HIGHWAY RESEARCH BOARD

Special Report 29

Landslides
and
Engineering Practice

By the

Committee on Landslide Investigations

National Academy of Sciences—

National Research Council

publication 544

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Edited by

Edwin B. Eckel

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The opinions and conclusions expressed in this publication are those of the authors and not necessarily those of the Highway Research Board.

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In Memory

Seward Ellis Horner

April 3, 1906 - July 8, 1954

As Chief Geologist, State Highway Commission of Kansas, he did more than any other to develop the application of all the disciplines of geology to a single practical end — better road building. As a member of this Committee he was a tower of strength. As an advisor, to us and to many other engineers and geologists, he was without equal. As a friend, he can never be forgotten or replaced.

Committee on Landslide Investigations

Drawn from photographs, by John R. Stacy

Contents

Page

Part II. Solution of the Problem

Page

Page

List of Illustrations

Figure | *Page*

List of Tables

Part I

Definition of the Problem

Chapter One

Introduction

Edwin B. Eckel[1]

Landslides are of profound interest to the common man. This interest stems from the fact that landslides, like volcanic eruptions, floods, and hurricanes, mean destruction of life and property by the forces of nature. Because landslides occur in a wide range of environments, they are seen and at least partly understood by almost everyone. It is little wonder that reports of "moving mountains," of rock avalanches, and even of trains or motor traffic held up by slides all capture the public imagination.

But to the scientist and the engineer, landslides are of even greater and more immediate interest than to the layman. The geologist is interested both academically and practically. He recognizes landsliding as one of the most widespread and effective agents in sculpturing of the earth's surface. To him, then, each landslide is an opportunity to understand a little better the makeup of the earth and the history of its surface. But his interest is also intensely practical, for his inquiry into the cause, character, and history of a landslide can and should provide the engineer with many of the answers that are needed for decisions as to effective methods of control, correction or prevention.

The engineer and the geologist who works with him are interested in landslides because their job is to build and maintain safe, economical, and useful structures on the earth's surface. A landslide, unforeseen or improperly provided for, may destroy their structure or impair its usefulness. Such a landslide may mean death to people who have trusted the structure; repair of the structure will most certainly cost money. Even an insignificant little slide, sloughing off into a roadside ditch, may wreck an automobile; or if it is continuous, it may in time run up enormous maintenance costs.

It is the purpose of this volume to bring together in coherent form and from a wide range of experience such information as may be useful to any engineer who must recognize, avoid, control, design for, or correct the more important types of landslide movement.

Because the book is designed for practical use, theoretical discussions are minimized, whereas those phases held to be of greater interest to the practicing engineer are emphasized over others. As shown in the table of contents, the book is divided into two parts. Part I, called Definition of the Problem, is intended to provide the engineer with the tools and methods he needs to solve an actual or potential landslide problem. Part II, called Solution of the Problem, summarizes the methods known to have been applied to the prevention and control of landslides; it also discusses the methods of making stability analyses and of using them in the solution of design problems. In this part every effort has been made to distinguish between those methods that have proved successful under given circumstances and those that have not. The brief closing chapter points out the kinds of information on landslides and their con-

[1] Publication authorized by the Director, U. S. Geological Survey.

trol that are still lacking and suggests methods by which such information may possibly be obtained.

In its attempt to cover the entire field of landslides, from causes to cures, the volume is, to the authors' knowledge, unique in the English language. In fact, the only foreign-language book known to the committee that is of comparable scope is that of Knorre, Abramow, and Rogosin (1951). There are available, of course, excellent books and articles that treat one or more facets of the entire problem far more fully than can be done here; these have been drawn on heavily, and are listed in the bibliographies that close each chapter.[2]

Partly because of its comparatively wide scope, and partly because of the spread of interests and knowledge within the committee, the book may seem to assume a knowledge of more specialties than are commonly held by any individual engineer or geologist. Without expanding the book unduly, the reader can only be referred to the standard texts and handbooks on geology, soils, hydrology, mechanics, foundations, and construction methods, or to some of the specialized glossaries of scientific and engineering terms.

