake to nearby storage areas:
   
   Pumping lake waters into adjacent areas to increase the evaporation surface is one means of controlling the water level. This alternative can be put in place quite quickly and is economically feasible if the costs of facilities and their operation are reasonably low compared to the benefits from the resulting reduction in lake level.

   This alternative has been studied by Allen et al. (1983) for controlling the Great Salt Lake water level. Because this alternative seems to be the only one that can have a substantial effect on lake level in a short time, it is being actively pursued by the Utah Division of Water Resources.

2. Store water in surface reservoirs:
   
   Storage in upstream surface reservoirs not only provides a means of regulating flows to the terminal lake, but also develops supplies for meeting the potential needs of all kinds of upstream uses.

   For the Great Salt Lake water level control, Allen et al. (1983) studied the feasibility of building additional dams and reservoirs to supply water to additional irrigation projects in the basin. For the 26 projects studied, the authors concluded that only two have a benefit-cost ratio greater than one considering only the irrigation project benefits. These two projects were the enlargement of Butler Dam to a total reservoir capacity of 1 million acre-feet, and the construction of Wyuta Dam with a reservoir capacity of 2.4 million acre-feet. The Utah Division of Water Resources is appropriately examining these sites and others as part of the surface storage alternative.

3. Divert water to another basin:
   
   Exportation can be a very effective method of protecting the exporting basin from flood damage if receiving basins are readily accessible and not subjected to increased flooding hazard as a consequence. This alternative sometimes requires building dams, pumping water to higher elevations, and constructing expensive water conveyance systems. If the economics of the project, the hydrologic and hydraulic conditions of the receiving basins, and the institutions and water rights can accommodate, this alternative can offer a promising solution.

   The diversion of water from the Bear River (the major basin tributary to the Great Salt Lake) to the Snake River Basin faces two major limitations: 1) the Snake River itself is usually at flood
stage when export of water from Bear River would be most expedient, and 2) flood waters in the Bear River are already appropriated for hydropower purposes. A possible route for a gravity diversion of water from Bear River to the Snake River Basin exists. The legal and institutional arrangements to protect Utah power rights and to mitigate flood damage would have to be worked out.

4. Diking:

Diking of all or individual segments of the lake shore is a long-term option for flood protection. Of course, this action does not control the lake water level, but only protects the shore area from inundation. Feasibility studies are needed to determine the cost-effectiveness of the various segments.

Various diking options for the Great Salt Lake were addressed in a recent feasibility-level engineering study completed by James M. Montgomery, Consulting Engineering, Inc. and a team of sub-consultants (Montgomery 1984). The study evaluates several on-shore (or perimeter) diking alternatives to protect specific facilities, such as wastewater treatment plants. In addition, the study looks at in-lake diking alternatives which provide additional management options by compartmentalizing the lake. The dikes were designed to protect significant structures at two particular lake levels, namely 4212 feet and 4217 feet.

5. Alternate drainage and recharge of surface depressions and underground aquifers in upper basins:

This alternative has not received significant attention in the past for controlling the water level of terminal lakes. However, the procedure might be a viable option when proper conditions exist. In certain instances, upper basin aquifers are large and often there are recharge opportunities. Recharge can occur in wet years with pumping in dry years to meet water demands. In this way, terminal lake levels can be controlled, flood hazards reduced, and irrigation demands met. Hydrogeological, institutional, and economical studies of the projects in each particular location are needed to determine their feasibility.

Analysis of the recorded water levels of Great Salt Lake shows the mean water level to be 4201.1 feet while the highest and the lowest levels ever recorded were 4211.50 and 4191.35 feet, respectively. From the elevation-volume curve of the Great Salt Lake, it can be seen that at the mean level, the volume of water stored in the lake is 16.6 million acre-feet. The greatest volume stored was 13.6 million acre-feet more than the mean volume and the least was 8.0 million acre-feet less than the mean. If the lake level is to be kept between 4195.0 and 4202.0, a total of 12.6 million acre-feet of water would have had to be stored elsewhere during the high flows of the 1870s. In the driest historical period of the 1960s, 2.4 million acre-feet would have had to be released to the lake to hold the level at 4195.0.
Thus, it would be necessary to store 2.4 million acre-feet of water in groundwater and surface water reservoirs in the wet cycles for pumping and release to the lake during dry periods. The remaining 10.2 million acre-feet (12.6 less 2.4) (less evaporation losses) would have to be pumped and used for irrigation of additional lands and other beneficial consumptive uses but at a rate sufficient to have the needed storage space available should the lake begin to rise again.

At first glance, the recharging of 12.6 million acre-feet and discharging of 2.4 million acre-feet from groundwater aquifers seem to be large volumes. However, if the historical hydrogram of the Great Salt Lake water level (Figure 2) is reviewed, there are apparently wet and dry cycles between 110 years and 25 years. If we consider the shortest cycle and assume that recharging and discharging should be done in half of the cycle, the maximum recharging rate would be 1.0 million acre-feet/year, and the discharging would reach a maximum of 192,000 acre-feet/year. These rates represent a rough estimate of the extreme values that might occur.

This use of the groundwater aquifers would both relieve flooding and conserve water during dry periods. The extra storage would take care of the additional dry cycle evaporation due to higher lake water level with some left over for irrigation. For wet and dry cycles which are shorter than above (more intermittent), the manipulated volumes decrease. Conjunctive management of ground and surface water would occur by pumping or draining the valley and flood plain aquifers and storage reservoirs during the periods of water shortage, and artificially recharging them during wet cycles.

If extensive marshlands were made a part of this store and release potential, the habitat in nesting areas of bird refuges could be adversely affected. Other questions of water rights and interstate compacts also would need to be studied carefully.

Many aquifers within the Bear River basin offer promising subsurface storage possibilities. In Cache Valley, the aquifers contain an estimated 40 million acre-feet of storage capacity, but not all of this capacity is economically feasible or legally available to use. It is estimated that the upper 20 feet of aquifer in Cache Valley could produce about 500,000 acre-feet of water. In the Bear River basin the upper 100 feet of saturated deposits in the valley groundwater reservoirs contain an estimated 12 million acre-feet of water which could be withdrawn by wells and drains (Schlotthauer et al. 1981).

In Cache Valley, reduction of 100 feet of the groundwater piezometric pressure would yield about 2 million acre-feet of water. To obtain this quantity, the pumping would require 1.93x10⁶ megawatt hours of energy (2 million acre-feet to be lifted 40 feet on average for discharging into the Bear River, and 50,000 acre-feet of water to be lifted 80 feet for agriculture). For comparison, to pump the same volume of brine from the Great Salt Lake into the Puddle Valley area requires 11.455x10⁶ megawatt hours of energy (2 million acre-feet lifted 250 feet). Thus, the brine pumping scheme would use more energy by a factor of six without considering potential
benefits accruing from additional crops that could be grown on lands supplied by the additional irrigation water under a conjunctive use management system.

ADVANTAGES OF DRAINING AND RECHARGING GROUNDWATER AQUIFERS FOR TERMINAL LAKE LEVEL CONTROL

1. Prevents the lake water level from rising above a selected maximum level or falling below a minimum level.
2. Provides room for recharging the excess water and thus reducing flood damages throughout the basin.
3. Provides security for investments on the shoreline areas.
4. Enhances the agricultural potential of wet and low-lying areas by lowering their water tables.
5. Reduces wasteful reservoir evaporation by storing water in underground reservoirs.
6. Provides increased flows in the rivers during dry periods and enhances power generation when it is most needed.
7. Replenishes the aquifers in areas where water levels are dropping due to pumping.

IMPORTANT GROUNDWATER AQUIFERS IN THE SALT LAKE BASIN

The following is a brief description of the major groundwater aquifers in the Salt Lake basin which could be used to control fluctuations of the Great Salt Lake within certain limits.

a. Gem and Gentile Valley. The formations underlying this valley consist of fractured basalt separated by lava flows. These formations are deep, and their transmissivities are high. Wells yield up to 3500 gpm. Depth to water table ranges from 60 to 500 feet with an average of 90 feet (USDA 1978). Depth of alluvium is estimated to be around 400 feet. There are 65,000 acres of land under irrigation, mostly by sprinkler systems. In the last few years the water table has dropped 30 feet. It is estimated that more than 200,000 acre-feet of water could be recharged annually. Recharge water might be diverted from Alexander Reservoir. This unit of the Bear River Basin contributes about 82,000 acre-feet of water annually to the Great Salt Lake (USDA 1978).

b. Soda Springs Area. The Soda Creek Basin is underlain by a rather thick formation of fractured basalt. The thickness of the formation is about 400 feet with a rather high hydraulic conductivity. Groundwater is moving westward from Alexander to Gem Valley. The demand has not been great enough to justify groundwater development,
but the area is promising from the water resources management point of view. This subbasin is contributing 104,000 acre-feet of water annually to Great Salt Lake.

c. Weber Delta District. This district has an aquifer capacity of 170 million acre-feet, not all of which is suitable for irrigation or municipal use. Thicknesses of the aquifers range from 50 to 250 feet, transmissivity is between 2,000-300,000 g/day/ft, and the storage coefficient is about 7.9 x 10^-4. Through lowering the groundwater level 50 feet, the nonartesian areas would yield 80,000 to 100,000 acre-feet of water. For the artesian areas, a piezometric head reduction of 50 feet produces 300,000 to 600,000 acre-feet of water (Feth et al. 1966).

