ER-3139

ENGINEERING GEOLOGY OF THE BATTLE MOUNTAIN LANDSLIDE SOUTH OF MINTURN, COLORADO

by

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ER-3139

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ABSTRACT

The Battle Mountain Landslide has existed below the cliffs south of Minturn, Colorado since late Wisconsin time. In the past few years, a portion of this original slide has reactivated. The costs of maintaining U.S. Highway 24 through this slide have become prohibitive, so a study of the engineering characteristics of the failure was initiated to determine if stabilization was economically feasible.

The landslide overlies a gently folded Paleozoic sedimentary sequence. The oldest unit beneath the slide is the Belden Formation, a marine shale and limestone sequence. It is overlain by the Minturn Formation, a nearshore accumulation of clastic terrestrial material resulting from the uplift of the Ancestral Rockies. Both units are of Pennsylvanian age. The landslide itself is composed of colluvial material derived from the cliffs of the Minturn Formation. During the Pinedale glaciation, a glacier advanced down Cross Creek, immediately west of the site, and diverted the Eagle River eastward, oversteepening the cliffs. Accumulations of colluvial materials at the base of the cliffs eventually failed and the original landslide formed. In recent years, a portion of this original landslide mass has reactivated, forming the modern slide.

The spoon-shaped sliding mass is characterized by a

circular failure which was located using 16 observation wells and an inclinometer installed on the slide. Movement, which was monitored in spring and summer of 1985, can be correlated with increases in the water table caused by spring snowmelt. Water table elevations varied by as much as 12 feet, and during this time up to 14 inches of movement were observed.

Maps of the water table and the failure surfaces, and a plane table topographic map were prepared as part of the study. These maps were digitized and used in volumetric calculations and a computerized stability analysis. Volume of slide materials is 1.08×10^6 cubic yards. Because soil strength values were unknown, a sensitivity analysis was run on soil strength values versus factor of safety. This showed that the angle of friction had more effect on the stability than did the cohesion. Failure zone strength parameters of zero cohesion with a 32° friction angle were chosen for comparison analyses. These values yielded a safety factor of 0.971 under current conditions, which could be increased to 1.065 by lowering the potentiometric surface by 10 feet.

The following procedures are recommended to stabilize the Battle Mountain Landslide:

1) Install 4,955 feet of horizontal drains from the five specified drill pads.

2) Rehabilitate surface drainage systems along the highway, to further reduce water infiltration rates.
3) Reduce the weight of the slide by minimal regrading of the excessively wide highway shoulder.
4) In conjuction with (3) above, consideration of a more extensive reconstruction of the road fill, using reinforced earth techniques to further reduce its weight.

Rough cost analyses indicate that the installation of the full set of horizontal drains would cost about $50,000; the regrading and removal of the excess fill might cost as much as $72,000, while the excavation and removal of up to half the failing slide would cost $6.5 million. Furthermore, assuming a 20 year life expectancy for these drains, their annual cost of $2,500 is substantially less than the current average maintenance cost for roadway resurfacing of $4,300. Accordingly, the installation of the horizontal drains appears to have the highest priority.

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1.0 INTRODUCTION

Since the time that U.S. Highway 24 was rerouted across the Battle Mountain Landslide in the late thirties, this section of road has required constant maintenance to keep it safe. In 1984, Mr. Robert K. Barrett of the Colorado Highway Department began a drilling program on this slide and two other landslides in the area. Dr. A. Keith Turner at the Colorado School of Mines was contacted and asked to conduct a water level and slide movement monitoring program over the next year. That request eventually resulted in this study of the Battle Mountain Landslide.

1.1 Objectives of the Study

This study was undertaken as part of a larger project commissioned by the Colorado Highway Department to investigate some troublesome landslides in the Vail area. Specifically, this report presents the results of an investigation conducted at the Battle Mountain Landslide from September, 1984 to September, 1985.

The objectives of this study were:

1. Definition of the slide and description of its physical characteristics;
2. Documentation of hydrologic and mass movement monitoring program;
3. Investigation of local geologic conditions that may

affect the slide;

4. Analysis of the relationships between geology, hydrology, and slide movement;
5. Analysis of slope stability; and
6. Recommendations concerning the most appropriate and cost effective remedial measures for slide control.

1.2 Methods Used in this Study

The slide was characterized by mapping the topography, potentiometric surface, and the failure surface, and by monitering slide movement. Observation well water levels were monitored on a monthly basis from September, 1984 to March, 1985. From March to May, measurements were taken every two weeks, then, beginning in May, potentiometric measurements were taken weekly until mid-June, followed by monthly measurements in July through September. Movement was monitored weekly from May 7 to June 13, the period of greatest slide activity, then monthly into September.

The surface topography was mapped by plane table methods at a scale of 1:480 (1 inch =40 feet). Scarps, seepage zones, and standing water locations, as well as cultural information, were mapped. The failure zones in the subsurface were located by analysis of data from a single inclinometer located in the slide. This information was supplemented by observations of depths to the failure zone at the observation wells, found by locating bends in the well casings.

The potentiometric surface was mapped by monitoring 16 observation wells and by locating the positions of standing water, seeps, and springs. In addition, groundwater discharge rates from the toe of the slide were measured.

Surface movements were measured by transit and differential leveling surveys or by direct measurement across zones of movement. The direction in which trees were leaning were plotted on the base map to give additional indications of directions of mass movement. Subsurface movement rates were monitored by data taken from the inclinometer.

The data analysis involved three phases. First, the observed data were reviewed to determine if there were correlations among different data types; for example, between water level changes and slide movements. As a consequence of these reviews, the slide was divided into four zones, each having a different rate or character of movement. Second, the maps of the topography, the potentiometric surface, and the major failure surface were digitized on an Autotrol CAD system. Volumetric calculations were then made with this system. Third, the stability of the slide was analyzed using the STABL2 slope stability program. A series of runs were made to establish the sensitivity of the results to the soil strength parameters for the slide materials. Typical soil parameters were selected from published sources, because accurate soil strength data were not available for this slide.

Based on the results of the stability analysis, alternative remedial measures were considered, and the use of horizontal drains selected as the most appropriate stabilizing measure. A series of priority ranked horizontal drain locations were specified, and a preliminary cost-benefit analysis was performed.

1.3 Location

The Battle Mountain Slide is located in the mountains of central Colorado, 110 miles west of Denver, and 5 miles west of Vail, immediately south of Minturn (see figures 1 and 2). Physiographically, the study area lies southwest of the Gore Range and just east of the northern flank of the Sawatch Range.

The landslide lies on the east wall of the Eagle River Canyon, the major drainage in the area. The east wall of the canyon, which rises 1800 feet over the river, is composed of clastic sedimentary beds dipping gently to the northeast. The toe of the landslide lies about 600 feet from the river and is some 80 feet higher. From this low point, the slide rises vertically 600 feet up the canyon

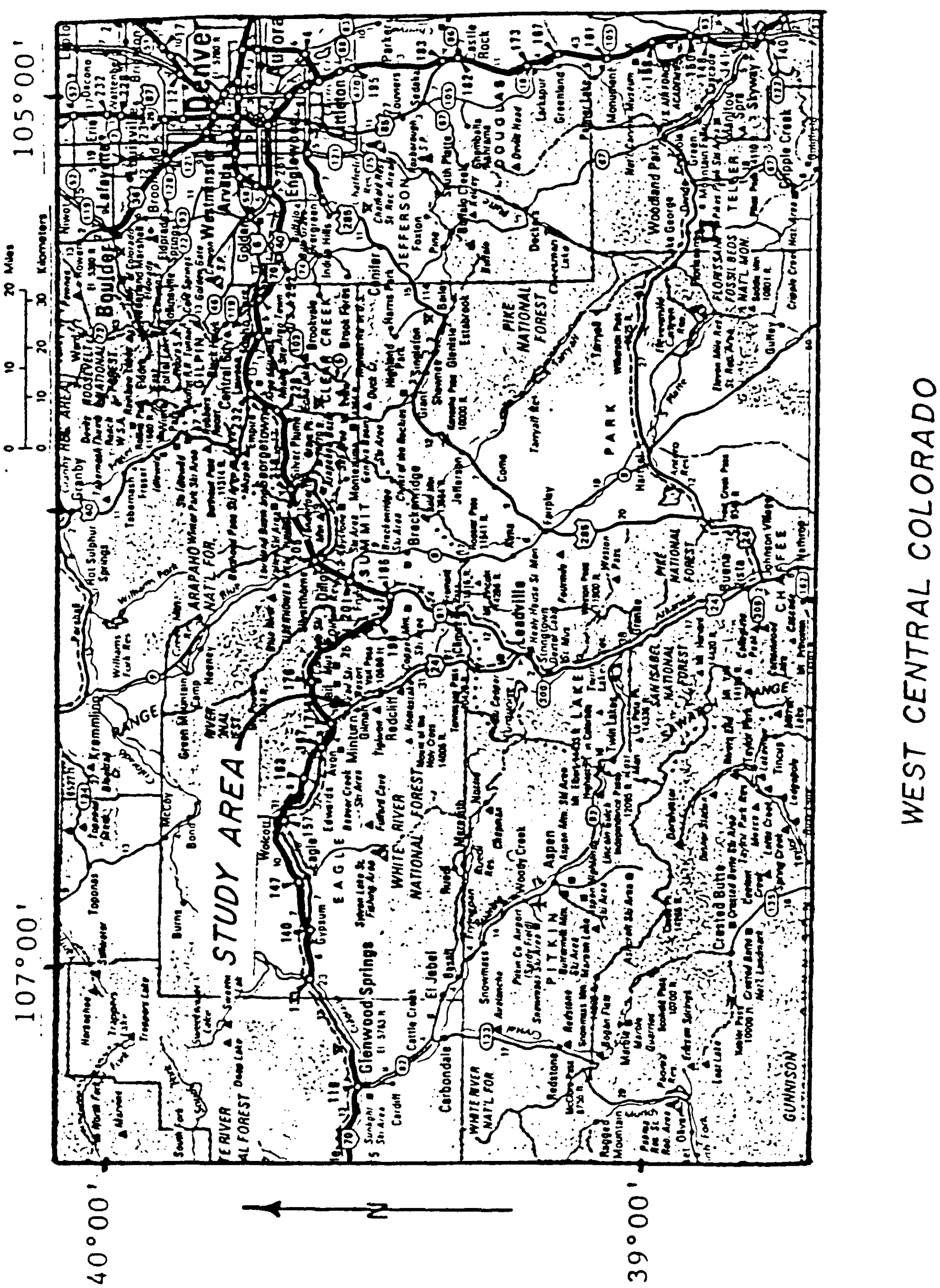


Figure 1. Study area location.

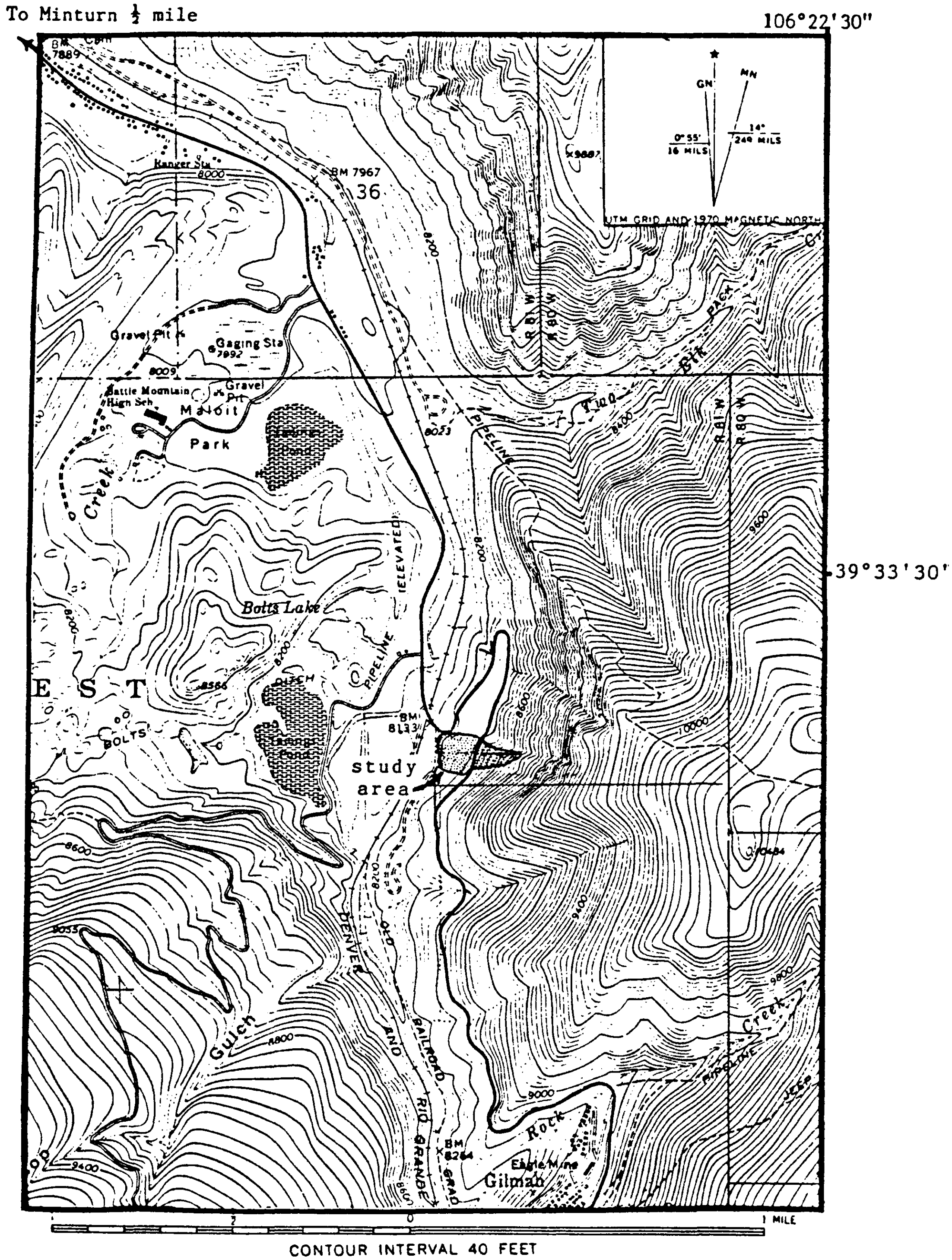


Figure 2. Study area topography from Minturn 7½' quadrangle

wall. Plate 1 presents a detailed topographic picture of the immediate study area.

1.4 Previous Work

Previous investigations of landslides in the area have been limited to site-specific studies of individual slides or debris-flows, especially in the Vail area (Mears, 1985). Of particular interest are the engineering reports pertaining to the construction of Interstate 70 over Vail Pass (Transportation Research Board, 1979). That highway was routed through several active landslides, and some of these slides are in the Minturn Formation, as is the Battle Mountain Slide.

The Minturn 15 minute Quadrangle has been extensively studied by Tweto and Lovering, who reported their results in a U.S. Geological Survey Professional Paper (Tweto and Lovering, 1977). They described the Minturn Formation type section and defined a section of the Belden Formation, both measured within one mile of the Battle Mountain Landslide. Bedrock and surficial geology units in the immediate area are described and the geologic history of the quadrangle is discussed in some detail. The report also includes some discussion on the landslides and related disturbances in this area.

The highway that crosses the slide was built in the late 1930's, but asphalt tonnage records have been kept by the Highway Department only for the past eight years. These records give a quantitative measure of the seasonality and gross amount of movement (see Appendix A).

A 1:480 scale plane table map of the slide was made in August of 1984 by the Highway Department. This map locates major scarps and seeps and was updated and used as a base map for this work (Coffee and Adler, 1984). The Highway Department had drilled sixteen observation wells and installed one inclinometer on, and immediately around, the slide. Drilling records were obtained for fourteen of the well installations and for the inclinometer installation. These were used in subsurface evaluation work.

Because of the large number of iterative calculations and variables involved in landslide analysis, computers have become an important tool in this type of work (Boutrup, 1977). One of the latest state-of-the-art programs written for slope stability analysis was developed over the last ten years at Purdue University (Lovell, Sharma, and Carpenter, 1985). This program, known as STABL4 in its latest version (an older version is STABL2), is the only program with routines that handle non-circular failures (Siegel, 1975a,b). Because of its versatility and availability, STABL2 was used in the study.

The Transportation Research Board, Special Report 176 (Schuster and Krizek, 1978) discusses stabilization by drainage and other methods. The use of horizontal drains to stabilize cut slopes is described by Newby (1953). The mechanisms for analyzing seepage and drainage of groundwater are described by Cedergren (1967), and methods of subsurface pavement drainage are discussed by Ridgeway (1982).

2.0 REGIONAL SETTING OF THE BATTLE MOUNTAIN SLIDE

The Battle Mountain Landslide is located on the site of a large prehistoric landslide that occurred subsequent to the retreat of the glaciers of Pinedale age. The end of the Pinedale glaciation along the Colorado Front Range has been dated at about 10,000 years ago (Benedict,1973). Accordingly, this is the maximum age for the original prehistoric landslide.

The current, or modern, landslide, which is the subject of this investigation, involves a remobilization of part of this original slide mass by a combination of natural and man-made events. It is unknown if the original landslide was ever truly stable, but movements became obvious and of concern only after U.S. Highway 24 was rerouted across the slide in the 1930's. Since that time, maintenance has been required more or less continuously to counteract the effects of the slide movements. Over the past few years these movements have intensified and the maintenance efforts have become a point of more serious concern.

The following sections describe some of the natural environmental factors which may affect the overall stability of both the prehistoric slide mass and the modern landslide.

2.1 Climate

The climate of the Minturn area is cool and semi-arid. Average annual precipitation is 20 inches, which is distributed evenly through the year with a slight increase in spring (Berry, 1968). In Dillon, 10 miles northeast at an elevation of 9065 feet, yearly average snowfall is 159 inches. The elevation of the study area is 8500 feet; thus, similar snowfall averages can be expected.

Average winter temperature is 22.8°F, and summer temperature is 46.8°F. Intense afternoon thunderstorms are common in summer. They may deliver rainfall at the rate of two to three inches per hour, but usually are of short duration.

An increase in precipitation over the last three years has helped initiate landslide movement in the area (Mears, 1985). Mears studied recent mudflows for the Town of Vail and concluded that the increase in landslide activity correlates with increased precipitation. There is also a strong inference that mudflow frequency increases with periods of high temperatures during the spring thaw (Mears, 1985).

Because of the rugged topography of the area, microclimatic variations are extreme. Slope orientation is very important. The landslide faces west, which promotes moderate snow accumulation and rapid snowmelt. To complicate the situation, the slide lies under steep cliffs which avalanche their snow accumulation though well-developed chutes, some of which run out directly onto the landslide. These avalanches act as a source of material deposited on the landslide.

2.2 Physiography and Geomorphology

The predominant geomorphic process of the area is mechanical weathering, either by frost action or by running water (Ritter, 1978). Valleys and canyons in the area are aggrading by deposition of colluvial and alluvial material under the present climatic conditions.

During Tertiary and Quaternary time, subsequent to the Laramide Orogeny that produced the modern Rockies, drainage evolution has been the predominant geologic process in the region. Glaciation has played a large role in shaping the present topography, with either erosional or depositional evidence of as many as nine distinct glacial advances in the area (Tweto and Lovering, 1977). Regional glacial deposits range in age from Pre-Bull Lake to Pinedale (Tweto and Lovering, 1977).

In the immediate study area, the relatively unweathered terminal moraines of the Pinedale glacial advances down Cross Creek Canyon are most important. All three stades of the Pinedale glaciers flowed eastward to form a terminal moraine that was deposited near the mouth of Cross Creek. The Eagle River was diverted to the east, around the moraine. This caused undercutting and subsequent oversteepening of the cliffs on the east side of the valley (Tweto and Lovering, 1977). Figure 3 shows the terminal moraine sequence that diverted the Eagle River toward the cliffs.

Because of the metastable conditions caused by the steep canyon wall and the varying lithologies of the stratigraphic sequence, this side of the canyon is subject to a number of geologic hazards. They include snow and debris avalanches, rockfalls, landslides, and soil creep.

2.3 Regional Geology

The geology of the region consists of a sequence of gently folded Paleozoic and Mesozoic sediments bounded on the east and west by uplifted crystalline Precambrian rocks. A geologic map of the immediate area, interpreted by the author in 1985, is shown in Figure 4. Tweto and Lovering (1977) have studied the area extensively.

The oldest stratigraphic unit underlying the slide is the Leadville Dolomite, of Mississippian age. This massive gray marine dolomite acts as a host rock for local economic mineralization.

At the landslide site, the Molas Formation is absent and the Leadville is unconformably overlain by the Belden Formation, a shallow sequence of interbedded shales, car-

Figure 3. Pinedale terminal moraine at the mouth of Cross Creek Canyon that diverted the Eagle River to the east, oversteepening the valley walls.

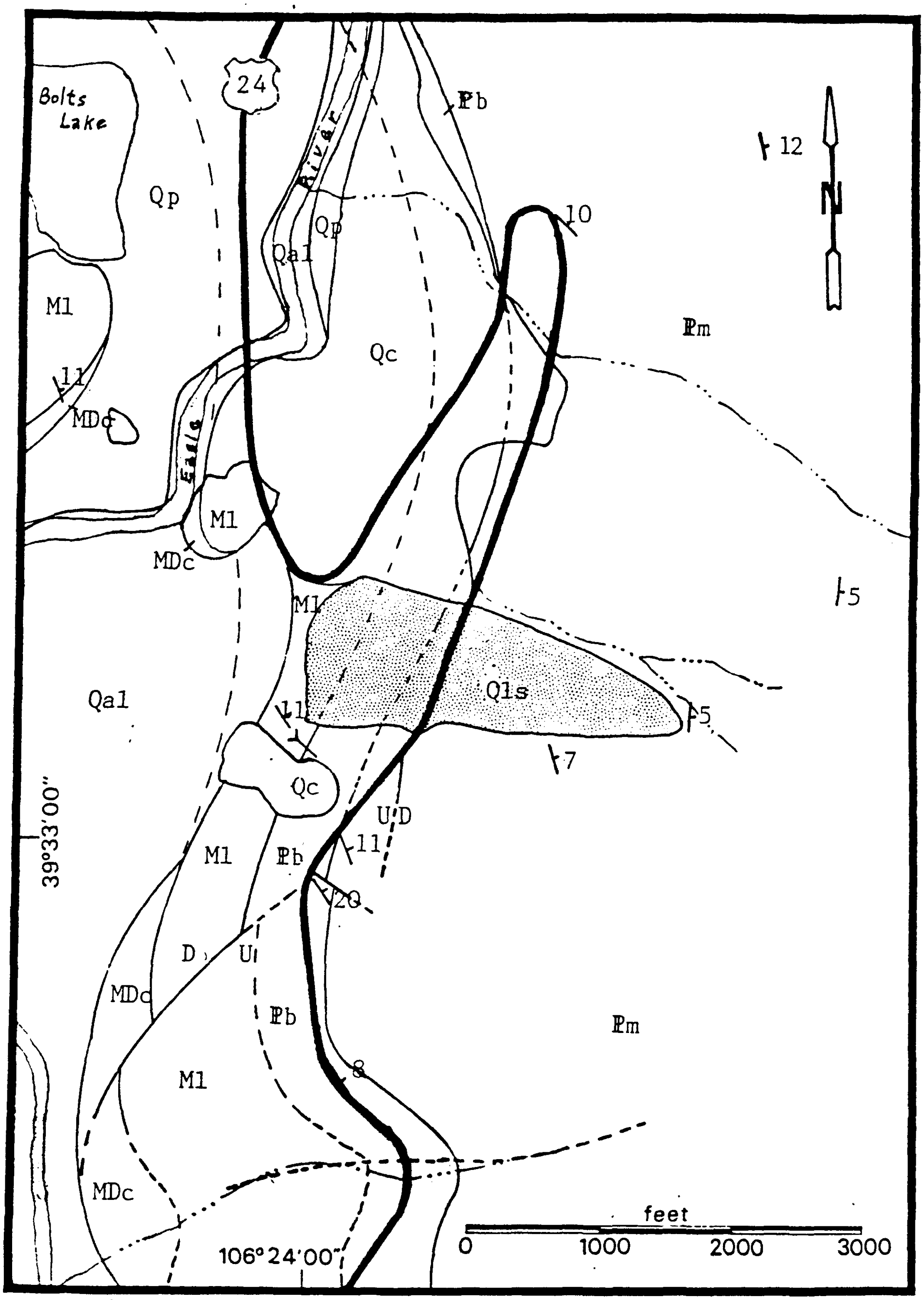


Figure 4. Geologic map of the Battle Mountain Landslide vicinity (see figure 5 for legend).