There is no expectation that the reader of this book will become an expert on all phases of the investigation and treatment of landslides. Rather, it has been the aim of the compilers to provide an introduction to all of the main factors that go into the solution of a given landslide problem. The average engineer, to whom a landslide is only one of many different problems that he encounters in his work, should be able to use the tools presented here himself or else should be able to determine from the facts given here when it is time to call in a specialist on one or another phase of his investigation. On the other hand, the specialist in some phase of landslide studies should gain an appreciation of the many facets of a landslide problem and of how his specialized knowledge of one facet can best be applied toward solution of the total problem.

[2] Just as the manuscript of this volume was ready for submittal to the Highway Research Board there appeared an English translation of a more than worthy forerunner of this book (Collin, 1846, 1956). Although the French original is more than a century old, this fascinating and remarkable volume bears much resemblance to the present one. To include adequate references to Collin's pioneer work would have required some revision of fully one-half the chapters in this book, a job that would have unduly delayed its appearance. Suffice it to call attention to it here and to commend it as a "must" to every serious student of landslides or of the application of soil mechanics and stability analyses to landslide problems.

Definitions and Restrictions

As described more fully in Chapter Three, the term "landslide" is defined for use in this volume as downward and outward movement of slope-forming materials — natural rock, soils, artificial fills, or combinations of these materials.

Normal surficial creep is arbitrarily excluded from consideration, as are subsidence without downslope movement and most types of movement due to freezing and thawing of water. Similarly, landslide phenomena in tropic and arctic climates, and their treatment, are almost entirely neglected here. A few examples are drawn from other countries, but as the writers and their informants are largely experienced in the United States, most of the descriptions of landslides and of engineering techniques are drawn from this country.

It was perhaps inevitable, considering the makeup of the committee and the sources of information easily available to it, that the volume should seem to stress the landslide problems related to highways and railroads almost to the exclusion of many other landslide problems, such as those of shorelines and waterways, of city, suburban, and resort developments, and of farmlands. This apparent neglect has not been intentional — nor should it necessarily detract from the applicability of the facts contained herein to the solution of landslide problems other than those encountered by highway and railroad engineers. The factors of geology, topography, and climate that interact to cause landslides are the same regardless of the use to which man puts a given piece of land.

The methods for examination of landslides are equally applicable to problems in all kinds of natural or human environment. And the known methods for prevention or correction of landslides are, within economic limits, independent of the use to which the land is put. It is hoped, therefore, that despite the narrow range of much of its exemplary material, this volume will be found useful to any engineer whose practice leads him to deal with landslides.

Method of Compilation

Early in May 1949 the present chairman was asked by Mr. Harold Allen, then Chairman, Department of Soils Investigations, Highway Research Board, to set up a committee on landslide investigations. This invitation was accepted soon after, but it was not until January 1951 that a committee was finally formed and the first meeting held.

In choosing the committee, and in adding to it from time to time, deliberate efforts were made to get wide geographic representation. An approximate balance between practicing engineers and geologists, as well as the spread of the membership between state highway, educational, and governmental organizations, was also a deliberate objective in setting up the committee.

By the end of 1951 the committee had sponsored publication of a bibliography on landslides (Tompkin and Britt, 1951) and had decided to put its main efforts into compiling the present volume. It had also adopted a general outline for the book and had made tentative assignments of authors to prepare the individual chapters. Since early 1952 progress and content of each chapter has been thoroughly discussed and reviewed by the entire committee, not only through correspondence but also at a series of semiannual meetings. Therefore, even though the individual chapters are credited to those authors who had primary responsibility for preparing them, each chapter is actually the product of the entire committee and represents its combined views.

The editor of the volume has merely woven the units into the whole, in an effort to make the book stand as a unit rather than as a symposium of related papers. Despite this effort, the critical reader may soon find that some if not most of the chapters tend to have a provincial flavor, in that much of the exemplary material comes from the state or region most familiar to the author of that chapter. This was almost inevitable, for each author quite naturally drew on his own experience in preparing his material. This provinciality of its parts need not, however, be an obstacle in the usefulness of the whole. As was mentioned previously in relation to the stress on highways and railroads, the basic problems presented by landslides are much alike everywhere, so that examples taken from one part of the country can usually be applied in another part. In addition, it is believed that if taken all together, the examples given in this volume provide a fairly representative cross-section of landslide problems throughout the United States.

