USDA FOREST SERVICE
RESEARCH PAPER INT-97
1971

INVESTIGATION OF SLOPE FAILURES IN THE IDAHO BATHOLITH

M. J. Gonsior and R. B. Gardner

INTERMOUNTAIN FOREST AND RANGE EXPERIMENT STATION
Ogden, Utah 84401

THE AUTHORS

Michael J. Gonsior is a Research Engineer for the Intermountain Forest and Range Experiment Station. He has been with the Station in Bozeman since 1965. He holds degrees of B.C.E. from Rensselaer Polytechnic Institute and M.S.C.E. from Montana State University, and has done additional graduate study in soil mechanics at the University of California at Berkeley.

Rulon B. Gardner is Engineer-in-Charge of the Forest Engineering Research Work Unit headquartered in Bozeman, Montana, at the Station's Forestry Sciences Laboratory, maintained in cooperation with Montana State University. For the past 18 years, he has been involved in the location, design, and construction of forest roads in the Rocky Mountain area. The past 8 years have been in research capacity.

USDA Forest Service
Research Paper INT-97
June 1971

INVESTIGATION OF SLOPE FAILURES IN THE IDAHO BATHOLITH

M. J. Gonsior and R. B. Gardner

INTERMOUNTAIN FOREST AND RANGE EXPERIMENT STATION
Forest Service
U.S. Department of Agriculture
Ogden, Utah 84401
Joseph F. Pechanec, Director

CONTENTS

ABSTRACT

Precipitation events in the winter and spring of 1965 caused significant erosion and numerous landslides in many parts of the Idaho Batholith. Most slope failures were associated with roads. An investigation of several representative failures in the Zena Creek sale area on the Payette National Forest was conducted. Details of the field and laboratory tests are given, and three examples of the stability analyses are presented. Causes of the failures are discussed and recommendations for future construction in similar terrain are made.

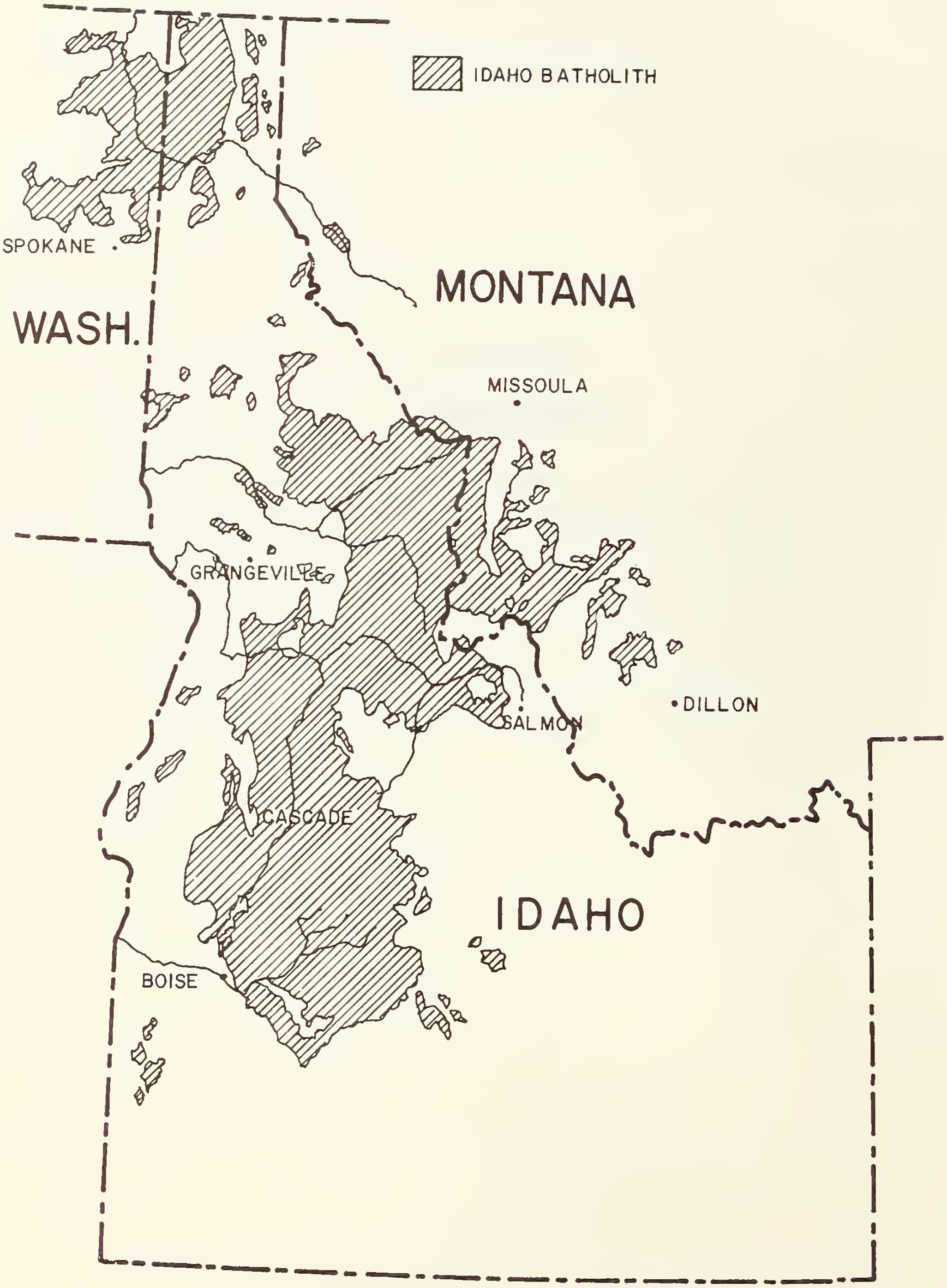


Figure 1.--The Idaho Batholith.

Introduction

As the uses of forest lands increase, the necessity to develop less favorable terrain also increases. It also becomes more difficult and complicated to provide access that is both economical and safe from contributing to watershed damage.

Because of the increased and justified emphasis upon environmental degradation, proper use of the many millions of acres of forest watersheds requires careful planning. The skills of many disciplines and the combined knowledge that covers all aspects of this complicated ecosystem--the forest--are necessary to assure proper use of forest lands. The silviculturist, ecologist, hydrologist, engineer, and others, must all function as a team to develop and carry out plans for management of forest lands.

The Idaho Batholith (fig. 1) is one of the most critical areas as to surface and subsurface stability (i.e., erosion and landslides) in the Intermountain West; also, this area provides prime spawning and rearing grounds for the Pacific salmon, good timber-producing land and outstanding recreation opportunities. Two major spawning rivers are the South Fork of the Salmon and the Selway Rivers.

The Batholith contains some of the most rugged and pristine scenery in America and includes such well-known areas as the Selway-Bitterroot Wilderness Area, Salmon River Breaks Primitive Area, and the Magruder Corridor. In such areas, those who are charged with the responsibility of multiple use land management face their most serious and complicated challenges. Within the scope of present knowledge and technological skill, proper planning can assure many uses for much of this vast land area. However, much of the Batholith must wait for the results of research to provide additional management guidelines, and it may have to be managed for less than total resource potential if adequate environmental safeguards are to be applied.

Considerable attention is currently being directed to the Batholith area. It contains valuable timber reserves as well as other resources which are important to the regional and national economies. However, many of its steep slopes are highly erodible and susceptible to landslides. More specifically, slopes are often greater than 60 percent and are commonly composed of highly fractured and weathered granite overlain by thin layers of loose, cohesionless soils. Developmental activities, especially roadbuilding, increase the occurrence of landslides and provide large exposed areas from which sediment can readily be removed by normal erosional processes. Initial increases in production of sediment have been measured and in some instances were more than a thousandfold greater than natural rates from undisturbed terrain.

The most serious consequence of this accelerated sedimentation is believed to be destruction of aquatic habitat, especially of the spawning and rearing areas of anadromous fish (fig. 2). In addition, the silting of downstream reservoirs is an important consideration, as is the visual impact of exposed cutbanks, fill slopes, debris piles, and slide scars.

This paper describes an investigation of slope failures in one small area of the Batholith and the conclusions and recommendations derived therefrom.

Figure 2.--Krassell Hole, Secesh River.

Road Location and Design Concepts

Traditionally, the concepts of road location and design have stressed the economics of transportation from origin to destination. This has encompassed not only the cost of the facility (fixed cost, depreciation, and maintenance) but also the cost to the user (principally concerned with speed of travel). More sophisticated analyses are evolving as more of the impacts of location and design are considered. However, this evolution has not generally developed on forest roads. Methods of analysis that could be helpful in evaluating effects of all major factors are available, but usually the inputs are not. For example, we do not have accurate values, nor in most cases, any value to assign to the influence that erosion may have on the functioning of the total system (ecosystem). Instinctively, from our ocular estimates, and in some instances from quantitative measurements of damage from erosion, we know that we must improve our present practices. Fortunately (or unfortunately, when viewed from its lack of use in the past), we are close to having sufficient technical knowledge to accomplish this objective.

To help illustrate what can be done and how it is done in areas such as the Idaho Batholith, information from a recent research study of logging roads in the Zena Creek drainage (tributary to the Secesh and South Fork Salmon Rivers) on the Payette National Forest will be used (figs. 3a and 3b).

Zena Creek Experimental Logging Study

In 1957, Region 4 (Intermountain) of the USDA Forest Service began a study of logging methods that could be used for harvesting timber on some of the steep slopes of the Batholith. The basic objective of the study was to determine if logging could be carried out economically without unacceptable damage to the watersheds. Tractors, jammers, and portable towers (skyline systems) were used for skidding timber during the study. In general, it was concluded that logging the steeper slopes with skyline systems caused little damage to the watershed and that the cost of logging, although high, still wouldn't prevent a logging operator from making an acceptable profit.

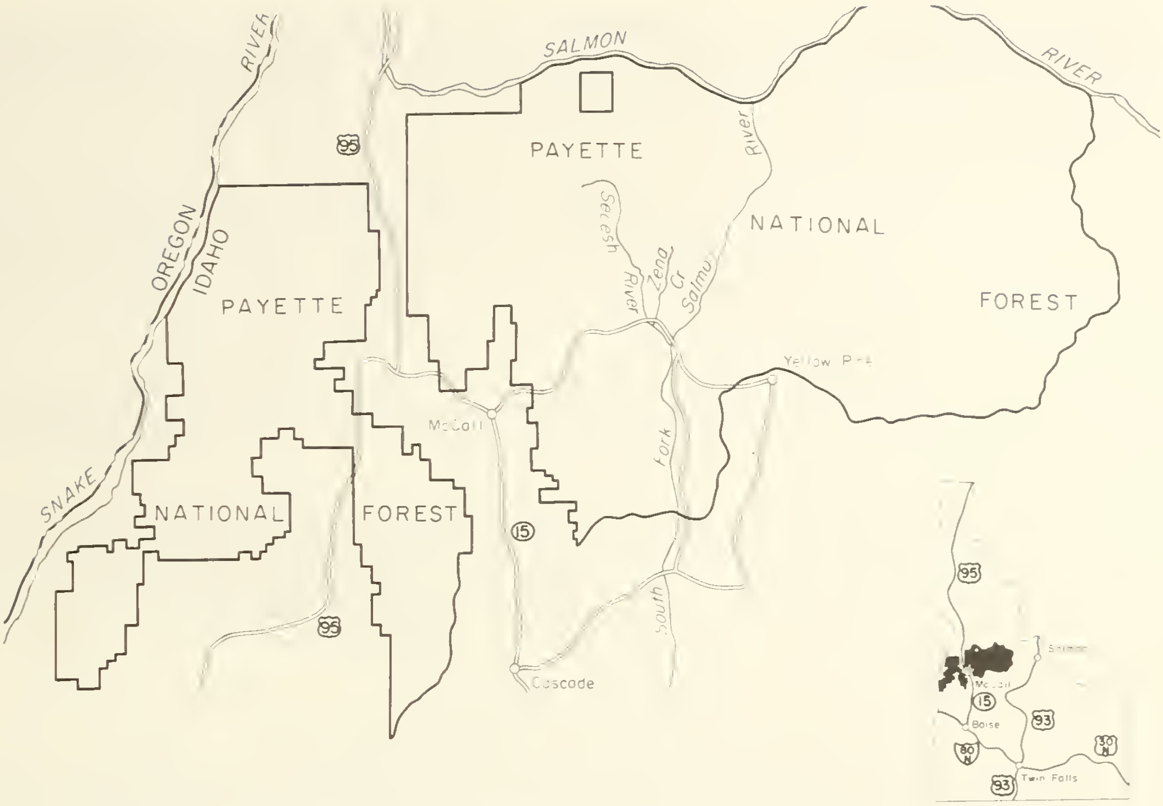


Figure 3a.--Location of Zena Creek in the Payette National Forest.

Figure 3b.--Oblique photograph of Zena Creek terrain illustrating its rugged and steep topography.

In the December-January 1964-65 period and in April of 1965, climatic events occurred that resulted in extensive damage to the road system. The probable frequency of these events is not well established. However, it can be concluded from past weather records that such events could happen as often as once in five years.

During late December and early January of 1964-65, over 6 inches of precipitation (mostly snow) fell in the Zena Creek area. Some road failures occurred below 5,000 feet as a result of this storm. Then, on April 19, 1965, 1.03 inches of rain fell. In the areas below about 4,200-foot elevation, where snowmelt had already been completed, little damage occurred. In areas where elevations were between 4,200 and 5,500 feet and the snowmelt was intense, extensive damage occurred, primarily to road fills.