Recharge studies by the USBR indicate that the Weber Delta district is a good place for a recharge-discharge project. "Artificial recharge experiments...indicate that artificial recharge on a large scale may be feasible" (Feth et al. 1966, p. 2).

d. Jordan Valley. The Jordan Valley offers good possibilities for groundwater recharge. Depths to water-table range from 500 feet below the ground at some locations on the periphery of the valley to the ground surface in the central and north sides of the valley. Transmissivity ranges from 1,000 to 50,000 ft^3/day/ft. The storage coefficient is between 0.15 along the valley edges to 10^-6 in the valley center. The thickness of the unconsolidated deposits is about 3,000 feet, and the active groundwater storage receives 367,000 acre-feet annually. This valley contributes 240,000 acre-feet and 4,000 acre-feet of water annually to Great Salt Lake from surface and underground sources, respectively (Schlotthauer et al. 1981).

e. Cache Valley. Cache Valley is one of the larger and more productive aquifers in the basin. The area of the valley is about 660 square miles of which 220 square miles are underlain by aquifers under artesian pressure. A minimum estimate of the storage capacity in the valley is 40 million acre-feet. The first foot of the aquifer yields about 25,000 acre-feet of water, but yields per foot decrease as the water table declines. For the first 20 feet the decrease in recoverable water is not significant. Transmissivity is from 10,000 to 330,000 ft^3/day/ft, the storage coefficient is from 2 x 10^-4 to 0.20, and the maximum fill depth is 6,000 feet.

Wells yield as much as 3,500 gpm. The average total groundwater discharge is 280,000 acre-feet per year through seepage, drains, flowing wells, and pumped wells. In Utah, only 26,000 acre-feet of water is pumped annually from the Cache Valley aquifer, and the rest of the discharge overflows from the aquifer. On the average, this valley contributes 523,000 acre-feet of water to Great Salt Lake annually (Bjorkland et al. 1978).
f. Malad Valley. Malad Valley, like Cache Valley, has one of the larger groundwater aquifers in the Bear River Basin. Irrigation wells yield, on the average, 2 cubic feet per second. The average groundwater discharge from the valley is 63,000 acre-feet per year, of which 17,000 acre-feet is pumped from irrigation wells and the rest comes from springs. This subbasin contributes 36,000 acre-feet of water annually to Great Salt Lake. Since 1964, agricultural development and pumping from the groundwater has lowered the water table sufficiently to make the area ideal for recharge projects (USDA 1978).

g. Bear Lake Valley. This valley is a vast low-lying area with a high water table. In much of the valley, water stands at the ground surface and forms marsh lands and swamps. Sediments are composed of relatively fine materials with low permeability. There are 430 wells in Bear Lake Valley and their yields are rather low. This valley is a promising site to be drained and reserved for storing excess water during wet cycles. Preservation of wildlife habitat is essential.

h. Upper Bear River Valley. In this valley, the water bearing aquifers are limited to flood plains which consist of deep and coarse alluvial deposits. Wells in this valley are 200 to 300 feet deep and do not penetrate the full alluvium. Their yield is 700-1500 gpm. The aquifer is estimated to have a thickness of 1000 feet. In the lower part of the valley, the water table is about 20 to 30 feet below ground level. The alluvium in this valley is a porous media that can be discharged in dry years and recharged in wet periods (USDA 1978).

ECONOMICS OF GROUNDWATER RECHARGING IN UTAH

Hansen (1983) developed a computer model to evaluate pumping lift decreases due to artificial recharge in the Weber River delta near Ogden, Utah. He concluded that when recharge project economics "are based solely upon the savings realized by a reduction in pumping lifts and fixed construction costs, the value of artificial recharge...appears to be marginal. Slight changes in the economic analyses could make a marked difference in the viability of such a project. For example, if pumping lifts were larger due to increased pumpage during dry years, the savings for a given quantity of recharge would be substantially higher. Alternate financing or charging interest rates...and rising energy costs would also change the economic worth of recharge."

A complete economic benefit analysis of a recharge project of this nature should include other benefits, such as in this case reduced Weber River flooding, reduced flooding potential at the Great Salt Lake, reduced treatment costs of water used for culinary purposes, and increased water availability during drought periods. By including these types of benefits, the economic viability of artificial recharge might be changes substantially.
In a summary report the U.S. Bureau of Reclamation (1970) examined 10 irrigation projects in the Bear River basin and estimated annual benefits and costs as follows:

<table>
<thead>
<tr>
<th>Project</th>
<th>Benefits/Costs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thomas/Smiths Fork</td>
<td>0.43</td>
</tr>
<tr>
<td>Bennington</td>
<td>0.57</td>
</tr>
<tr>
<td>Caribou</td>
<td>1.77</td>
</tr>
<tr>
<td>Oneida Narrows 1</td>
<td>1.46</td>
</tr>
<tr>
<td>Oneida Narrows 2</td>
<td>1.36</td>
</tr>
<tr>
<td>Oneida Narrows 3</td>
<td>1.46</td>
</tr>
<tr>
<td>East Cache</td>
<td>1.16</td>
</tr>
<tr>
<td>Blacksmith Fork 1</td>
<td>0.86</td>
</tr>
<tr>
<td>Blacksmith Fork 2</td>
<td>0.82</td>
</tr>
<tr>
<td>Honeyville</td>
<td>1.67</td>
</tr>
</tbody>
</table>

Groundwater storage and pumping were not considered specifically in the project designs. Flood control benefits were included only along streams, but not around the Great Salt Lake. Addition of groundwater aspects to the plans and the value of flood protection around the Great Salt Lake could change the benefits and costs, and thus increase some of the ratios cited above.

INSTITUTIONAL ASPECTS OF THE PUMPING AND RECHARGE ALTERNATIVE

While drainage and recharge technologies might be effective ways of regulating inflows to the Great Salt Lake so as to dampen the lake level extremes, such management measures would have to be implementable within the basinwide framework of water rights ownership and the multitude of institutions currently engaged in water management activities. The apprehension about how new management measures might affect existing water entitlements and availability would need to be addressed. Shifts and expansion of upstream storage and out of basin transfers designed to alter inflow regimen and quantity in the benefit of lakeshore interests could not result in adverse impacts to the conglomerate of existing water users. The array of institutions whose water rights and operating patterns may be affected by the introduction of new management measures includes federal and private waterfowl sanctuaries, power companies, canal companies, irrigation and drainage districts of various kinds, and municipalities. Technological solutions must be politically acceptable, legally permissible, and institutionally adaptable. Therefore, the kinds of accommodations and adjustments that existing infrastructures might face as major new management measures are implemented would need to be determined. Prevailing institutional arrangements can be changed if the benefits of doing so are clearly shown to increase as a result. The nature of institutional changes required would be somewhat specific to the kind, location, and magnitude of the solution proposed.
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SECTION V - EMERGENCY PREPAREDNESS/RESPONSE
PRE-FLOOD PERIOD

Prior to the Flood of 1983, Salt Lake City had a proposed Emergency Management Plan. The plan was by all means not comprehensive. It addressed only flood and not earthquakes, civil disorders, strikes, and many other types of incidents. The plan consisted of four basic parts: (1) an emergency board and the board's responsibilities; (2) identified internal and external logistical resources; (3) a basic flood control plan; and (4) the definition of the different phases of an emergency. The plan had not been tested until the Flood of 1983.

FLOOD PERIOD

Just prior to Memorial Day the emergency board was activated and a command center established. The Mayor declared a phase 3 emergency and declared the City in a state of emergency. This gave emergency powers to the Mayor and enabled the City to expedite purchasing and contracting requirements.

Alternatives Considered

The emergency board with advice from the Public Works Department considered the alternatives for handling the anticipated volumes of water. It was decided that the water from Red Butte and Emigration Canyons could be handled using the City's storm drain system. However, the volume of water in Mountain Dell Reservoir was determined to exceed the capacity of the storm drain system. Primarily the 1300 South conduit was not large enough to accommodate the anticipated flow. It was decided to dike 1300 South and run the excess volume of water on top of the street. The City used all its heavy equipment plus all the heavy equipment of contractors to haul dirt. In three days, working around the clock, 1300 South was bermed from State Street to the Jordan River.
City Creek’s Surprise

The City planned on handling the flow of City Creek as it usually did. The flow would be directed down the North Temple underground conduit. However, City Creek surprised the city crews by peaking with larger water volume than was predicted. The tremendous flow washed out creek banks and uprooted trees. The debris quickly blocked the North Temple storm drain. The City improvised by calling on volunteers and diverting the water from City Creek down State Street to 400 South and into the storm drain system. It became quickly apparent that the storm drain system at 400 South was not adequate to handle the volume; consequently, the City improvised once more by continuing the “State Street River” from 400 South to 800 South down State Street. This also required the construction of two four-lane, temporary bridges over 500 and 600 South. These two streets are the two main access streets to and from Interstate 15.

Fight to Save Cottonwood Canyon Water Treatment Plant

While all of the other flooding activities were going on within the city limits, water flows from Big Cottonwood Canyon were threatening the City’s water treatment plant. The City sent crews to sandbag and dig channels in order to divert water away from the plant. Otherwise, drinking water to many county and city residents would have adversely been affected.