Qal	Unconsolidated alluvial deposits of gravel, sand, and silt
Qc	Unconsolidated colluvial deposits, mainly derived from the Minturn Formation
Qls	Unconsolidated landslide material composed of colluvium
₽m	Minturn Formation (Pennsylvanian)-Gray, tan, and red sandstone, shale, and conglomerate up to 6000 ft. thick
₽b	Belden Formation (Pennsylvanian)-Gray to black carbonate, shale, and sandstone, about 900 ft. thick
Ml	Leadville Dolomite (Mississippian)-Gray, massive dolomite, about 150 ft. thick in this area
MDc	Chaffee Formation (Mississippian-Devonian)-Banded light tan and gray sandstone and carbonate,about 150 ft. thick in this area
	Contact, dashed where concealed or implied
	Fault, dashed where implied
12	Strike and dip of beds
	Adit

Figure 5. Legend for geologic map shown in figure 4.

bonates, and fine-grained sandstones. The Belden underlies the lower half of the slide, from Highway 24 to the toe area.

The youngest bedrock unit found under the slide is the Pennsylvanian Minturn Formation. The Minturn is a nearshore clastic accumulation of debris shed from the Ancestral Rockies, which rose about 15 miles to the east of the study area. The contact between the Minturn and Belden Formations is gradational and runs almost directly under the highway. The Minturn underlies the upper half of the slide and forms the cliffs, some 1800 feet high, which occur above the slide.

Geologic structures found in the area generally are related to the uplifted Gore and Sawatch Ranges. In the vicinity of the landslide, no major faulting and folding exists, but faulting due to the Sawatch intrusion, immediately to the west, is apparant.

In general, surficial drainage appears to follow the northeast-southwest trending faults in the basement rocks of the Sawatch Range, to the west. These faults are mapped in the 1978 1° x 2° Leadville Quadrangle by Tweto, Moench, and Reed. Drainages to the east, in the Paleozoic sedimentary rocks, although not mapped as faults, appear as lineaments on aerial photographs. These lineaments can readily be extended back to the Precambrian rocks to the west.

In this respect, the location of the landslide is structurally controlled. Its orientation is similar to that of surrounding lineaments, and it rests on the northwest facing slope of a large drainage. However, on a qualitative level, sandstone and conglomerate strata are relatively undeformed and no preferential jointing patterns were observed.

2.4 Geology of the Immediate Landslide Area

The geology of the immediate area of the landslide was examined in some detail. The site was mapped using conventional surveying and geologic mapping techniques.

Bedrock geology, as noted in the Regional Geology section, consists of gently dipping beds of the Minturn and Belden Formations situated on a steep slope overlooking the Eagle River. Some minor folding and faulting is apparent in the roadcut immediately south of the slide.

The moving slide mass overlies the Minturn and Belden Fromations, with the Minturn-Belden contact running roughly under the highway. The toe of the slide rests about twenty feet below the Belden-Leadville contact.

The landslide is composed of weathered colluvium and detached segments of bedrock derived from the cliffs of the Minturn Formation that overlook the slide. Although the source material cannot be correlated with specific bedrock strata, it can be inferred from the slide lithology found in the well logs that the source material is largely from the non-marine facies of the Minturn, which begins about 365 feet above the highway.

3.0 FIELD INVESTIGATIONS OF THE BATTLE MOUNTAIN LANDSLIDE

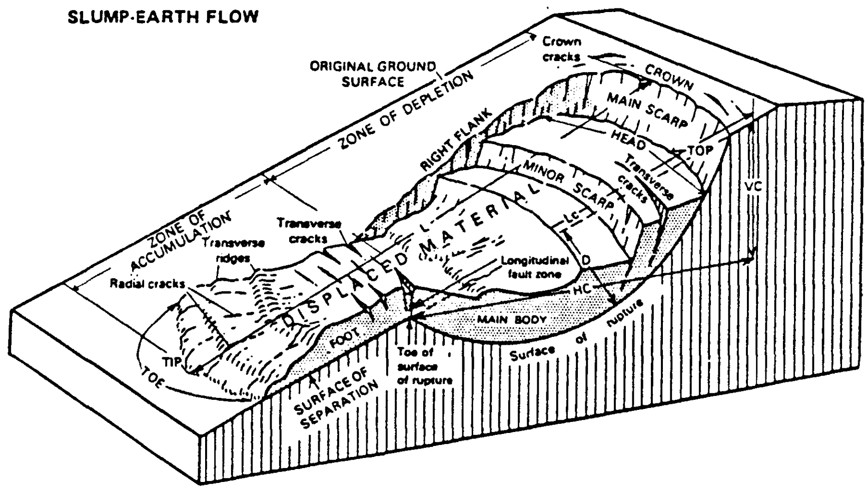
In this report, the terminology for describing landslides, developed in the Transportation Research Board Special Report 176 (Schuster and Krizek, 1978), will be used. Figure 6 shows the principal features of a landslide similar to Battle Mountain. The standard method of specifying direction on any landslide is to consider the observer to be standing at the head of the slide looking down the failure. Then the "left flank" will be on his left, and the "right flank" on his right. Accordingly, at Battle Mountain, the right flank is the northern edge of movement, nearest Minturn, and the left flank is the southern edge of movement, since the slide is generally moving to the west-northwest.

3.1 Frequency and Types of Observations

As described previously, observations were conducted at intervals throughout a one year period (September, 1984 to September, 1985). However, observations were limited in the winter, when several observation wells could not be found in the deep snow. Also, a number of observation wells were not installed until after November, 1984, so their data do not cover the entire year.

The frequency of observations also varied by season. The most intense observation effort was concentrated in a

SLUMP-EARTH FLOW



NOMENCLATURE

MAIN SCARP—A steep surface on the undisturbed ground around the periphery of the slide, caused by the movement of slide material away from undisturbed ground. The projection of the scarp surface under the displaced material becomes the surface of rupture.

MINOR SCARP—A steep surface on the displaced material produced by differential movements within the sliding mass.

HEAD—The upper parts of the slide material along the contact between the displaced material and the main scarp.

TOP—The highest point of contact between the displaced material and the main scarp.

TOE OF SURFACE OF RUPTURE—The intersection (sometimes buried) between the lower part of the surface of rupture and the original ground surface.

TOE—The margin of displaced material most distant from the main scarp.

TIP—The point on the toe most distant from the top of the slide.

FOOT—That portion of the displaced material that lies downslope from the toe of the surface of rupture.

MAIN BODY—That part of the displaced material that overlies the surface of rupture between the main scarp and toe of the surface of rupture.

FLANK—The side of the landslide.

CROWN—The material that is still in place, practically undisplaced and adjacent to the highest parts of the main scarp.

ORIGINAL GROUND SURFACE—The slope that existed before the movement which is being considered took place. If this is the surface of an older landslide, that fact should be stated.

LEFT AND RIGHT—Compass directions are preferable in describing a slide, but if right and left are used they refer to the slide as viewed from the crown.

SURFACE OF SEPARATION—The surface separating displaced material from stable material but not known to have been a surface on which failure occurred.

DISPLACED MATERIAL—The material that has moved away from its original position on the slope. It may be in a deformed or undeformed state.

ZONE OF DEPLETION—The area within which the displaced material lies below the original ground surface.

ZONE OF ACCUMULATION—The area within which the displaced material lies above the original ground surface.

Figure 6. Terminology used to describe the parts of the landslide in this study (from Schuster and Krizek, 1978).

six week period in May and early June, when the slide movements were the greatest. Field investigations included the following activities:

1) Installation of boreholes;
2) Construction of a plane table map;
3) Definition of the failure surface or surfaces;
4) Hydrologic observations, and;
5) Movement observations.

The following sections describe these activities in more detail.

3.2 Installation of Boreholes

Two sets of observation wells were drilled at this site. In the late summer and fall of 1984, the Colorado Highway Department installed ten observation wells, identified as TH-1 through TH-10, and one inclinometer (TH-11). These holes were located, drilled and logged prior to the involvement of any CSM personnel.

In October 1984, after CSM personnel were contacted, the locations of six additional observation wells were agreed to and drilling of these holes, identified as CSM-1 through CSM-6, began in November of 1984. Snow and cold weather conditions delayed the completion of these additional observation wells. The holes were logged by several different personnel, some from CSM and some from the Highway Department.

Wells TH-1 through TH-10 were drilled using a 4-inch rotary drilling bit and ordinary drilling mud to wash cuttings. They were cased with 3/8 inch PVC pipe, backfilled with sand and capped with bentonite. Spoon samples were taken at infrequent intervals, but neither samples or records of the samples could be located.

The inclinometer boring, TH-11, was also drilled with a 4-inch rotary bit and drilling mud. One hundred sixteen feet of Sinco 3-inch PVC inclinometer pipe was placed and backfilled with a concrete slurry to within four feet of the ground surface. For this reason, the inclinometer did not give an accurate representation of fluctuations in the water table.

Wells CSM-1 through CSM-6 were drilled using a 4-inch rotary bit with compressed air used to flush the cuttings. The wells were cased with ½-inch inside diameter steel pipe and completed similarly to the other wells.

For reasons unknown, records were not kept on the perforated sections of any of the well casings and some may not have been perforated at all. Well logs were recorded by at least 4 different individuals and are sometimes incomplete or inconsistent. The logs for CSM-5 and CSM-6 could not be located, and the remaining logs are found in Appendix B.

3.3 Slide Definition

The slide is defined as the colluvial and detatched bedrock material that is part of a moving mass with distinct boundaries. These boundaries are the subsurface failure plane, the lateral scarps, crown scarp and toe bulge, all of which are readily discernible in aerial photographs and by ground reconaissance. Figure 7 shows an aerial photograph of the study area. The major failure surface approximates a circular arc. Its location was confirmed later in during the stability analysis of the slide.

Using the original plane table map drawn by Coffee and Adler in 1984 as a base, the slide was defined using conventional plane table surveying techniques. The original 1:480 (1 inch=40 feet) scale map was amended to include the locations of cracks, seeps, bulges, mudflows, topography, observation wells, monitoring stations, and other features of interest that did not appear on the original map. Transient features such as mudflows are dated as nearly as possible. Monuments placed by Coffee and Adler in 1984 were used to locate and align the updated map. The updated version of the plane table map can be found as Plate 1.

3.3.1 Mudflows

A number of relatively small scale mudflows have occurred in recent years. These flows appear to be confined

Figure 7. Aerial photograph of study area. Scale is approximately 1:40,675.

to disturbed area such as roadcuts, scarps, and embankments. They are relatively high velocity events, occurring in a few minutes to a few days. Locations for these flows are in saturated areas, with seeps and springs oozing from the resultant scarps. They are generally less than 100 feet long and 40 feet wide, and once they have failed, further serious failure at the same location seems unlikely. The failures appear to open an outlet for the release of localized excess pore pressure by allowing seeps to ooze freely through the failed material. Seeps flowing from these scarps were used for water table elevation control.

3.4 Definition of the Failure Surface

The inclinometer provided useful data concerning the location of failure surfaces at one location in the slide. The inclinometer data were processed using standard techniques and the results are shown in Figure . Several surfaces are evident at this location. The major failure surface is at a depth of 108 feet, but there are several other zones at depths of 95, 55, 38, and 20 feet. Near the surface, some rotational movement is indicated by the progressively decreasing displacements from about 20 feet of depth to the surface. In general, however, the movements at this location are largely translational. This is not unusual in the center of a large complex slide of this type.

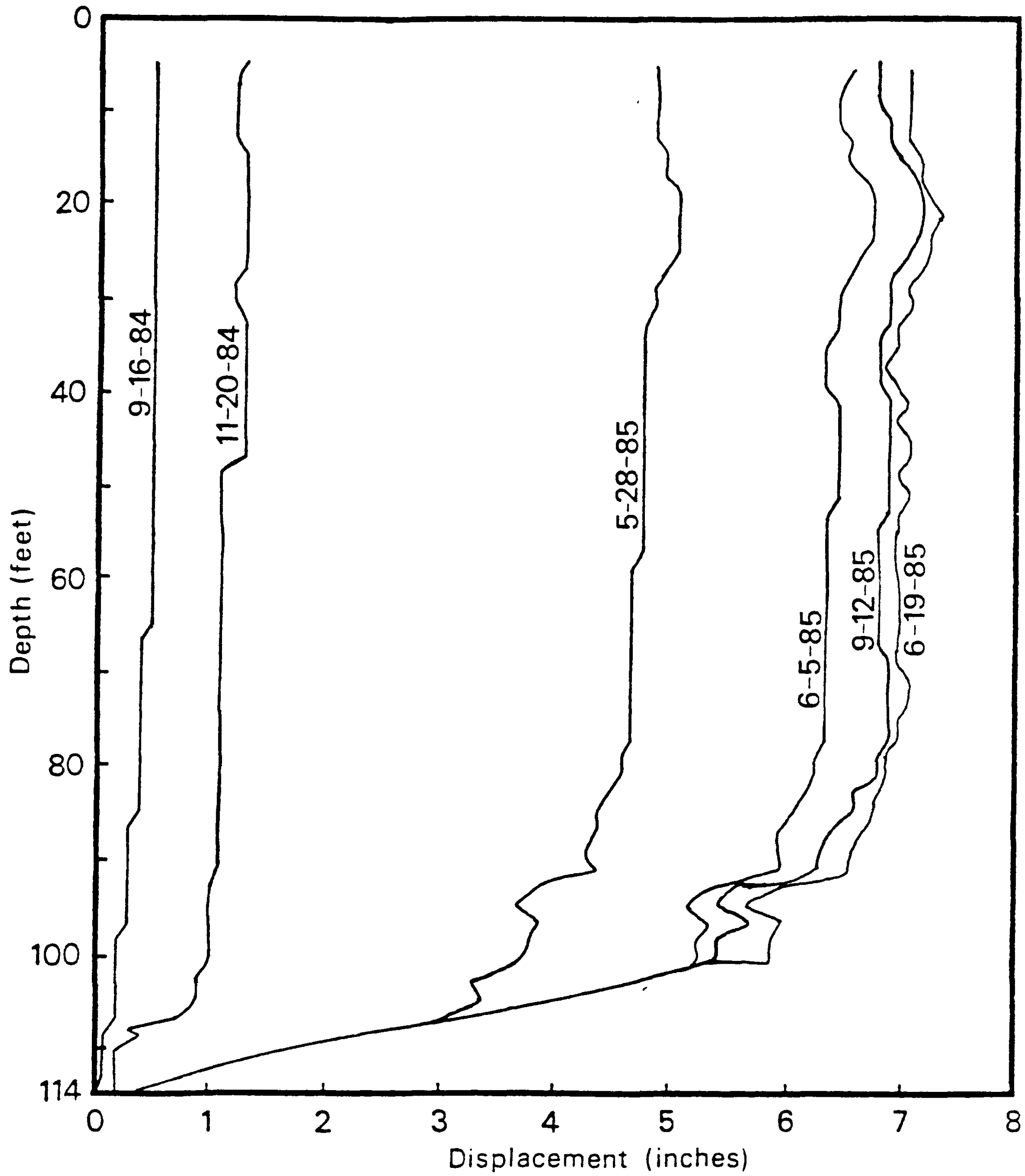


Figure 8. Plot of movement from inclinometer data. Original readings were taken 9-9-84. Inclinometer casing sheared after 5-28-85; subsequent readings use the 5-28 data as a baseline.

Additional locations of these failure surfaces were desired. Where they intersected the ground surface, cracks or scarps might result, so the locations of all such features on the map were carefully checked. The remaining observation wells were not designed as inclinometers, however, if movements occurred, their casings would bend or break at the shear zones. Accordingly, a "deadman", shown in figure 9, was lowered down these holes to locate such bends. Only the uppermost failure surface, if indeed there were more than one such surface, could be located in this way, but the data were believed to be of value.

Well logs were examined for fifteen of the seventeen observation wells (the logs for CSM-5 and CSM-6 were not available). Quality of the well logs varies, but rough correlations can be made in some parts of the slide. The lithology of the primary failure surface, as defined by inclinometer and bent well casing depths, is mostly a micaceous, sandy, gravelly maroon silt.

Once these upper and lower boundaries were located, it was possible to compute the volume of the slide using techniques that will be described later. The slide volume was found to be 1.08×10^6 yards^3.

3.5 Observation of Subsurface Hydrology within the Slide

Hydrologic conditions of the landslide were monitored

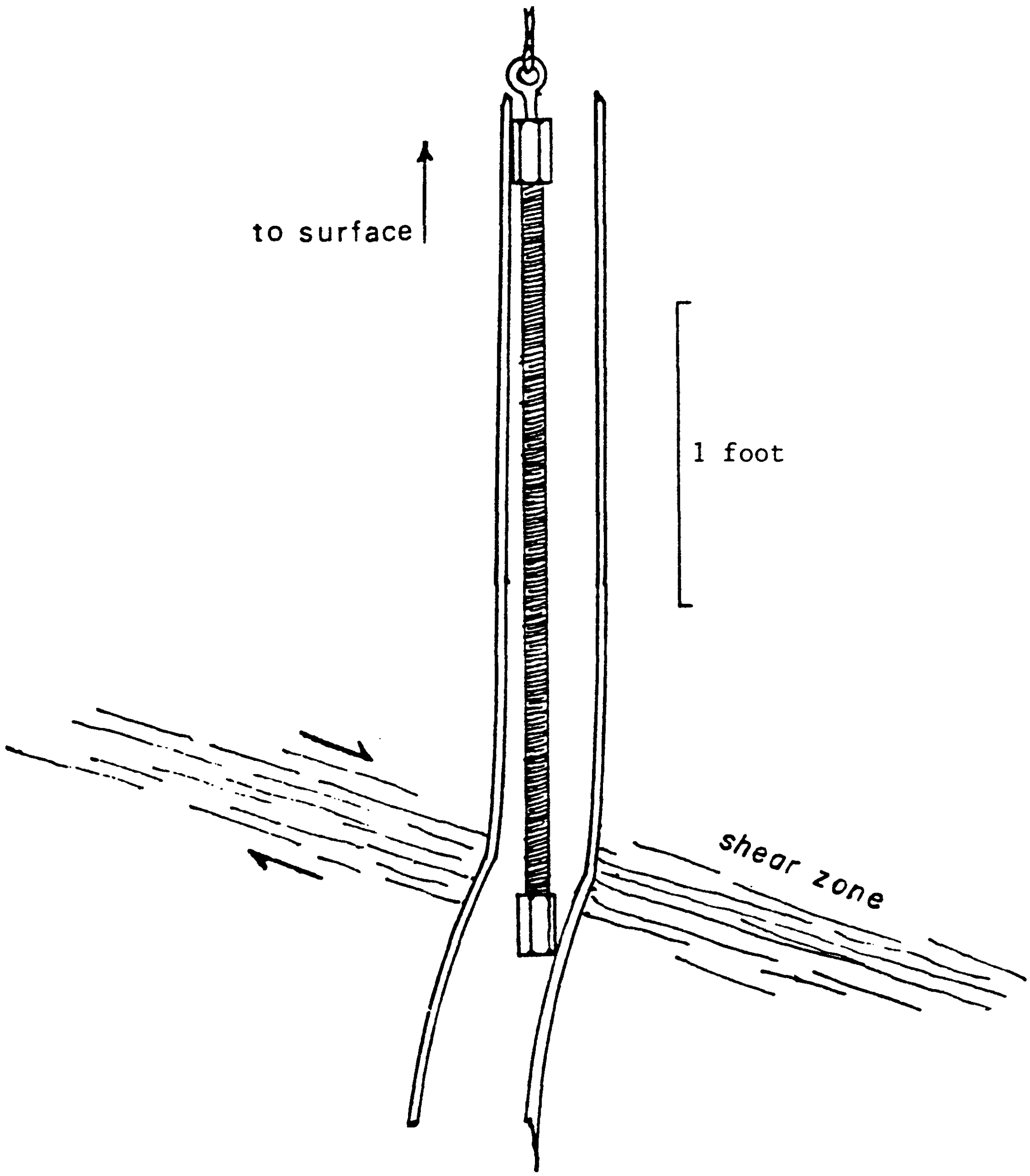


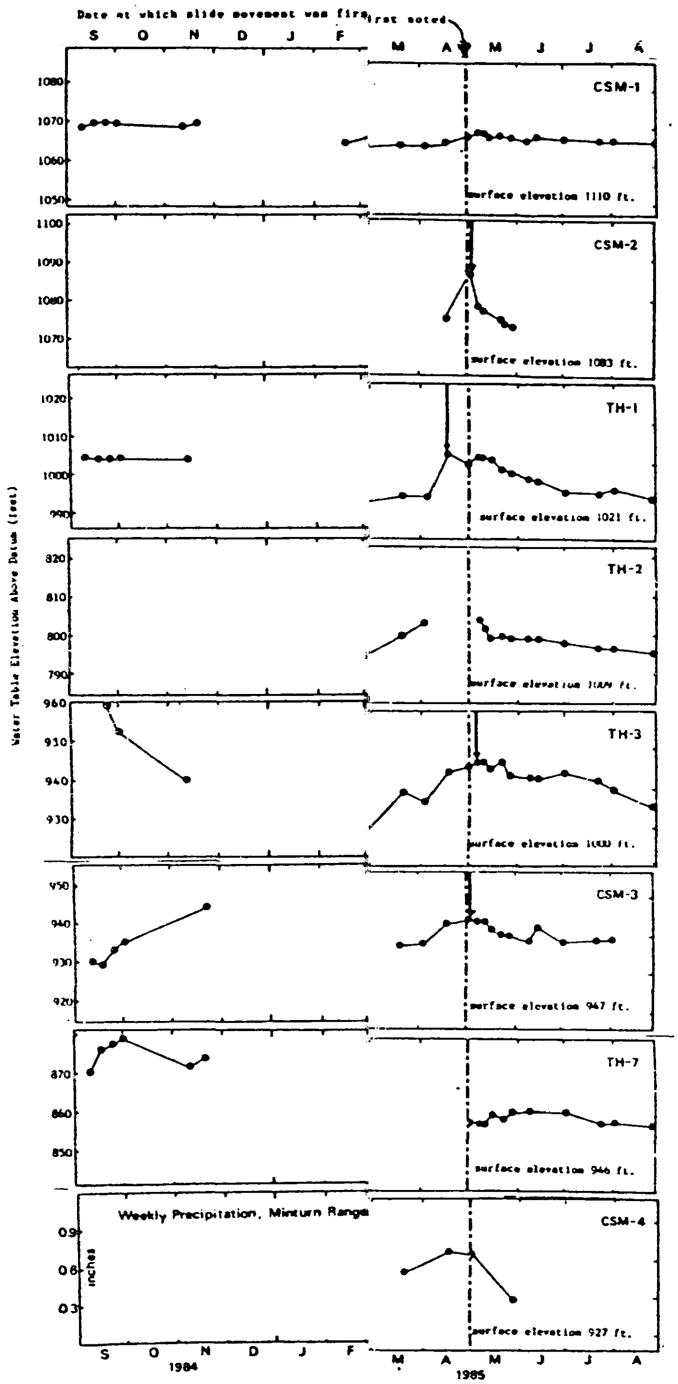
Figure 9. "Deadman" used to detect bends or distiotions in observation well casings.

by using the sixteen observation wells (TH-1 to TH-10 and CSM-1 to CSM-6), distributed around the lower two-thirds of the slide. The inclinometer, TH-11, was monitored, but was not an effective piezometer. Locations of the wells can be found in Plate 1. The discharge rate from the toe of the slide was also measured, and seeps, springs, and standing water were mapped on the topographic base.