The major landslides and erosion on the watersheds in the Zena Creek drainage were judged to have resulted from road construction. The balance of this report will deal primarily with the road systems that were constructed to transport timber from the Zena Creek drainage.

In figure 4, a planimetric map of the Zena Creek drainage shows the road system used to gain access to the harvest area. The arrows and circled points mark locations of sites that were investigated as part of the Zena Creek study. The Secesh River is a tributary to the South Fork of the Salmon River, as previously noted.

Failure Investigations

The major purpose of these investigations was to determine the principal causes of slope failures, both road-associated and nonroad-associated, in the Zena Creek study area. Surface erosion, or the movement of single grains by wind and water, was excluded from consideration except where this type of movement contributed to mass failures. Whether surface erosion by runoff is an important mode of soil detachment is a matter for study. Nevertheless, the importance of surface water as a transporting medium is well recognized, and movement of soil particles in roadside ditches and ephemeral channels takes place readily at Zena Creek.

Raveling from weathered rock surfaces in cut slopes was also excluded from this investigation. This, too, is recognized as an important contributor of soil grains to surface water in roadside ditches.

The analysis of surface erosion remains a highly empirical art, and is beyond the scope of this study. Only the movement of soil *en masse* was considered.

FIELD AND LABORATORY TESTS

A detailed discussion of the field and laboratory tests and the results thereof may be found in the original report.[1] The procedures followed were relatively routine for an engineering investigation of this type, and only the more significant findings will be discussed herein.

---

[1]R. B. Gardner, M. J. Gonsior, and G. L. Martin. Zena Creek road and logging system investigation. September 1969, 173 p., illus. (Unpublished report on file at USDA Forest Serv., Intermountain Forest and Range Exp. Station, Forestry Sciences Lab., Bozeman, Montana.)

Figure 4.--Planimetric map, Zena Creek Roads.

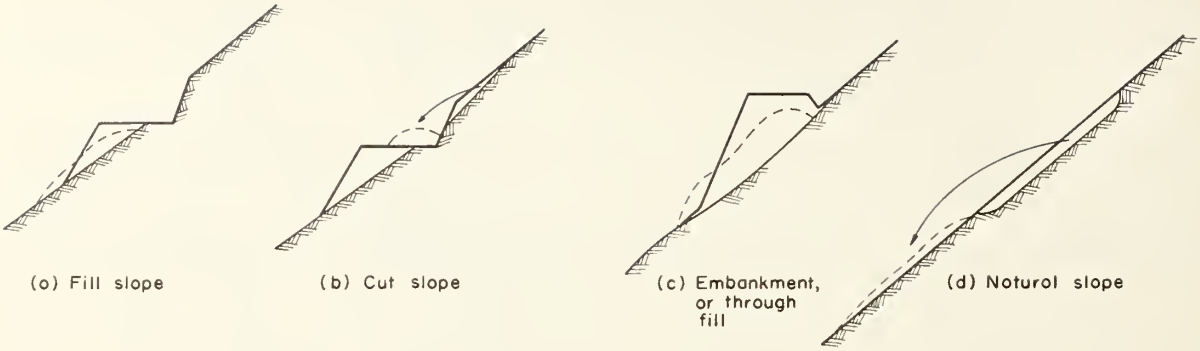


Figure 5.--Failure types.

A preliminary reconnaissance of the Zena Creek study area was made in May of 1967 to identify the extent and types of failures that occurred during the previously discussed 1965 storm event. As a convenience for analysis and discussion, four general failure types were identified, as shown in figure 5.

Based upon the preliminary reconnaissance, eight sites representing the three failure types shown in figure 5(a), (b), and (d) were selected for intensive study (i.e., fill-, cut-, and natural-slope failures, respectively). None of the embankment failures (c) were deemed amenable to analysis, but one stable embankment was selected for sampling and analysis. In addition, four other sampling sites were chosen for analysis of material properties representative of alluvial and bank slough deposits. All 13 sites are identified and briefly described in figure 4 and table 1.

During the months of July and August of 1967, sampling, field testing, and mapping were conducted at the selected sites.

Seismic subsurface exploration was conducted at each of the failure sites and also at the site of the stable embankment. In addition, some verification work was performed by trenching with a tractor-mounted backhoe. Representative, disturbed samples were obtained by manual excavation, some with the aid of the backhoe. Depths, sizes, and other descriptions of these samples are given in table 2. In-place density tests were performed with a balloon volumeter; these results are also shown in table 2. Finally, the geometry at each of the sites was mapped and photographs were taken.

Laboratory tests were performed to determine grain-size distribution, liquid limits, and plastic limits for classification purposes. Results of these tests are given in table 3. For the material passing the 3/8" sieve, the gradations of all samples, excluding samples which were obtained from alluvial or slough deposits, were within the range shown by dashed lines in figure 6. Further, the extremes represented by the dashed lines represent two samples obtained only 20 feet apart in natural soil. The gradations of all other samples, representing both fills and natural deposits and representing a wide range of other conditions, were within the range shown by the solid lines in figure 6. Because these materials are easily broken down, and thus the sieve analyses are subject to error simply from variations in abrasive action during preparation and testing, the variabilities in grain-size distributions for samples passing the 3/8" sieve were judged negligible.

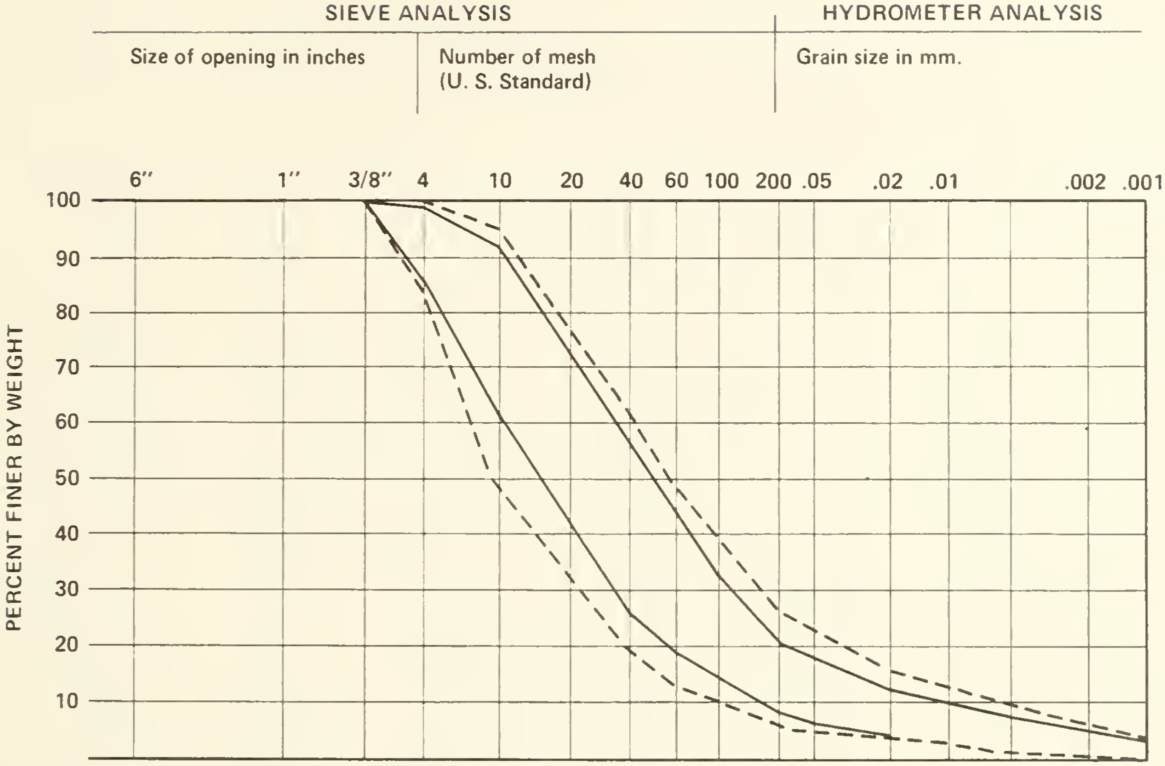


Figure 6.--Gradation curves,-3/8" materials.

Table 1.--*Site descriptions*

Site	Failure type and/or description
917-1	Embankment section in small gully, not a failure
917-2	Cut-bank slough deposit in roadside ditch
917-3	Incipient fill-slope failure
917-4	Cut-slope failure
917-5	Alluvial deposit in streambed
917G-1	Combination fill- and cut-slope failure
917H-1	Fill-slope failure and incipient fill-slope failure
917H-2	Fill-slope failure
917H-3	Cut-slope failure, may also be considered natural-slope failure
977-1	Natural-slope failure
977-2	Fill-slope failure
977-3	Cut-bank slough deposit, saturated by natural spring
977-4	Alluvial deposit on road surface

Table 2.--*Sample descriptions and field test data*

Sample no.	Sample weight (lb.)	Depth (ft.)	In-place γ_t (p.c.f.)	w (%)	e***	**** Relative density (%)
917-1-1†	30	1.0	102.4	2.3	0.65	79
917-1-2	31	1.0	102.2	4.4	.69	77
917-1-3	32	2.5	98.3	6.5	.79	73
917-1-4	54	2.5^F	126.8	6.1	.38	94
917-1-5	41	2.0^F	113.1	6.4	.56	83
917-2-1	108	1.0	87.4	4.7	.98	66
917-3-1	106	3.5^F	106.3	8.4	.69	77
917-3-2	47	2.5	95.5	3.6	.79	73
917-3-3	47	1.0^F	105.4	2.7	.61	81
917-4-1	29	2.0	107.2	8.1	.67	78
917-5-1	105	1.5	105.8	8.4	.69	77
917G-1-1	30	2.0	98.8	3.6	.73	75
917G-1-2	41	1.5^F	108.0	1.3	.55	84
917G-1-3	29	$.5^F$	104.8	.6	.59	82
917H-1-1	36	1.0	96.2	2.1	.76	74
917H-1-2	102	1.5^F	99.1	5.5	.76	74
917H-1-3	41	1.0^F	114.4	2.5	.48	88
917H-2-1	30	2.5	117.3	6.5	.50	87
917H-2-2	30	6.0	142.5	5.2	.22	107
917H-2-3	28	2.5	**	**	**	**
917H-2-4	30	1.0^F	104.6	2.4	.62	80
917H-3-1	35	1.5	92.1	2.3	.84	71
917H-3-2	50	1.5	90.0	2.5	.88	69
977-1-1	42	2.5	103.0	4.8	.68	76
977-1-2	28	2.0	128.1	3.2	.33	98
977-2-1	48	6.0	**	**	**	**
977-2-2	42	2.5	113.1	3.5	.51	86
977-2-3	41	1.0^F	96.1	1.7	.75	74
977-2-4	70	1.0^F	95.1	5.8	.84	71
977-3-1	108	.5	102.5	14.2	.84	71
977-3-2	*	.5	130.1	21.1	.54	85
977-4-1	100	.5	102.8	1.7	.64	79

F Indicates fill material.

* Same site as 977-3-1. Increased density and moisture content reflect changes caused by hand-tamping

** Fractured bedrock--too coarse and hard for in-place density and moisture content tests.

*** Void ratio, $e = G_s\,(1+w)\,\frac{\gamma_w}{\gamma_t}$, where G_s = specific gravity of soil particles, γ_w = unit weight of water (62.4 p.c.f.).

**** Relative density, based upon figure 5, $= \frac{\gamma_d}{127 \text{ p.c.f.}} \times 100\%$, where

γ_d = dry unit weight $= \frac{\gamma_t}{(1+w)}$.

γ_t = unit weight of soil-water system.

w = water content relative to dry unit weight.

† Last number identifies sample, first and second numbers identify failure site (e.g., sample no. 917-1-3 indicates sample no. 3 recovered at site 917-1).