LESSONS LEARNED

A number of lessons were learned from the flood experiences. Many of these lessons were later incorporated into the creation of a new Emergency Management Plan. Many of these lessons were problems and obstacles which had to be overcome during the flooding situation. The lessons learned were: communications, physical facilities, chain of command, logistic support, supervision and management of volunteers, financial documentation, and inter-governmental relations.

Communications

Communication quickly became a critical problem. The problem took on two dimensions. One dimension was equipment problems, the other dimension dealt with the process. The City discovered a number of “dead” spots where communications were not possible. The batteries could not always meet the demand of around-the-clock use on the hand-held radios. Batteries could not be charged fast enough and extra batteries could not be purchased fast enough.

The reports to the command center were coming from too many individuals. This often created confusion for those required to make decisions within the command center.

Command Center Physical Facilities

The command center was one large room within the Police Department. Command center staff and decision makers were not separated. It became
apparent that this caused two primary problems, besides a great amount of confusion. First, staff members often deferred making decisions on petty problems because major decision makers were readily available. Second, staff members made some decisions that they should not have. Lack of training also contributed to this problem.

Chain of Command

Two situations developed out of the flooding situation that raised questions about the chain of command. Initially all members of the emergency board were included in the chain of command. This became too cumbersome. Effective communications were hampered because information was coming from too many sources. Everyone had somebody in the field. Second, as the emergency progressed there was not a need for all the members of the emergency board. Only certain departments were affected by the current situation. Additional people in the chain of command slowed the ability to react swiftly. On site, in the field, there was often no one person in charge with whom to coordinate.

Logistical Support and Management of Volunteers

The logistics of obtaining flood fighting supplies, tools, equipment, food and then delivering it to the proper place in a timely manner were enormous. There was no one department assigned to handle volunteers or feeding of volunteers. The demand on the purchasing division from the departments was never ending on a 24-hour basis. It was hard to direct supplies and food to the necessary places in the field because situations changed so rapidly. Food was brought to one location and attempts were made to distribute it. There was never enough food because the head accounts were never accurate or the crew sizes would increase or decrease according to the situations. The number of volunteers would also affect the number of meals needed. Additionally, most of the food or meals were donated and the amount varied greatly.

The majority of volunteers who reported to help out were gainfully employed. However, because the numbers were so great at times it was impossible to control all the volunteers’ activities. Consequently some complained because they were not properly supervised and directed to areas where they might have been more beneficially used. Since there were, many times, no designated incident or site commanders, such complaints were often valid.

Financial Documentation and Intergovernmental Relations

Two situations that went hand-in-hand were the need to document the cost of fighting the floods and proving to county, state and federal agencies the legitimacy of such flood fighting expenses. The normal day-to-day accounting system was unable to handle the volume and variety of expenses incurred during this emergency period.

The second situation which required communication and coordination between the City and County, the City and the State, and the City and FEMA were difficult at times primarily because there were several people involved and procedures were all new and not well understood.
NEW PLAN IMPLEMENTED

As a result of the lessons learned a new Emergency Management Plan was developed, adopted, and implemented. The plan had several significant changes. Those changes are briefly discussed below.

Emergency Board Restructured

The original emergency board consisted of 15 individuals. Most were members of the Mayor's cabinet (department heads) and some additional deputy department heads. The board was restructured to consist of five people. The Mayor, Chief Administrative Officer, Chief of Police, Fire Chief, and the Department Head specified by the plan were to have primary responsibility for the emergency. This greatly streamlined the structure and the chain of command. A primary department is defined as the department assigned to handle a predetermined emergency.

Intergovernmental Relations and Volunteers

The Mayor's Office is now tasked with the responsibility of handling intergovernmental relations and volunteer participation. Because lower level staff had difficulty circumventing bureaucratic red tape, it was felt the Mayor's Office could expedite coordination with other governmental entities.

The Mayor's Office will also work with organized volunteer groups rather than with unorganized and unsupervised volunteer groups. Additionally, site commanders will be responsible to supervise volunteers at that site.

Logistic Support

Several changes were made here. The Personnel Department is responsible to see that city employees and volunteers are fed during the emergency period. The Finance and Administrative Services Department will handle purchasing, pre-contracting for services (i.e., damage assessment, private contractors, heavy equipment), financial documentation, and city equipment maintenance.

Communication Support

The Police Department is responsible for communication at the stationary command center and the Fire Department is responsible to provide a mobile command center and communications within the mobile command center. More phone lines will be phased in to accommodate better communications in the stationary command center.

On-Site or Incident Commander

One very significant improvement in the chain of command was the designation of one site commander at each major problem area. This single commander is primarily responsible for decisions at the scene and communication with the command center. This individual directs all activities at the scene. Additionally, the levels of emergency and responsibilities were more clearly defined. (See figure 1 and 2.)
Command Center Restructured

The command center has been restructured to separate the decision makers and support staff thus eliminating unnecessary confusion and expediting information and decisions.

CONCLUSION

The experiences of the 1983 flood have made the handling of the 1984 flood effort much more manageable. The new Emergency Management Plan when tested proved workable.
LEVEL 1 EMERGENCY

INCIDENT OCCURS

RESPONSE
1) PRIMARY DEPARTMENT

COMMAND RESP.
PRIMARY DEPT.
NOTIFICATIONS
NONE

RESOLVED

NO

DOCUMENTATION
CLAIMS/GRANTS
EVALUATION
REVIEW/REVISE PLAN
FILE

LEVEL 2 EMERGENCY

INCIDENT REMAINS UNRESOLVED

RESPONSE
1) PRIMARY DEPT.
2) OTHER CITY DEPT.
(Routine assists)
3) OUTSIDE AGENCIES
(INTER/LOCAL/REGION)

COMMAND RESP.
PRIMARY DEPT.
NOTIFICATIONS
ALERT CAO

RESOLVED

NO

DOCUMENTATION
CLAIMS/GRANTS
EVALUATION
REVIEW/REVISE PLAN
FILE

LEVEL 3 EMERGENCY

INCIDENT ESCALATES

RESPONSE
1) PRIMARY DEPT.
2) ALL CITY DEPT.
(NON-Routine assists)
3) OUTSIDE AGENCIES
(NON-Routine)

COMMAND RESP.
EMERGENCY BOARD
NOTIFICATIONS
ALL

RESOLVED

RESOLVED

DOCUMENTATION
CLAIMS/GRANTS
EVALUATION
REVIEW/REVISE PLAN
FILE
LEVEL 3 EMERGENCY
CHAIN OF COMMAND

LEVEL 1 & 2 EMERGENCIES

DEPARTMENT HEAD

INCIDENT COMMANDER

SITE COMMANDER

SITE COMMANDER

SITE COMMANDER

LEVEL 3 EMERGENCY

EMERGENCY MANAGEMENT BOARD

DEPT. HEAD | CAO | MEMBERS

INCIDENT COMMANDER

SITE COMMANDER

SITE COMMANDER

SITE COMMANDER
Learning from the Past: Developing Increased Preparedness

AN OVERVIEW OF DISASTER RESPONSE, RECOVERY, AND PREPAREDNESS: LANDSLIDE AND FLOOD DISASTERS IN CALIFORNIA AND UTAH, 1978-84, AND IMPLICATIONS FOR THE FUTURE

by William M. Brown III

ABSTRACT

California and Utah were beset by a series of natural disasters originating from severe winter storms in 1978, 1980, 1982, 1983, and 1984. These storms produced, at different times and places, high-intensity rainfall, large volumes of stream runoff, heavy snowfall, and large ocean waves. The consequences of these events were widespread debris flows and avalanches, deep-seated landslides, riverine and coastal flooding, and coastal erosion. The magnitude of these processes generally was considered to be very rare in most affected areas to the extent that some of the American West's largest communities were ill-prepared for what occurred, despite decades of meteorological observations and the building of immense storm-defense systems. Whereas the death toll from the storms was almost miraculously low, direct property damage exceeded $1.5 billion, and costs continued to accumulate during the storms' aftereffects, many of which persist in 1984.

This paper will review the 1978-84 sequence of federally declared storm disasters in California and Utah, notable successes and failures in dealing with them, and the recent and proposed evolution of preparedness for future, similar events. Specifically, structural, monitoring, and warning systems will be appraised, and disaster-prone areas will be identified. The relations among population, development, geographic area, and storm-disaster occurrence will be reviewed. Similarities and differences as to storm impacts in different regions will be discussed with a view toward speeding up information transfer and making best use of other communities' experiences.
COMMUNITY "HELPING THY NEIGHBOR" PLAN

by David L. Scott, M.D.

ABSTRACT

The purpose of the plan is to coordinate and communicate a plan for disaster response in a community, before, during and after a disaster.

The plan provides an organization whereby any individual can communicate his or her needs to the community by passing through at most 3-4 levels in the chain and receive assurance that the community is well enough organized to meet family and individual needs.

The plan considers most disaster situations and has provided an organized response that is preplanned, rehearsed and understood by everyone prior to any conceivable situation. This plan includes means for dealing with evacuation, mass feeding, aged and handicapped, medical needs, crisis intervention and multiple casualties. Strong links have been developed with the county program to provide the plan with cohesiveness and a strong base.

The plan recognizes that perhaps the most important element is a quick, organized response. A well organized communications system, which is being implemented, is a necessity.