3.5.1 Potentiometric Measurements

Water levels were measured in wells TH-1 through TH-11 starting in September 1984 and continuing on a regular basis until September 1985. Wells CSM-1 through CSM-6 were also monitored subsequent to their completion dates. Completion dates can be found in Appendix B, which documents the drilling records for TH-1 through TH-11 and CSM-1 through CSM-4.

Water levels were taken using a Slope Indicator Co. water level indicator, model #51453, a wireline device that gives an audio signal when the water level is reached. Levels were measured with a tape measure to the nearest eighth of an inch to the top of the well casing pipe, then converted to a ground level datum. Water level data from the observation wells are summarized in figure 10. Enlarged versions of the data shown in this figure, and the associated tabular water level data can be found in Appendix C.



ɔmetric levels.
aphs refer to
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3.5.2 Definition of Potentiometric Surface

A potentiometric surface was constructed from the data taken on June 13, 1985, when the most complete and accurate potentiometric information existed. This surface can be found on Plate 3. Some interesting observations and implications can be taken from the shape of this surface.

As one would expect, local flow is generally downhill and to the north and west, towards the Eagle River to the west and towards the ephemeral drainage immediately to the north of the slide. There are some minor fluctuations in the potentiometric surface. These are probably due to the lack of homogeneity of the aquifer material.

The details of groundwater flow are not entirely clear from the available data. The most obvious pattern shown in Plate 3 is the flattening of the potentiometric surface gradient in the lower portions of the slide. Figure 11 shows why such a flattening may occur.

The groundwater flow within the slide mass is but part of a larger regional groundwater flow regime. Regionally, groundwater is believed to move more easily through the Minturn than through the Belden. This difference in permeability will cause the groundwater to migrate laterally and discharge above the Minturn-Belden contact into the slide mass. Coupled with seepages at higher elevations in the

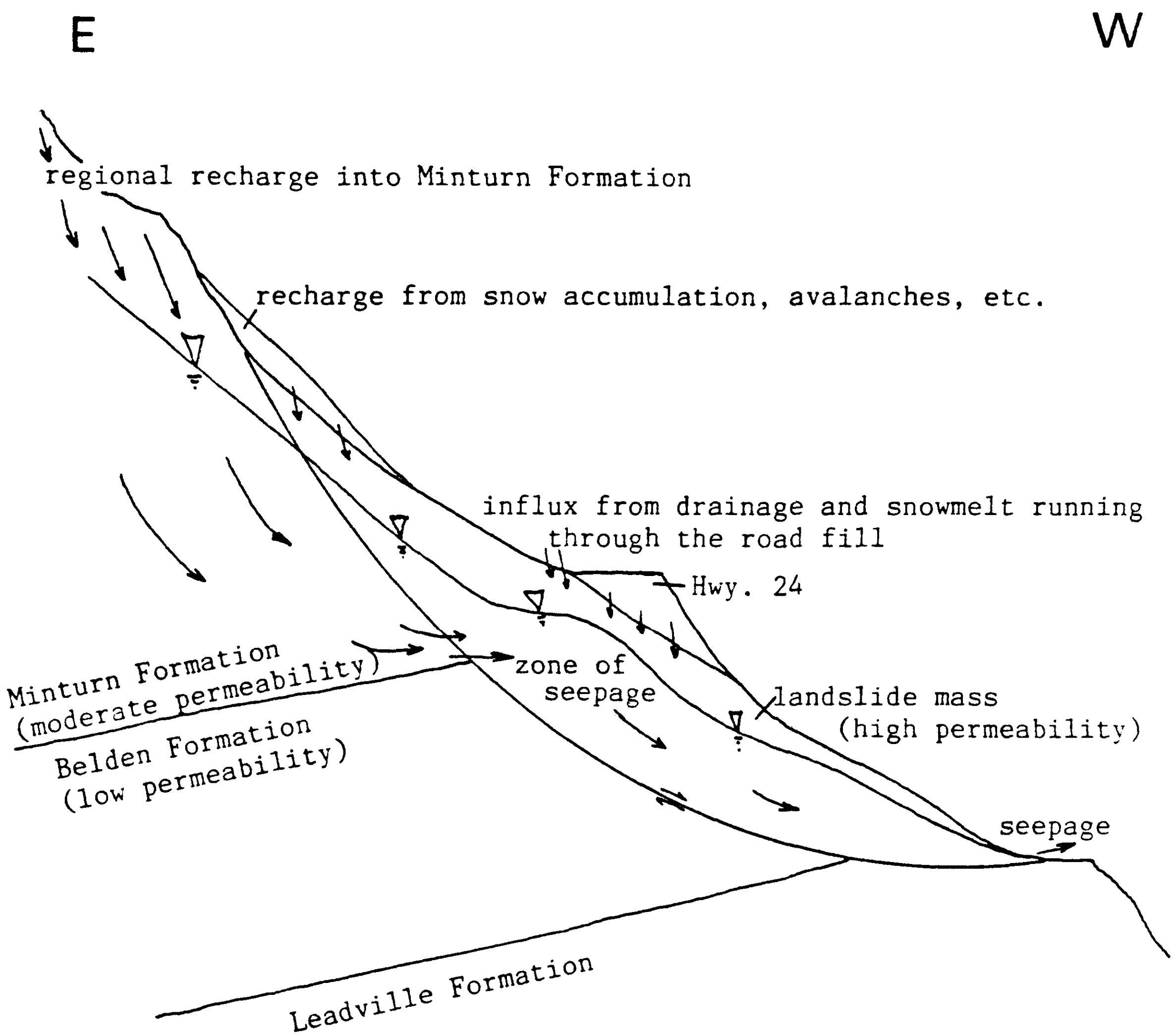


Figure 11. South-looking section of suspected hydrologic conditions existing in the slide.

Minturn, and the infiltration of large volumes of water from avalanche deposits on the upper portions of the slide, this subsurface seepage seems to cause a "mounding" of ground-water within the slide mass which can only drain by moving through the lower portion of the slide materials. Because the slide materials are more permeable than the Belden, the net result is a high level of saturation throughout the slide mass.

Other factors may be important: If some faulting occurs under the slide, as appears possible, then additional seepage conditions may exist at this location. This may, in fact, have been a factor in triggering the original pre-historic failure.

The road fill, being composed of large size aggregates, is an excellent aquifer, sometimes channeling water from the southern part of the slide under the road, where it enters the slide material and contributes to the saturation.

Along with the primary water table, a deeper potentiometric surface was observed in wells TH-7 and CSM-4. This surface runs about 50 feet above the Leadville-Belden contact and probably represents a confined artesian aquifer in a sandy zone of the Belden. Casing perforation depths were not recorded, so that the exact source of the water is uncertain. This potentiometric surface lies well below the deepest failure surface and probably has no effect on the ground water aquifer as it applies to the slide.

3.5.3 Drainage From the Slide

Discharge from the base of the slide was measured by diverting the surface flow originating from the toe area. The flow was directed through a rectangular section galvanized steel gutter down pipe measuring 2 x 3 inches. Flow was directed over the edge of the bank of the gravel road, where the discharge could run directly into a plastic bucket. The bucket was filled repeatedly, and average times were used to determine the flow rate. Discharge rates were determined only in dry weather in an effort to eliminate contributions from overland flow. The flow rates, as a function of time, can be found in figure 12 and table 1. The flow rate decreases as available snow melts from the hillside. All snow was gone by June 5 and flow completely ceased by September 1.

3.5.4 Estimation of Hydraulic Conductivity

If Darcian conditions are assumed, it is possible to estimate the hydraulic conductivity in the slide. Assuming isotropic, homogeneous conditions, a rough approximation of the ability of the slide to transmit water can be made by examining hydrograph peaks as a function of time and distance.

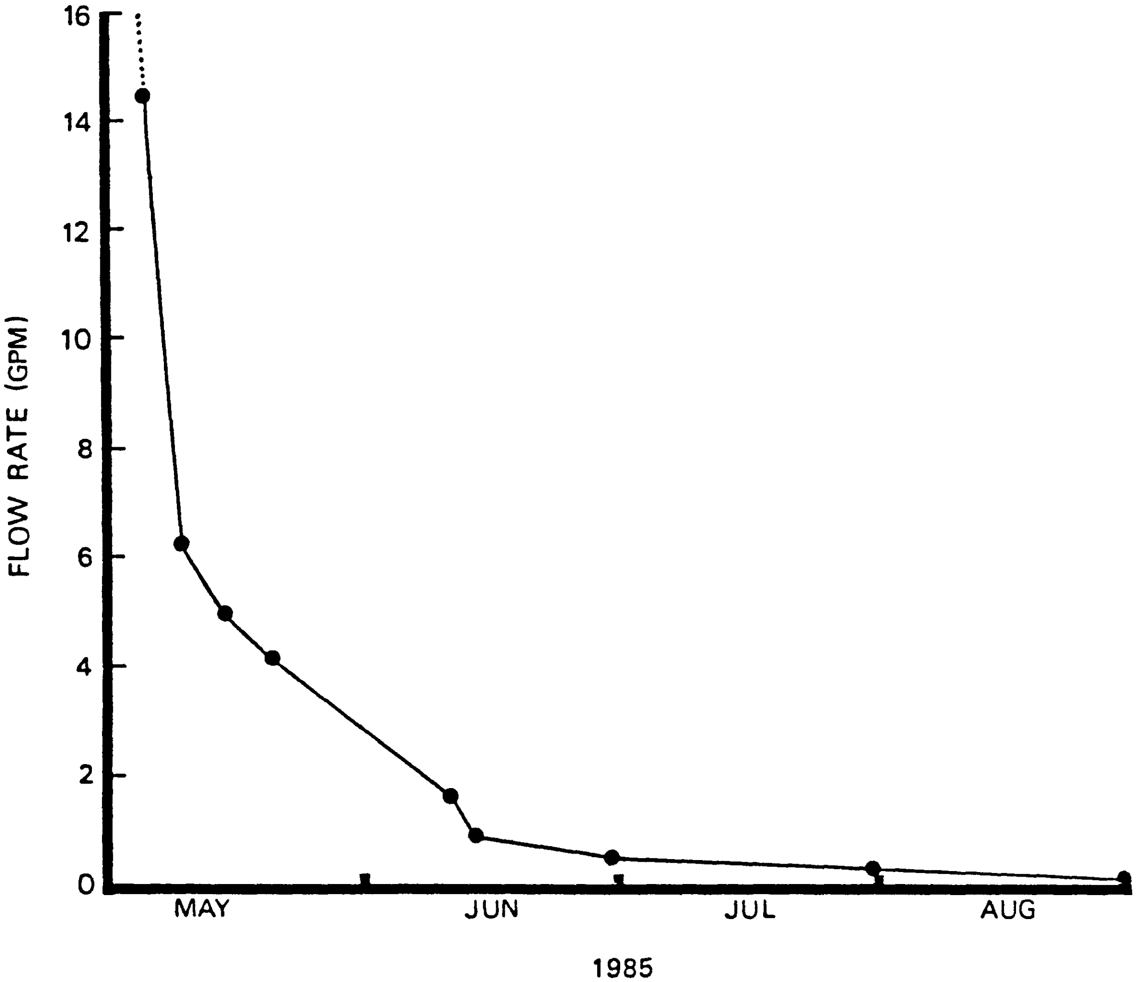


Figure 12. Discharge at toe of landslide.

Table 1. Discharge at Toe of Landslide

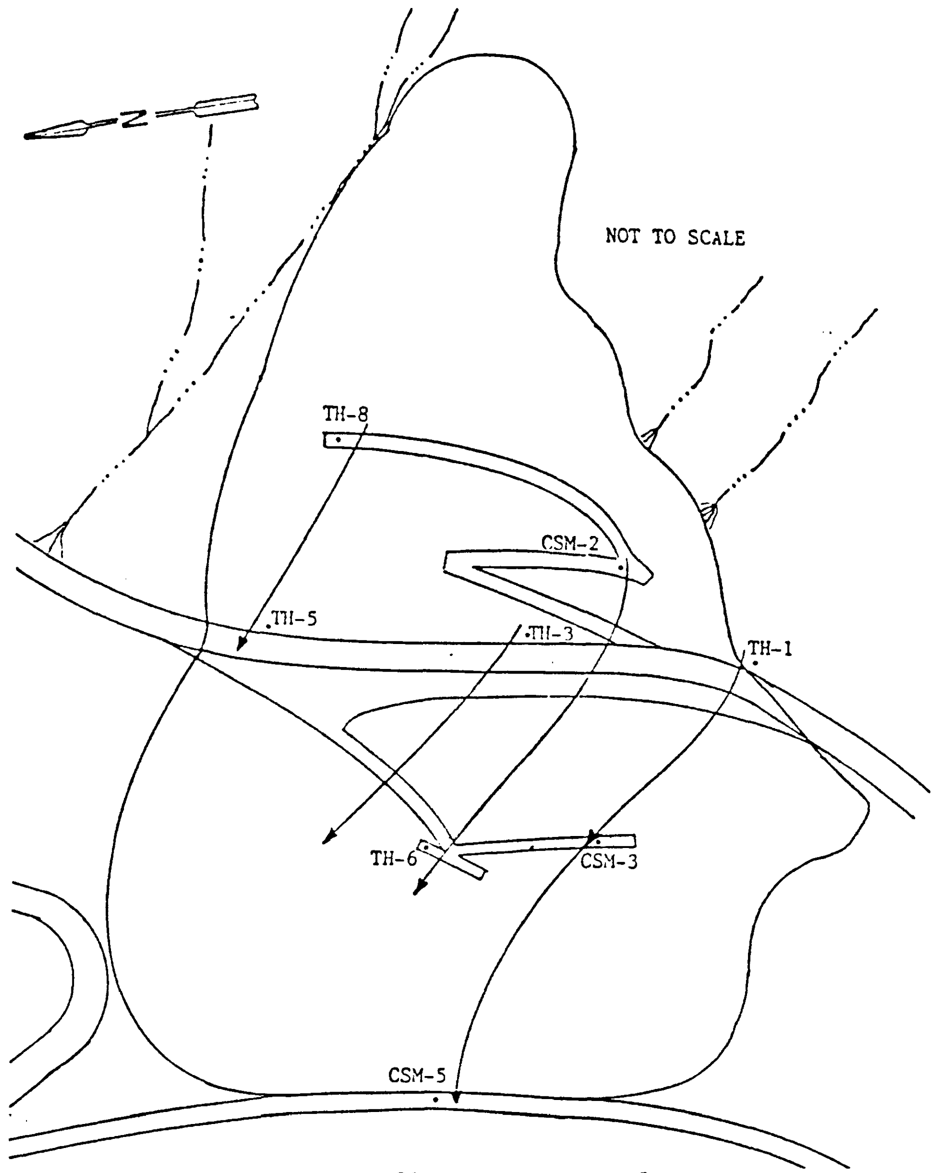
Date	Flow Rate (gpm)
5-8	14.4
5-14	6.25
5-22	4.95
5-30	4.16
6-10	1.61
6-13	0.86
6-30	0.51
7-31	0.29
9-3	0.00

A flow net, constructed from the potentiometric surface (Plate 3), was used to find travel distances for hydrograph peaks in wells along the same flow path. These flow paths can be found in figure 13, which also summarizes the data used in calculating the hydraulic conductivity. Potentiometric peaks that occured in upstream wells were detected later in downstream wells. These peaks are shown in figure 10. Travel times versus flow distance estimates for the peaks in five sets of wells were made and using the data in figure 13 and Darcy's law, and an average hydraulic conductivity of 13 feet per day was arrived at. It should be noted that the peaks used in this calculation may not be related to one another.

3.6 Observations Concerning the Surface Hydrology of the Slide

Because of the steepness of the terrain above the slide, a number of well developed snow and debris avalanche chutes have formed. Some of these chutes lead directly onto the slide, where material is deposited. Figure 14 shows one of the avalanche runs that deposited snow in the vicinity of TH-9.

A constant flow of water into the landslide was observed at three locations during the spring thaw. Peak flow into the slide was estimated to be about 20 gallons



well interval	date of spikes	flow distance (feet)	travel time (days)	potentiometric change (feet)
CSM2-TH6	4-30 to 5-9	340	10	90
TH3-TH6	4-30 to 5-9	280	10	70
TH1-CSM3	4-16 to 4-30	280	14	90
TH8-TH5	4-30 to 5-9	200	10	160
CSM3-CSM5	6-7 to 6-13	370	6	120

Figure 13. Flow paths and data used in hydraulic conductivity calculations. Flow is perpendicular to equipotential lines on Plate 2, hydrograph spikes are shown in figure 10.

Figure 14. Drainage above TH-9 running directly into the slide. Note snow avalanche debris in the foreground.

per minute. Water flows down the channels above the left flank, then disappears into the slide mass when it reaches the left flank scarp. The location of these channels is shown in Plate 1. Flow into the slide was not measured because of unstable snow and difficulty in reaching the channels. Flow in these channels ceased as soon as all of the snow on the hillside had melted.

These channels also acted as snow avalanche chutes in the winter and early spring months. This avalanched snow is believed to be a major source of water infiltrating into the slide mass. The avalanche shown in figure 15 ran down the slide, crossed Highway 24, and ran out over the bank of the highway. Snow deposited along the uphill gravel shoulder of the highway also percolated into the slide. At TH-1, the sound of underground flowing water could be heard during spring thaw.

In the summer, precipitation in the study area occurs in the form of rain. Records of precipitation are kept by the White River National Forest Ranger Station in Minturn, about 2½ miles from the landslide. Precipitation is recorded from mid-May until mid-September, as part of a program to calculate fire hazard in the area. Records for rainfall in 1985 can be found in Appendix C These records are plotted in figure 10, where they can be compared with

Figure 15. Remains of a snow avalanche that ran across Highway 24 near the south flank. Photograph taken April 16, 1985.

observation well water levels. Even with the large increase in precipitation in July, only slight increases in observation well levels are noted.

When compared with the significant increase in observation well levels due to the spring thaw, it can be inferred that most of the rainfall never contributes to recharge in the slide. Most of the rain runs across the surface and into the local drainages, which feed directly into the Eagle River. Furthermore, summer rainstorms are usually of short duration, interrupted by dry periods that allow for the evaporation of surface water. Snowmelt, on the other hand, is largely constrained from flowing away by the snowpack and percolates directly into the soil. Thus, there is a constant source of water to the soil during the spring thaw.

3.7 Observations of Slide Movements

3.7.1 Conventional Surveying Techniques

Conventional surveying techniques were used to quantify surface movements. These included:

1) the laying out of several stake lines and resurveying them at intervals.
2) the observation of isolated control points (usually the observation wells) by angular measurements with a transit, and;

3) differential leveling.

Table 2 describes the techniques used for the individual stations, which are located on Plate 4.

Seven stake lines (lines A through F on Plate 4) were laid out in various orientations from the crown of the slide to the toe. Lines A, B, C, and D were transit lines originating across the slide from permanent hub locations to reference points located some distance off of the slide. The hubs were four-foot long, ½ inch diameter steel reinforcement bars, pounded 3½ feet into the ground and center punched. In addition, instrument height was reproduced at each station by sighting on a vertical control mark on a nearby surface to allow for reproduction of vertical angles with some reasonable degree of accuracy.

Lines A, B, C, and D were set by sighting the transit on a specified distant object and setting stakes or nails in the ground surface along the line of sight. Movement was measured by resighting the line some time after it was set and recording the offset distances for the points. Vertical angles were recorded on lines C and D, but data proved inconclusive.