Table 3.--*Grain-size distributions and unified and AASHO soil classifications*

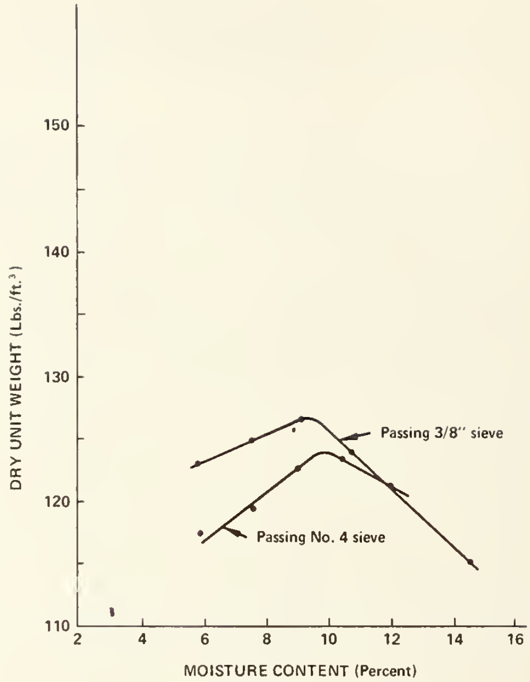
Sample	Cumulative percent passing sieve sizes											Unified	AASHO
	3/8"	#4	#10	#40	#60	#200	.05mm.	.02mm.	.01mm.	.005mm.	.001mm.		
917-1-1	97	91	69	33	25	12.6	11.1	7.6	5.8	4.2	1.8	SW-SM	A-1-b
917-1-2	95	87	63	30	23	12.4	7.7	6.3	5.3	3.1	2.0	SW-SM	A-1-b
917-1-3	95	90	68	32	25	12.1	10.0	8.0	6.1	5.0	2.5	SW-SM	A-1-b
917-1-4	71	64	48	22	16	6.4	5.6	3.5	2.9	2.2	.2	SW-SM	A-1-a
917-1-5	86	78	61	29	21	8.8	7.3	4.5	3.6	2.7	.8	SW-SM	A-1-b
917-2-1	94	88	72	34	23	11.1	10.4	6.4	5.2	3.7	1.9	SW-SM	A-1-b
917-3-1	90	79	58	31	24	14.4	13.1	9.5	8.6	6.5	3.0	SM	A-1-b
917-3-2	95	84	58	32	27	17.6	15.4	11.4	7.9	4.9	3.0	SM	A-1-b
917-3-3	85	76	56	28	22	12.1	9.4	7.4	5.0	3.3	1.5	SW-SM	A-1-b
917-4-1	97	86	66	33	25	12.8	11.6	8.3	7.3	4.7	1.5	SW-SM	A-1-b
917-5-1	94	90	75	28	15	2.9	2.4	1.0	.7	.0	.0	SP	A-1-b
917G-1-1	94	90	74	39	31	15.4	11.7	7.8	6.4	3.8	.9	SM	A-1-b
917G-1-2	83	79	66	35	26	11.7	9.8	4.8	3.8	1.9	.5	SW-SM	A-1-b
917G-1-3	79	72	54	23	16	6.5	6.1	3.9	2.8	1.9	.0	SW-SM	A-1-b
917H-1-1	93	83	59	23	17	7.9	7.0	4.5	3.8	2.2	.7	SW-SM	A-1-b
917H-1-2	89	80	58	24	16	6.7	5.8	4.5	3.9	2.3	1.2	SW-SM	A-1-b
917H-1-3	84	76	57	27	19	9.2	7.4	5.0	4.3	3.2	..7	SW-SM	A-1-b
917H-2-1	100	100	95	61	48	26.0	22.2	15.5	13.1	9.5	3.8	SM	A-2-4
917H-2-2	100	99	92	54	41	20.4	17.4	11.6	9.2	4.8	1.3	SM	A-2-4
917H-2-3	100	84	49	19	13	6.0	5.0	3.8	3.4	2.5	1.1	SW-SM	A-1-a
917H-2-4	85	78	64	31	23	9.7	8.0	4.1	3.6	2.0	.7	SW-SM	A-1-b
917H-3-1	91	86	70	37	28	13.7	11.1	8.9	6.8	4.6	2.0	SM	A-1-b
917H-3-2	72	66	50	26	20	10.7	8.9	5.0	4.0	2.2	.6	SW-SM	A-1-a
977-1-1	99	95	78	40	29	14.4	12.3	9.2	6.4	4.5	2.1	SM	A-1-b
977-1-2	91	82	62	30	22	10.3	6.9	5.3	2.8	1.0	.4	SW-SM	A-1-b
977-2-1	37	31	23	10	7	3.2	2.2	1.8	1.4	.8	.3	GW	A-1-a
977-2-2	96	91	75	43	34	17.3	12.1	8.7	5.1	3.6	1.0	SM	A-1-b
977-2-3	95	90	75	35	25	10.6	7.8	4.6	2.6	1.3	.0	SW-SM	A-1-b
977-2-4	76	69	54	24	18	7.9	6.1	4.2	4.0	2.4	.7	SW-SM	A-1-b
977-3-1	95	90	71	38	29	15.0	9.7	9.1	4.9	3.5	1.1	SM	A-1-b
977-3-2	95	90	71	38	29	15.0	9.7	9.1	4.9	3.5	1.1	SM	A-1-b
977-4-1	100	97	80	26	14	1.8	1.1	.5	.5	.5	.0	SW	A-1-b

Because of the near identity of all the samples, and because all further testing was to be conducted on only the finer portions of the samples, the remainder of the testing program was confined to determination of the pertinent engineering properties of a single sample from site 917-3. Included were tests to determine moisture-density relationships, shear strength, and permeability.

Results of the moisture density tests, conducted according to AASHO Test Designation T99-57, Method A, except for variations in maximum grain size, are shown in figure 7. Note that for material passing the 3/8" sieve, a maximum dry unit weight of approximately 127 pounds per cubic foot (p.c.f.) was obtained, while for the fraction finer than a #4 sieve, a maximum dry unit weight of approximately 124 p.c.f. was achieved.

Numerous direct shear and triaxial compression tests were conducted to investigate the effects of unit weight and moisture content upon the shear strength of these materials. However, strengths based upon effective stress parameters showed negligible variation over a wide range of conditions. Typical results are shown in figure 8, which represent the Mohr stress circles at failure for consolidated, undrained triaxial tests on saturated specimens wherein pore pressures were measured. These tests were conducted at an axial strain rate of 0.02 inch per minute, and failure was defined as the maximum ratio of major to minor principal effective stress. All tests were conducted on

Figure 7.--Moisture-density test results.



materials passing a #16 sieve, and all specimens were approximately 5 inches in height and 2-1/2 inches in diameter before testing.

Finally, falling head permeameters were used to determine the relationships between void ratio and permeability. Typical results of these tests are shown in figure 9. Unfortunately, tests could not be satisfactorily conducted at void ratios as high as are commonly found in the field, due to channel formation along the smooth walls of the permeameters. Nevertheless, the results indicated that these materials are capable of being relatively impermeable when disintegrated and compacted.

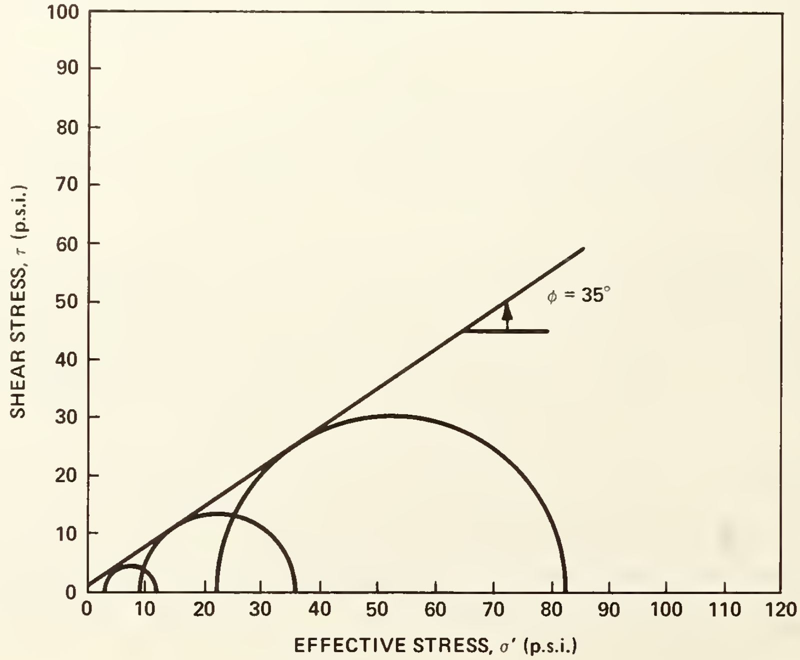


Figure 8.--Mohr stress circles and failure envelope from consolidated, undrained triaxial compression tests.

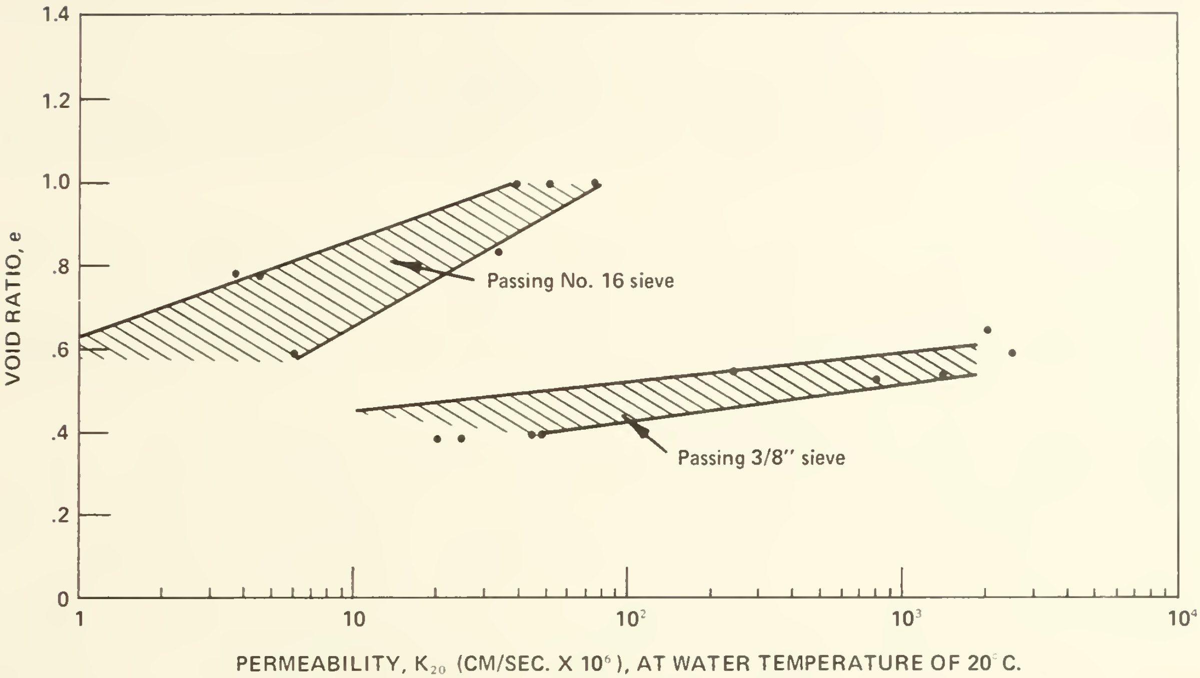


Figure 9.--Permeability test results.

DISCUSSION OF TEST RESULTS

In any investigation of this type, the major objective of the laboratory testing program should be to simulate the actual field conditions as closely as possible. Insofar as available equipment and time permitted, this was done. However, as has been previously indicated, a period of over two years had elapsed between the rain-on-snow event that caused failures at Zena Creek and the beginning of investigation. Erosion and growth of new foliage had significantly altered both the geometry and soil conditions at the failure sites in the interim, and in most cases hardly any traces of the original material remained at or near the sites. Furthermore, fabrication of samples of this material in the laboratory at densities as low as are found in the study area was an extremely difficult task and often met with failure. Thus, in regard to the tests and their results, considerably more work must be done to fully describe these materials and to establish adequate criteria for the design of stable roads and safe harvesting practices in the Zena Creek and similar areas.

For design purposes in this area, it appears that reasonable values are 35° and 0 for the angle of internal friction (ϕ) and cohesive strength (c), respectively. These values are based upon the results of the triaxial compression tests, some of which are shown in figure 8. Although the strength envelopes indicated slight cohesive strength in most of the tests on saturated specimens, it is doubtful that any permanent cohesion can be assumed for long-term design purposes. Further, in most cases, it is believed that some of the apparent cohesion measured in the triaxial tests on saturated specimens could be accounted for by corrections for membrane resistance; no corrections were made for this factor.

The value of 35° for the friction angle is undoubtedly conservative, but not excessively so. For analyses of the type conducted here, the plane strain test is more suitable for determining strength parameters and for simulating conditions along the failure plane, but the apparatus necessary for conducting such tests was not available. It is known, however, that the friction angle measured in plane strain tests will be appreciably greater than that determined from triaxial tests on cohesionless materials. Generally, the difference in values depends upon density, and will be found to decrease with decreasing density. Therefore, it can be concluded that 35° is a reasonable lower limit for the value of the friction angle for the materials investigated in this study. (As will be noted from subsequent discussions of the stability analyses, it appears that a value of 38° for the friction angle more nearly complies with the conditions for most of the failures.)

A complete series of tests that would give an accurate determination of the critical void ratio-confining pressure relationship was not conducted; however, volume change measurements from both direct shear and triaxial tests indicated that, for the relatively shallow depths to the failure planes in most cases, the critical void ratio is in the range of from 0.55 to 0.65. The significance of this is discussed more fully in the conclusions of the failure investigations.

Permeabilities in the range from 10^{-6} to 10^{-3} cm./sec. were measured in tests wherein sample size, maximum grain size, and void ratio (density) were varied. Few of these tests adequately simulated the loose conditions and grain-size distributions actually existing in the field, and thus the measured permeabilities are probably lower limits for the actual field conditions. Nevertheless, these materials are capable of good water tightness and due to compaction and rapid disintegration they are undoubtedly continually decreasing in permeability both in road fills and in the natural state. This presumed reduction in permeability over a period of time is important in several ways. First, infiltration capacity is reduced, thus tending to increase overland flow and scour, or erosion; in the absence of vegetation, the damage is many times greater. Second, culverts and other facilities designed to handle surface water based upon reasonably permeable fills at the time of construction may become inadequate in later years as the quantities of surface water increase. Third, for a given quantity of subsurface flow, one or both of two factors must increase in response to a loss in permeability. Either the cross-sectional area of flow must increase, hence the phreatic surface must rise closer to the ground surface, or else the hydraulic gradient must increase, thereby creating higher seepage forces. A further danger from an increase in hydraulic gradient is that of piping, or the washing away of soil at a point of emergence. When this occurs, for example at the toe of a fill slope, the piping action causes removal of soil which is necessary for support of the remaining material upslope, thus leading to possible mass failure.

In connection with the prognosis of this investigation, a comparison of the in-place densities (table 2) and the results of the moisture-density test results (fig. 7) reveal the significant disparity between measured field conditions and the attainable conditions.