Included with this abstract is a 20 page insert which is part of the heart of the program in that it contains the information the family will use to respond to a disaster situation. It requires families to form into neighborhood groups which are also grouped into areas. Areas are then grouped into districts. A community, depending on its size, will have from one to many districts. Bountiful, for example, has 11 districts. The insert has been distributed to every family in Davis County in the back of the South Davis Telephone Directory or the North Davis Telephone Directory which will be distributed in August.

The presentation of the program includes also a 10 minute video-taped dramatized description of how the program works. This tape is intended to be the first of many such video presentations designed to help implement one aspect or another of the program.

The program is exciting in its simplicity and yet far-reaching effect. It is presently being used as Davis County communities deal with the pressures of the spring rapid water runoff.

David L. Scott, M.D. is with Pediatrics, University of Utah, and Davis County Emergency Service.
The objective of this paper is to illustrate the utility and application of the EPA ENVIROPOD aerial camera system in identifying hazards and mapping impacts of events.

During the past year the Center for Remote Sensing and Cartography has been operating under a memorandum of understanding with the Environmental Protection Agency (EPA) and the State of Utah, to test the effectiveness of the ENVIROPOD in monitoring environmental conditions and changes. With the outset of landsliding and flooding throughout the state, an ample number of targets arose. Some 22 missions were flown for a variety of purposes: flood mapping along the Weber, Ogden, Jordan, and Sevier Rivers; debris flow detection and mapping along the Wasatch piedmont; high water mapping around Great Salt Lake, Utah Lake, and Bear Lake; dike damage detection along causeways and waterfowl areas in Great Salt Lake; and landslide pattern detection at Thistle and elsewhere in the state.

The paper will describe the aerial camera operating system and demonstrate its flexibility and utility in responding to emergency needs. A variety of applications, as indicated above, will be presented. Factors of film type (natural color and color infrared), flying height and scale, flight planning, indexing, and interpretation will be illustrated. The remarkable detail that is detectable from the very high quality film will be emphasized, as will the flexibility of placing the camera in the optimal position with regards to viewing angle, time of day, and exposure conditions.

Access to the system has been fortuitous for the State of Utah during the 1983 season, and promises to be of equal or greater value during the 1984 season. In addition to the above applications pertinent to the purposes of the Specialty Conference, a further application has been suggested by the USU landslide study team in a possible pre-snowmelt detection of ground failure in canyon environments. Such detection may provide a kind of "early warning system," and is certainly worth pursuing as an ideal application of the ENVIROPOD system.

The paper will be illustrated with 35 mm slides, overhead transparencies, and handouts.

Merrill K. Ridd is with the Center for Remote Sensing and Cartography, University of Utah Research Institute.
THE ROLE OF TECHNICAL PEOPLE/COMMITTEES DURING CRISIS PERIODS OF GEOLOGIC HAZARD EVENTS

Bruce N. Kaliser

The Utah Geological and Mineral Survey has provided on-site assistance to political subdivisions throughout the state before, during, and after the occurrence of geologic hazard events. 1983 was no exception except that the phenomena were so large in magnitude and impact that two Technical Committees were established: one for the Thistle Landslide, in April, and one for the Davis County debris flow and debris flood events, in May and June. Individuals were sought with expertise in complementary fields, normally from the public sector where their availability could be sustained. Priorities were reestablished on a daily or twice-daily basis. A Technical Committee spokesperson addressed elected officials following each Technical Committee meeting. At the pleasure of the committee of elected officials, the media and the public was addressed by the spokesperson. In open meetings the public was given the opportunity to query any matter on their minds. In this manner the geologic phenomena were explained and removed from the realm of the mysterious.

Assistance ensued through the disaster recovery phase with additional expertise being brought in as well. Needs in the engineering and non-engineering spheres were identified and explored with city, county, and state emergency preparedness and recovery officials.

Bruce N. Kaliser is with Utah Geological and Mineral Survey.
Different Perspectives Create Different Responses

IMPACTS OF THE WEBER BASIN RESERVOIR SYSTEM UPON THE OGDEN AND WEBER RIVER DRAINAGES DURING THE SPRING RUN-OFF AND FLOODS OF 1983

by Mark D. Anderson

ABSTRACT

Weber Basin Water Conservancy District dictates reservoir releases for flood control purposes within the Weber and Ogden watersheds. Higher than expected runoff during late May and early June produced flood conditions causing damage to communities, industries, utilities, water districts and agriculture land. Through efficient operation of the reservoirs, it was possible to reduce the peak run-off and effectively prevent additional damage.

INTRODUCTION

The Weber Basin Project, built by the Bureau of Reclamation and now managed by the Weber Basin Water Conservancy District (District), was planned to conserve and utilize for multiple purposes practically all the presently unused flows of streams in the natural drainage of the Weber River, including the Ogden River basin its principal tributary. Prior Federal reclamation developments include the Weber River Project with its Echo Reservoir on the main stem of the Weber River, and the Ogden River Project with its Pineview reservoir on the Ogden River.

Stream flow for project purposes is regulated by four new reservoirs and two enlarged reservoirs and the correlated operation of project reservoirs and the old Echo Reservoir. Three of the six project reservoirs: Wanship (Rockport Lake), Lost Creek and East Canyon (enlarged), as well as the Weber River Project's Echo Reservoir, regulate the flow of the Weber River before it emerges from its mountain watershed to the East shore.

Mark D. Anderson is the District Engineer for Weber Basin Water Conservancy District in Layton, Utah.
area where the principal water utilization occurs. Two project reservoirs: Causey and Pineview (enlarged), regulate the flow of the Ogden River before it emerges from the mountains to join the Weber River in the East shore area. Willard (off-stream) is the lowest reservoir of the system and receives water from the Weber River, diverted at Slaterville Diversion Dam, below the mouth of the Ogden River, and conveyed through the Willard Canal.

The three project reservoirs on the Weber River and its tributary creeks are operated to supply water for irrigation, municipal and industrial purposes in the East shore area and for power production at Gateway and Wanship power plants. In addition, the reservoirs are operated to provide supplemental irrigation water and replacement water for residential purposes in mountain valleys along the Weber River and its upper tributaries. The reservoirs are also used to provide flood control and for the maintenance of stream flows for supporting game fish.

The District makes the decisions on flood storage and releases based on forecasts received by the snow survey staff of the Soil Conservation Service and modified slightly by the Bureau of Reclamation. The flood operation of the reservoirs are reviewed and are subject to the Bureau of Reclamation and the Army Corps of Engineers.

CLIMATIC CONDITIONS 1982-1983

Utah's 1982 water supply was better than average. Later than usual snow melt produced more streamflow than forecasted. Much heavier than usual summer precipitation improved late season streamflow and reduced water use resulting in October 1, 1982 carry-over reservoir storage at near record high levels. Some reservoirs were reported full and spilling. The total of 24 (4) key Utah reservoirs were at 138% of the 1963-77 average for October 1st and 146% of 1981 storage.

Snow cover on the Ogden and Weber watersheds ranged from 94 to 142 percent January through May 1983. However, late May snowstorms that were reported in the June snowfall survey indicated snow cover at 423 and 518 percent for the Ogden and Weber watersheds respectively as shown in Table 1.

TABLE 1 Snow Cover on the Ogden and Weber Drainages for 1983 (Percent of Average)

<table>
<thead>
<tr>
<th></th>
<th>JAN</th>
<th>FEB</th>
<th>MAR</th>
<th>APR</th>
<th>MAY</th>
<th>JUNE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ogden</td>
<td>120</td>
<td>101</td>
<td>108</td>
<td>120</td>
<td>142</td>
<td>423</td>
</tr>
<tr>
<td>Weber</td>
<td>122</td>
<td>94</td>
<td>104</td>
<td>119</td>
<td>114</td>
<td>518</td>
</tr>
<tr>
<td>Soil Moisture</td>
<td>AA</td>
<td>AA</td>
<td>AA</td>
<td>AA</td>
<td>AA</td>
<td>WAA</td>
</tr>
</tbody>
</table>

Legend: W=WELL, AA=ABOVE AVERAGE
A heavy snowfall in late May and ensuing high temperatures produced high volume run-off over a short time duration. The 1982 run-off hydrograph shows a longer duration with lower peaks. The two contrasting run-off hydrographs for one point on the Weber River are shown in Figures 1 and 2.

FIGURE 1 Snowcourse and Run-off Hydrograph for the Weber River near Oakley for 1982

FIGURE 2 Snowcourse and Run-off Hydrograph for the Weber River near Oakley for 1983.
Releases for 1983 were scheduled in accordance with the U.S. Corps of Engineers' reservoir regulation for flood control (3). As shown in Tables 2-5, the actual inflow in May exceeded the projected inflow at every reservoir site. With the exception of Echo-Wanship, the percent difference between the forecasted inflow and actual inflow ranged from 76 to 135 percent. The forecast in June also increased from the May forecast at every site. With the excessive late May run-off the unused reservoir capacities were soon filled. The reservoirs did, however, absorb the peak run-off in every case. Stream channel capacities, particularly on the Ogden River, were effectively reduced by housing encroachments. This problem diminishes somewhat the flexibility of the system and requires that forecasts be more accurate in order to schedule releases.