Lines E and F consisted of a string run across the slide from trees originally thought to be located outside the movement. Distances were measured from the end of the

Table 2. Total Movement From May 14 to July 22, 1985

Station	Horizontal (feet)	Vertical (feet)	Instrument Sta.	Notes
TH-1	0.10	-0.03	Hub-A	1,2
TH-2	0.46	-0.35	Hub-A	1,2
TH-3	0.39	-0.41	Hub-A	1,2
TH-4	0.25	-0.29	Hub-A	1,2
TH-5	0.56	-0.30	Hub-A	1,2
TH-11	0.42	-0.35	Hub-A	1,2
LC	0.42	-0.60		4
L	--	-0.38		2,5
A-1	0.27	-0.29	Hub-A	1,2
A-2	0.30	-0.36	Hub-A	1,2
B-1	0.31	-0.27	Hub-B	1,3
B-2	0.30	-0.27	Hub-B	1,3
B-3	0.29	-0.36	Hub-B	1,3
B-4	0.54	-0.23	Hub-B	1,3
B-5	0.49	-	Hub-B	1,6
C-1	0.21	-	Hub-B	1,6
C-2	0.33	-	Hub-B	1,6
C-3	0.31	-	Hub-B	1,6
C-4	0.28	-	Hub-B	1,6
C-5	0.35	-	Hub-B	1,6
C-6	0.31	-	Hub-B	1,6
C-7	0.25	-	Hub-B	1,6
D-1	0.15	-	Hub-C	1,3
D-2	0.14	-0.04	Hub-C	1,3
D-3	0.00	0	Hub-C	1,3
D-4	0.07	0	Hub-C	1,3
E-1	0.19	-0.35	-	7
E-2	0.23	-0.40	-	7
F-1	0.34	-0.33	-	7
F-2	0.25	-0.36	-	7
F-3	0.31	-0.27	-	7
F-4	0.24	-0.33	-	7

Notes:

1. Horizontal movement perpendicular to line from instrument station.
2. Vertical movement measured by differential leveling.
3. Vertical movement measured by change in vertical angle from instrument station.
4. Stake location, measurements were taped across crack.
5. No horizontal control.
6. No vertical control.
7. Vertical and horizontal control taken from a line fixed across the slide.

string and stations were set by dropping a plumb bob from that point on the string. String tension was set using a spring scale, and the height of the string above the point was recorded. Subsequent measurements were compared with the original measurements to determine movement parallel and perpendicular to the string and in a vertical direction. The northernmost terminus of line F was found to be moving, consequently, only the south four points, where error was small, were used.

Another method was to turn an angle with the transit from a permanent point to a point on the slide. This method was used to monitor movement of the observation wells along the highway. Angles were turned and recorded to the nearest 20 seconds to the original location of the point. Later, the same angle was turned and the distance that the point had moved was recorded. These angles were not measured from any other location because errors from the next closest possible transit station would have been excessive.

Slide movement has caused subsidence along the road, therefore, several points along Highway 24 and in the shoulder area were differentially leveled to obtain vertical control. Rather than readjust the contours and elevations shown on the 1984 map, these observations were plotted seperately and compared. The 1984 map appears accurate in areas where movement has not occurred, and thus is believed to accurately represent the elevations along Highway 24 as they existed in the late summer of 1984. Comparison with the new data revealed up to eight feet of subsidence. Accordingly, the two profiles were plotted on a new plate (Plate 5).

3.7.2 Direct Observations

Other measurement techniques included direct measurement, using a tape measure, of points located on opposite sides of the crack across the highway. Although these points were continually being patched over by Highway Department crews, data could be extrapolated from the old point to the new one by plotting movement as a function of time and connecting the slope of the old point to the new one. This location is shown in figure 16. Horizontal and vertical measurements taken at various points along the crack are shown in figure 17, which summarizes rates of movement taken from the points used at this location.

3.7.3 Observation of Leaning Trees

Numerous mature conifer trees are found in the slide area. Many are leaning or have curved trunks indicating direction and degree of movement (figure 18). Such trees were located on the base map and the directions of movement for each tree were recorded. These data were also

Figure 16. Before and after shots of a measurement point along the south lateral scarp on the highway. The top photo was taken May 14, 1985, and the bottom photo was taken July 31, 1985.

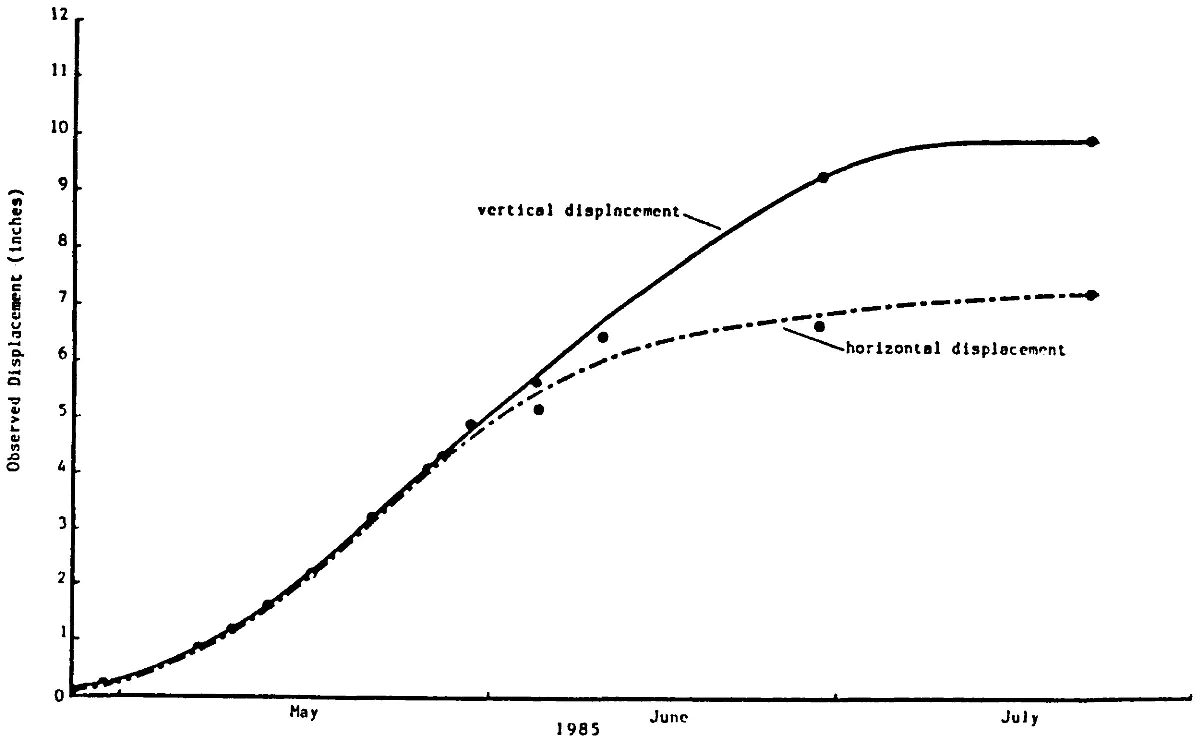


Figure 17. Observed displacement as a function of time at station located across the south lateral scarp, where it crosses Highway 24. Station was paved over and replaced on 5-13, 5-21, and 6-6

Figure 18. Looking north from the upper drill road at a distressed conifer, typical of the older trees found on the slide.

plotted on Plate 4, as directional information, to provide supplemental movement data.

3.8 Subsurface Movements

Movement below the ground surface was quantified by the inclinometer, TH-11. Figure 8 is a plot of movement recorded from September 9, 1984, when the original readings were taken, to September 2, 1985. Total displacement is the distance the inclinometer casing has moved from its original position, based on readings taken on September 9, 1984.

These reading are adjusted by the program (provided by the Highway Department) used to reduce the raw data. The program uses the greatest depth reached by the probe as the stable bottom of the hole, regardless of whether or not the casing has sheared. It then compares this data with the original readings. If the casing has sheared, it adds the last known displacement at the shear depth to the data taken after the casing had sheared.

Between May 28 and June 5, the inclinometer casing sheared off at just below 100 feet. Readings taken after May 28 show displacements relative to the original inclinometer casing profile, but total movement of any point is not known.

Surface surveying techniques used on TH-11 show 2.4

inches of movement between May 28 and June 6, 1985. The inclinometer data, by way of comparison, shows 1.8 inches of displacement 4 feet below the surface during the same period. This is a difference of about $\frac{1}{2}$ inch, which is within the error of the transit readings. It is also possible that the surface moved more than the subsurface.

The inclinometer data were useful in locating failure zones in the subsurface. The main failure is located at about 108 feet, where the inclinometer casing was sheared. Other failure zones are evident at around 95, 55, 38, and 20 feet, and are shown in figure 8.

4.0 DATA ANALYSIS

The data analysis was conducted in three phases. First the observational data were reviewed to determine if there were correlations among the data items which would indicate differential rates, directions, or mechanisms of movement within the slide. As a consequence of these reviews the slide was divided into four zones, each having a different rate or character of movement.

The subsequent two analysis phases involved the computation of volumes and an analysis of the slide stability.

4.1 Relationship Between Potentiometric Surface and Movement

Figure 19 was constructed to demonstrate the effect that increases in the water table elevation have on slide movement. Movement data were taken from measurement points across the crack in Highway 24, near TH-1 (see figure 15 for photographs of the crack). No movement was observed on this crack until late April. From that time until August, movement was regularly monitored. Movement was converted into inches per day and compared with water levels in the adjacent observation wells, TH-1 and TH-2.

Figure 18 shows that movement lags behind rises in the water table. This suggests that once a threshold water level is reached, movement is initiated. After the water level

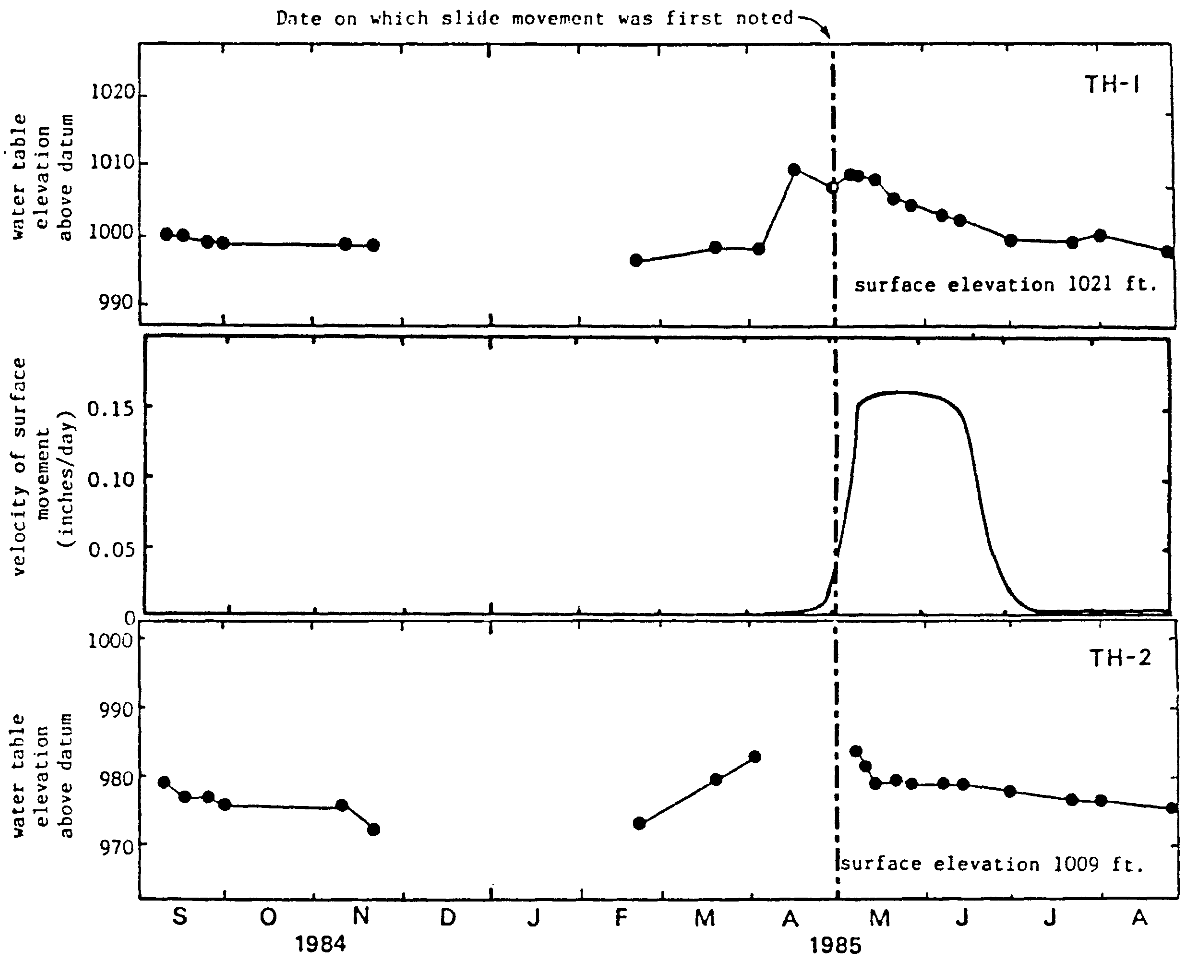


Figure 19. Comparison of water table elevations vs. slide movement. Slide velocity data taken from south lateral scarp movement, across crack in Highway 24, immediately adjacent to TH-1.

drops, loss in soil strength and internal structural changes allow the slide to continue to fail. Finally, when water levels drop sufficiently to stabilize the slide, movement abruptly decreases.

4.2 Definition of Zones

Even before the surveying data from the spring of 1985 were analyzed, it was apparent that differential movement existed within the slide. Transverse cracks mapped by Coffee and Adler in 1984 attest to the existence of smaller failures within the main slide mass. Plates 1, 5, and 6 show the location and probable subsurface extensions of these failure surfaces. Inclinometer data and bent observation well casings were used to locate these failures at depth. From these subsurface locations, circular failures can be extrapolated back to the transverse cracks at the surface.

Surveying data, gathered from May to August of 1985, confirms the existence of these secondary failures. The boundaries of these zones are well defined near their crowns but become nebulous in their toe areas.

Plate 4 shows surface movement surveyed over the entire slide. Differences in directions and amounts of movement are apparent at different locations on the slide. Secondary data were defined by using the movement data plotted on

Plate 4, scarp and crack locations, and inclinometer and well casing deformation depths. Figure 20 shows the approximate boundaries for the larger failures.

The toe of the slide shows little horizontal displacement, and vertical measurements were inconclusive. In September 1985, vertical displacement was observed along cracks immediately northeast of CSM-5. The sense of movement on these cracks implied that the toe was moving up relative to the stable ground immediately to the west of the slide. Similar movement is to be expected in the toe areas of the secondary slides. Because the main slide is generally subsiding (except in the toe area), any positive vertical movements in the toe areas of these secondary slides are superimposed on this gross overall subsidence, making them difficult to detect.

Two possible alternatives for such superimposed failure zones are shown in figure 21a and b. In the situation represented by figure 21a, movement first occurs at the head of the slide with failure occuring in the upper zone. This loads the next lower zone which fails in turn. Thus, the movement progresses down the slide in an incremental fashion.

The situation shown in figure 21b fails in a different manner. Failure begins in the lower portion of the slide,

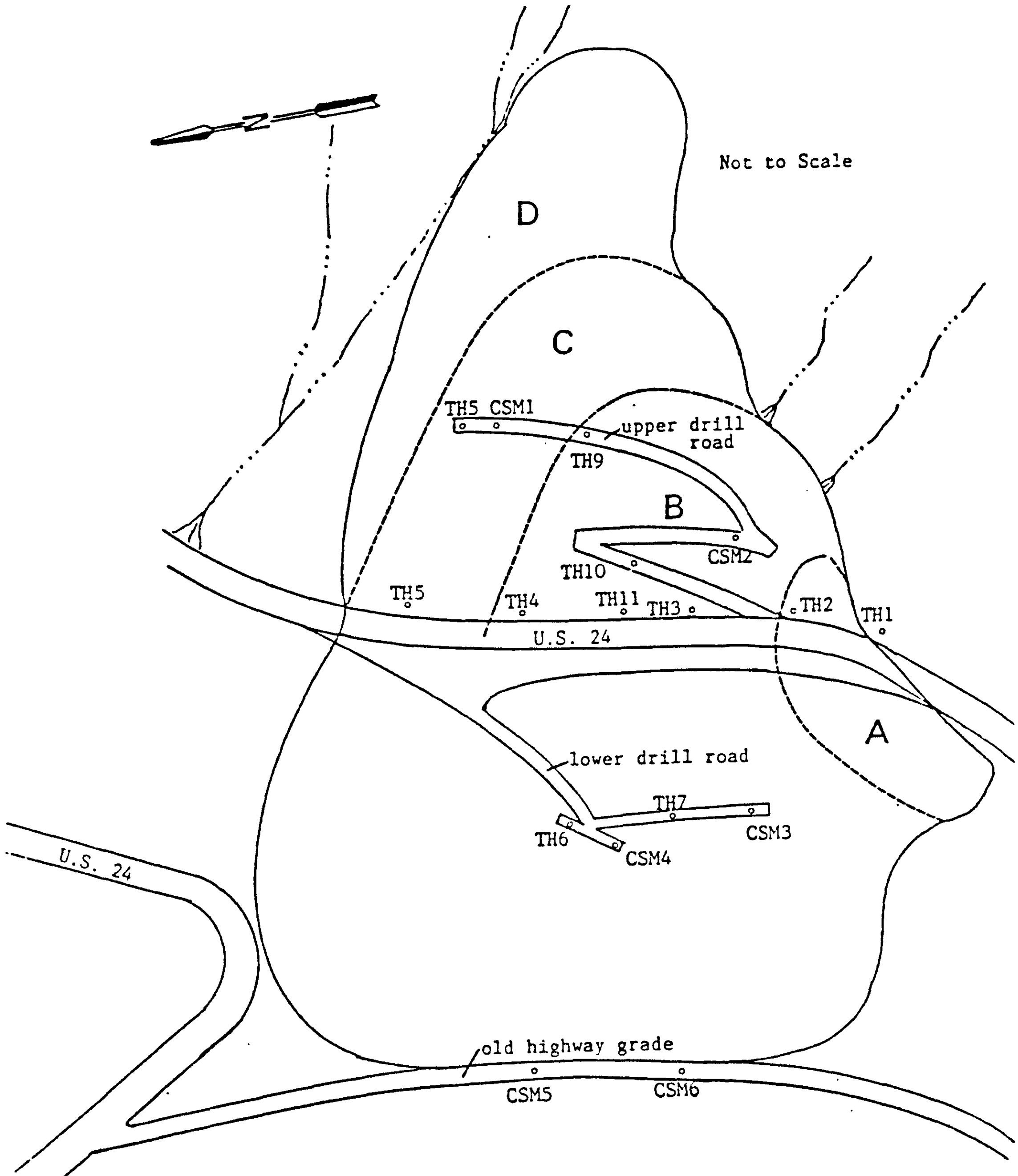


Figure 20. Major zones of movement.

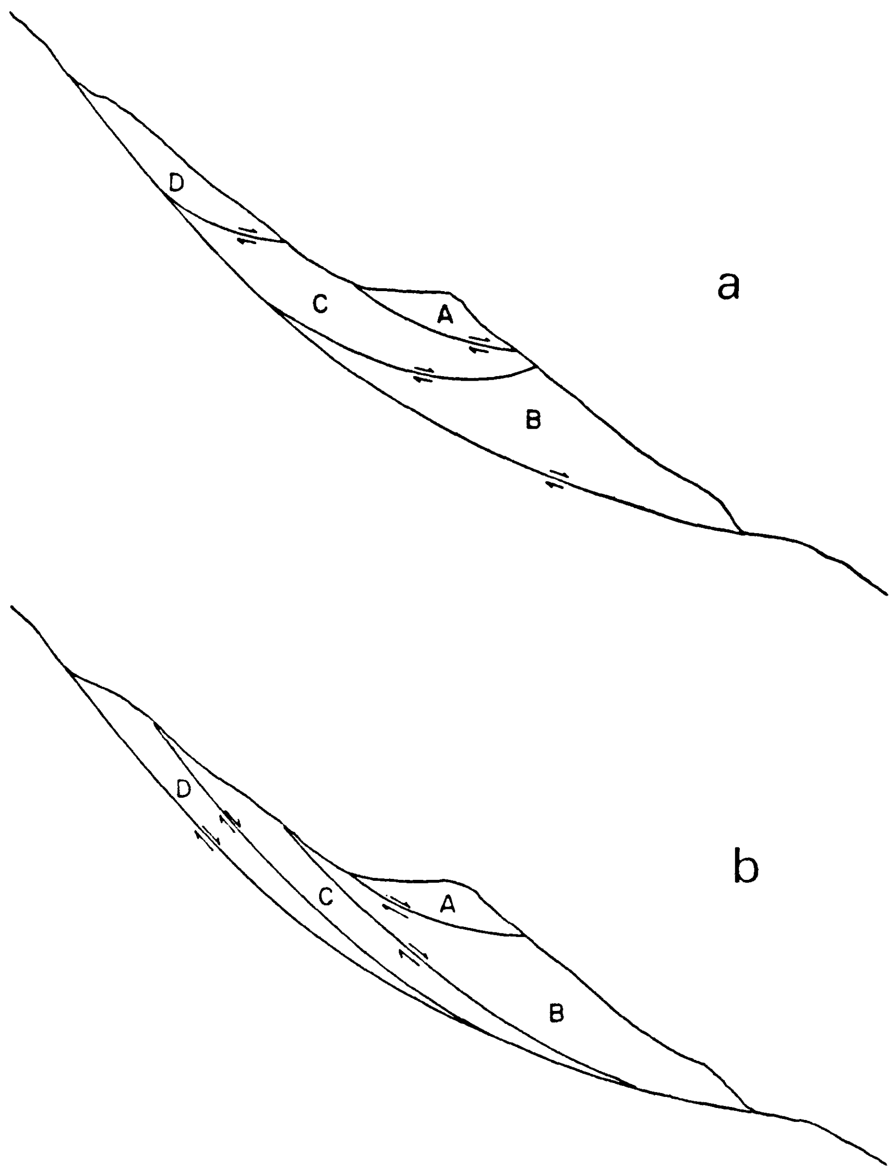


Figure 21. Cross section showing two possible modes of failure.

not necessarily right at the toe, but perhaps as a more generalized remobilization of the lower part of the slide mass. This could be caused by saturation. As this lower portion moves, the upper portions become unsupported and fail in turn. In this situation the failure retrogresses up the slide.

Comparison among the zones shown in figure 20, shows that Zone D, at the top of the slide, moves slowly, movements increase down the slide to Zone B, and Zone A moves the most rapidly. These movements suggest that the slide is failing in accordance with the concept of figure 21b. This movement also seems to be in agreement with what might be expected if groundwater was seeping into the slide mass from the lower Minturn, and saturating the lower parts of the slide. This observed potentiometric surface geometry suggests that such a groundwater condition is likely (see figure 11 on page of this report). However, saturation by snow avalanche debris on the head of the slide would favor the situation shown in figure 21a.

On the other hand, the observed scarp locations and the lack of observable toe bulges within the slide make the situation shown in figure 21a unlikely.

The possibility remains, however, that snowmelt may begin the slide movement by initiating failures in the crown area causing increasd loads on the central slide mass. Such movements would occur early in the season when observation is difficult or impossible. Once lower portions of the slide become saturated, its failure rate increases, and the bulk of the movements then occur according to the scenario shown in figure 21b.