STABILITY ANALYSIS

Neither the field and laboratory procedures nor the types of analyses performed in this study were very complex or unique. The ordinary method of slices, assuming circular failure surfaces, was the predominant procedure followed in most of the analyses. A variation of this procedure was programmed for a small digital computer and used for a more thorough search for the critical failure arc for some of the fill slopes; slices of differential thickness were assumed in order to derive a closed-form solution for the safety factor. The infinite slope stability theory, as developed by Taylor,[2] was used to analyze the natural slope failure, and was also applied to some of the shallow

---

[2]Donald W. Taylor. Fundamentals of soil mechanics. New York: John Wiley & Sons, Inc., 700 p., illus. 1948.

cut-and-fill-slope failures. In addition, a sliding block or wedge analysis was used to analyze some of the cut- and fill-slope failures wherein the failure surfaces could reasonably be considered planar.

To satisfy requirements of completeness and brevity, only three stability analyses are presented and discussed in this paper. Those included were selected primarily for their illustrative value. Complete descriptions of all the sites and analyses may be found in the original report of this investigation.[1]

The reliability of any analysis of the types conducted herein depends upon five fundamental factors:

1. Accurate description of the geometry (i.e., topography and cross section);

2. accurate knowledge of the soil properties (i.e., shear strength and unit weight);

3. correct definition of external loads, if any (e.g., vehicle or structural foundation loads);

4. correct description of ground water or seepage conditions (i.e., pore water pressures); and,

5. correct method of analysis.

As was mentioned previously, there had been significant alterations of the topography during the period between time of failure and time of investigation. Undoubtedly, subsurface conditions had also undergone some changes during this interim. Nevertheless, reasonable confidence is placed in the accuracy of the topography and subsurface measurements.

The tests for soil properties in the Zena Creek study area have previously been described, and their results discussed. Admittedly, there are questions about the accuracy of the measured soil properties. The reader is cautioned not to extrapolate, or place excess confidence in these results for purposes of decisionmaking in other regions of the Idaho Batholith. Nor should any of these results be considered absolutely correct within the Zena Creek study area. Often in studies of this type, there is a strong tendency to question the method of analysis when discrepancies between the "prototype" and "model" are discovered. However, it must always be remembered that the assumed strength properties have been determined under laboratory conditions which never perfectly duplicate the actual field conditions. Further, it must be realized that the field conditions are continually changing due to geologic and climatic variations, and thus the soil properties are also continually changing. Therefore, it will be noted that the analyses have been conducted to account for variations in the soil properties.

External loads were not considered in the analyses because of poor definition. However, it is believed that the "rain on snow" so often referred to by many observers constituted a significant load on the road fills and denuded slopes. Most references to this condition emphasize the soil saturation aspects and neglect the importance of its weight.

One other factor, which is discussed briefly in the conclusions of the analyses, is the effect of sonic booms. There is a strong suspicion that these shock sources might also have played an important role as external loads leading to failure.

The description of seepage conditions in this region is undoubtedly the most questionable and complex factor detracting from the reliability of the analyses. There is no doubt that better and more rigorous assumptions regarding the subsurface flow might have been made, or that more exact and detailed field tests might have been conducted to improve the reliability of the analyses. However, neither time nor funds would permit these desirable refinements, nor were such refinements necessary to demonstrate the probability and predictability of the occurrences of failure in the Zena Creek study area. The matter of ground water flow characteristics in the Idaho Batholith is deserving of perhaps the greatest immediate research endeavor if development of this region is to be continued and accelerated.

Finally, the reliability or exactness depends upon the correctness of the analysis procedures. All of the techniques used in this investigation are widely used and accepted. However, as mentioned in connection with soil properties and seepage conditions, many assumptions and idealizations are made which, to greater or lesser degrees, depart from the actual field behavior. It is important to know whether these assumptions are made for convenience (ease of computation) or out of ignorance; but this knowledge is not nearly so important as the fact that these assumptions are made and that, therefore, another source of error must be recognized. It is beyond the scope of this report to elaborate upon this matter except to note that, in general, the safety factors obtained by the ordinary method of slices are conservative. That is, for design purposes, this method usually will yield a lower computed safety factor than will most other more refined methods. More important, the actual factor of safety will usually be higher than the computed factor. Therefore, in analyses of the type performed in this study, a computed safety factor less than 1.0 may, or may not, mean that failure actually would occur.

It must also be understood that the methods used in these analyses are based upon the assumption that the soil mass behaves, or moves, as one or more discrete, solid units. In fact, most soils behave plastically. Further, it is strongly suspected that the dominant mode of failure in most of the cases discussed was liquefaction, wherein the soil mass behaves neither elastically nor plastically, but rather as a heavy fluid. No analytic techniques are yet available for completely describing this phenomenon, although the conditions leading to liquefaction are reasonably well understood and avoidable.

Before proceeding with the analyses, it is appropriate to briefly discuss the typical subsurface conditions found in the study area and, in particular, the assumptions regarding ground water conditions. A typical subsurface profile in the natural, undisturbed state, is shown in figure 10.

Referring to figure 10, the major portion of the flow that affects the stability of slopes occurs above the competent rock, although this deeper bedrock is also weathered and fractured and undoubtedly conducts considerable flow. What is referred to as the weathered zone is actually a transition zone composed of highly fractured, weathered material near its top and increasing in content, angularity, and size of stones, and also in unit weight, from the top to the bottom of the layer. The topsoil is almost without stones. Under certain climatic conditions and during certain times of the year, it is probable that all of the subsurface materials are saturated, with the top flow line coincident with the ground surface. This seepage condition would be the most critical in, and was assumed in the analysis of, natural slopes.

When the natural slope is interrupted by a road prism, the subsurface profile appears typically as shown in figure 11. In this situation, part or all of the relevant flow is interrupted. Above the cut slope, under the most critical conditions, the top flow line is distorted as shown in figure 11. This distortion, or drawdown, is not so critical as would be the case if the top flow line could be maintained coincident with the surface of the slope. Hence, in analyses of the cut slopes, a conservative assumption that no drawdown occurs was used.

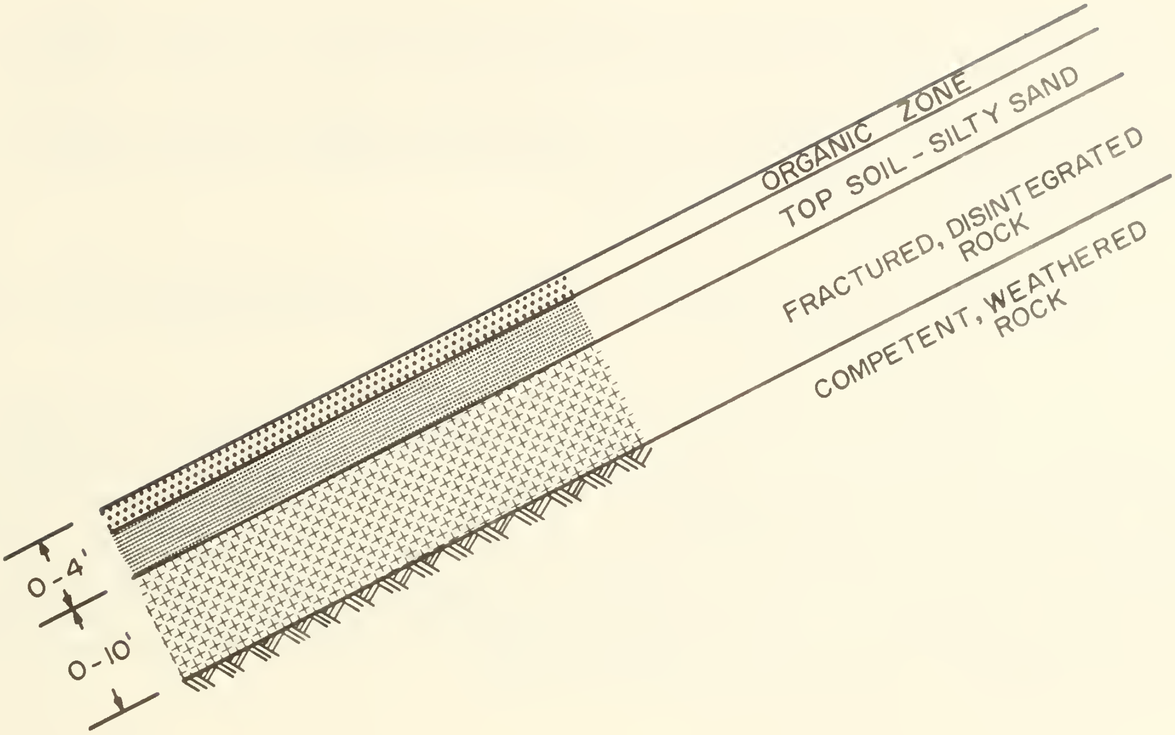


Figure 10.--Typical soil profile.

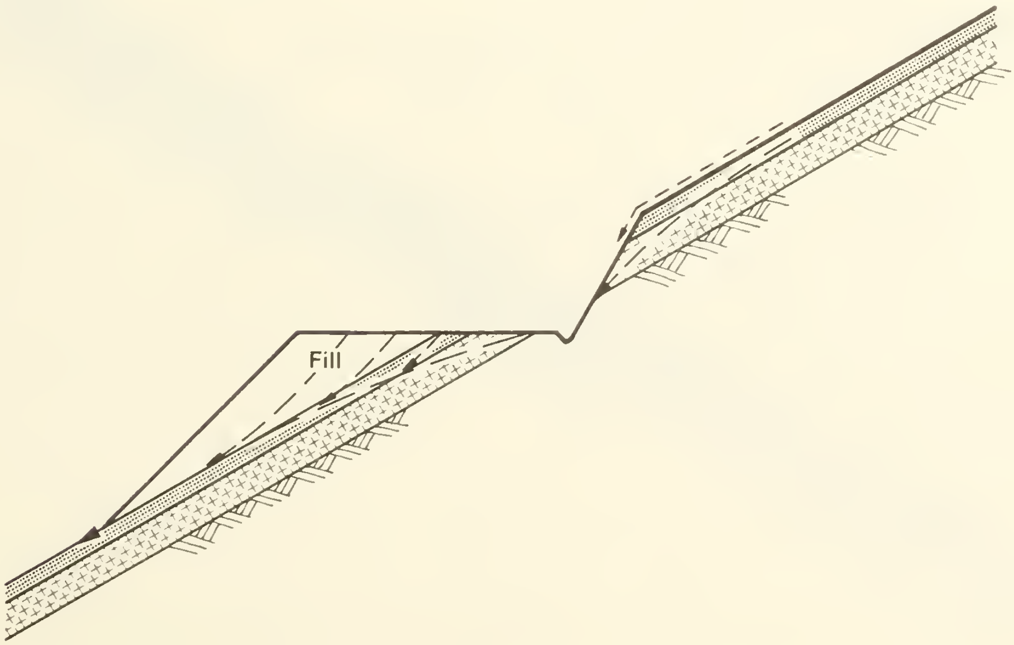


Figure 11.--Post construction profile and flow conditions.

Of course, overland flow may be occurring simultaneously, as shown in figure 11, some of which is caused to seep vertically downward to reach the phreatic surface.

Below the road surface, the top flow line probably appears as shown in the fill section of figure 11. Represented is an equilibrium situation, wherein the ditch is full of water. Water on the road surface and fill slope tends to seep vertically downward to reach the flowing ground water.

Situations can arise, however, where water accumulates in the road surface. In such cases, active seepage throughout the section, with flow roughly parallel to the fill slope, is possible. It is this extreme situation which was assumed in the analysis of fill-slope failures.

Where an embankment or through fill crosses a draw, the situation can be represented as shown in figure 12. The culvert installation may interrupt part of the subsurface flow, thus causing some drawdown of the top flow line above the embankment. Below the culvert inlet, subsurface flow continues, and at some location downslope may again coincide with the ground surface. It is possible for the top flow line to rise above the original ground surface and into the fill material immediately downslope from the culvert inlet. Thus, a conservative assumption in such a case would be one in which the embankment was considered as an earth dam; furthermore, this "dam" would have a free water surface upslope at the elevation of the culvert entrance, and tailwater elevation at the toe of the downstream slope.