The Questionnaire

Early in its work the committee realized that it needed many more data, based on actual experience, than were available in the literature or in the minds and files of the dozen or so committee members themselves. After exploring a number of possible methods for gathering additional data it was decided to prepare and circulate a questionnaire to all available geologists and engineers whose work was likely to bring them into contact with the landslide problem. This decision was not taken without misgivings; preparing and distributing the questionnaire meant much additional work and delay for the committee, and far more work in analyzing and using the results. More important was the imposition on the time, energy, and good will of countless busy men who would be asked to contribute to the

questionnaires in some way. The results were gratifying beyond all expectations; they form the basis of much that is new and worthwhile in this book.

Questionnaires were sent to the state highway departments, state geologists, the larger railroads in the United States, the Canadian railroads, and all Federal Government agencies concerned with major engineering construction work. By personal requests of committee members and through announcements in the technical press, several turnpike authorities and many company and private engineers and geologists, as well as the civil engineering and geology departments of some colleges and universities, were also asked to help.

Of some 250 questionnaires that were sent out to individuals and groups, about 75 were returned. Naturally, these varied in their degree of completeness, but all contained information of value to the committee. In addition to the data contained in the completed questionnaires, there were many special reports on individual landslide problems that would not have come to the attention of the committee or of the engineering profession except as a result of the questionnaire. A great number of useful facts also came in letters, with or without completed questionnaires. Even those that were confined to negative statements were valuable. The fact that certain states have no landslide problems, for instance, is just as useful in a study of this kind as full descriptions of landslide problems in some other states.

Doubtless because the committee worked under the auspices of the Highway Research Board and because its work emphasized the problems of highways and railroads, responses from the state highway departments and from the United States and Canadian railroads were more numerous and comprehensive than the responses from most other sources. Some Federal agencies, however, were not far behind the railroads and highways; some of the most useful specific descriptions and illustrations came from them and from a few state geologists and private engineers.

Because it is of possible interest to the reader in seeing the kind of information made available to the committee, the entire questionnaire is reproduced in the Appendix.

Permanent File of Questionnaires

Every effort was made to wring the last drop of value from the questionnaires received. All of them were studied by each member of the committee, and pertinent material was abstracted by the authors of the several chapters for use in their compilations. Even so, the completed questionnaires, together with numerous letters relating to them, obviously must contain far more data than could possibly be condensed into a book of this kind. Moreover, they represent an incalculable investment of time, energy, and good will by a great many engineers and geologists. To do justice to these men and to their data, it has seemed essential that the material be preserved in its original form. Accordingly, except for the few that contained confidential information and that have been returned to their authors, all questionnaires, together with pertinent extracts from letters received by the committee, have been deposited with the Highway Research Board Library in Washington, D. C. There they will be accessible for research purposes to any future student of landslides, whether he be interested in generalizations such as are represented in this book or in the details of particular case histories with which much of the material abounds.

Acknowledgments

The efforts of this committee, large though they were, would have been well-nigh valueless without the assistance of countless others. Literally hundreds of engineers and geologists, living and dead, have contributed in greater or lesser degree to the quantity of facts condensed within these pages. Many such contributions are in the international literatures of geology and of engi-

neering; many others were in the minds of individuals or in official files and come to light now through the medium of the committee's questionnaire or of conversations and correspondence with members of the committee. To list all the contributors, direct and indirect, is impossible; to list those who contributed most would be unfair to others. All that can be done is to express deep appreciation to all who added anything whatever to the wealth of technical and scientific facts that have been considered. All of them are assured that the committee, individually and collectively, considers itself a compiler of their information, rather than an originator of new information.