Another exception is Zone A, where the road fill may be failing due to its inherent instability when large volumes of meltwater infiltrate into the fill. Its failure would tend to load the main mass of the slide and initiate additional failure. However, it can also be argued that movement of the slide will tend to remove support for the road fill and so cause movement in Zone A. Zone A moves the most rapidly of all of the zones and it is suspected that it is failing by a combination of causes, including both scenarios shown in figure 21a and b.

4.2.1 Description of Zone A

Zone A is the fastest moving part of the slide. It is composed mostly of road fill of unknown composition, including a shoulder made up of uncontrolled fill. The failure is defined by cracks across the highway pavement that continue northward to a mudflow along the south lateral scarp of the main slide. The north boundary is undefined, but the toe of this failure is at the base of the rock fill beneath the shoulder. At this location, large cracks surface in the road fill. Above the cracks, the fill slope is oversteepened. Direction of movement is about N70W, directly downslope.

This zone has been analyzed for stability using the STABL2 program. Results of this analysis were inconclusive. Boundary conditions for this failure were not readily quantifiable for such an analysis. These conditions include shear along a vertical plane between the road fill and the main slide mass. The fill also rests on top of the main slide and is no doubt subject to subsidence as the slide moves out from under it.

As it was mapped in 1984 by Coffee and Adler, the left flank of the slide, in the area of Zone A, was apparently not well defined (see Plate 1). They approximately located the flank across the highway about 160 feet north of the 1985 location. The location of the 1985 flank is very clearly defined by a major offset in the pavement surface. This implies that movement in this area was first initiated or accerated in the spring of 1985.

4.2.2 Description of Zone B

Zone B is the next fastest moving part of the slide. It is composed of colluvial material similar in composition to the rest of the slide. This zone is defined by a transverse crack above the upper drill road. This crack intersects the left lateral scarp where it continues downhill to Zone A. The north boundary and the toe areas show no distinct surface expressions but the inclinometer data and bent well casings in TH-2, TH-3, TH-9, TH-10, and CSM-2 can be used to define the shear surface at depth. The toe is somewhere below the lower drill road, or the failure surface might intersect the primary failure at depth. Zone B underlies Zone A and apparently contributes to its instability. The direction of movement of this zone is perpendicular to Highway 24.

This part of the slide is subject to the highest infiltration rate and hence, has the highest water levels in spring. It is the main runout area for snow avalanches and recieves considerable flow from two drainages that feed directly into its crown and left lateral scarp.

4.2.3 Description of Zone C

Zone C moves more slowly than Zone B. Zone C is composed of colluvial material and is defined by a significant transverse crack originating above Zone B along the left flank. The right flank area is heavily forested and any surficial expression is obscured.

Subsequent stability analysis with the STABL2 computer program suggested that a circular surface with a maximum depth of about 94 feet under the highway, or roughly 10 feet above the primary failure zone, was very likely to fail. Such a surface was evident in the inclinometer data, and so was used to set the limits of Zone C.

Although surveying data in this zone are lacking, a clear divergence of movement directions can be observed on Plate 4 at the south end of line F, between points 3 and 4. The transverse crack can be extended uphill between these points. Above line F, the ground is covered with colluvial and mudflow debris.

Lower on the slope, the zone B-C boundary lies between point C-1 and C-2 (on Plate 4) along the lower drill road. Along the highway, the boundary lies between TH-11 and TH-4. Zone C underlies Zone B.

4.2.4 Description of Zone D

Aside from the toe, Zone D is the least active part of the slide. It underlies the other zones and acts as a basal unit, moving at a fundamental velocity along the primary failure surface. This basal unit daylights above Zone D, and probably in the toe, where differentiation between zones is not possible. The boundaries of Zone D begin at the right flank of the slide, just above the highway, and continue uphill, around the main crown, then down the left scarp where it intersects the transverse crack of Zone C.

It moves about N60W beneath the other zones.

4.3 Volume Calculations on Autotrol Computer System

Volumetric data for the landslide were computed on an Autotrol three dimensional computer aided design (CAD) system. The surface topography, water table, and primary failure surface (Plates 1, 2, and 3) were digitized using a digitizing table connected to the system.

Slide volume and saturated slide volume were computed by taking 20 sections of the data on 40 foot centers, roughly parallel to the longitudal axis of the slide. The sections were rotated 90° to display a simple view of each vertical section. A planimeter built into the CAD system was used to calculate the section area. Total volumes were then computed using the average end area method. Slide volume was found to be 1.08×10^6 yards^3, and the saturated volume on June 13,1985 was found to 6.48×10^5 yards^3.

4.4 The STABL2 Computer Program

Because of the large numbers of calculations and variables involved in landslide analysis, computers have become an important tool in slope stability analysis (Boutrup, 1977). One of the state-of-the-art programs written for slope stability analysis has been developed over the last ten years at Purdue University (Lovell, Sharma, and Carpenter, 1985). A series of updated versions have been issued periodically. All are called STABL with numbers to indicate the version. The STABL2 version was used in this study.

The STABL2 program can handle non-circular failure surfaces (Siegel, 1975a, b). It is written in FORTRAN and calculates the safety factor against slope failure by a two-dimensional limiting equilibrium method, either the Simplified Bishop Method of Slices (for circular failure surfaces), or by the Simplified Jambu Method of Slices, for failure of a general shape (Bishop and Morgenstern, 1960). It is also capable of analyzing failures along specified surfaces, which proved useful in this study since the failure surface had been mapped in the field (see Plate 2).

4.5 Analysis of Main Failure

Because soil properties were unknown, values were estimated from the general soil characteristics of the failure zone and standard engineering data tables (Hunt, 1985). Initial values of 300 pounds per square inch cohesion and a friction angle of 28° were used.

Since the failure surface had previously been defined, the STABL2 failure surface searching routine was used on the established topography and water table. This routine searched for a failure containing the known crown and toe of the landslide. One hundred trial surfaces were generated using the Jambu method for irregular failure shapes.

The actual failure surface found in the field was in close agreement with the second most likely surface generated by the program. A section of this surface is shown in Plate 6.

This initial run yielded a factor of safety of 0.917 for the most critical surface (see Table 3). This value seemed too low for the observed field conditions. Because the landslide was failing, but the failure did not seem to be accelerating, a factor of safety of between 0.95 and 0.97 seemed appropriate.

Accordingly, the soil strength parameters of cohesion and friction angle were modified and a series of runs were undertaken (Table 3). Values of zero for the cohesion and 32° for the friction angle yielded a factor of safety of . 0.971 run 10, Table 3).

Using these soil parameters, the effect of lowering the potentiometric surface in the slide was examined with some additional runs. The elevation of the potentiometric surface was lowered uniformly by five feet to simulate drainage from the slide. This yielded a factor of safety of 1.019 (see run 11, Table 3), a small but significant increase in stability. Further lowering of the water table by five additional feet (ten feet total) gave a factor of safety of 1.065 (see run 12, Table 3), an increase of 9.7 percent over the original 0.971 factor of safety.

Table 3. Stability Analysis of Main Failure Surface

Moist density=115 lbs./ft.^{3}

saturated density=125 lbs./ft.^{3}

Run	cohesion (lbs./in^2)	friction angle (degrees)	Factor of Safety	comments
1	300	28	0.917	initial run
2	300	15	0.495	
3	300	24	0.771	
4	300	34	1.127	
5	0	24	0.692	
6	0	34	1.048	
7	1000	24	0.954	
8	1000	34	1.311	
9	300	30	0.976	
10	0	32	0.971	values used in analysis
11	0	32	1.019	water level drop of 5 ft.
12	0	32	1.065	water level drop of 10 ft.
13	300	30	1.069	water level drop of 10 ft.

It should be noted that, because soil conditions were unknown, the results of this analysis are only estimates. They are designed to show the relationships among soil strength, potentiometric surface levels, and the factor of safety. The numbers reported in this section are intended only to show such relative changes.

4.5.1 Sensitivity Analysis

The analysis described above used a range of values for both cohesion and friction angle. The failure surface occurs within a sandy maroon silt zone. Accordingly, cohesion values are likely to be very low or zero. However, if significant amounts of clay materials occur in parts of the failure zone, then the cohesion would be significant.

Figure 22 shows the relationship between cohesion and the factor of safety for various angles of friction. The shaded zone shows the area between the angles of friction of 24° and 34°. The limits of this area (cohesion of zero to 1000 psi and angles of friction of 24° to 34°) encompasses all likely subsurface conditions. The change in factor of safety, represented by the height of this area, as the angle of friction varies from 24° to 34°, is much greater than the change in factor of safety as cohesion varies from zero to 1000 psi. Therefore, these analyses show that the angle of friction is the most critical soil parameter in

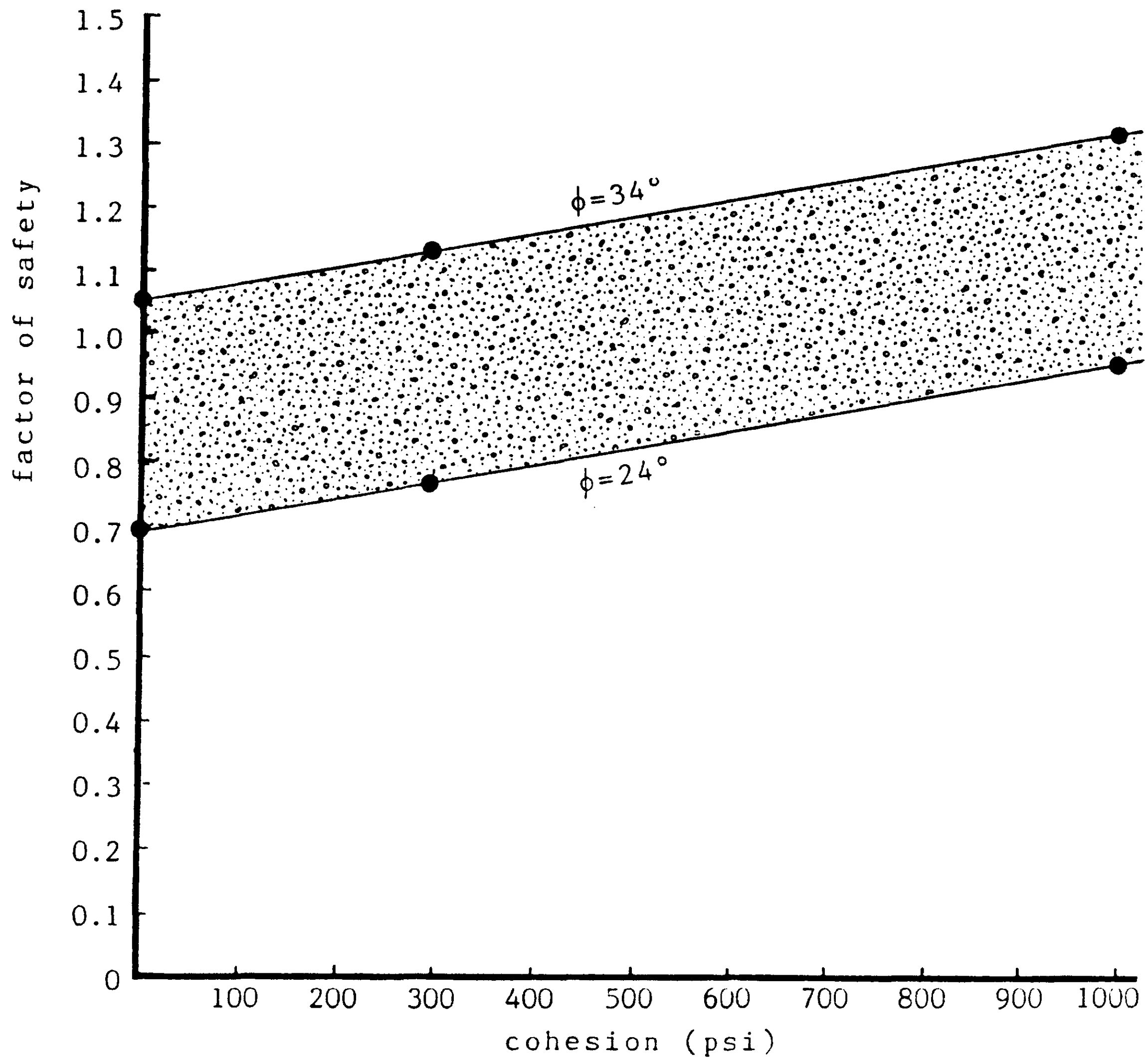


Figure 22. Sensitivity analysis of soil strength vs. factor of safety. Factor of safety is more sensitive to changes in friction angle than cohesion. Stippled area includes conditions likely to be found in the failure zone.

these factor of safety calculations.

4.6 Analysis of Road Fill Failure

The failure of the road fill under the highway and shoulder area adjacent to the south lateral scarp was also analyzed with the STABL2 program. The fill under the road is of unknown composition, but probably consists of large angular sandstone and siltstone blocks cut from the bedrock roadcut south of the slide. The shoulder area is composed of uncontrolled fill of sandstone and siltstone boulders and blocks, with a matrix of sandy silt. There are numerous void spaces between the boulders.

The failure in the road crosses the asphalt diagonally, then parallels the pavement as it goes south (see Plate 1). It resurfaces at the base of the fill slope, but because of the nature of the road fill material, the toe is difficcult to define. The fill slope is disturbed and oversteepened in this area.

The subsurface geometry of the road fill is unknown, so a circular analysis was attempted with only the crown and toe limits specified. Initial soil parameters were selected with zero cohesion and an angle of friction of 32°. A bedrock surface was located and bedrock strength values were selected to keep the failure zones from extending into the bedrock.

A series of iterations were performed (Table 4). The results are plotted on figure 23. The factor of safety becomes 1.0 at an angle of friction of 28°. This value seems unreasonably low for the type of material because angular rip-rap typically has a friction angle of 34° or more (Hunt, 1985). At a friction angle of 34°, the computed factor of safety is around 1.884.

These analyses suggest that the fill may not be inherently unstable, but that the distress observed in the fill may be largely due to its position on top of a moving slide mass.

Table 4. Stability Analysis of Road Fill Failure

Moist density = 125 $lbs./ft^3$
Saturated density = 130 $lbs./ft^3$
Cohesion = 0 $lbs./in^2$

Run	Friction Angle (degrees)	Factor of Safety
1	34	1.884
2	33	1.716
3	29	1.171
4	28	1.028

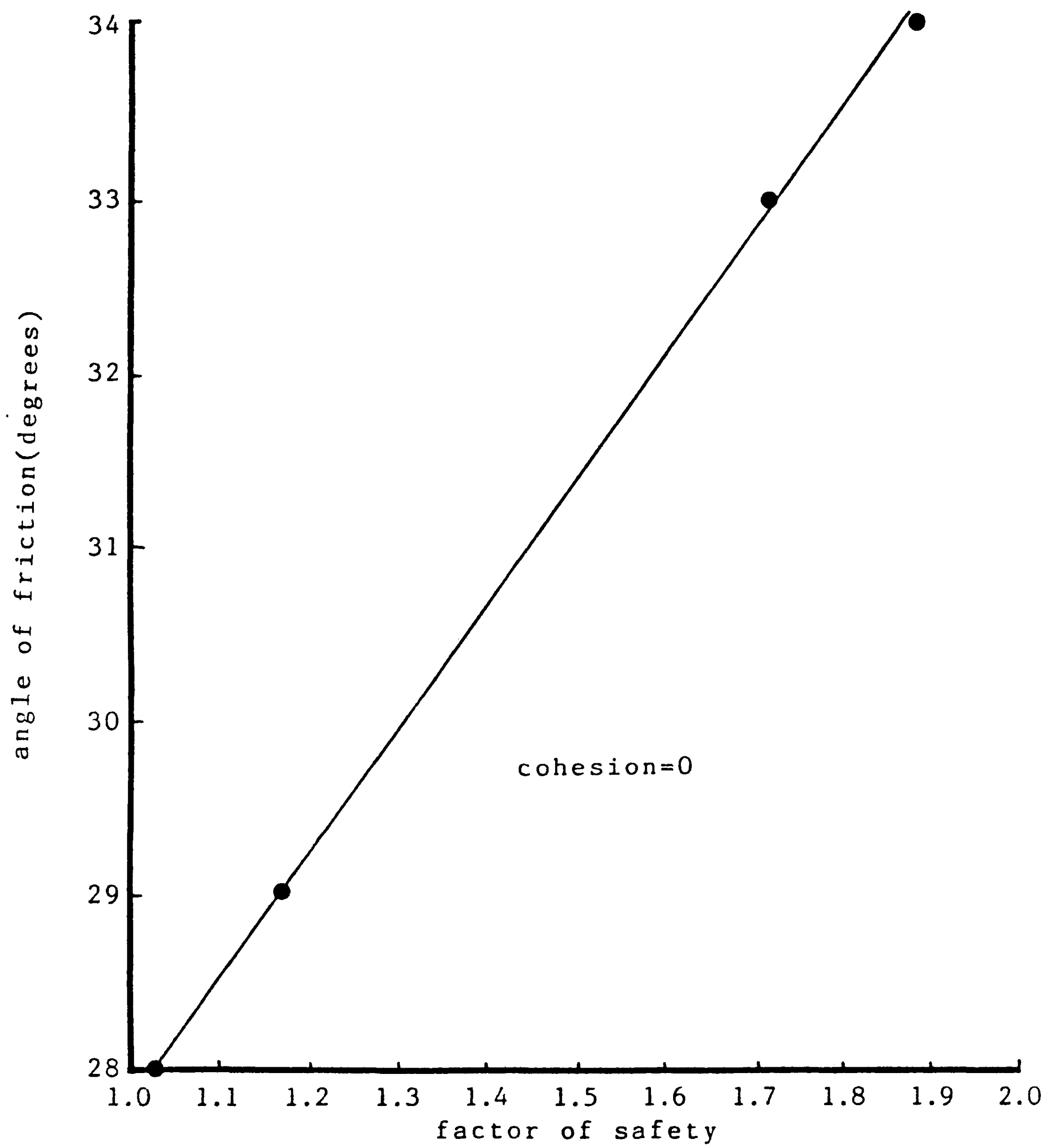


Figure 23. Plot of factor of safety vs. angle of friction for road fill stability analysis.

5.0 SUGGESTED REMEDIAL MEASURES

The ultimate purpose of this study was to determine the most economic method of stabilization of the landslide. In general, there are five basic engineering alternatives to consider in highway construction through a landslide area (Schuster and Krizek, 1978):

1) Avoid the landslide by rerouting;
2) Excavate dangerous materials;
3) Regrade the area to stabilize zones of movement;
4) Use restraining structures to control movement; and,
5) Stabilize the area by hydrologic alteration.

Each of these categories was considered and their relative costs and potential for success were assessed.

5.1 Rerouting

Because of the steep canyon walls and the lack of suitable alternate routes, construction costs for a new right of way would be prohibitive. Therefore, rerouting is not a valid alternative.

5.2 Excavation of Slide Materials

The total volume of the present slide was calculated as slightly more than one million cubic yards. Typical costs for excavation and haulage to a nearby disposal site, assuming a suitable nearby site can be found, are estimated to be $12 per cubic yard. In this case, a total removal of all materials is probably not feasible, since the highway would then have no simple method of traversing this area. Therefore, this alternative would more likely involve removing about half the material at a cost of $6.5 million.

The costs for this are much higher than the other alternatives described in the following sections.

5.3 Regrading the Slide

The regrading alternative refers to a greatly reduced volume of earthwork excavation compared to the volume in the excavation alternative. Regrading is often used to reduce driving forces and increases resisting forces by excavating the upper portions and adding material to the toe areas of slides. Its success and practicality depends on the geometry of the slide.

The Battle Mountain Slide, in general, does not seem to be especially favorable to regrading. The construction of a large toe berm is not practical within the space available between the toe of the slide and the Denver and Rio Grande Western railway tracks. The upper portion of the slide is not particularly steep.

Some regrading of the highway fill may aid in the stability of the highway grade. Analysis of stability for Zone A, the highway fill, showed that this fill was probably not failing on its own account, but was failing due to loss of support by the underlying slide materials. However, knowledge of the strength characteristics of the fill materials is very poor, so the computed factors of safety are open to question.

The present fill is much larger than needed and has a very steep face. This undoubtedly adds considerable loads and stresses to the underlying slide materials. Narrowing the excessively wide shoulder and regrading this slope would involve the removal of about 6,000 cubic yards of material, removing a load of at least 10,000 tons from this portion of the slide. This measure should cost $72,000. By itself, it will not stabilize the slide, but since it is a probable contributing factor to the distress and distortions exhibited in the pavement, it may be an appropriate measure to undertake if maintenence costs rise significantly.

5.4 Restraining Structures

Restraining structures include various types of walls, berms and pile structures. Because of the size and depth of this slide, retaining walls and piles are not viable. Since there is limited space available between the toe and the railroad tracks, an adequate toe berm cannot be constructed.

One type of restraint might be feasible, however. In conjunction with the road fill regrading suggested in section 5.3 above, a thorough reconstruction of the road fill using reinforced earth technology should be considered.

Reinforced earth techniques were successfully used on Vail Pass during the construction of Interstate-70 to dramatically reduce the volumes, and thus the weights, of road fills in landslide areas. The design and cost of this type of reconstruction is beyond the scope of this report, but it is believed this alternative should be studied.

5.5 Alteration of Hydrologic Conditions

A common, and frequently effective, method of slide stabilization is the alteration of the hydrologic conditions to promote drainage of the slide mass. Such drainage can involve both the diversion of surface waters to reduce the infiltration of water into the slide, and the installation of drainage systems to assist the removal of ground water contained within the slide.

Due to the steepness of the slopes, diverting the drainages that lead into the Battle Mountain Slide would prove difficult. If any diversion structures were constructed, the snow avalanches which inundate the drainages in winter and spring would probably damage or destroy them. Finally, much of the water that saturates the slide mass comes from snowmelt percolating directly down through the soil (see section 3.6).

In the past, horizontal drains, constructed of perforated PVC pipe, have been used successfully in dewatering saturated slopes. One slide in California produced one million gallons of water in one day from horizontal drains. The slope proved to be stable after the drains were installed. The Colorado Department of Highways has used horizontal drains to control landslide movements on Vail Pass and the Whiskey Creek Slide, north of Minturn. One of the Whiskey Creek Slide installations is shown in figure 24. In most cases, such drains are hidden by small rock fills. These are permeable enough not to interfere with the drainage process, but help prevent vandalism and freezing of the drainage pipes' discharging ends, while also improving the aesthetics.

The Battle Mountain Slide appears to be favorable to treatment by horizontal drains. Generally, these drains are drilled in roughly fan-shaped arrangements from a small number of "drill pads". Economic and technical considerations mandate the use of drill pads which should be located in accessible areas to maximize the potential drainage while minimizing the total amount of drilling.