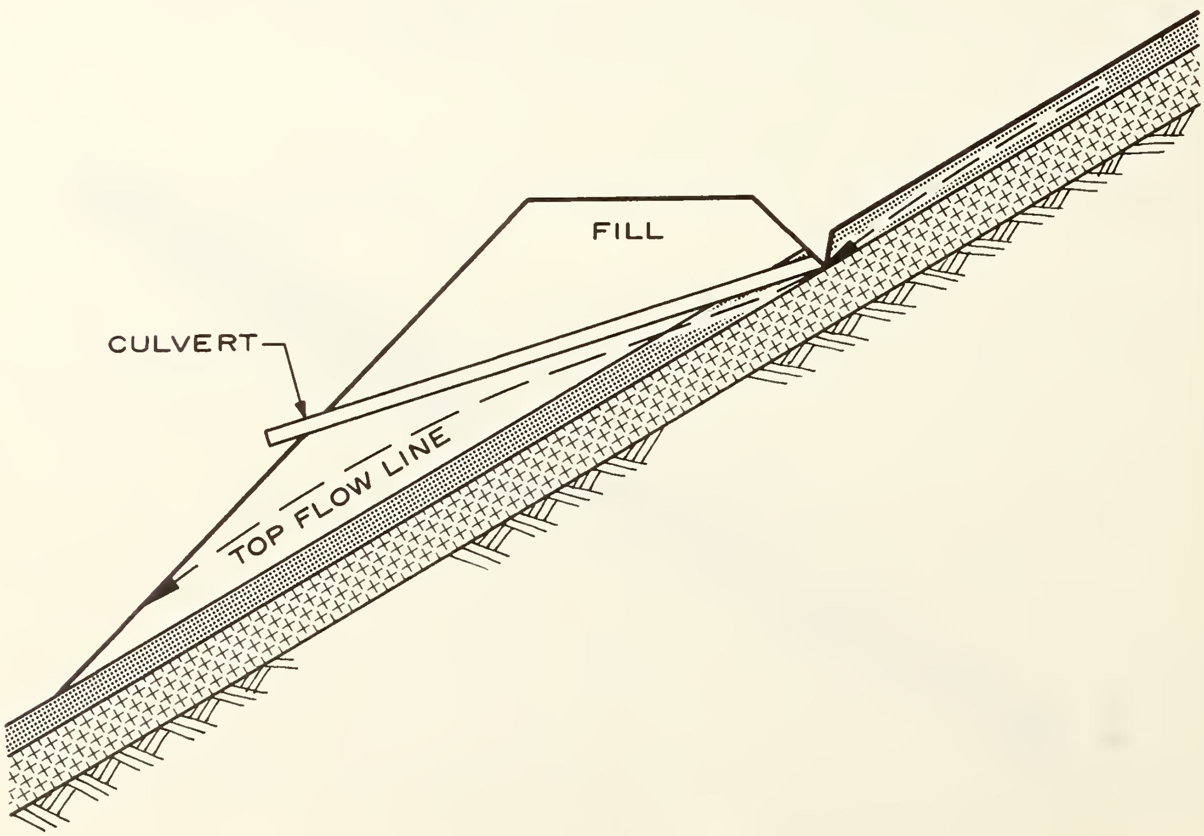


Figure 12.--Subsurface flow conditions for through fills.

GONSIOR M. J. and GARDNER R. B.

1971. Investigation of slope failures in the Idaho Batholith, USDA Forest Serv. Res. Pap. INT-97, 34 p., illus.

An investigation of slope failures in the Zena Creek sale area on the Payette National Forest is described. Conventional methods of field and laboratory testing were applied to describe soil properties. Stability analyses were conducted to demonstrate the predictability of failures. Recommendations for future construction in similar terrain are made.

GONSIOR M. J. and GARDNER R. B.

1971. Investigation of slope failures in the Idaho Batholith, USDA Forest Serv. Res. Pap. INT-97, 34 p., illus.

An investigation of slope failures in the Zena Creek sale area on the Payette National Forest is described. Conventional methods of field and laboratory testing were applied to describe soil properties. Stability analyses were conducted to demonstrate the predictability of failures. Recommendations for future construction in similar terrain are made.

GONSIOR M. J. and GARDNER R. B.

1971. Investigation of slope failures in the Idaho Batholith, USDA Forest Serv. Res. Pap. INT-97, 34 p., illus.

An investigation of slope failures in the Zena Creek sale area on the Payette National Forest is described. Conventional methods of field and laboratory testing were applied to describe soil properties. Stability analyses were conducted to demonstrate the predictability of failures. Recommendations for future construction in similar terrain are made.

GONSIOR M. J. and GARDNER R. B.

1971. Investigation of slope failures in the Idaho Batholith, USDA Forest Serv. Res. Pap. INT-97, 34 p., illus.

An investigation of slope failures in the Zena Creek sale area on the Payette National Forest is described. Conventional methods of field and laboratory testing were applied to describe soil properties. Stability analyses were conducted to demonstrate the predictability of failures. Recommendations for future construction in similar terrain are made.

Figure 13.--This illustrates proximity of site 917-1 to ridgetop.

Site 917-1. As shown in figures 13 and 14, this site is a stable embankment crossing a small gully on Road 917 and is located approximately 1.7 miles from the junction of Road 917 and the main Secesh Road (fig. 4). A plan view and three profiles are shown in figure 15.

Provisions for surface drainage seem to be adequate at this site. A culvert has been properly installed in the embankment with a suitable downspout and wire mesh catchment to dissipate the hydraulic energy. The close proximity of the site to a ridgetop can be seen in figure 13, illustrating the relatively small drainage area contributing to the culvert flow. The road has a 9 percent favorable grade at this location and a similar embankment with drainage equally well provided for is located only about 100 yards up the road (east) from this site. Therefore, surface water flowing down the road does not present a drainage problem at site 917-1. A soil-cement berm, which can be seen in figure 14, prevents road-surface water from eroding the outside slope of the embankment.

Figure 14.--This illustrates berm and culvert downspouts for proper handling of water at site 917-1.

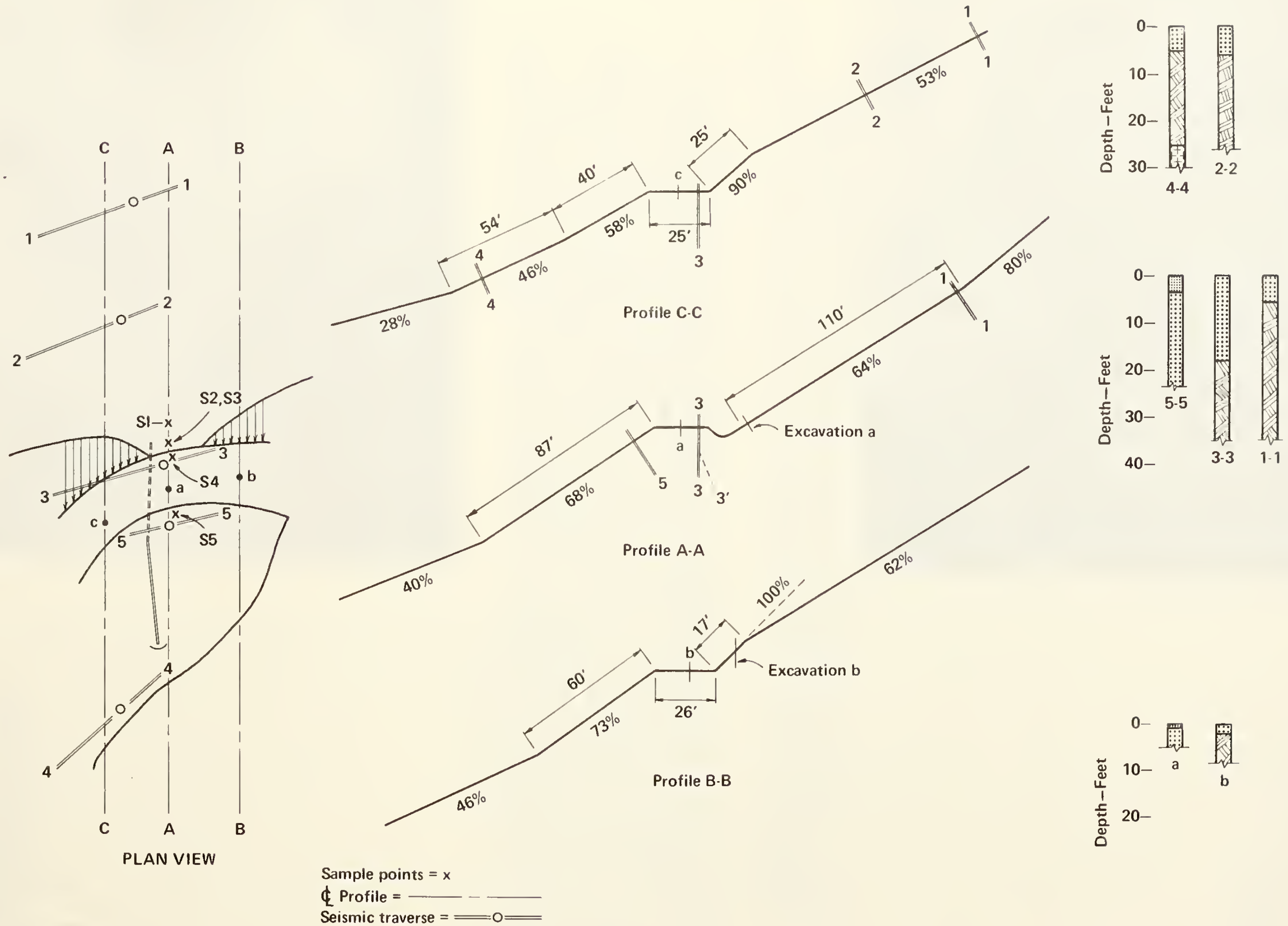


Figure 15.--Description of test site 917-1.

Five seismic traverses were run at site 917-1; the approximate locations are shown in figure 15. The small circles on the double lines represent the approximate locations at which the seismic interpretations are valid. Following are the interpretations, with the accompanying seismic velocities included:

Traverse 1: 5.5 feet of loose material, velocity = 1,050 feet per second (f.p.s.) overlying fractured rock (3,125 f.p.s.) which extends at least to an additional 30-foot depth.

Traverse 2: 6.2 feet of loose material (850 f.p.s.) overlying fractured rock (2,350 f.p.s.) which extends at least to an additional 20-foot depth.

Traverse 3: 5 feet of fill material (900 f.p.s.) overlying 13.2 feet of loose material (900 f.p.s.), beneath which is fractured rock (4,500 f.p.s.).

Traverse 4: 5.15 feet of loose material (540 f.p.s.) overlying a 20-foot thick layer of fractured rock (2,330 f.p.s.), beneath which is sound rock (20,000 f.p.s.).

Traverse 5: 3.5 feet of very loose material (540 f.p.s.) overlying at least 20 feet of denser material (1,100 f.p.s.).

Also shown in figure 15 are the approximate test pit locations.

Lines A-A, B-B, and C-C in the figure are the locations of the profiles sketched adjacent to the plan view in figure 15.

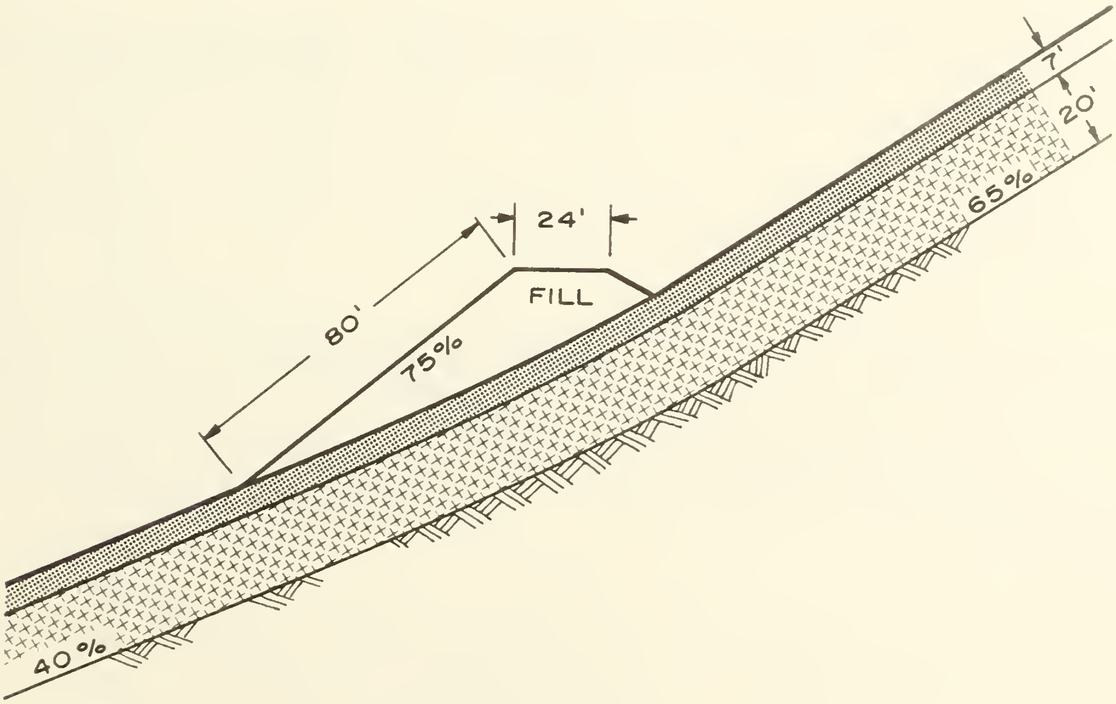


Figure 16.--Representative profile, site 917-1.

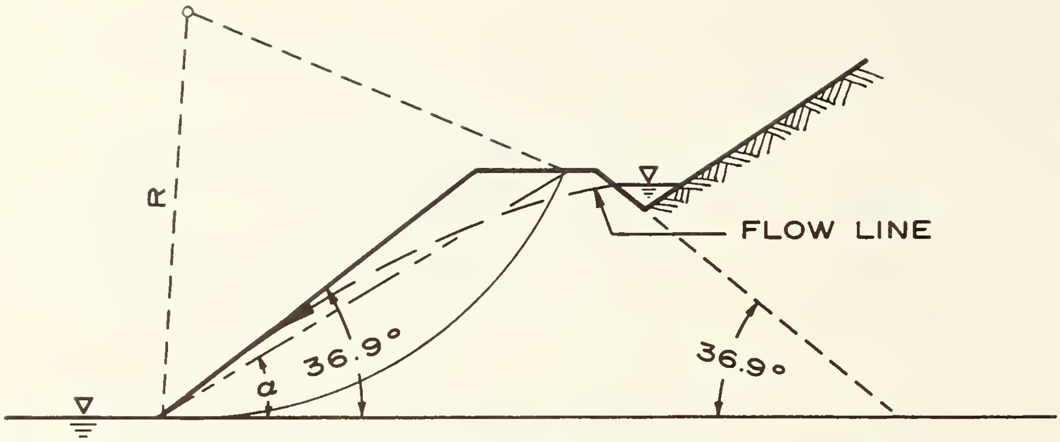


Figure 17.--Assumed conditions for L. Casagrande solution at site 917-1.