### TABLE 2 - Pineslew Reservoir

<table>
<thead>
<tr>
<th></th>
<th>March</th>
<th>April</th>
<th>May</th>
<th>June</th>
</tr>
</thead>
<tbody>
<tr>
<td>March</td>
<td>37.1</td>
<td>35.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>April</td>
<td>37.6</td>
<td>42.2</td>
<td>45.8</td>
<td></td>
</tr>
<tr>
<td>May</td>
<td>45.6</td>
<td>51.0</td>
<td>56.5</td>
<td>99.3</td>
</tr>
<tr>
<td>June</td>
<td>16.0</td>
<td>26.2</td>
<td>33.2</td>
<td>135.0</td>
</tr>
<tr>
<td>July</td>
<td>5.7</td>
<td>9.6</td>
<td>7.3</td>
<td>30.0</td>
</tr>
<tr>
<td>Total</td>
<td>122.1</td>
<td>129.0</td>
<td>97.0</td>
<td>165.0</td>
</tr>
</tbody>
</table>

### TABLE 3 - Lost Creek

<table>
<thead>
<tr>
<th></th>
<th>March</th>
<th>April</th>
<th>May</th>
<th>June</th>
</tr>
</thead>
<tbody>
<tr>
<td>March</td>
<td>2.0</td>
<td>2.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>April</td>
<td>5.5</td>
<td>3.2</td>
<td>3.8</td>
<td></td>
</tr>
<tr>
<td>May</td>
<td>7.5</td>
<td>7.6</td>
<td>7.5</td>
<td>13.5</td>
</tr>
<tr>
<td>June</td>
<td>2.5</td>
<td>4.8</td>
<td>4.5</td>
<td>25.0</td>
</tr>
<tr>
<td>July</td>
<td>.3</td>
<td>.3</td>
<td>1.0</td>
<td>5.0</td>
</tr>
<tr>
<td>Total</td>
<td>18.0</td>
<td>16.0</td>
<td>13.0</td>
<td>38.0</td>
</tr>
</tbody>
</table>

Notes: Underlined number indicates actual inflow
DIFFERENT PERSPECTIVES CREATE DIFFERENT RESPONSES

TABLE 4 - Echo - Wanship

<table>
<thead>
<tr>
<th></th>
<th>March</th>
<th>April</th>
<th>May</th>
<th>June</th>
</tr>
</thead>
<tbody>
<tr>
<td>March</td>
<td>33.9</td>
<td>49.6</td>
<td></td>
<td></td>
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<tr>
<td>April</td>
<td>54.8</td>
<td>63.8</td>
<td>52.4</td>
<td></td>
</tr>
<tr>
<td>May</td>
<td>96.5</td>
<td>112.0</td>
<td>104.8</td>
<td>119.0</td>
</tr>
<tr>
<td>June</td>
<td>91.3</td>
<td>105.0</td>
<td>154.2</td>
<td>253.0</td>
</tr>
<tr>
<td>July</td>
<td>32.6</td>
<td>38.3</td>
<td>39.0</td>
<td>64.0</td>
</tr>
<tr>
<td>Total</td>
<td>309.1</td>
<td>319.1</td>
<td>298.0</td>
<td>317.0</td>
</tr>
</tbody>
</table>

TABLE 5 - East Canyon

<table>
<thead>
<tr>
<th></th>
<th>March</th>
<th>April</th>
<th>May</th>
<th>June</th>
</tr>
</thead>
<tbody>
<tr>
<td>March</td>
<td>5.6</td>
<td>6.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>April</td>
<td>8.9</td>
<td>9.1</td>
<td>11.2</td>
<td></td>
</tr>
<tr>
<td>May</td>
<td>11.7</td>
<td>10.7</td>
<td>12.6</td>
<td>24.5</td>
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<tr>
<td>June</td>
<td>8.1</td>
<td>8.6</td>
<td>7.6</td>
<td>41.4</td>
</tr>
<tr>
<td>July</td>
<td>1.7</td>
<td>1.6</td>
<td>1.0</td>
<td>5.6</td>
</tr>
<tr>
<td>Total</td>
<td>34.0</td>
<td>30.0</td>
<td>21.0</td>
<td>47.0</td>
</tr>
</tbody>
</table>

Note: Underlined number indicates actual in flow

IMPACTS OF THE SPRING RUN-OFF

High run-off produced damage along portions of the Ogden and Weber Rivers. Numerous homes received extensive damage while several others incurred minor damage. Several hundred thousand acres (2) of farm ground were inundated with water causing erosion and siltation damage ranging from moderate to severe. Many public roads, bridges and buildings were damaged along with several portions of water, storm and canal systems. Several industrial properties were also damaged.
The U. S. Corps of Engineers developed flow damage curves for different reaches of the Weber and Ogden Rivers. These curves were based on 1970 prices and economic conditions. Using the consumer price index and a 1 percent average annual building increase rate, these curves were adjusted to reflect 1983 prices and economic conditions. Based on these curves, estimated damages were $2.9 million dollars. Due to the reservoir operations, the Weber and Ogden drainages were spared an additional cost of $7.1 million dollars in damages, see Table 6.

<table>
<thead>
<tr>
<th>Reach/Location</th>
<th>Park</th>
<th>Echo</th>
<th>Lost</th>
<th>Canyons</th>
<th>Pineview</th>
<th>Willow</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weber River:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rockport Lake to Echo Res.</td>
<td>436</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>436</td>
</tr>
<tr>
<td>Echo Res. to Mouth of Lost Cr.</td>
<td>224</td>
<td>271</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>495</td>
</tr>
<tr>
<td>Mouth of Ogden River</td>
<td></td>
<td>1,400</td>
<td>1,700</td>
<td>456</td>
<td>1,100</td>
<td></td>
<td>4,266</td>
</tr>
<tr>
<td>Ogden River to Great Salt Lake</td>
<td>92</td>
<td>113</td>
<td>30</td>
<td>72</td>
<td>10</td>
<td>165</td>
<td>328</td>
</tr>
<tr>
<td>Echo Res. to Pineview Reservoir</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>47</td>
<td>47</td>
</tr>
<tr>
<td>Pineview Reservoir to Mouth</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>Mouth area, Upper Weber and Lost Cr.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>272</td>
<td>272</td>
</tr>
<tr>
<td>Lost Creek to Weber River</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>2,152</td>
<td>2,092</td>
<td>493</td>
<td>1,544</td>
<td>53</td>
<td>355</td>
<td>7,125</td>
</tr>
</tbody>
</table>

CONCLUSION

1983 produced an abnormally high water year, which began with near record reservoir storage carry over from 1982. High late May snowfall and ensuing high temperatures brought the run-off down off the mountains quickly producing high flows. Inflows that varied as much as 130 percent from the forecast allowed the reservoirs to fill and spill sooner than planned. The reservoirs were able to absorb peak flows and help reduce flooding damage. Based on the U. S. Corps of Engineer's flow damage curves, which were adjusted to current 1983 prices and economic conditions; the operation of the Weber Basin project reservoirs and dams reduced the damage by 7.1 million dollars.
REFERENCES


Responsing to the Fury of the Colorado River -- A Public Affairs Perspective

by Kathy Wood Loveless

Objective

In meeting the challenges of any natural disaster, one of the most important tasks is disseminating timely, accurate, and relevant information to the public and media. The objective of this paper is to document and analyze how that task was handled in 1983 when the Colorado River Basin received 210 percent of normal flows, resulting in: (1) unique operating procedures at many dams, (2) flooding downstream to numerous residents and businesses, (3) rumors that ran the full gambit of reasonable to ridiculous, (4) nearly $23 million worth of damage to the Glen Canyon Spillways alone, (5) a public and media awareness that was not only national, but international in scope, and (6) a series of hearings and investigations by the U.S. Congress and the Government Accounting Office that revealed areas requiring change as well as those that must remain the same to prepare for any similar future occurrences.

Summary

The natural phenomena that resulted in the Colorado River Basin's receiving 14.6 million acre-feet when just prior to the runoff period it was expected to receive only 6.7 million has been well documented elsewhere. This paper attempts to explore the human response to that record runoff. As soon as the sudden above-average temperatures of June began to melt, the heavy snowpack of April and May, runoff into Lake Powell began to rise so quickly that its full elevation of 3,700 feet was quickly reached. To minimize damage to the spillways from high flows, 8-foot high metal flashboards were added to the top of the spillway gates. These flashboards allowed Lake Powell to hold an additional 1.5 million acre-feet. Notifying concessionaires around the reservoir, as well as recreationists that the reservoir could rise 8 feet above its "full" level required immediate action and a reassuring strategy. Other related issues had to be addressed such as the impact 8 additional feet of water would have on the delicate Rainbow Bridge, having long been embroiled in litigation.

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This action concerned Reclamation because it wanted not to set a precedent of operating reservoirs with the use of flashboards. Additionally, spillways were used at dams throughout the west where they had rarely, if ever, been used before. Those of Crystal and Morrow Point in western Colorado were used while only the outlet tubes of Blue Mesa were used. Special concern surrounded the operation of Fontenelle Dam in southwestern Wyoming which was seeping water and had been drawn down for monitoring.

Rumors that flashed throughout the west ranged from the Bureau of Reclamation's ostensible decision to blow up Glen Canyon Dam as was reported in the Los Angeles Times, to the Red Cross Chapter Chairman of Sweetwater County, Wyo., telling residents of Green River, Wyo., that they had 72 hours to evacuate because he "knew" Fontenelle Dam was failing, to radio reports in western Colorado that a "6-foot-wall of water" was coming over the Crystal spillway (6 cfs of water being released). The handling of these kind of rumors was a special challenge because: a) people for some perverse reason during unusual times want to believe the worst is happening, and b) there is always the suspicion of a "government cover-up" of what is really happening.