Five drill pad sites have been selected. Four are acc-

Figure 24. Horizontal drains used on the Whiskey Creek Landslide, north of Minturn.

essible from the highway and the fifth would require an easily constructed access road. Figure 25 and Plate 1 show the locations of these proposed drill pad sites, and define the pad elevations and orientations of each drain hole. These locations and patterns were selected to meet the following criteria;

1) The drains should not cross any shear surfaces, to ensure their continued operation and prevent the pipes from acting as a source of water in the event of slide movement;
2) A minimum upward gradient of 5% in the drain pipe, which seems a practical limit to ensure efficient flow through the pipes;
3) The maximum drill depth is 200 feet, due to equipment limitations, and;
4) Drains are located to maximize the interception of groundwater in the slide.

It is proposed that two-inch inside diameter PVC pipes be used, with mill slot perforations in the appropriate sections. Figures 26 through 29 show the drain profiles for each drill pad while Table 5 quantifies their orientation and design.

The washing out of fines from the annulus around the drain pipe may be a significant problem in this type of in-

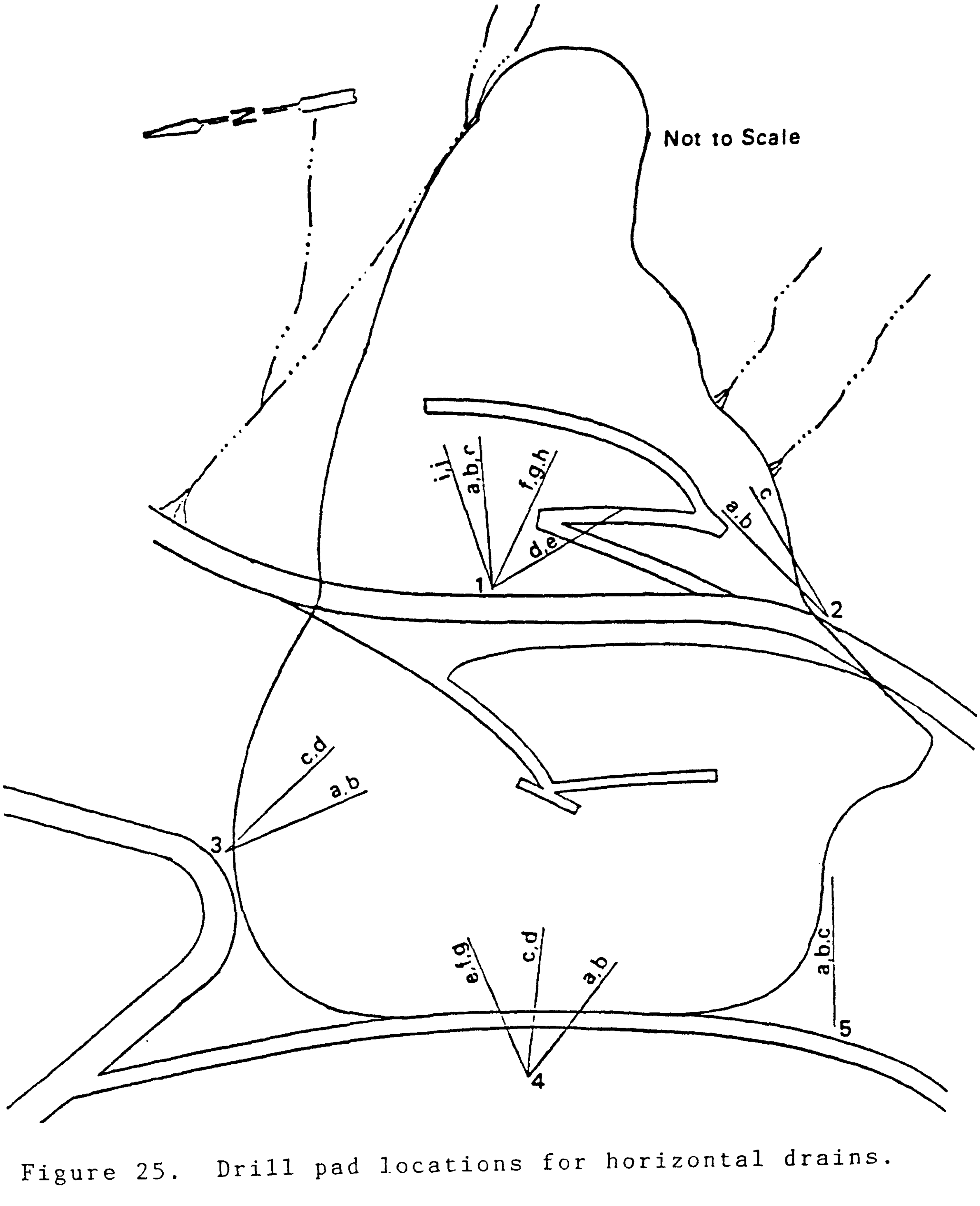


Figure 25. Drill pad locations for horizontal drains.

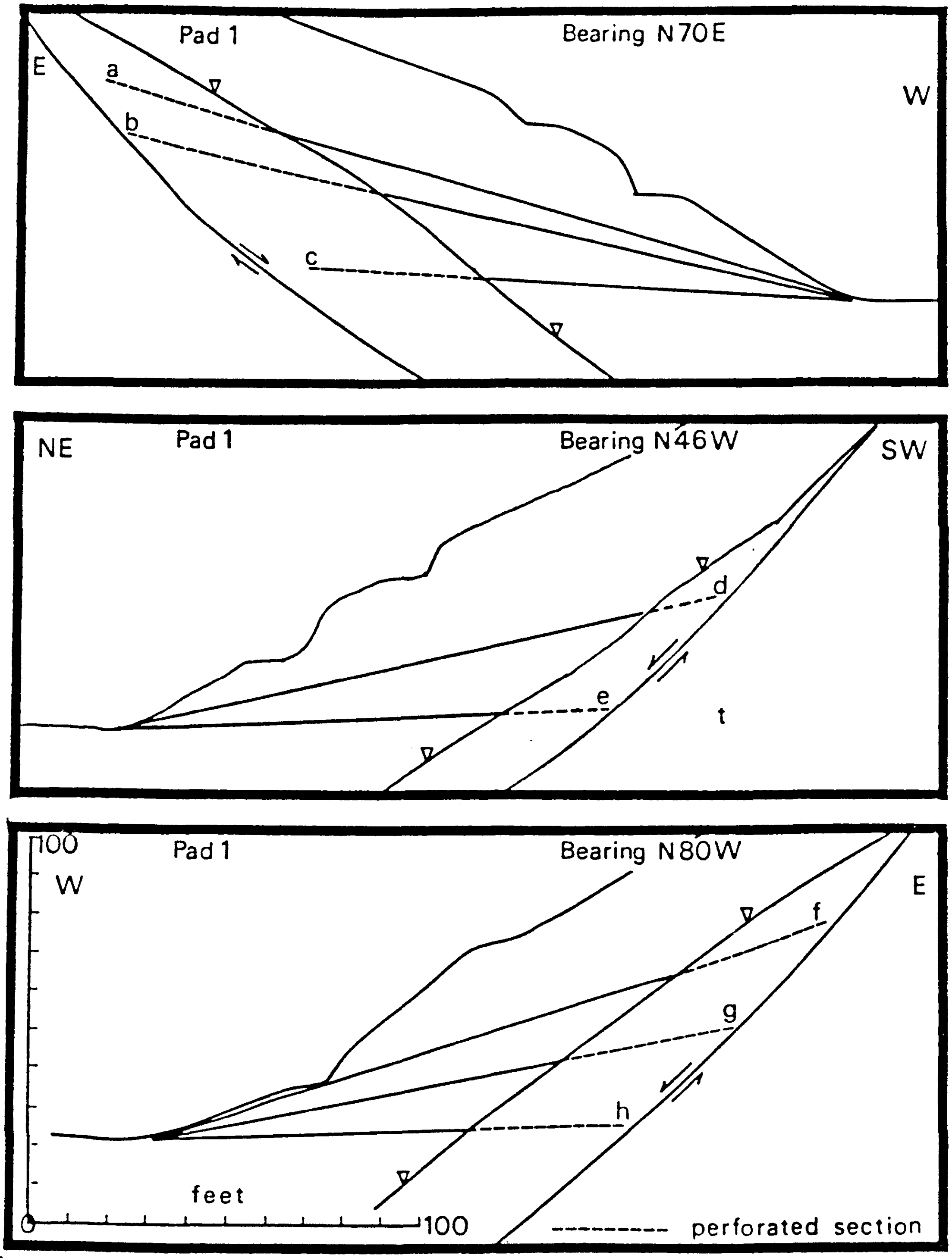


Figure 26. Cross section showing proposed drain profiles from drill pad 1.

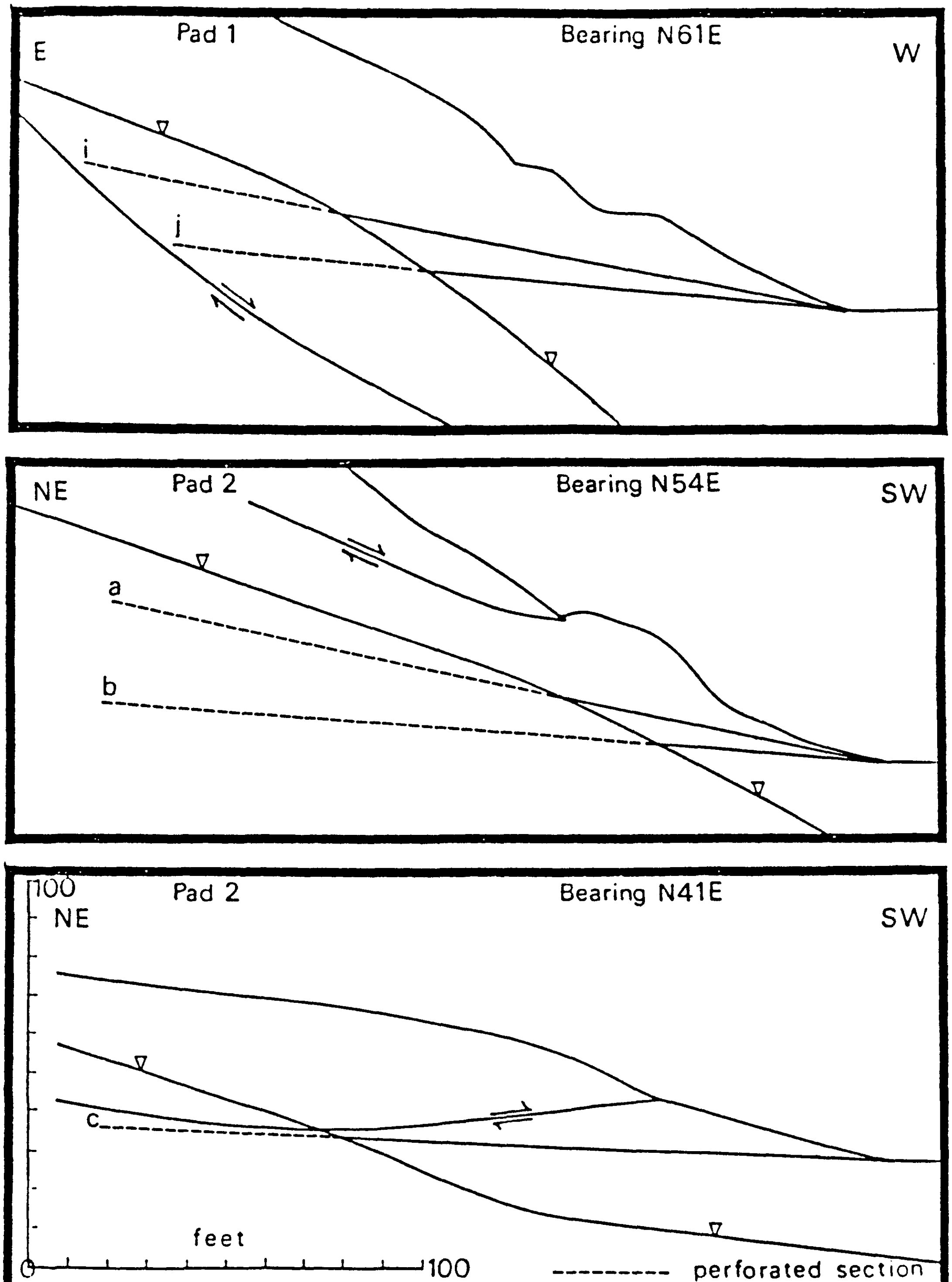


Figure 27. Cross sections showing proposed drain profiles from drill pads 1 and 2.

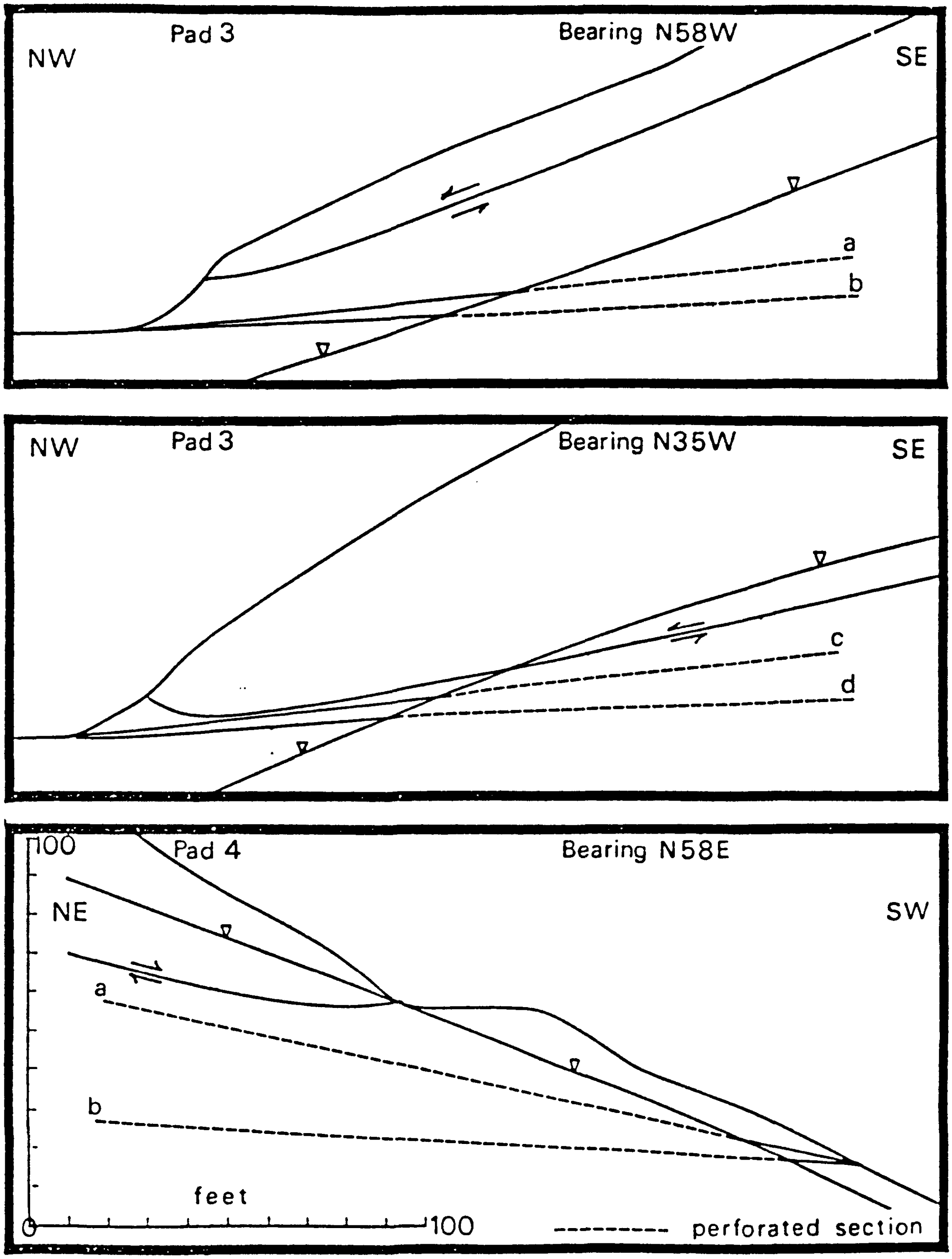


Figure 28. Cross section showing proposed drain profiles from drill pads 3 and 4.

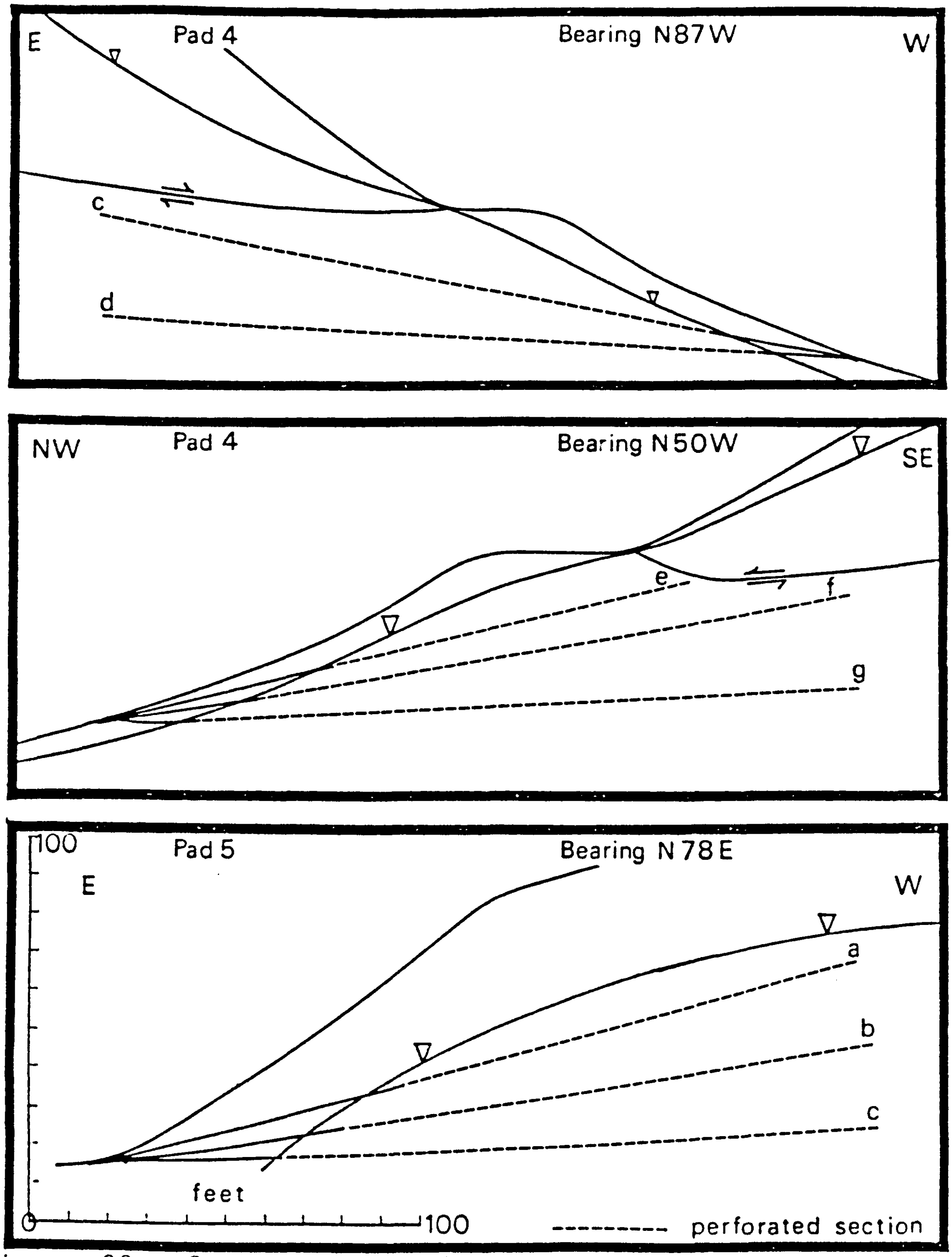


Figure 29. Cross section showing proposed drain profiles from drill pads 4 and 5.

Table 5. Horizontal Drain Design Data.

Pad	Pad Elevation	Drain	Bearing	Elevation Angle	Length	Perforated Section	Priority
1	1007	a	N70E	16.0	200	45	2
		b	N70E	12.5	190	55	3
		c	N70E	2.9	145	50	1
		d	N46W	12.33	160	20	2
		e	N46W	2.9	125	25	1
		f	N80W	2.9	125	40	1
		g	N80W	10.6	150	45	3
		h	N80W	16.7	190	40	2
		i	N61E	10.5	200	60	2
		j	N61E	5.7	170	60	1
2	1025	a	N54E	10.5	200	100	2
		b	N54E	2.9	200	150	1
		c	N41E	2.9	100	60	1
3	795	a	N58W	2.9	200	85	1
		b	N58W	6.0	200	105	2
		c	N35W	6.3	200	105	2
		d	N35W	2.9	200	120	1
4	760	a	N58E	11.9	200	170	2
		b	N58E	2.9	200	175	1
		c	N87W	11.3	200	130	2
		d	N87W	2.9	200	140	1
		e	N50W	13.0	200	100	2
		f	N50W	9.4	200	150	3
		g	N50W	2.9	200	165	1
5	830	a	N78E	14.8	200	120	2
		b	N78E	8.8	200	145	3
		c	N78E	2.9	200	155	1

stallation (Cedergren, 1967). Because of the nature of the installation, it is impossible to place a filter around the drain. However, when the slide material is fairly coarse, as is believed to be the case at Battle Mountain, the slotted portion of the drain may provide an adequate filter (Cedergren, 1967). Thus, it is believed that horizontal drains should operate satisfactorily at Battle Mountain.

The maximum net flow into the slide during spring runoff is estimated to be 105 to 150 gallons per minute. This is based on the maximum change in the potentiometric surface, from April 2 to April 16, and on a conservative estimated average void ratio of 0.15 in the saturated soil zone. In order for the drain system to work, it obviously should have a discharge capability greater than or equal to the maximum rate of infiltration of water into the slide (Ridgeway, 1982).

The actual discharge due to these drains cannot be accurately predicted. Some rough calculations suggest that the maximum rates of drainage through the pipes under normal operating conditions would be about 90 gallons per minute. However, rising heads would increase this flow rate, so it is believed that the system, as outlined, is adequate to handle the groundwater volumes likely to be encountered.

By reason of the observed geologic and hydrologic conditions, along with length and other economic factors, some drain orientations are more likely to be successful than others; hence, each drain in Table 5 has been given a priority ranking of 1, 2, or 3.

5.5.1 Estimated Cost for Horizontal Drains

The Colorado Department of Highways suggested that a cost of $10 per foot is a reasonable estimate for installing horizontal drains in the Battle Mountain Slide. The total footage of priority 1, 2, and 3 drains shown in Table were summed and multiplied by the $10 per foot estimate to compute the cost for this proposed drilling program. Table shows three costs: the cost for the priority 1 drains only; for the priority 1 and 2 drains, and; all the proposed drains.