Site 917-1 was included in the investigation primarily for its comparative value and also to serve as a check on the stability analysis techniques and measured soil-strength parameters. Because a safety factor of unity separates stability from instability, theoretically no hypothetical failure surface with a safety factor less than 1.0 should exist in this embankment.

Based upon the data available, a representative profile of this site is shown in figure 16.

For an initial analysis, an in-place dry unit weight of 115 pounds per cubic foot (p.c.f.) is assumed (noting from table 2 that the dry unit weights found at sampling sites 917-1-4 and 917-1-5 were 119 p.c.f. and 106.3 p.c.f., respectively). The corresponding void ratio, e, would be 0.44, and saturated unit weight, γ_t, would be 134 p.c.f.

A conservative analysis can be performed by assuming a plugged culvert and subsurface flow as shown in figure 12. In this analysis, the L. Casagrande solution for the top flow line through a triangular dam is used[2] (see Taylor, p. 181). It is arbitrarily assumed that the upstream and downstream slopes of the dam are equal. Figure 17 depicts the assumed cross section.

A series of analyses were conducted, assuming circular failure arcs and using the method of slices. In all cases, failure arcs were assumed to pass through the toe of the fill slope. The material above the top flow line was assumed to weigh the same as that below the top flow line, or 134 p.c.f. The results of these analyses, assuming that c = 0 and $\phi = 35°$, are shown in figure 18 and, alternatively, in figure 19, wherein the parameters R and α are as shown in figure 17. Safety factors significantly lower than 1.0 are noted in figures 18 and 19. Three major conclusions can be drawn from these analyses:

(1) The assumed friction angle of 35° may be too low.

(2) The material actually possesses a minimum cohesive strength, even when saturated.

(3) The assumed seepage conditions are significantly more severe than would ever arise naturally.

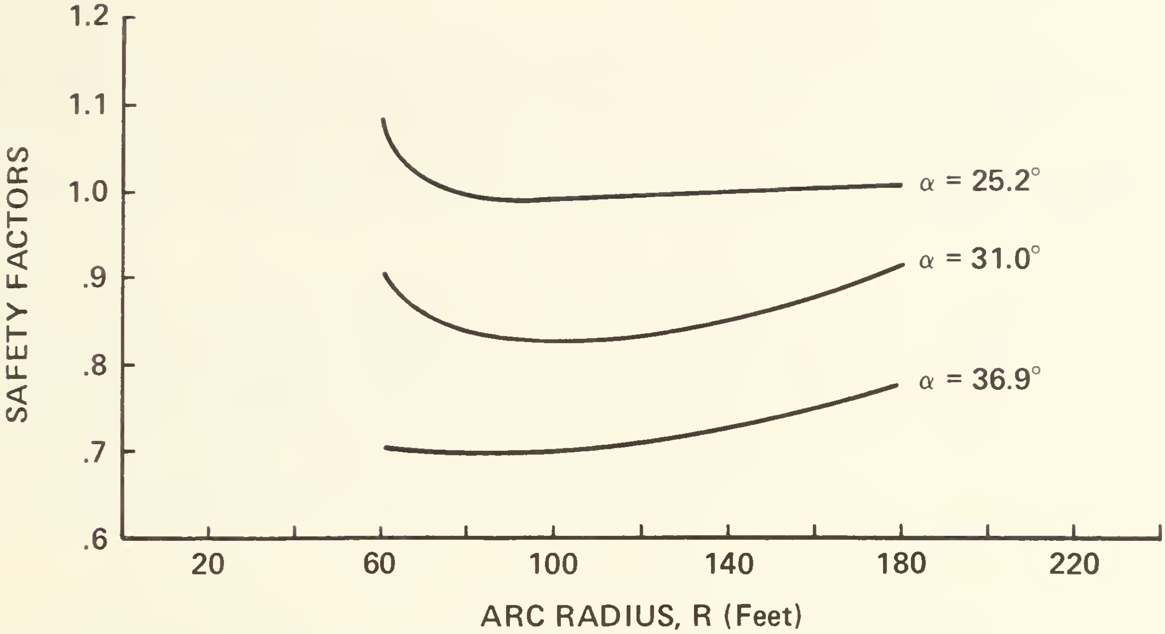


Figure 18.--Safety factors vs. arc radius, R, for conditions given in figure 17.

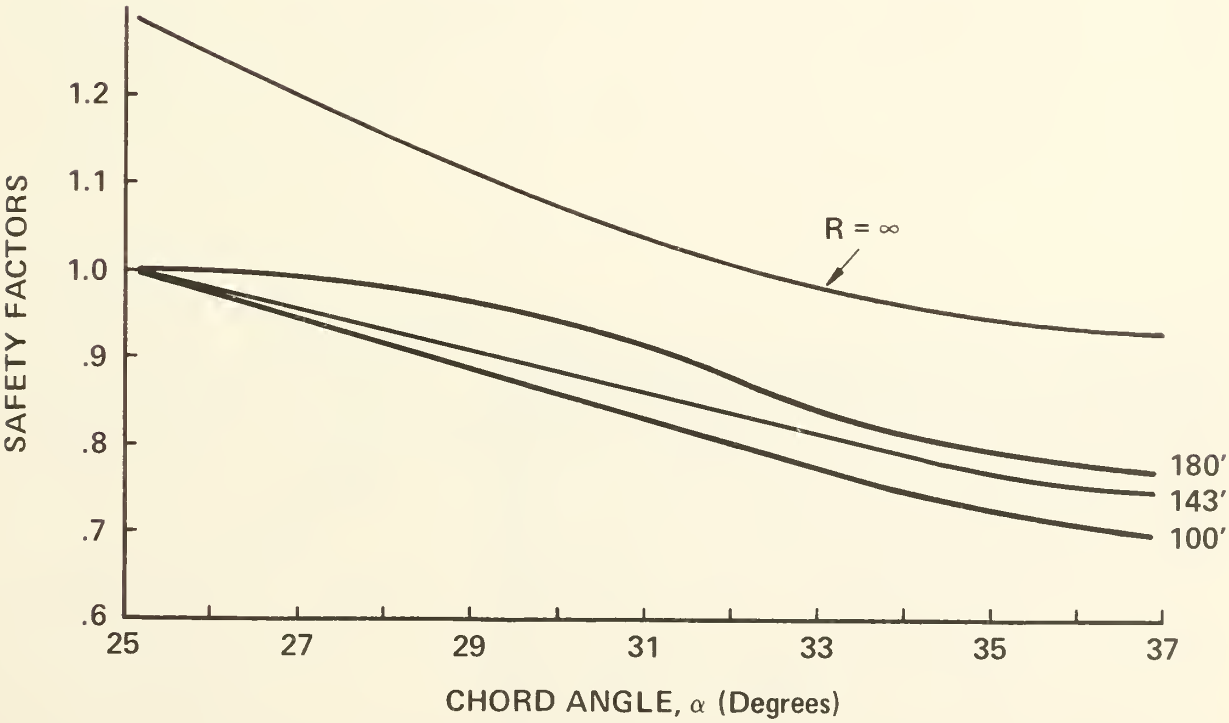


Figure 19.--Safety factors vs. arc chord angle, α, for conditions given in figure 17.

Although the effective friction angle in the field may exceed 35°, it is believed that the angle stated in the first conclusion is a reasonable estimate based upon the consistency of the laboratory results. Further, where the safety factors are as low as 0.7, a friction angle of 45° would be necessary in the absence of cohesion to increase the safety factor to 1.0. It seems inconceivable that the estimate of the friction angle would be this much in error.

With regard to the second conclusion, it seems possible that this material may possess sufficient cohesive strength, even when saturated, to assure stability at this site. The computed maximum cohesion required for a safety factor of 1.0, among all of the trial failure arcs, was only 175 pounds per square foot (p.s.f.), or about 1.2 pounds per square inch (p.s.i.), assuming $\phi = 35°$. Thus, although it has been concluded that no cohesive strength should be used for design purposes, it must be recalled that even in the saturated triaxial tests on loose material, a cohesive strength of 1 p.s.i. was estimated. Obviously, therefore, some moderate cohesion, combined with a slightly greater estimate for the friction angle, would assure computed safety factors in excess of 1.0 for all the trial failure arcs in this analysis.

The third conclusion is undoubtedly correct. There is no evidence that the culvert has ever become plugged at site 917-1. Even in the event of such an occurrence, it is doubtful that a stable seepage condition as severe as that assumed in these analyses would arise. However, since the exact flow pattern cannot be predicted, a series of analyses were conducted assuming no seepage. This was done in order to establish a minimum upper limit for the safety factors at this site. The results of these analyses are shown in figure 20, where the parameters R and α are as defined in figure 17.

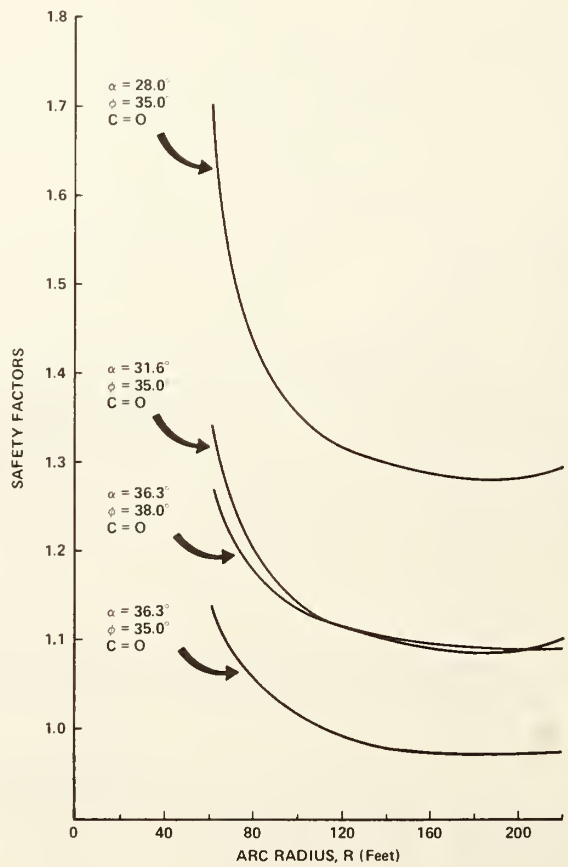


Figure 20.--Safety factors vs. arc radius, R, without seepage for site 917-1.

Again, it is noted that either the friction angle (ϕ) or cohesion (c) has been underestimated, since safety factors less than 1.0 were calculated. In these analyses, an estimated cohesive strength of 1.0 p.s.i., or 144 p.s.f., would have resulted in a minimum computed safety factor of approximately 1.4.

To evaluate the influence of error in the estimate of ϕ, a series of analyses, again assuming no seepage, were conducted with $\phi = 38°$, $c = 0$, and $\alpha = 36.3°$. The resulting safety factors are shown in figure 20. Thus, only moderate increases in either ϕ or c, or in both ϕ and c, result in computed safety factors indicating stability.

Finally, an extreme seepage condition wherein the top flow line is coincident with the fill slope was analyzed by assuming the validity of the infinite slope stability theory. Ordinarily, an arbitrary requirement that the length of failure be in excess of 20 times its depth is made. Noting that the fill slope is 80 feet long, a depth to failure of 4 feet is assumed. From the relationship:

$$c_d = \gamma_t H \cos^2 i \left(\tan i - \frac{\gamma_b}{\gamma_t} \tan \phi_d\right),$$

where

γ_t = saturated, or total, unit weight

H = vertical depth to failure surface

i = ground slope

γ_b = buoyant unit weight

ϕ_d = friction angle

and letting

$\gamma_t = 134$ p.c.f., $i = 36.9°$, $\phi_d = 35°$, $H = 4'$

$\gamma_b = 71.6$ p.c.f.

we compute a developed cohesion of 129.3 p.s.f., or

$c_d = 0.9$ p.s.i.

Figure 21.--Evidence of incipient failure at site 917-3, cracking and settlement of fill.

Figure 22.--Additional evidence of incipient failure, site 917-3 (test pit excavation shown in top of photo).

Of course, this is the most extreme seepage condition imaginable for this site, but the computed required cohesion is compatible with the laboratory strength test results.

Thus, although the assumption that these soils are cohesionless is conservative and appears desirable for design purposes, it may be necessary to recognize some minimum cohesion in the remainder of the site analyses. It may also be necessary to perform the analyses assuming various values for ϕ.

Site 917-3. This site, shown in figures 21-23, is located on Road 917 approximately 4.2 miles from the junction of Road 917 and the main Secesh Road (fig. 4). Considerable importance is placed upon the analysis of this site, primarily because failure is impending, as evidenced by cracking in the road surface and settlement of the outside portion of part of the fill (figs. 21 and 22). In addition, a narrow portion of the fill has failed, as shown in figure 23. Thus, this site offered the best opportunity for material sampling and definition of the mode and shape of failure.

Figure 23.--Narrow failure on fill at site 917-3.

Plan view and profile sketches appear in figure 24, along with pertinent seismic profile and test pit information.

Following are the seismic interpretations for site 917-3:

Traverse 1: 21.5 feet of loose material (1,150 f.p.s.) overlying at least 40 feet of fractured rock (3,000 f.p.s.).