Reporters from all major newspapers, the three television networks, principal newsmagazines in the United States, and many media outlets from England, Australia and other countries either called or visited the Reclamation's Salt Lake City Public Affairs Office to obtain information in covering the event. Managing the flow of information required constant updating by operations officials, use of the latest code-a-phone radio actuality equipment, toll-free telephone numbers, and a staff of writers and documentary photographers.

Finally, following the crisis period, Congressman Mo Udall called for a series of hearings to bring to light all that occurred, how officials responded, and what the public's views were. Meetings of the Seven Basin State Governors to review the operating criteria of the river ensued. The Government Accounting Office conducted its investigation of the occurrences. In short, the findings of all were that the Bureau of Reclamation had operated the system of dams according to law, regulation, and prudent judgment, but perhaps some alterations should be made in the Operating Criteria. Additionally, it was suggested that the various data collecting agencies of the Colorado River Forecasting Service needed more funds and personnel to improve snowpack data collection.

Upon conclusion of the flooding emergency, the Upper Colorado Region Public Affairs Office produced two videotapes that have since received wide circulation and acclaim. The first, "1983: The Record Water Year," outlined the flooding as it affected Utah and the Upper Colorado River Basin. The second, "The Spillways of Glen Canyon," details the damage and repair of the huge spillways, and is updated periodically as the repair nears completion.
LOCAL FLOOD FIGHT AND CLEAN UP RESPONSE

BY CHARLES H. CALL, Jr., P.E., LEE RITZMAN, P.E.,
and DUANE FULLER, Jr. 1

This paper discusses Salt Lake City’s (SLC) response to the floods of 1983 and 1984. Management of flood fighting and cleanup is discussed. This includes communication, lines of responsibility, coordination, decisive response, and cleanup process. Both effective and non-effective aspects of SLC’s response to flooding is discussed in an effort to help others tailor their own workable plan. SLC’s current emergency flood control plan will be discussed.

BACKGROUND

Five streams drain into Salt Lake City, City Creek on the north, Red Butte, Emigration, and Parley’s Creeks on the east, and the Jordan River drains from Utah Lake, some 45 miles to the south. Red Butte and Emigration merge near Liberty Park, in the City, and flow in an underground storm drain, which is joined by Parley’s Creek about one mile further downstream. The three streams then jointly occupy a storm conduit under Thirteenth South Street for an additional mile and a half until they join the Jordan River. City Creek flows into the City from the north and in a conduit directly west about two miles to the Jordan River. The Surplus Canal, which was built originally by the pioneers in the 1800’s and later enlarged by the U.S. Army Corps of Engineers, is capable of accepting nearly all of the flow from the Jordan River at the south city limit and channeling it directly to the Great Salt Lake. These conveyance systems described as shown on Figure 1.

There is only one reservoir on the system of any significance. Mountain Dell reservoir in Parley’s Canyon is capable of holding 3200 acre-feet of water. This reservoir is not very large, in that it could have been filled nineteen times by the 1983 spring runoff alone. It does, however, play a key role in the water supply for Salt Lake City, and can he used to some extent for flood control. If the level gets much below 500 acre-feet, bottom mud is drawn into the City’s water treatment plant.

1 The authors all work for Salt Lake City Corporation. They are respectively, drainage engineer, operations engineer and assistant streets superintendent.
Salt Lake City experienced heavy snowmelt runoff during the springs of 1983 and 1984. In both years the snow pack was very high in water content and total depth at the start of the spring runoff.

In 1983 the extreme flooding potential was not evident until a series of snowstorms occurred the first few weeks of May. This delayed the snowmelt runoff, increased low elevation snowpack and cooled off temperatures. The weather warmed into the nineties just before the Memorial Day Weekend and very high flood peaks resulted. The City was forced to react to flood emergencies as they occurred. City Streets - 1300 South, North Temple and State Street - were diked mostly by volunteers to create rivers for the rushing flood water. Flood fight costs within SLC topped $5 million during 1983 with an additional $5 million being paid out for restoration.

FIGURE 1. Major Storm Water Systems in Salt Lake City, Utah
More apprehension was displayed during the spring of 1984. Snowpack and temperatures were monitored closely and preparations were made to fight potential flood problems. As part of this preparation, a detailed emergency flood control plan was prepared by the City Public Works Department. This plan defined responsibilities for each City department and established response action plans for the major problem areas. It also defined the relationships between the City and Salt Lake County Flood Control, and the levels of emergency at which various political entities should become involved. City engineers met to discuss all of the possible scenarios and alternatives to relieve the flows from the most critical locations. No idea was rejected as impossible until it was thoroughly discussed and investigated. The plan also addressed means to dike residential areas from the Great Salt Lake.

Weekly meetings were held with City personnel and County officials, wherein weather forecasts, stream flow forecasts, and current conditions were reviewed. A computer model was developed which simulated the stream flows and level of the Mountain Dell Reservoir on Parley's Creek. With various weather/temperature scenarios, the optimum storage of the reservoir could be determined. It was found that if 130 cubic feet per second could be diverted from any combination of the streams leading into the 1300 South Storm conduit, and if the weather remained somewhat close to normal, the stream drainage system could be controlled to handle the peak runoff. If, however, all three streams were to peak at the same time, 1300 South would have to be turned into a river again at a cost of over $1 million.

In 1983 the City reacted to flooding. In 1984 the City responded, the big contrast being the amount of advance planning - establishing the "what if's". This required pre-defining and evaluating problems and assessing the City capabilities. By planning properly in advance, the City was able to respond in a decisive and immediate manner. Flood fight costs for Salt Lake City in 1984 were about $600,000 with an additional $100,000 needed for restoration. This was despite the fact that combined peak flows in 1984 were nearly as high as 1983.

Salt Lake City experienced snowmelt floods during 1983 and 1984. Peaks during those years were very high with historic records being set in both years -- City Creek, Red Butte and Parley's during 1983 and Emigration and The Surplus Canal during 1984. Comparing the two years, damages within Salt Lake City during 1984 were significantly reduced while flood magnitudes were similar. This was the result of more effective operation of conveyance systems and diverting flood waters into local drains which normally do not handle snowmelt runoff. Implementation of this plan was effectively set in motion by advance planning and development of an Emergency Flood Control Plan.
Prior to the 1984 flood, a detailed flood plan was developed by Salt Lake City. This plan established and assigned City department responsibilities and set forth policies and procedures. The various City department flood fight responsibilities were modeled after their normal duties within the City. Some examples of these are as follows:

**Mayor** - Overall responsibility to control and direct flood control efforts.

**Chief Administrative Officer (CAO)** - Advise the Mayor on flood control programs and insure proper functioning of staff and operation center.

**City Attorney** - Prepare legal documents necessary for flood control operations. Process all claims for flood damage. Advise other departments on legal matters pertaining to flood fight operations.

**Public Works Director** - Responsible for the planning and preparation of flood control. Advise CAO and Mayor on all matters related to flood control. Control field operations.

**Chief of Police** - Maintain law and order. Provide necessary traffic control. Provide flood control documentation, video and still photos.

**Director of Finance** - Establish and maintain flood control cost accounting. Locate, purchase or lease necessary supplies and equipment.

**Chief of Fire Department** - Plan for and assist property owners in evacuation operations. Provide medical assistance.

**Superintendent of Parks** - Provide necessary personnel and equipment to the Public Works Department. Plan for and operate sandbag filling operation. Remove trees and debris from stream beds.

**Director of Personnel** - Plan for and provide health, food and drink comfort items to flood control personnel and volunteers. Prepare awards and recognition for outstanding service.

**City Engineer** - Administer necessary contracts for flood fighting. Plan for and supervise the construction of emergency sandbag berms as the situation dictates, using volunteer sandbaggers. Establish and operate the Public Works coordination duties.
Streets Superintendent - Monitor and clean streams and storm drains running through the City. Construction berms to control flood water. Keep open ditches and bridges clear of debris.

Traffic Engineer - Operate and maintain normal traffic operations. Coordinate barricading and signing of streets affected by flood control operations.

Fleet Management - Set up and operate water pumps as necessary. Deliver fuels and service to all stationary equipment. Maintain list of site equipment.

Under this organization plan the appropriate decision makers were able to meet in the command center to review operations and make decisions. A system of centralized decisions and decentralized implementation was set up. A quasi-military communication system was set up to provide for decision implementation.

A number of issues should be well thought out as part of an effective emergency flood control plan. These include:

Preplan - Evaluate where problems will occur. Listen to your people in the field, they know where your problems are.

Inventory - Evaluate available personnel and equipment resources. Locate additional needed equipment and personnel.

Establish staff responsibilities - Provide for planners, commanders and workers. Use normal lines of responsibility.

Coordination - Establish close contact with utility companies - gas, electrical, water, railroads, etc., volunteer groups, other flood agencies, etc.

Communication - Provide proper training and equipment. This can be the key to effective implementation.

Response - Cut thru red tape. Be assertive. This fosters community support. A bad decision is sometimes better than no decision.

Clean Up - Put as much effort into clean up as flood fight.

CONCLUSIONS

(1) An effective emergency flood plan requires advance planning and evaluation of potential problems.
(2) To respond effectively to a flood or any other emergency, potential scenarios should be listed and response action plans established for each.