5.5.2 Cost-Benefit Analysis

Table 6 also shows the anticipated drainage rates for each option. When the increases in cost are compared to the increases in anticipated drainage, an estimate for relative efficiency can be calculated. In the final column of Table 6, the estimated relative efficiency has been computed for each class of drains in terms of gallons per minute of drainage achieved for each $1,000 of drilling costs. These calculations show that the priority 1 holes are expected to perform more drainage for the dollars spent than either the priority 2 or 3 holes. Actually, the priority 3 holes appear to have a better drainage-to-cost ratio than do the priority 2 holes, but are believed to be more risky because of geological uncertainities.

According to Mr. Jim Henderson, a Highway Department crew foreman, maintenance costs for patching the slide are approximately $45 per ton for materials, labor, and equipment. Based on the maintenance records in Appendix A, this works out to about $4,300 per year.

Some horizontal drains in California have been working for over 20 years. Using this figure as a design life, the total cost of installing all the drains ($49,550) averages $2,477 per year. This represents a savings of over $1,800 per year compared with the current maintenance costs. It should be noted, however, that this comparison does not differentiate between current and future expenditures.

5.6 Other Drainage Facilities

All water discharging from these horizontal drains should be routed away from the slide by appropriate surface drainageways. Discharges from pad 2 should be drained along the highway shoulder to pad 1, then off the slide to the north. This drainage can be accomplished with a ditch

Table 6. Cost-Benefit Analysis

Priority	Total Length (feet)	Cost (dollars)	Estimated Drainage Rate (gpm)	Benefit-Cost Ratio (gpm/$1000)
1	2,065	20,650	43	2.09
2	2,150	21,500	33	1.54
3	740	7,400	14	1.87
1+2	4,215	42,150	76	1.81
1+2+3	4,955	49,550	90	1.82

or french drain, preferably lined with a flexible, impermeable geomembrane to prevent infiltration of the water back into the slide mass. Drainage water collected at pads 3, 4, and 5 can be diverted away from the slide by gravel drains to prevent surface erosion.

Water infiltrating through the shoulder of the highway should also be considered. Snow plowed off of the road melts and percolates directly into the slide from the shoulders. Conventional drainage gutters, made of bituminous material, should be installed from 100 feet south of TH-1 to the north lateral scarp, on both sides of the road. For the same reason, the road should be repaved. The drill roads should be revegetated, as should any disturbed soil on the slide.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The Battle Mountain Slide is a complex slide, with a volume of approximately one million cubic yards. Movement is triggered by saturation of the mass by snowmelt in the spring. Intense rainfall events which occur during the summer do not appear to affect the slide's stability.

The slide has at least four zones which move at different rates. The dominant sequence of movement appears to begin with the failure of the lower central portions of the slide which become saturated. These movements cause a loss of support for zones higher in the slide causing them to fail in turn. It is possible that this movement may also be triggered by initial movements in the crown area, as the upper slide mass becomes saturated by avalanche deposits and moves downhill early in the spring. This process would increase loads to the central lower portions of the slide mass. Such movements could not be confirmed due to snow cover and difficult access.

The saturation in the lower portions of the slide appears to be due in part to seepage of groundwater from lower portions of the Minturn Formation into the more permeable slide material.

The most rapid movements during the Spring of 1985 were in the road fill on the south flank of the slide. This failure appears to be due to several causes, including the removal of support from beneath the fill by movement of the main slide, by saturation of the road fill from snowmelt, and by the weight of the road fill, which is much larger than is required for the highway.

Analyses using the STABL2 computer program indicated that the slide stability could be increased significantly with drainage. For this reason, the major stabilization measures listed below are recommended to provide proper drainage. Regrading and reconstruction of the road fill, thereby reducing its mass and increasing the stability of the highway, is also recommended.

6.2 Recommendations

The following procedures are recommended to stabilize the Battle Mountain Slide:

1) Install 4,955 feet of horizontal drains from the five specified drill pads.
2) Rehabilitate surface drainage systems along the highway, to further reduce water infiltration rates.
3) Reduce the weight of the slide by minimal regrading of the excessively wide highway shoulder.
4) In conjunction with (3) above, consideration of a

more extensive reconstruction of the road fill, using reinforced earth techniques to further reduce its weight.

Rough cost analyses indicate that the installation of the full set of horizontal drains would cost about $50,000; the regrading and removal of the excess fill might cost as much as $72,000, while the excavation and removal of up to half the failing slide would cost $6.5 million. Furthermore, assuming a 20 year life expectancy for these drains, their annual cost of $2,500 is substantially less than the current average maintenance cost for roadway resurfacing of $4,300. Accordingly, the installation of the horizontal drains appears to have the highest priority.

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U.S. Geol. Survey, 1979, Minturn, Colorado 7½ minute topographic quadrangle.

APPENDIX A

HIGHWAY MAINTENANCE RECORDS

STATE OF COLORADO

DEPARTMENT OF HIGHWAYS

606 So. Ninth St. P O. Box 2107
Grand Junction, Colorado 81502-2107
(303) 248-7389

August 7, 1985

Mr. Brend Shine
P.O. Box 1042
Golden, Colorado 80402

Mr. Shine;

In responce to your request for information on Battle Mtn. Slide, Hwy. 24, Mile Marker #149.30 to 149.50, I was able to gather the following information.

The location in question has been re-paved nearly every year since our records were started in 1978.

7-78 thru 8-78	93 tons of premix applied
9-79 thru 10-79	86 tons of premix applied
7-80	360 tons of premix applied
7-81	10 tons of premix applied
82	no premix recorded for this location
83	no premix recorded for this location
8-84 thru 10-84	72 tons of premix applied
6-85	146 tons of premix applied

Material placed on the shoulder is impossible to trace since it would have been removed cleaning ditches within two or three miles, and not recorded as to where placed. Also, since the road seems to sink gradually, our records do not reflect an exact day of settlement, except that it occurs in the springtime. March 15, thru May 31, seems to be the most active time.

I hope this information is of value to you. If I may be of further assistance, please let me know.

Merle J. Long
M.M.S. Coordinator
Grand Junction, Colorado

APPENDIX B

OBSERVATION WELL LOGS

STATE OF COLORADO
DEPARTMENT OF HIGHWAYS
DIVISION OF HIGHWAYS
DOH Form No. 267
Revised, September, 1978

Project
Location Battle Mountain
Structure Slide
Route US 24 County Eagle
Date Drilled 7/23/24

FOUNDATION BORING LOG

Top Hole Elev. ______ Logger ~~Geologist~~ Driller Station ______ Boring No. TH-1

Elev.	Depth	Description of Material	BPF*	Remarks
	0-5'	Fill		
	5'-28.5'	Sandstone w/ interbedded shales		TD

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ______ Elev. ______ Date ______ Time ______
Water level (24 hrs.) ______ Elev. ______ Date ______ Time ______

STATE OF COLORADO
DEPARTMENT OF HIGHWAYS
DIVISION OF HIGHWAYS
DOH Form No 267
Revised. September, 1978

Project
Location Battle Mountain
Structure Slide
Route US 24 County Eagle
Date Drilled 7/24/74 - 7/25/74

FOUNDATION BORING LOG

(TH-2)

Top Hole Elev. ______ Geologist ______ Station ______ Boring No. 2

Elev.	Depth	Description of Material	BPF*	Remarks
	0-3'	Organic top soil		7/24
	3'-12'	black silt w/ subangular gravel		
	12'-21'	Maroon silt w/ boulders, cobbles and gravel		
	21'-28'	Wet - No returns		Stopped for day
	28'-35'	Spoon sample attempted - No returns		7/25
	35'-38'	White dust (?) - Poor returns		Slow drilling
	38'-42'	Maroon silt w/ cobbles, gravel and boulders		
	42'-48'	Dark brown to gray black shaley, sandy clay		TD
		Ran PVC pipe to TD		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ______ Elev. ______ Date ______ Time ______
Water level (24 hrs.) ______ Elev. ______ Date ______ Time ______

STATE OF COLORADO
DEPARTMENT OF HIGHWAYS
DIVISION OF HIGHWAYS
DOH Form No. 267
Revised, September, 1978

Project
Location Battle Mountain
Structure Slide
Route US 24 County Eagle
Date Drilled 7/23/84

FOUNDATION BORING LOG

(TH-3)

Top Hole Elev. ________ Geologist Pihl Station ________ Boring No. 3

Elev.	Depth	Description of Material	BPF*	Remarks
	0-11'	Fill		Logged by driller
	11-20'	Maroon silt		
	20'-22'	Dark brown silt		
	22'-23'	Salmon color silt		
	23'-28'	Poorly to moderately lithified cobbles and gravel		
	28'-33'	Maroon silt w/ subrounded to well rounded		
		gravel in 1" to 6" layers		
	33'-45'	As above Moist		
	45-75'	As above. Wet		
	75'-80'	As above. Very Wet		
	80'-82'	No returns		
	82-84'	No returns		Difficult to drill
	84'-86'	No returns		Less difficult to drill
	86'-94'	No returns		Difficult to drill
		Set 3/4" PVC pipe to TD		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ________ Elev ________ Date ________ Time ________
Water level (24 hrs.) ________ Elev. ________ Date ________ Time ________

STATE OF COLORADO
DEPARTMENT OF HIGHWAYS
DIVISION OF HIGHWAYS
DOH Form No 267
Revised, September, 1978

Project ____
Location Battle Mountain
Structure Slide
Route US 24 County Eagle
Date Drilled 7/24/84

FOUNDATION BORING LOG

(TH-4)

Top Hole Elev. ____ Geologist Pihl Station ____ Boring No. 4

Elev.	Depth	Description of Material	BPF*	Remarks
	0-2′	Fill		
	2′-13′	Maroon silt w/ subangular to subrounded gravel		
	13′-14′	Tan silt w/ few gravel. Moist		
	14′-22′	Maroon and tan interbedded silt. Sandstone		
		boulder at 19′		
	22′-24′	White to light tan medium grain sandstone		
	24′-70′	Maroon silt w/ occasional sandstone boulders		
	70′-82′	Maroon silt. Moist		
	82′-127′	Interbedded white to tan medium grained		
		sandstone and maroon silt		
	127′-140′	Rounded to subrounded sandstone gravel w/		
		trace of clay balls		
		Set PVC to 128′		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ____ Elev. ____ Date ____ Time ____
Water level (24 hrs) ____ Elev. ____ Date ____ Time ____

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Date Drilled 7/25/84 - 7/30/84

FOUNDATION BORING LOG

(TH-5.)

Top Hole Elev. ____ Geologist Pihl Station ____ Boring No. 5

Elev.	Depth	Description of Material	BPF*	Remarks
	0-2'	Fill, sand and gravel		Logged by 7/25
	2'-100'	Maroon silt w/ gravel and cobbles of sandstone		Driller
	100'-105'	Medium to Coarse grained sandstone. Weathered		Stopped for Day
	105'-110'	Moderately well sorted, subangular, medium grained		7/26
		sandstone cobbles		
	110'-119'	Silty sand w/ gravels and cobbles		
	119'-123'	Interbedded maroon silt and slightly		
		micaceous sandstone gravel		
	123'-139'	Salmon colored silty fine grained sand w/		
		micaceous sandstone gravel		
	139'-162'	Salmon siltstone w/ cobbles and gravel		
	162'-168'	As above. Moist		Stopped for Weekend
	168'-178'	Silty subrounded, medium grained quartz sand		7/30
	178'-185'	subrounded, medium grained quartz sand w/		
		trace of black shale and dark maroon to		
		blackish sandy shale		
		Set PVC pipe		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ____ Elev. ____ Date ____ Time ____
Water level (24 hrs.) ____ Elev. ____ Date ____ Time ____

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FOUNDATION BORING LOG

(TH-6, 1 of 2)

Top Hole Elev. Geologist Pihl Station Boring No. 6½

Elev.	Depth	Description of Material	BPF*	Remarks
	0-2'	organic top soil		
	2'-6'	cobbles and subrounded gravel of quartz sandstone		
	6'-8'	Red silt w/ subrounded gravel. Moist		
	8'-11'	Red silt w/ fine to medium grain sand		
	11'-17'	Interbedded red and green mudstones and		
		medium grained subrounded quartz gravel		
	17'-18'	Light colored fine grained sandstone		
	18'-22'	Interbedded red and green mudstones, black		
		shale and sandstone		
	22'-27'	Interbedded sandy green mudstone, quartz		
		gravels and red talc-like greasy silt. Moist		
	27'-44'	Red silt w/ light colored fine grained sandstone		
		cobbles and boulders w/ trace of Shale. Moist		
	44'-53'	As above w/ more silt and moisture		
	53'-63.5'	Maroon silt		Easier to drill
	63.5'-68'	Red silt w/ fine grained sandstone cobbles and		Stopped for day
		boulders w/ trace of shale. Moist		
	68'-81'	Red sandy silt w/ med grained sandstone gravel		8/2
		high water content		
	81'-87'	No returns		Easy to drill
	87'-94'	Maroon sandy silt w/ some gravel		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion Elev. Date Time
Water level (24 hrs.) Elev. Date Time

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FOUNDATION BORING LOG

(TH-6, 2 of 2)

Top Hole Elev. Geologist Pihl Station Boring No. 6 2/2

Elev.	Depth	Description of Material	BPF*	Remarks
	94'-98'	As above. Very Wet		Drill rig jumping
	98'-101'	Interbedded green mudstone (40%), black		Drilling smoothed out
		shale (20%) and maroon micaceous medium		
		grained sandstone (40%)		
	101'-103'	No returns		
	103'-108'	Interbedded green mudstone, black shale		Drilling rig jumping
		and maroon medium grained sandstone		Stopped for weekend
	108'-125'	Silt w/ cobbles and gravel		8/6
	125-128'	No returns		Stopped for day Need more circ.
	128'-135'	Silt w/ cobbles and gravel		8/7
	135'-138'	Interbedded dark gray shale (30%), green		
		mudstone (20%) and medium grained		
		sandstone (50%)		
	138'-143'	Silty sand w/ cobbles and gravels		Stopped for day
	143'-152'	Cobbles and gravel - Lost circ.		8/8
		Set PVC pipe to 152'		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ______ Elev. ______ Date ______ Time ______
Water level (24 hrs.) ______ Elev. ______ Date ______ Time ______

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FOUNDATION BORING LOG

(TH-7)

Top Hole Elev. ______ Geologist Pihl Station ______ Boring No. 7

Elev.	Depth	Description of Material	BPF*	Remarks
	0-2'	Black peat		
	2'-9'	clayey, sandy silt. Moist 4'-9'		
	9'-22'	Green to tan shaley mudstone and salmon		
		to light maroon silt and sand		
	22'-25'	As above w/ occassional dark brown to black		
		silty sand and cobbles and boulders		
	25'-35'	White medium grained sandstone gravel w/		
		some tan silt		
	35'-55'	Maroon sandy silt w/ sandstone gravel		
		cobbles and boulders. Moist		
	55'-93'	Tan to red slightly sandy silt w/ sandstone		
		and dark green to black shale gravel		
	93'-98'	Light brown to dark tan very fine grained sand		
		and silt w/ subrounded to well rounded sandstone		
		and shale gravel. Moist		
	98'-128'	Interbedded gray brown sandy silt and		
		white sandstone. Occassional gravel composed		
		of very hard black shale		
	128'-133'	Light colored fine grained sandstone, Salt and		
		Pepper texture.		
	133'-145'	Black Shale w/ some sandstone lenses		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ______ Elev. ______ Date ______ Time ______
Water level (24 hrs.) ______ Elev. ______ Date ______ Time ______

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Date Drilled 8/10/84

FOUNDATION BORING LOG

(TH-8)

Top Hole Elev. Geologist Pihl Station Boring No. 8

Elev.	Depth	Description of Material	BPF*	Remarks
	0-2'	Black peat w/ sandstone cobbles		
	2-15'	Maroon to brown sandy silt w/ angular to		
		subangular sandstone gravel, cobbles and		
		boulders		
	15'-32'	Angular to subangular med. grain quartz		
		sandstone w/ salmon to dark tan silt and		
		a little green mudstone		
	32'-40'	Well sorted, subangular to subrounded, fine to		
		medium grained quartz sand		
	40'-42.5'	As above w/ gray sandy mudstone and a		
		trace of sandy shale		
	42.5-48'	Fine to medium grain sand		
	48'-58'	Interbedded green mudstone, quartz sand		
		and shale w/ subrounded gravel and cobbles		
	58'-79'	Interbedded red, medium grained sandstone,		
		black shale and salt and pepper micaceous sandstone		
		Traces of maroon sandy siltstone and green		
		mudstone. Moist		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ______ Elev.______ Date______ Time______
Water level (24 hrs.)______ Elev.______ Date______ Time______

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FOUNDATION BORING LOG

(TH-9)

Top Hole Elev. ______ Geologist P.M.I Station ______ Boring No. 9

Elev.	Depth	Description of Material	BPF*	Remarks
	0-4'	Black peat w/ gravel and cobbles. Moist		
	4'-9'	Dark brown silty medium grain sand w/		
		gravel, cobbles and boulders. Moist		
	9'-16'	Silty sand and sandstone gravel w/ maroon		
		silt		
	16'-22'	Micaceous, medium grain subrounded sand		
		w/ gravels and siltstone. Very moist		
	22'-25'	Maroon silt, sand and subrounded sandstone		
		gravel.		
	25'-26.5'	Maroon, clayey silty sand interbedded w/		
		gravels, black shale and green mudstone		
		Very wet		
	26.5'-51'	As above w/ more clay.		
	51'-74'	Interbedded black shale, green mudstone		
		and thin sandstone lenses.		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ______ Elev. ______ Date ______ Time ______
Water level (24 hrs.) ______ Elev. ______ Date ______ Time ______

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FOUNDATION BORING LOG

(TH-10)

Top Hole Elev. Geologist Station Boring No. 10

Elev.	Depth	Description of Material	BPF*	Remarks
	0-2'	Black peat w/ sandstone gravel and cobble		8/14
	2'-9'	Brown silty sand and sandstone gravel and		
		cobbles. Moist		
	9'-18'	Maroon silty fine grained sand w/ sandstone		
		gravel and cobbles. Very moist		
	18'-23'	Dark brown silty sand and gravel. Very Moist		
	23'-25'	Maroon slightly micaceous silty sand and		
		gravel.		
	25'-47'	Interbedded black shale, green mudstone		
		and fine to medium grain sandstone occassional		
		gravel and angular conglomerate		
	47'-69'	Maroon siltstone w/ gravel, medium to		
		coarse grain sandstone and green mudstone		
	69'-73'	Interbedded green mudstone and maroon		Stopped for clay
		siltstone		
	73'-84'	Maroon siltstone		8/15
		Set PVC pipe to 84'		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ______ Elev.______ Date______ Time______
Water level (24 hrs.) ______ Elev.______ Date ______ Time

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FOUNDATION BORING LOG

(TH-11, 1 of 3) 1/3

Top Hole Elev. ____ Geologist Pihl Station ____ Boring No. 11

Elev.	Depth	Description of Material	BPF*	Remarks
	0-4'	Fill, gravel and cobbles w/ black organic silty		8/15
		sand		
	4'-10'	Brown silty fine grained sand w/ gravel and		
		cobbles		
	10'-13'	Sanstone boulder		
	13'-25'	Brown to reddish, slightly clayey, sandy silt		
		w/ mudstone and sandstone subrounded		
		to rounded gravel, cobbles and boulders		
	25'-26.5'	25'-26' Brown silty sandy clay w/ some		Drive #1
		green mudstone gravel and a trace		recovered 14"
		of black shale		saved 10"
		26'-26.5' Red silt w/ angular medium grained		
		sandstone gravel and a trace of		
		angular black shale cobbles.		
	26.5'-31'	As above w/ sandstone and green mudstone		
		cobbles and boulders. Moist		
	31'-43.5'	Maroon silty sand w/ sandstone, mudstone		
		and shale cobbles, boulders and gravel		Stopped for day
	43.5'-48'	As above w/ higher gravel content		8/16
	48'-49.5'	Maroon clayey, silty sand w/ subangular to		Drive #2
		well rounded green mudstone, black shale		recovered 5"

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ____ Elev. ____ Date ____ Time ____
Water level (24 hrs.) ____ Elev. ____ Date ____ Time ____

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FOUNDATION BORING LOG

2/3

(TH-11, 2 of 3)

Top Hole Elev. ______ Geologist Pihl Station ______ Boring No. 11

Elev.	Depth	Description of Material	BPF*	Remarks
	48'-49.5' cont	and sandstone gravel		not saved
	49.5'-53'	As above		
	53'-58'	As above, drilling smoothed out		
	58'-59.5'	58'-59' Maroon clayey, sandy, silt w/ green		Drive #3
		mudstone, shale and sandstone gravel		recovered 13"
		59'-59.5' Brown medium to coarse grained poorly		not saved
		cemented sandstone (weathered)		
	59.5'-81'	As above, less gravel. Wet		
	81'-84'	Mudstone		
	84'-86'	No recovery		Drive #4
	86'-87.5'	No recovery		Drive #5
	87.5'-89'	Maroon silt and gravel		
	89'-92'	Black shale		Drive #6
	92'-93.5'	92'-92.5' Maroon sandy silt w/ sandstone and		recovered 10"
		shale gravel		saved sample
		92.5'-93' Subangular quartz sandstone gravel		
		93'-93.5' Micaceous fine grained sand w/ some		
		maroon silt		
	93.5'-99'	As above, very soft		
	99'-106'	Interbedded sandstone and mudstone/		
		siltstone		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ______ Elev. ______ Date ______ Time ______
Water level (24 hrs.) ______ Elev. ______ Date ______ Time ______

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FOUNDATION BORING LOG

(TH-11, 3 of 3) 3/3

Top Hole Elev. ____ Geologist Pihl Station ____ Boring No. 11

Elev.	Depth	Description of Material	BPF*	Remarks
	106-116	Brown shale interbedded w/ thin		
		sandstone lenses		
		Set 116' of inclinometer pipe		
		Pumped 150 gal of super gel and water		
		to seal off hole. Could hear commun.		
		w/ hole #3.		
		Pumped 150 gal slurry of 2 sacks of		
		Portland type II cement and 1½ sacks of		
		super gel.		
		Repeated w/ 3 additional 150-gal		
		tanks of above slurry.		
		Grouted to within 4 feet of surface.		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ____ Elev. ____ Date ____ Time ____
Water level (24 hrs.) ____ Elev. ____ Date ____ Time ____

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Project
Location [illegible]
Structure [illegible]
Route US 24 County Eagle
Date Drilled [illegible]

FOUNDATION BORING LOG

Top Hole Elev. ________ Geologist D. Jurich Station ________ Boring No. CSM-1

Elev.	Depth	Description of Material	BPF*	Remarks
	0-3'	black, loamy organic rich soil		
	3'-11'	red silty sandy micaceous soil w/ cobbles		
		of well rounded to semi rounded quartz		
	11'-13'	red arkosic sandstone boulder		difficult drilling
	13'-40'	red silty sandy micaceous soil w/ pebbles		
		cobbles and boulders of arkosic sandstone		
	40'-52'	same as above, higher moisture content		
	52'-58'	red silty micaceous arkosic sandstone		bedrock
		Set ½" steel pipe, lower 20' perforated		
		Backfilled w/ sand		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ________ Elev. ________ Date ________ Time ________
Water level (24 hrs.) ________ Elev. ________ Date ________ Time ________

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FOUNDATION BORING LOG

Top Hole Elev. ______ Geologist D. Jurich Station ______ Boring No. CSM-2

Elev.	Depth	Description of Material	BPF*	Remarks
	0-4'	red clayey soil w/ red silty micaceous		
		cobbles		
	4'-8'	red clayey soil w/ coarse grained arkosic		
		sand and red silty micaceous cobbles.		
	8'-20'	red to tan arkosic sandstone w/ siltstone		Bedrock
		stringers		slow drilling
		Set perforated ½" steel pipe		
		Backfilled w/ sand		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ______ Elev. ______ Date ______ Time ______
Water level (24 hrs.) ______ Elev. ______ Date ______ Time ______

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FOUNDATION BORING LOG

Top Hole Elev. ______ Geologist D. Jurich Station ______ Boring No. CSM-3

Elev.	Depth	Description of Material	BPF*	Remarks
	0-3'	dark brown to black organic rich soil		
	3'-17'	red cobbley, clayey soil w/ boulders of buff		
		quartz sandstone		
	17'-19'	red clayey, silty soil		
	19'-21'	red silty soil w/ angular cobbles of		
		gray-green shale and fine to med. grained		
		green quartz sandstone		
	21'-22.5'	fine to med. grained green qtz sandstone		
	22.5'-25'	as above higher moisture content		
	25'-42'	red clayey, silty soil - very moist		TD
		set perforated ½" steel pipe		
		Backfilled w/ sand		

* Standard Penetration Test (AASHTO T 206-74)

Water level upon completion ______ Elev. ______ Date ______ Time ______
Water level (24 hrs.) ______ Elev. ______ Date ______ Time ______

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FOUNDATION BORING LOG

Top Hole Elev. Geologist D. Jurich Station Boring No. CSM-4.