Traverse 2: 4.8 feet of very loose material (830 f.p.s.) overlying at least 25 feet of fractured rock (2,500 f.p.s.).

Traverse 3: 3.1 feet of very loose material (770 f.p.s.) overlying at least 20 feet of loose material (1,100 f.p.s.).

A berm, although apparently not of soil cement, is located along the outside edge of the road surface, as at site 917-1. However, the road has negligible grade through this section, and the berm apparently acts as a small dam. This causes water to collect on the road. In fact, there may even be a slight dip in the longitudinal axis of the road, as evidenced by the fact that water had been flowing along the cracks and into the failed section from both directions.

Apparently the incipient failure surfaces, as is the scar left by the narrow failure, are at relatively shallow depths below the fill slope. Ignoring for the moment any water infiltrating from the road surface, the ground water conditions represented by figure 11 would seem to have minimal influence in the outer portion of the fill where failure occurred and is impending. To evaluate the accuracy of the laboratory estimates for ϕ and c, a series of analyses were conducted assuming circular failure arcs, with an assumed saturated unit weight of 125 p.c.f., but without excess boundary neutral stresses along the failure arc. (From table 2, test pits 917-3-1 and 917-3-3, an average dry unit weight of 100 p.c.f. and e = 0.66 provides for a saturated unit weight of 125 p.c.f.). The analyses were made under the condition that all failure arcs passed through the toe of the fill slope, where the fill slope was 70 feet in length and at a slope of 38.6° (profile A-A).

The results of these analyses are shown in figure 25, wherein R and α are again defined as in figure 17.

Under the assumption that $\phi = 35°$ and c = 0, several trial failure arcs were found with a safety factor less than 1.0. As discussed in connection with site 917-1, it is possible that this material retains some cohesive strength, even under the condition of saturation with seepage. Noting that the lowest safety factors were obtained when $\alpha = 38.6°$, a series of analyses were conducted at this steepest chord angle when $\phi = 35°$ and c = 1 p.s.i. As seen in figure 25, the safety factors were significantly increased. However, these increases were more pronounced at a greater R value. Since the appearance of the failure indicates a failure arc of low curvature, i.e., of long radius, the safety factor vs. radius curve for $\phi = 35°$ and c = 1 p.s.i., in figure 25, does not seem to comply with the actual failure surface nearly as well as do the curves for $\phi = 35°$ and c = 0.

Finally, a series of analyses were conducted with ϕ increased to 38° and c = 0. The results of these analyses are also shown in figure 25, and seem to be most compatible with the observed conditions at the site.

The low depth-to-length ratio of the failure at this site suggests that the infinite slope stability theory might be applied. Assuming a depth no greater than one-twentieth the length, or about 3.5 feet, developed cohesion values of only 0.18 and 0.03 p.s.i. are necessary for stability in the absence of seepage forces, for $\phi = 35°$

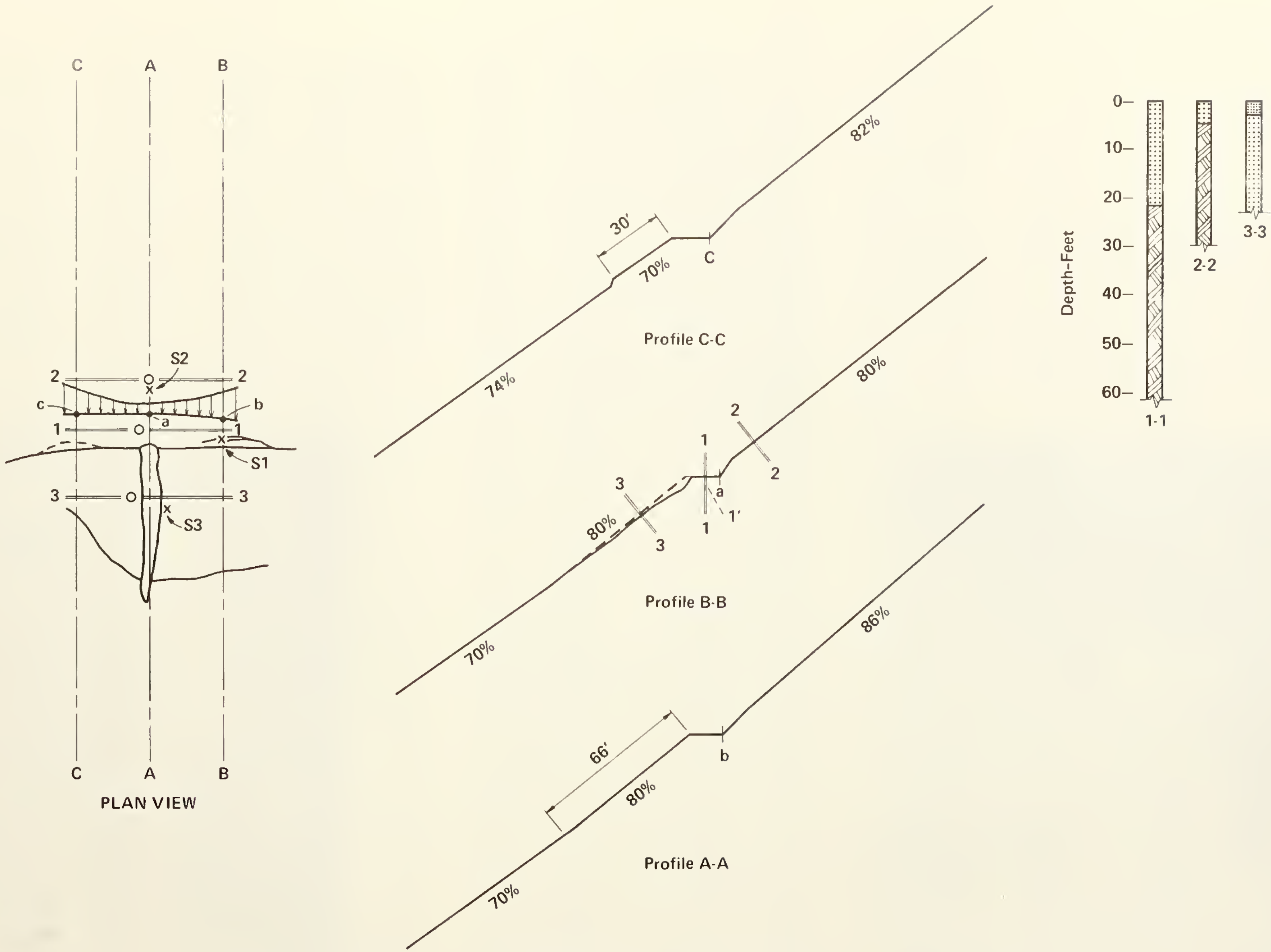
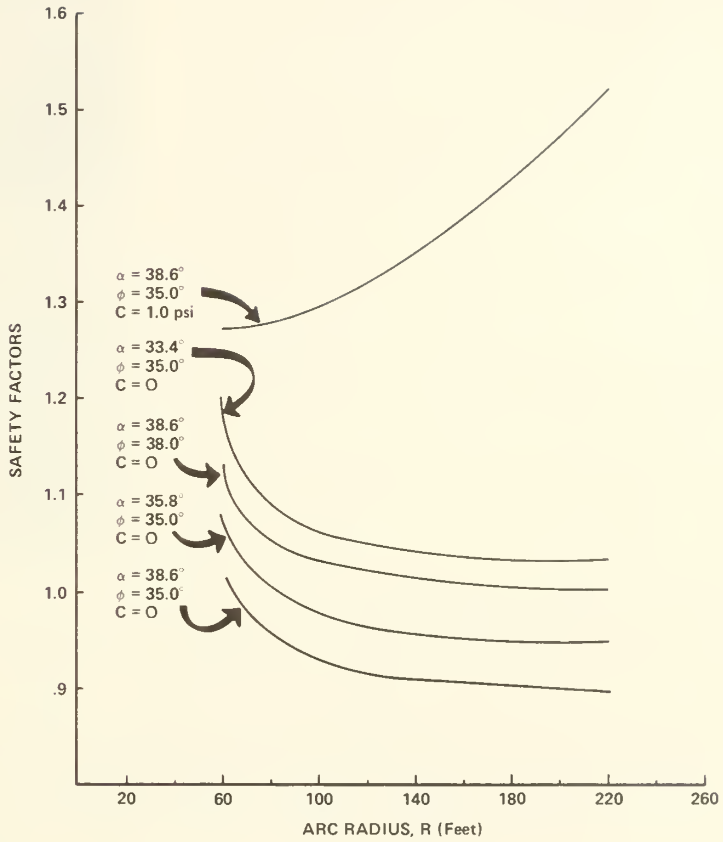


Figure 24.--Description of test site 917-3.

Figure 25.--Safety factors vs. arc radius, R, for site 917-3, with zero boundary neutral stresses.



and 38°, respectively. Under the extreme condition that would permit seepage throughout the fill, with a top flow line coincident with the fill slope, the necessary developed cohesion values would be 0.83 and 0.75 p.s.i. for $\phi = 35°$ and 38°, respectively. Such an extreme seepage condition seems unlikely at this site, although it has already been noted that water collected on the road at this location. It is possible, however, that water infiltrating from the road surface combined with the ground water, resulting in emergence of the water at some location above and near the toe of the fill slope. A subsequent "blow out," or quick condition might have occurred locally, and the resultant loss of support at the toe would have allowed the upslope material to slide.

In any case, the analyses of this site and of site 917-1 seem to indicate that a more realistic value for the friction angle in this material might be about 38°.

One additional factor should be noted in connection with site 917-3. Timber has been left standing at and below the toe of the fill slope. It is believed that these live trees and their fully developed root systems significantly enhance the stability of the site. Most certainly they were instrumental in preventing complete translation downslope of the material which partially failed.

Site 917H-3. This site is a relatively long, narrow failure of material lying in a swale above Road 917H at a location within 100 yards of site 917H-2. It is pictured in figures 26 and 27, as viewed from the bottom and top of the failure, respectively.

Figure 26.--View from below site 917H-3, illustrating its long, narrow, and shallow characteristics.

Plan view and profile sketches of the site are presented in figure 28. The seismic interpretation at traverse 1 is: 6.7 feet of loose material (1,140 f.p.s.) overlying at least 35 feet of solid rock (10,000 f.p.s.).

Although the road cut intersected the bottom of the swale, it is difficult to state with certainty that this resulted in sufficient loss of support to cause failure. By using appropriate techniques for the analysis of stability of a cohesive, infinite slope, it is apparent that the natural slope might have failed in the absence of road construction. Such an analysis follows:

Assume a natural slope, i = 70 percent (35°), void ratio, e = 0.86 (γ_d = 89 p.c.f.), and average depth to failure surface, H = 5 feet. The appropriate relationship for the dry condition is:

$$\frac{c}{\gamma H} = \cos^2 i\ (\tan i - \tan \phi).$$

Since $i = \phi$, no cohesion is required for stability in the dry condition.

Figure 27.--View from above site 917H-3, illustrating road at bottom.

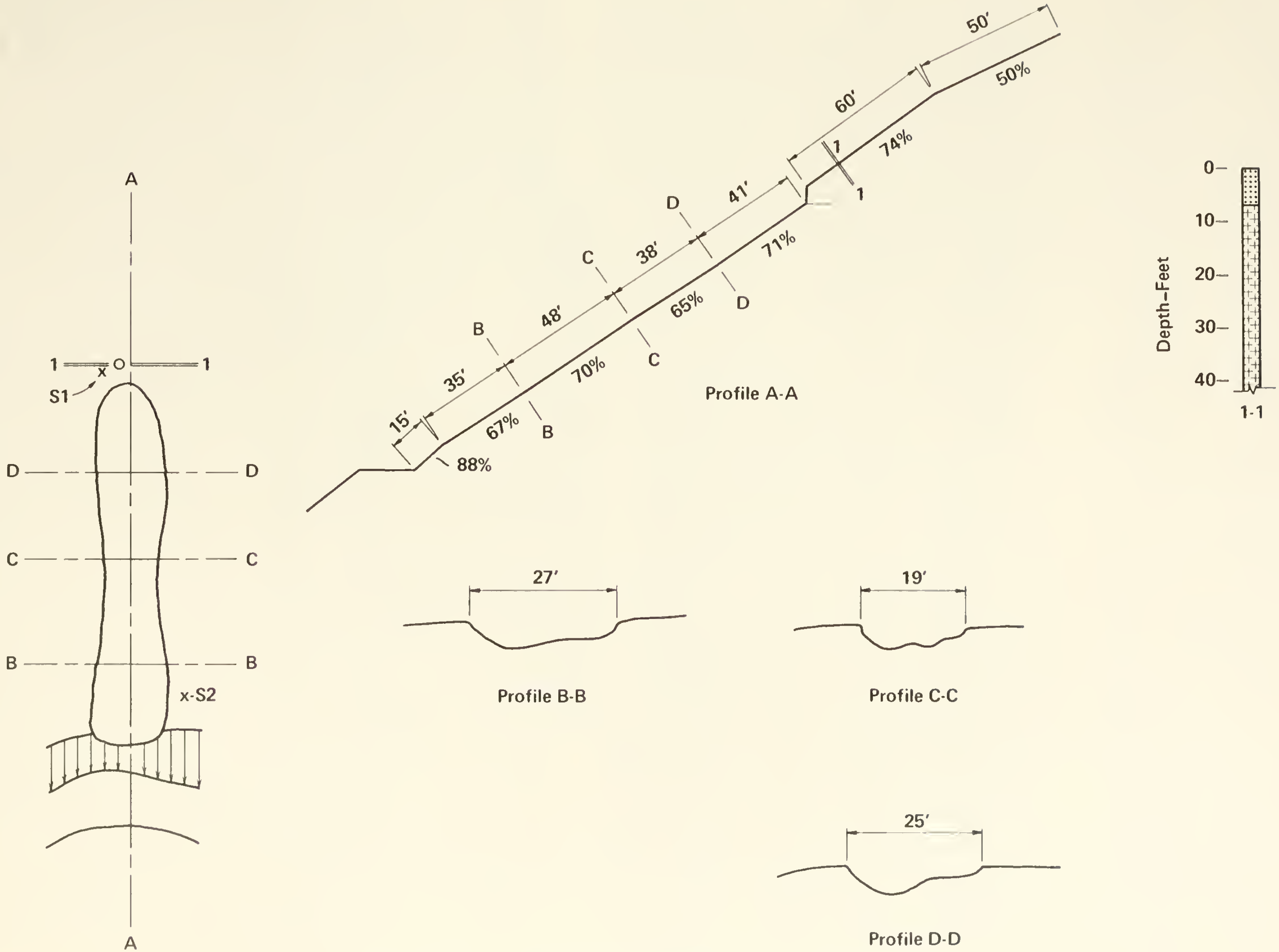


Figure 28.--Description of test site 917H-3.