(3) Action is more effective if major decisions are centralized and execution is decentralized. Extra involvement of the decision makers in flood fight execution can be detrimental.
SECTION VI - THISTLE LANDSLIDE
Movement of the Thistle landslide on the west side of the Spanish Fork Canyon, 0.5 mile downstream from Thistle, has been documented over a period of many years. Relatively minor movements of the toe of the slide had affected the railroad tracks which were on the west side of the canyon. Evidence of headward regressive movement had been reported. The slide had been mapped as 8000 feet long, 900 feet wide at the toe where it is confined between ridges of Jurassic sandstone. Volume was estimated at 25 million cubic yards. The primary geologic unit involved is the North Horn Formation of Cretaceous-Tertiary age. In April 1983, a major part of the slide began to move into the Spanish Fork Canyon elevating both the railroad tracks and the highway on the opposite side of the canyon. After attempts to maintain road and rail traffic through the canyon failed, an attempt was made to keep a channel open for the Spanish Fork River. When this proved impossible, emphasis was placed on controlling the lake that was developing behind the slide. The filling rate was calculated and construction was started on an overflow tunnel through the Jurassic sandstone on the east side of the canyon 170 feet above the river bed. The main mass of the landslide continued to move rapidly, greater than 70 feet of measured lateral movement per day. This mass movement and stabilization efforts which included the transfer of material from the lower foot onto the toe resulted in an engineered "dam" 200 feet high. Earth was moved on the surface of the slide to maintain access to the tunnel portals and prevent overtopping by the lake waters. After major movement of the slide had stopped, the downstream face of the slide was engineered to provide greater stability. Subsequently a drain tunnel was constructed and the lake drained. In addition to the transcontinental rail and highway traffic disrupted by the slide, the lake flooded the community and railyards at Thistle and the highway and railroad into south central Utah. The costs of relocating the highway and railroad, attempting to stabilize the slide, and draining the lake will total well in excess of $100 million. The railroad to central Utah has not been reopened. The economic loss to the railroad and to the economy of Utah was large. Further movement of portions of the Thistle landslide is likely in the near future, entirely probable in the long-term future, and inevitable in the geologic future.

Genevieve Atwood and Bruce N. Kaliser are with Utah Geological and Mineral Survey.
THISTLE LANDSLIDE EMERGENCY RAILROAD RELOCATION
THISTLE, UTAH

by D. E. Hilts

ABSTRACT

On April 14, 1983, a huge landslide slowly moved 80 to 90 million cubic yards of soil and rock into the Spanish Fork Canyon in central Utah, destroying the Denver & Rio Grande Western Railroad mainline tracks and U.S. Highways 6/89. The slide formed a 220-foot-high dam and the resulting lake which eventually submerged the town of Thistle, threatened the stability of the dam, and jeopardized the town of Spanish Fork.

Shannon and Wilson, Inc., designed and supervised construction of an emergency spillway to prevent overtopping of the dam. By predicting the eventual height of the still-growing dam and by estimating how fast the water would rise behind the dam, Shannon & Wilson, Inc., was able to locate the spillway to minimize the amount of water impounded behind the dam, while still allowing sufficient time for construction before the lake level reached the intake. Round-the-clock construction of the combination rock tunnel and steel pipeline began on April 25 and was completed 20 days later, just three days before the water reached the inlet. The location of the emergency spillway also determined the maximum lake level, thus allowing rail line relocation to begin. Shannon & Wilson, Inc., evaluated the local geology and determined the alignment, support requirements, and construction method for the 3,000-foot-long twin tunnel. The first tunnel was completed on July 3. Rail service was restored on July 4, 1983, completing in just 81 days a project which, under normal circumstances, would have taken more than a year to design and construct.

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Many people from both the public and private sectors were involved in the effort to overcome the difficulties created by the massive mudslide near the town of Thistle. The cooperation and coordination which occurred were tremendous. Speakers at this session will give some of the flavor for those events. I will limit my remarks to the efforts and accomplishments of the workers and contractors employed by UDOT. In so doing, in no way do I minimize the achievements of others which were necessary for the success of our project.

On April 14, 1983, the uplift and movement of the soils underlying U.S. 6 about one-half of a mile north of Thistle were so great that traffic was disrupted. Thistle has been the junction of U.S. 6 and 89 and the D&RGWR and the Marysvale Branch, since before the turn of the century. The highway, the railroad, and the streams have been joint tenants in the canyon since their earliest existence.

Our maintenance crews tried to repair the pavement, keep traffic moving and the stream within its banks. Four-foot metal pipe was hurriedly placed to carry the stream flow. Before it could be covered it was literally torn apart by the earth uplifting 8 to 10 inches per hour, and flowing horizontally some 2 feet each hour. Shortly the road was closed and highway traffic from Carbon, Emery, Grand, San Juan, Sanpete, and Sevier counties was forced to seek inconvenient detours which added 25 to 50 miles of length and an hour or more of time to each direction of travel. Ultimately, some 3 1/2 miles of U.S. 6 along Soldier Creek, and 3 1/2 miles of U.S. 89 along Thistle Creek, were covered by water and silt.

With the road closure we began to investigate ways to restore traffic. Mr. Archie Hamilton of District Four in Price was given overall responsibility for design and plan preparation. Our first hopes of going over the slide or benching into the cliffs above it were soon ruled out, as the 1 1/2 mile long, 9 million cu. yd, sliding mass relentlessly filled the canyon to a height of some 220 ft.
Our designers, surveyors, materials engineers, and geologists virtually lived in the field observing the slide and assessing the alternatives for getting around it. At the request of our District Four geologist and materials engineer, the U.S. Geological Survey extended their soon to be published geologic mapping to cover Billie's Mountain, and adjoining areas of our concern. We obtained copies of the 500 ft./inch with 10 ft. contour maps prepared by DARG. This mapping became the basic design survey information for not only the railroad, but UDOT and Water Resources.

While all the engineering, geologic, and materials information showed about the worst conditions imaginable, it soon became evident that our only solution was to cut through Billie's Mountain. Routes through other canyons and passes, benching into the cliffs, and tunneling were all considered before the staff recommended to the Transportation Commission on May 6, 1983, that we proceed in all haste through Billie's Mountain. The Commission, the state administration, legislative officials, and local leaders supported the decision as the most feasible and fastest way to permanently restore traffic flow and give relief to the economic strangulation of Southeastern Utah.

In preparing our plans we found ourselves in the unfamiliar situation of having our highway right-of-way being taken by the railroad through condemnation action. We felt it necessary to cooperate in their efforts to restore rail service. They were able to proceed faster than we were to at least partially restore the flow of commerce. To accommodate the railroad, it was necessary that we adjust our preferred alignment further into unstable hillside, and make longer approaches to the big cut.

In less than two weeks, after virtual round the clock design effort and three walks over the alignment by our engineers and geologists, our plans for construction were completed. Quantities for all items of work were included, right-of-way was designed, property owners were contacted, and right-of-entry given by each one without condemnation; coordination with state and federal agencies was accomplished, an Environmental Assessment was prepared by UDOT and approved by FHWA, and the U.S. Forest Service completed their own Environmental Assessment to provide us with right-of-way across their lands. Ordinarily, we would have been fortunate to have completed plans for such a project in three years. We would have performed investigative drilling at the cut from one end to the other. As it was, we had no time for even one drill hole. Outstanding effort and cooperation by District Four, the central office, and FHWA were required to accomplish so much in such a short time.

One week before bid opening we held a pre-advertising review for interested contractors. Three days later we advertised. Four days after that, on May 24, 1983, we opened the bids submitted by five contractors. W. W. Clyde of Springville was the low bidder at $22.7 million which was 11 percent above our estimate. On June 8, the contract and bond were approved and the contractor was directed to proceed. Mr. Arthur Chidester was assigned as our project engineer.
The project was 6.6 miles long with the 300 ft. deep open cut through Billie's Mountain at the mid-point. The largest item of work was 5.5 million yards of roadway excavation at $2.37/yard. Five percent grades from both directions elevated the new road to a height of about 600 ft. above the canyon floor. Pavement is bituminous. The typical section consists of four 12 ft. traffic lanes, a 16 ft. median, and 8 ft. shoulders. This typical section is necessary to allow safe passing opportunities on the steep grades and to meet traffic demands in the future.

A special provision required that two paved lanes be opened to traffic by December 1, 1983, in order to avoid $50,000 per day liquidated damages. With this incentive, I think the contractor started work even before award of the contract. He worked round the clock and, during the summer and fall, earned money at the fastest rate we have ever experienced.

Considering the time we had for design and the fast pace of construction, the job has gone very well. There have been extreme problems with cut stability and the weather. The unstable North Horn Formation has fulfilled our worst fears. It has been necessary to flatten slopes, modify grades, and shift alignment to solve the problems caused by rock slides. Even with these problems and with clear hindsight, we make no apology for the decision to go over Billie's Mountain rather than around and to open cut rather than tunnel. Had there been the luxury of a normal three-year design period, and a three-year construction time we would have reached virtually the same alignment, and design and construction decisions.

By October 10 the contractor had actually moved and placed the 5.5 million cu. yds. of roadway excavation called for in the plans. In December, before the cut was ready for the two paved lanes, slides had made it necessary to remove an additional 1,000,000 cu. yds. of material. The weather was very good through the first week of November. In the second week it changed completely and we began to experience one of the worst winters on record. Paving of the two lanes was accomplished with about a foot of snow on the ground. The road was officially opened to traffic on New Years Eve. Though this was 30 days beyond the deadline, no liquidated damages were assessed because of the extenuating circumstances.