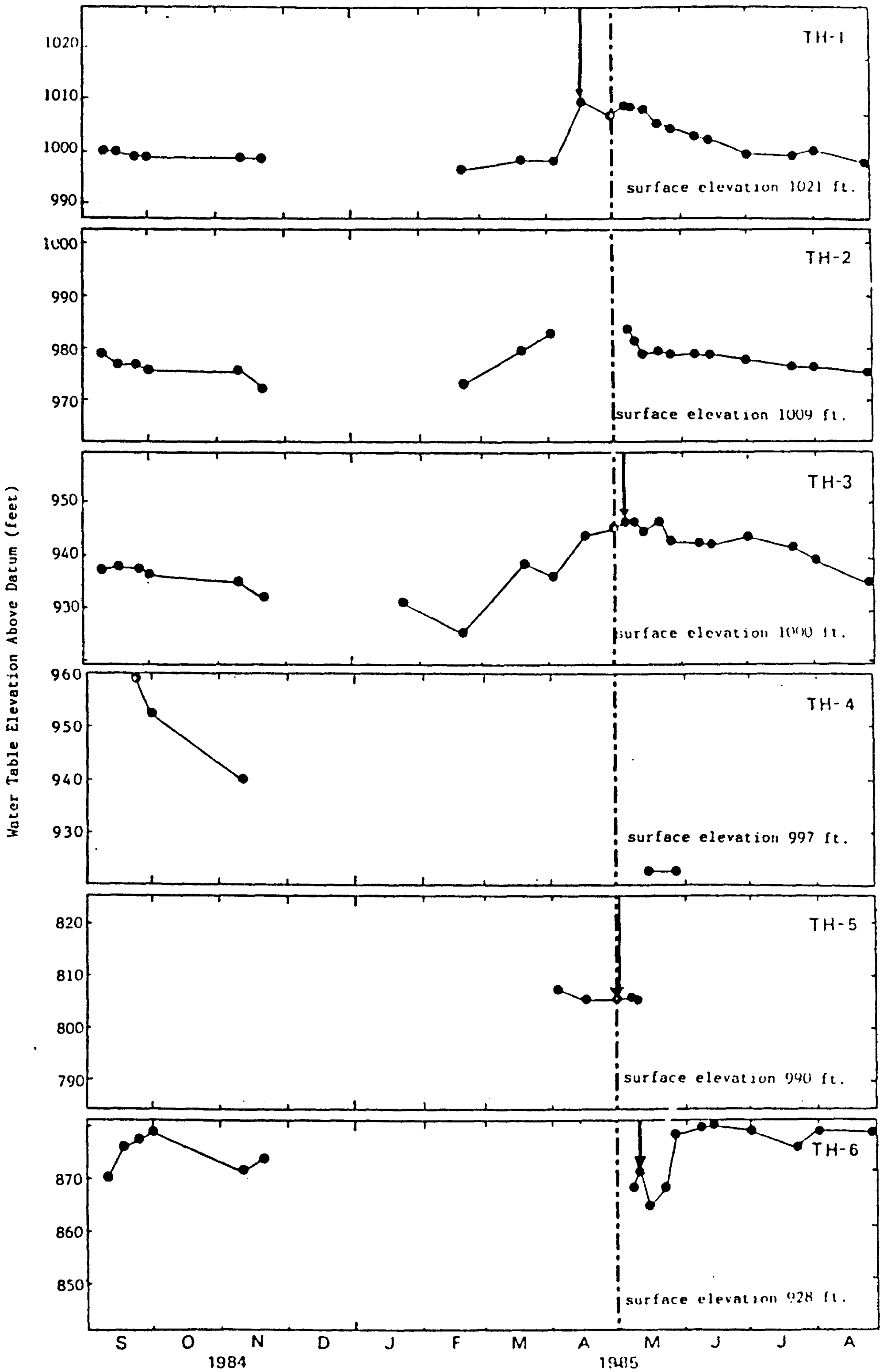
Elev.	Depth	Description of Material	BPF*	Remarks
	0-3'	black loamy organic rich soil		
	3'-5'	buff quartz sandstone		
	5'-8'	black loamy organic rich soil		
	8'-25'	red clayey soil w/ a few cobbles of		
		red micaceous arkosic sandstone and clean		
		quartz sandstone		
	25'-83'	red clayey, silty soil w/ cobbles and		
		boulders of arkosic sandstone and		
		stringers of sandy siltstone/shale		

tandard Penetration Test (AASHTO T 206-74)

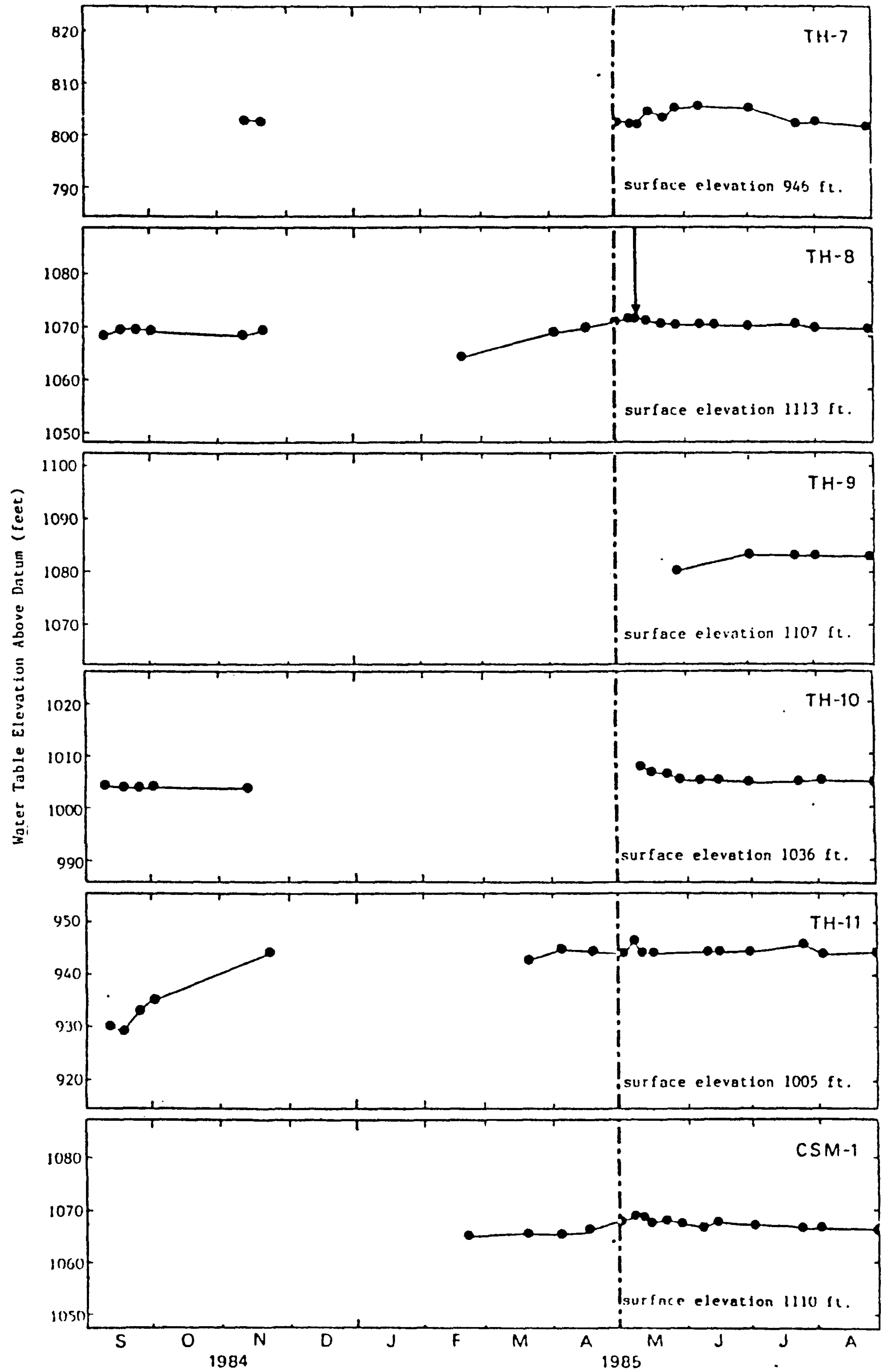
ter level upon completion Elev. Date Time
ter level (24 hrs.) Elev. Date Time

APPENDIX C

HYDROLOGIC AND PRECIPITATION DATA



Water Table Elevation Above Datum (feet)
TH-1
1020
1010
1000
990
surface elevation 1021 ft.
TH-2
1000
990
980
970
surface elevation 1009 ft.
TH-3
950
940
930
surface elevation 1000 ft.
TH-4
960
950
940
930
surface elevation 997 ft.
TH-5
820
810
800
790
surface elevation 990 ft.
TH-6
870
860
850
surface elevation 928 ft.
S
O
N
D
J
F
M
A
M
J
J
A
1984
1985


TH-7
surface elevation 946 ft.
TH-8
surface elevation 1113 ft.
TH-9
surface elevation 1107 ft.
TH-10
surface elevation 1036 ft.
TH-11
surface elevation 1005 ft.
CSM-1
surface elevation 1110 ft.
Water Table Elevation Above Datum (feet)
S O N D J F M A M J J A
1984
1985

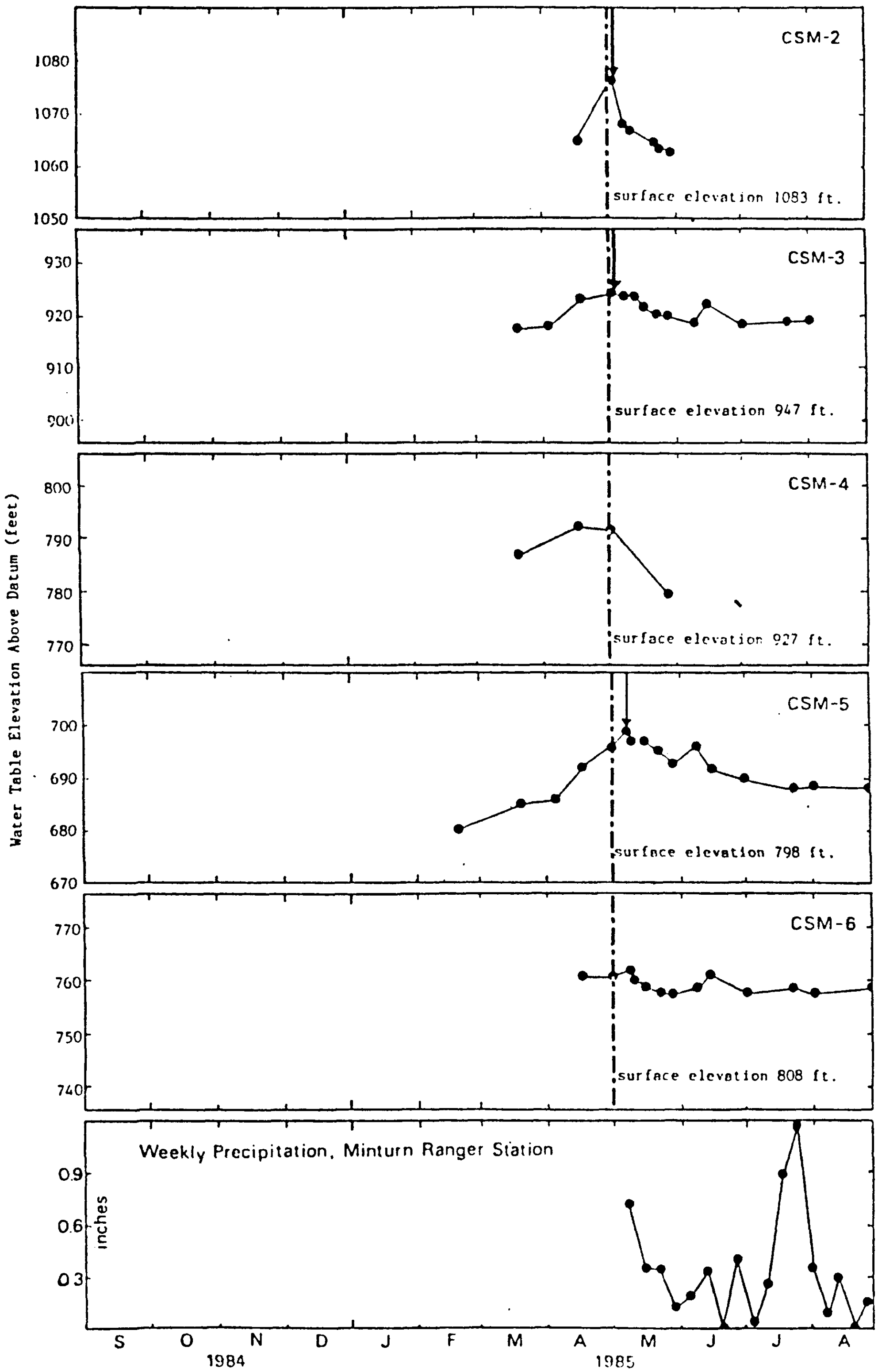


FIGURE . TEST HOLE WATER LEVELS

Drill Hole Water Level Summary

Date → Drill Hole	9-9-84	9-16-84	9-23-84	9-30-84	11-11-84	11-20-84
TH-1	20.46'	20.92'	21.50'	21.67'	22.08'	22.22
TH-2	30.00	31.00	31.25	31.83	32.50	35.67
TH-3	62.67	61.75	62.21	63.50	64.58'	67.48
TH-4	37.46	37.21	42.04	49.54	61.63	Ice in Pipe
TH-5	29.50	19.00	19.83	25.42	25.58	>150?
TH-6	58.21	52.38	51.04	49.88	56.79	54.79
TH-7	44.33	26.08	53.75	58.42	143.00	142.83
TH-8	44.75	44.00	44.00	44.00	45.00	43.90'
TH-9	-	-	-	-	-	-
TH-10	32.00	32.25	32.08	32.17	32.42	Ice in Pipe
TH-11	68.79	68.00	71.83	73.25	-	82.50

Feet Below Ground Surface

Date → Drill Hole	1-22-85	2-19-85	3-19-85	4-2-85	4-16-85
TH-1	Covered with snow	24	21.00	22.00	10.92
TH-2	- - -	35	28.42	24.92	Burried by Avalanche
TH-3	68.42	74	61.42	63.92	56.25
TH-4	Dry at 82.00, probe	Dry at 82.00	Dry at 82.00	Dry at 82.00	Dry at 78.75
TH-5	>150.0	>150.0	>150.0	182.50	180.42
TH-6	Covered with snow	Covered with snow	Covered with snow	Covered with snow	107.50
TH-7	Covered with snow	Covered with snow	Covered with snow	Covered with snow	21.10
TH-8	Covered with snow	49	Covered with snow	44.50	43.75
TH-9	Covered with snow	Covered with snow	Covered with snow	Covered with snow	Covered with snow
TH-10	Covered with snow	Covered with snow	Covered with snow	Covered with snow	Covered with snow
TH-11	Covered with snow	Locked	82.83	83.00	82.50
CSM-1	Covered with snow	45	44.50	45.00	44.17
CSM-2	Covered with snow	Plugged by ice at 9	Plugged by ice at 19	Plugged at 25.0	18.50
CSM-3	Covered with snow	Plugged by ice at 5	30.08	29.42	24.08
CSM-4	Covered with snow	Plugged by ice at 6	140.67	Dry at 141.5, TD	135.50
CSM-5	Covered with snow	117	111.83	111.21	105.00
CSM-6	Covered with snow	Dry at 57, TD	Dry at 52	Dry at 58.58	47.08

Drill Hole Water Level Summary

Feet Below Ground Surface

Date → Drill Hole	4-30-85	5-7-85	5-9-85	5-14-85	5-21-85
TH-1	11.42	11.50	12.00	13.25	14.82
TH-2	Buried by Avalanche	24.33	26.58	28.75	28.49
TH-3	54.75	53.75	53.92	55.25	56.33
TH-4	Dry at 78.25	Dry at 78.25	Dry at 78, TD	78.21	Dry at 78.52
TH-5	180.83	181.00	180.75	Blocked and Dry at 34.27	34.41
TH-6	94.17	60?	57.08	83.21	60.26
TH-7	143.00	143.33	143.25	140.92	142.06
TH-8	42.75	42.00	42.00	42.04	42.75
TH-9	Covered with snow	Covered with snow	Covered with snow	Covered with snow	Covered with snow
TH-10	Covered with snow	Covered with snow	29.13	30.17	30.32
TH-11	82.50	83.17	82.50	82.50	Not taken
CSM-1	42.58	41.75	41.92	43.00	42.54
CSM-2	6.00	14.67	15.58	18.08	19.00
CSM-3	22.92	23.67	23.67	25.79	26.83
CSM-4	Moisture at 135.67 TD-13600	Dry at 93.58	Dry at 93.50	93.25	93.19
CSM-5	101.42	98.58	100.42	100.25	102.29
CSM-6	47.50	46.50	48.00	49.54	50.42

Feet Below Ground Surface

Date → Drill Hole	5-27-85	6-7-85	6-13-85	6-30/85	7-22-85
TH-1	16.00	17.23	17.96	20.67	19.69
TH-2	28.78	28.90	29.21	29.92	31.33
TH-3	56.99	57.69	57.83	56.35	58.19
TH-4	78.80	Dry @78.75, TD	Dry @78.08, TD	not found	not found
TH-5	Mud only @185 TD	Dry @34.46, TD	Dry @34.40, TD	Dry @33.15, TD	Dry @34.25, TD
TH-6	50.51	48.92	48.31	49.61	52.75
TH-7	140.10	139.60	not taken due to shear condition in pipe	140.31	143.00
TH-8	43.06	43.10	42.98	43.40	42.67
TH-9	26.26	Dry @24.00, TD	Dry @12.42, TD	23.81	23.89
TH-10	31.23	31.27	31.29	31.39	31.58
TH-11	Not taken	82.65	82.75	82.73	83.79
CSM-1	42.92	43.85	42.98	43.38	43.67
CSM-2	19.52	Dry @19.42, TD	Dry @19.48, TD	Dry @19.44	Dry @19.43, TD
CSM-3	27.21	28.97	25.42	28.65	28.40
CSM-4	147.74	Dry @94.26, TD	Dry @93.21, TD	Dry @93.17, TD	Dry @93.13, TD
CSM-5	104.29	101.38	105.75	107.40	109.13
CSM-6	50.85	49.42	47.33	50.50	49.98

Drill Hole Water Level Summary

Date → Drill hole	7-31-85	9-3-85
TH-1	19.90	22.00
TH-2	31.60	32.70
TH-3	60.38	64.32
TH-4	Lost	Lost
TH-5	Dry @34.33, TD	Dry @34.35, TD
TH-6	49.88	49.57
TH-7	142.96	143.99
TH-8	43.67	44.08
TH-9	23.92	24.13
TH-10	31.46	31.56
CSH-1	43.65	44.00
CSH-2	Dry @19.42, TD	Dry @19.46, TD
CSH-3	28.02	29.21
CSH-4	Dry @93.10, TD	Dry @93.10, TD
CSH-5	108.65	109.03
CSH-6	50.58	49.76

1985 Precipitation Data (Inches)

Day	May	June	July	Aug.	Sept.
1	NR	0.12	0	0	0.02
2	NR	0	0.02	0.19	0.07
3	NR	T	0	0	
4	NR	0.17	0	0	
5	0.03	0	0	0	
6	0.02	T	0	0.06	
7	0.14	0	0	0	
8	0	0	0	0	
9	0	0	0	0	
10	0.06	0.33	0	0	
11	0.35	T	0	0.02	
12	0.10	0	0.01	0.27	
13	0.10	0	0.25	0	
14	0	0	0	0	
15	0	0	0	0	
16	0	0	0.03	0	
17	0.03	0	0.01	0	
18	0.05	0	0.07	0	
19	0.15	0	0.16	0	
20	0.18	0	0.34	0	
21	0	0	0.25	0	
22	0.12	0	0.45	0	
23	0.02	0	0.44	0	
24	NR	0	0.16	0	
25	NR	0.31	0.10	0	
26	NR	0.07	0	0	
27	NR	0.02	0.02	0.01	
28	0	0	0	0.03	
29	NR	0	0.09	T	
30	0	0	0.04	0	
31	T	-	0	0	

Note: Data taken from White River National Forest Ranger Station, 1.75 miles N.W. of the study area. Data are taken during the spring and summer months only, starting on May 5 this year. Total rainfall from May 5 to Sept. 2 was 5.48 inches.
NR = not recorded.
T = trace.