In the case of saturation, however, the appropriate relationship describing stability is:

$$\frac{c}{\gamma_t H} = \cos^2 i \ (\tan i - \frac{\gamma_b}{\gamma_t} \ \tan \phi).$$

where now $\gamma_t \cong$ 118 p.c.f. and $\gamma_b \cong$ 55 p.c.f. Assuming $\phi = 35°$, a cohesive strength of

$$c = \gamma_t H \cos^2 i \ (\tan i - \frac{\gamma_b}{\gamma_t} \ \tan \phi) = 148 \text{ p.s.f.}$$

$$\cong 1.0 \text{ p.s.i.}$$

is required for stability. Even if $\phi = 38°$, a cohesive strength of 133 p.s.f., or approximately 0.9 p.s.i., would be needed. It is reasonable, therefore, to assume that failure might have occurred without road construction.

An additional factor to consider is that all timber had been removed from site 917H-3. Yet, adjacent to this site is equally steep terrain also intersected by the road cut, but this terrain has not been denuded and remains stable. There is evidence throughout the Zena Creek and adjacent watersheds that timber harvesting on steep slopes, with subsequent destruction of the stabilizing effects of live root systems, may contribute to the occurrence of shallow landslides.

Finally, as mentioned above, this slide occurred in a moderate natural swale. The existence of the swale indicates some form of structural weakness in the subsurface, which ultimately might have led to deeper dissection. Thus, the failure at this site, though undoubtedly hastened by construction and timber harvesting activities, might easily be considered to be merely one of a series of inevitable geomorphic events leading to a deeper gully.

Conclusions From Failure Investigations

For the failures investigated, it must be emphasized that no single cause can be isolated. Indeed, each individual failure was probably a result of a combination of circumstances.

Perhaps the natural instability of the terrain in the Zena Creek and similar drainages is of greatest importance. In the absence of human activities, a climatic event such as that which occurred in 1965 could be expected to result in landslides even on undisturbed terrain, though the frequency of such occurrences is undoubtedly increased manyfold as a result of road construction and timber harvesting operations. The bedrock and soils are extremely frangible, and landslides are undoubtedly very important as landforming processes in this region. Thus, it should be recognized that soil is continuously undergoing displacement in the relatively rapid natural dissection of the terrain. This recognition, however, should not provide rationalization for the accelerated soil movement resulting from human exploitation. Instead, it should supply an impetus for considerably greater care in the use of these lands.

In every road-associated failure, slope steepness was an important factor contributing to instability. As a consequence of the laboratory tests, it appears that no unretained slopes should be allowed in excess of 35°, or 70 percent, on road fills. Although both natural and artificial slopes in excess of 35° can be found in many locations, it is believed that most of the necessary additional strength required to maintain these slopes is from more or less temporary sources. For example, live tree roots extending into the deeper fractured zones undoubtedly provide some structural reinforcement to the upper soil layers. Also, large fragmentary boulders embedded in newly placed fills provide an interlocking and keying effect that permits creation of steeper slopes. However, the rapid disintegration of the larger stones, and the death and rotting of tree roots can result in significant stability reductions.

Aside from mass stability requirements, erosion of finer particles from the surface of steep slopes is considerably enhanced when the slopes are at, or near, the angle of internal friction. Further, since revegetation of these slopes appears to be a difficult and slow process, such steep slopes are serious contributors of sediment for extended periods.

Water was another important factor contributing to instability. As demonstrated in several of the stability analyses, complete saturation with equipotential lines assumed normal to the ground surface results in appreciable reductions of safety factors. Although seepage conditions are probably never quite so severe as assumed for some of these analyses, neither are the safety factors in the assumed dry conditions sufficiently large to compensate for even moderately adverse water conditions. Furthermore, there is good evidence that with the passage of time after construction and considering the weathering and compaction, the seepage conditions may worsen from the standpoint of stability. Undoubtedly many of the road-associated failures could have been prevented if surface water had been better handled.

Minimal compactive effort in the road fills was evidenced by the low in-place densities. The problem of liquefaction, a result of high void ratios in combination with saturation, or near saturation, in these soils is apparent throughout the Zena Creek area. There is little doubt that liquefaction occurred at practically every large failure, as evidenced by the long transport distances (often several hundred yards) and "mud" splattered on tree trunks up to heights of several feet. This is not just a problem in the road fills, however, as the in-place densities at shallow depths in the natural materials were equally low. Nevertheless, even nominal compaction of the fills during construction should have prevented some failures, and certainly should have reduced both the size of some of the failures and the ease of transport to the streams.

Aside from the liquefaction problem, significantly lower strain energy is required for failure in these loose, uncompacted materials. In denser materials, shear strain must be accompanied by a corresponding volumetric increase. But in loose materials, either at or above the critical void ratio, shearing can occur without volume change, and volume decreases usually accompany failure. Thus, lack of compaction considerably increases the chances for failure even in the dry condition.

Besides the apparent lack of compaction control, other examples of poor construction practices were in evidence in the Zena Creek road system. At one site, a large log was found embedded in the fill. The inclusions of logs and stumps in embankments and fills should never be permitted. Loose surface materials should be compacted and keyed if clean, or removed if not. It is doubtful that adequate clearing and grubbing were performed in most locations.

Cyclic displacement resulting from sonic booms is another factor which may be of significance, but one for which very little information is presently available. During the field investigation the authors experienced several unnerving shocks caused by the supersonic aircraft that passed over the study area, usually two or three times each day. It is suspected that these shocks may provide sufficient ground motion to initiate liquefaction in some of the more critical slopes. The authors believe that this deserves some additional study; however, they do not believe that sonic booms would be of much consequence to the behavior of adequately designed and compacted slopes.

Finally, as discussed in connection with sites 917-3 and 917H-3, live trees probably are important factors influencing the stability of both natural slopes and road fills. It is suspected that the live tree root systems exert a significant stabilizing influence due to both mechanical reinforcement and moisture depletion. To avoid future distress, all stumps and large roots should be removed before placing road fills. It is believed, however, that barriers of live trees should remain undisturbed immediately below the toe of the fill slope and above the cut slope. In addition, it is suspected that weathering of the subsurface materials may be accelerated after removing the timber since daily temperature extremes are undoubtedly more severe after timber harvest. Thus, it is not sufficient to consider only surface disturbances in the planning of harvesting operations but, in addition, long range mass stability must also be an important consideration.

Summary and Recommendations

An investigation of several slope failures in the Zena Creek watershed, located in the Idaho Batholith, successfully explained the causes of failure and provided an opportunity for a more critical evaluation of current road construction and harvesting practices.

A natural instability exists in the terrain at Zena Creek and in similar drainages. In the absence of human activities, a climatic event such as occurred in 1965 could be expected to result in landslides even on undisturbed terrain, though the frequency of such occurrences is increased manyfold as a result of certain harvesting and road construction practices. It is this accelerated damage that must be greatly minimized.

In every road-associated failure, the steepness of both natural ground slope and fill slope was an important factor contributing to failure. As a result of the laboratory tests and subsequent stability analyses, it can be concluded that no man-made slopes exceeding 35° should be constructed without special provisions to assure stability.

Aside from the mass stability problems, erosion of finer particles from surface slopes is considerably enhanced for cohesionless materials when slopes are at or near the angle of internal friction.

The problems of liquefaction, a result of high void ratios in combination with saturation, were common. The long transport distances of failed fill materials and the presence of "mud" splashed high on tree trunks were irrefutable evidence that liquefaction had occurred in conjunction with most failures. High void ratios were not just a problem found in road fills, but were common at shallow depths in natural material. Nevertheless, even nominal compaction of the fills could have prevented some of the failures.

In addition to the low in-place densities and inadequate control of surface and subsurface flows, the presence of logs and other debris in the fills may have contributed to many of the road failures.

Live roots and trees are probably important factors influencing the stability of natural and artificial slopes. In addition, it is suspected that disintegration of the subsurface materials may be accelerated by exposure to greater daily temperature extremes as a result of timber harvesting.

The type of investigation conducted and reported upon herein probably cannot be adopted as a routine procedure for all situations. Certainly one may find stable natural and man-made slopes in the Zena Creek and similar drainages which are greater than 70 percent. Similarly, failures of slopes less than 70 percent are possible. It should be understood that the tests and analyses conducted in this type of investigation are based upon extremely simplified models and assumptions of uniformity and homogeneity.

Where engineers are called upon to design and analyze man-made earth structures, and where such structures can be constructed to closely approximate the models of analysis, then the methods of analysis employed can be considered reasonably reliable. However, when dealing with the vagaries of nature or with complex and uncontrolled man-made structures, the confidence one places in most methods of investigation and analysis must necessarily be reduced.

It is important, however, that some measure of soil strengths and depths, and their degree of variability within an area for which development is planned, be obtained before development proceeds. Such parameters may be determined by intensive exploratory and laboratory techniques, by correlations between topography and movement, or simply through experience gained in other similar areas; but without this knowledge the location and design of roads cannot proceed on a completely rational basis.

Similarly, knowledge of surface and subsurface water conditions must precede road location and design if erosion and landslides are to be avoided or controlled. Beyond these areas of information, the designers must exercise their professional judgments regarding the appropriate specifications, methods of analysis, and intensity of inspection and control during construction.

As a consequence of this investigation, the following tentative criteria are recommended:

1. Fill slopes should be specified by the design engineer and based upon suitable stability analyses. In no cases should unretained fill slopes exceed 70 percent.

2. Cut-bank slopes should generally be as steep as possible consistent with subsurface conditions. Slope rounding should be practiced when overburden depths require it.

3. In general, alignment should be sacrificed whenever possible to avoid deep fills and/or cuts.

4. All fill slopes should be compacted to a degree consistent with design standards and material properties.

5. Drainage facilities should be provided to prevent damaging concentrations of surface runoff and to avoid high pore pressures in cuts and fills.

6. All fill sections should be insloped with an adequate berm on the outside, unless adequate means for protecting the fill slopes against erosion can be provided. Benched sections can be outsloped providing adequate vegetative cover or other erosion protection remains after construction.

7. Specifications requiring log and debris removal from fill slope sections must be rigidly enforced.

8. All fill surfaces should be revegetated with suitable species as soon as possible after construction. In addition, rip-rap or other suitable debris should be placed at or near the toes of fill slopes to halt the downhill movement of sediment from the fill surfaces.

9. Roadbeds should be stabilized to prevent erosion and rapid deterioration.

10. Provisions should be made to prevent raveled cut-slope materials from reaching live streams.

Note that these recommendations merely constitute improvements, not complete solutions, and merely reflect engineering considerations pertaining to the management and use of this and other similar areas. Future knowledge about the behavior of these materials and the flow characteristics of surface and subsurface waters may require alterations of and additions to these recommendations.

Beyond these criteria, there are a great number of other environmental responses which must be understood; and knowledge of these responses must be applied if these lands are to be wisely managed. In addition, there appears to be a need for a subtle philosophical change in the traditional engineering approach to problem solving and design. Usually, the integrity of a road, dam, or any other structure is viewed as the primary goal, and thus natural processes such as erosion, seepage, and settlement are considered as impositions on the structure which must be controlled or withstood. Instead, the road or structure might better be viewed as an imposition upon the various natural processes, and location and design might better be oriented toward assuring the continuity of, or at least compensation for changes in, these natural processes. By so reorienting design philosophy not only should the integrity of roads and structures be better guaranteed, but the chances for causing undesirable changes in the functioning of natural systems should be considerably reduced. Of course, by changing the question from "What are the natural processes which will endanger the road's integrity?" to "How will the road influence natural processes?" the designer is forced to consider a broader spectrum of environmental factors. Thus, multidisciplinary cooperation and teamwork become not only desirable, but absolutely essential to the completion of the planners' and designers' work.

Headquarters for the Intermountain Forest and Range Experiment Station are in Ogden, Utah. Field Research Work Units are maintained in:

- Boise, Idaho
- Bozeman, Montana (in cooperation with Montana State University)
- Logan, Utah (in cooperation with Utah State University)
- Missoula, Montana (in cooperation with University of Montana)
- Moscow, Idaho (in cooperation with the University of Idaho)
- Provo, Utah (in cooperation with Brigham Young University)

AFLC/HAFB, Ogden