Because of the unstable cut slopes we stationed watchmen to give warning and close the road whenever slide activity was observed. Closure was necessary on January 3 and 6, and again from March 16-30. Since then the contractor has proceeded to flatten and stabilize the cut. He has now moved about 2,000,000 more yards of roadway excavation than originally planned. With the contract time remaining we are hopeful that the full four-lane project can be completed this fall.

Even with the rush to prepare plans, we were able to incorporate recommendations from the Division of Wildlife Resources to provide a special baffle design in the floor of the large box structure at Diamond Fork. This was to allow the passage of the fish upstream through the box. In recognition of this design we received an award from American Fisheries Society in February, 1984.
The effort that has gone into restoring traffic flow on U.S. 89 has also been significant and carried on concurrently with that on U.S. 6. Lake Thistle was the major problem. We looked at alternates with and without the lake drained. The various alignment proposals and the decision to drain or not to drain the lake became very controversial. This made it impossible for us to proceed with U.S. 89 construction as rapidly as on U.S. 6. However, after the Division of Water Resources was able to make a commitment that the lake would be drained by fall, we were able to keep the commitment that at least the temporary reconnection of U.S. 89 would occur concurrently with the opening of U.S. 6.

We hired a consultant to review our proposals for both the permanent and temporary reconnection, and to obtain a divers report of the conditions of the submerged roadway and bridges. The divers reported that silt deposition was minimal and removal was feasible. The bridges had not failed. We immediately negotiated a $1.5 million supplemental agreement with W. W. Clyde Co. for a temporary reconnection to serve until a permanent solution could be found. They were to proceed immediately with the construction of a new U.S. 89 - U.S. 6 intersection and a short connection back to the old road about 1 1/2 miles east of the old junction. This required a temporary grade railroad crossing of the new tracks and the removal of silt from the roadway as the water receded. This work commenced in late September and was accomplished without unexpected difficulty.

Initially it was felt that the most feasible method of permanently reconnecting U.S. 89 (with or without a permanent lake) was to construct a new road which stayed above the high water level. It would have crossed Soldier Creek just east of the old junction on a 1300 ft. bridge some 250 ft. above the bottom of the lake. As we proceeded with geologic and foundation investigation it became clear that this alternate was unfeasible. The foundation conditions on the east slope of Billie's Mountain were so poor that costs would have been prohibitive and the bridge would have always been at risk from foundation failure and the unstable hillside.

We are now proceeding with plans for a permanent reconnection which will utilize much of the old road in the canyon bottom and the intersection with U.S. 6 which is now serving as the temporary connection. New bridges to cross over the new D&RG mainline, and their proposed reconstruction of the Marysvale Branch, will be required. Some grade raising and channel stabilization work will be required at the Thistle town site. We hope to have plans ready and start construction of this $9-10 million project before this summer ends.

When the work is finally completed, we will have expended about $45 million in getting U.S. 6 and U.S. 89 back into operation. In the words of Governor Matheson as he contemplated the devastation of the floods and slides and the fact that Utah is the second most arid state in the union, "This is a helluva way to run a desert!"
THE EMERGENCY DRAINING OF THISTLE LAKE

Robert L. Morgan, State Engineer  
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During the first part of April, 1983, a major landslide involving some 15-25 million cubic yards of material began moving on the west side of Spanish Fork Canyon. The landslide, which was located about one-half mile north of Thistle, eventually formed a 225-foot high dam across the canyon and blocked U.S. Highways 50 and 6, the Denver and Rio Grande Western Railroad, and eventually inundated U.S. Highway 89. The slide created a natural lake which filled very rapidly by the melting snows, and eventually created an impoundment totaling approximately 62,000 acre feet of water to a depth of 180 feet.

Because early undertakings to keep the slide breached had failed, Mother Nature was allowed to take her course, and the slide very rapidly became a dam. As soon as the water started to be impounded, a major concern was raised as to the stability of this dam. It was feared that a rapid breach could cause major damage in the downstream floodplain and within the city of Spanish Fork, Utah. On April 27, 1983, Governor Matheson requested that the landslide and flooded area be declared a federal disaster. This request was granted by President Reagan on April 30, 1983. Granting of the request resulted in federal monies being made available to the State of Utah to cope with this disaster.

On April 29, 1983, Woodward-Clyde Consultants was retained by the State of Utah, Department of Public Safety, to design and oversee the construction of: (1) a permanent drain for the lake; and (2) a permanent means of diverting the Spanish Fork River around Thistle slide. Because the slide had essentially become a natural dam, the State Engineer, Dee Hansen, was asked to supervise and monitor the construction of the drainage facility and the draining of the lake.

One of the first tasks was the selection of a contractor to perform the construction work. Proposals were solicited from several contractors based upon a preliminary approach to the project. Based upon these proposals, the firm of Morrison-Knudson (M-K) of Boise was selected as the contractor.

Concurrent with the selection of a contractor was also the investigation process to determine the drainage method best suited for this locality. Basic factors involved in the selection of the drainage method were: (1) flexibility to deal with unexpected emergency situations; (2) a simple construction process to avoid confusion; and (3) a short construction time frame.
to lessen the emergency nature of the project. It was decided that the drainage facility would consist of a tunnel driven into Billy's Mountain around the east side of the Thistle slide. The tunnel would then terminate at a vertical shaft to be located immediately adjacent to the impounded water. Water from the natural lake would then be diverted into the shaft by successive benching efforts which would involve removal of the material between the vertical shaft and the lake.

The actual tunnel involved the construction of a 2,260-foot long, 13-foot wide by 12-foot high, finished diameter, concrete-lined, horseshoe-shaped, tunnel. The downstream portal elevation was set at 5000.55 feet MSL, and the upstream entrance was constructed to elevation 5030 feet MSL. The tunnel was excavated through Navajo (Nugget) sandstone and was supported by steel sets on 6-foot centers. The tunnel excavation terminated approximately 300 feet from the canyon wall and 1200 feet upstream from the dam. A horizontal bench was excavated in the canyon wall close to the edge of the lake and vertically above the upstream end of the tunnel. A 180-foot deep by 16-foot diameter vertical shaft was excavated using a raised bore machine. Upon completion of the tunnel and shaft, the tunnel was lined with concrete. Tunnel excavation began on May 24, 1983, and was completed on July 22, 1983. The shaft was completed on August 5, and cast-in-place concrete for the tunnel lining and valve bulkhead was completed on September 19. The control valves used to release waters into the tunnel were installed, and the actual drainage of the lake started October 1.

The lake was drained by benching downward in successive 10-foot to 45-foot lifts to form an inlet channel between the lake and the shaft. Reservoir water discharge volumes were controlled by two knife gate valves mounted on the concrete bulkhead installed at the entrance of the tunnel.

Figure 8, which is attached, shows the drainage schedule and the lift sequence as constructed.

Some special design considerations that were utilized in locating the outlet portal were:

1. to minimize the length of the drainage tunnel but still keep the use of any possible northward movement of the slide;

2. to provide separation of the outlet portal and the emergency spillway discharge to prevent water from interfering with the tunneling operations;

3. to try and minimize interference between the cut slopes of the outlet channel and the muck piles generated by the tunneling effort of the Denver and Rio Grande Railroad located upslope from the drainage tunnel;

4. to minimize the length of the outlet channel and to provide a smooth transition in the natural river channel.
(5) to minimize excavation volumes to facilitate rapid excavation of the outlet portal under emergency conditions.

The general construction of the project started on May 11 as M-K began mobilizing equipment and personnel to the site. On May 24, after removing some 44,000 bank cubic yards of earth and rock to develop the portal access, M-K commenced tunnel excavation. Driving of the tunnel progressed on a 3-shift, 7-day per week basis until July 22, when excavation was completed. The tunnel was driven by conventional drill and shoot methods. Overall, the progress averaged 12.8 feet per shift, and 38.8 feet per day, with peak rates of 28 feet in one shift, and 60 feet in one day.

Frontier-Kemper served as the shaft drilling subcontractor to M-K, and mobilized equipment at the site in mid-July. A pilot hole was drilled from the previously-described bench to the tunnel below. Drill steel was inserted in the pilot hole and a 16-foot diameter raised bore bit was assembled onto the drill steel. Raised drilling began on July 28 and was terminated August 2, approximately 16 feet below the working bench surface. A total of approximately 154 feet of shaft was drilled at an average daily rate of 25.7 feet per day. The remaining work in the shaft "plug" was drilled and shot by conventional methods following the demobilization of the raised bore.

Excavation of the inlet channel and drainage of the Thistle Lake was essentially completed by December 25, 1983. After a Christmas and New Year's holiday, M-K's bolting and excavation work continued. On January 18-19, 1984, the bulkhead and control valves were removed. The final channel trim shot was made on January 27, 1984. Contractors' crews and equipment were demobilized by February 17, 1984.
INUNDATION OF THISTLE - APRIL 1983

COMPLETE EXCAVATION OF ROCK CUT BETWEEN SHAFT AND THISTLE LAKE
JANUARY 1984
GENERAL OVERALL VIEW OF THISTLE LAKE - JUNE 1983