To the entire staff of the Highway Research Board, thanks are owed, not only for moral and financial support but also for many other tangible and intangible aids given throughout the work. In particular, mention should be made of Messrs. Fred Burggraf and Roy W. Crum, Director and former Director; Frank R. Olmstead and Harold Allen, Chairman of the Department of Soils, Geology and Foundations, and former Chairman of the Department of Soils Investigations; and A. Walton Johnson, Engineer of Soils and Foundations, for their constant encouragement and advice. W. A. Warrick, the only committee member not specifically credited with authorship of any of its parts, nevertheless contributed much to the volume. Acting as friendly advisor and critic, he kept the rest of the committee on the track of practicality.

Two members of the U. S. Geological Survey — John R. Stacy, who prepared many of the illustrations, and Bernice M. Peterson, who acted as secretary to the chairman, hence to the committee — contributed much. To them and to the technical and secretarial staff members of each of the committee members we are grateful, as we all are to the officers of our parent organizations who permitted us to devote so much official time and energy to this undertaking.

Finally, not as chairman of this committee, nor as editor of this book, but as a person, I want to express my deepest gratitude and thanks to all the members of the committee. For more months than I care to count each one has worked earnestly; each has put into the job far more than he could hope to take out of it. From all our discussions, formal and informal, has come a comradeship and a mutual understanding of the problems of engineers and geologists that is all too rare. We have worked together — and we have had fun doing it.

References

Knorre, M. E., Abramow, S. K., Rogosin, I. S., "Erdrutsche und ihre Bekampfung (Landslide and Its Control.)" Moscow, 1951. German translation: Schriftenreihe des Verlages Technik, Band 89, 141 p., 60 figs., 15 bibl. refs., VEB Verlag Technik, Berlin, 1953.

Tompkin, J. M., and Britt, S. H., "Landslides." A selected annotated bibliography. Highway Research Board Bibliography 10, 53 p., 1951.

Chapter Two

Economic and Legal Aspects

Rockwell Smith

The Economic Importance of Landslides

Costs to the Nation

Reliable estimates as to how much landslides cost the nation are difficult to obtain. It can be stated confidently, however, that the average yearly cost of landslides in the continental United States runs to hundreds of millions of dollars. This money, paid out every year by taxpayers and private companies, includes not only the direct costs of corrections and repairs of damage caused by landslides, but also very large sums for such items as delays of traffic, interruptions of service, and claims for damages (see Fig. 1).

Highways, railroads, and public utilities as a group receive the greatest direct damages; but even here, full costs can be assessed only in individual cases. Heavy losses are also sustained by local governments and homeowners in some cities (see Figs. 2, 3). In parts of the country very severe losses are involved in destruction of farmlands, resorts, and homes, particularly along rivers and lakes where undercutting and slipping occurs (see Fig. 4). The damages in these categories are extremely difficult to evaluate, but they are obviously large. The filling of the reservoir behind the Grand Coulee dam has cost taxpayers and private property owners at least $20,000,000 during the past 20 years in avoidance and correction of damage from landslides (Jones, 1956). A single oil company must have spent well over $1,000,000 in partially controlling slides in the Ventura Avenue oil field, California, to say nothing of its losses in production (Mineral Information Service, 1954).

The nationwide questionnaire yielded data on the typical costs of landslides to highway and railroad organizations. Thus, among the highway departments one state reported annual costs of more than $1,000,000; three, between $500,000 and $1,000,000; one, $250,000 to $500,000; five, $100,000 to $250,000; six, $25,000 to $100,000; and eleven, less than $25,000. These figures apply largely to maintenance costs, because costs of reconstruction and damage claims are not usually accessible. In fact, the figures reported are probably low even for maintenance costs, because many highway department accounting methods are not such as to disclose fully maintenance costs that are directly related to landslide problems.

Railroad accounting procedures, on the other hand, are prescribed by regulation and special projects over and above routine are commonly handled under an authority for expenditure. This results in full application of expenditures to a given project and probably means that the following figures, as far as they go, are somewhat more accurate than those furnished by the highway departments.

Twelve railroads, representing approximately 22 percent of the total roadway mileage in the United States and 30 percent of that in Canada, reported their annual costs. One road showed $500,000 to $1,000,000; two, $250,000 to

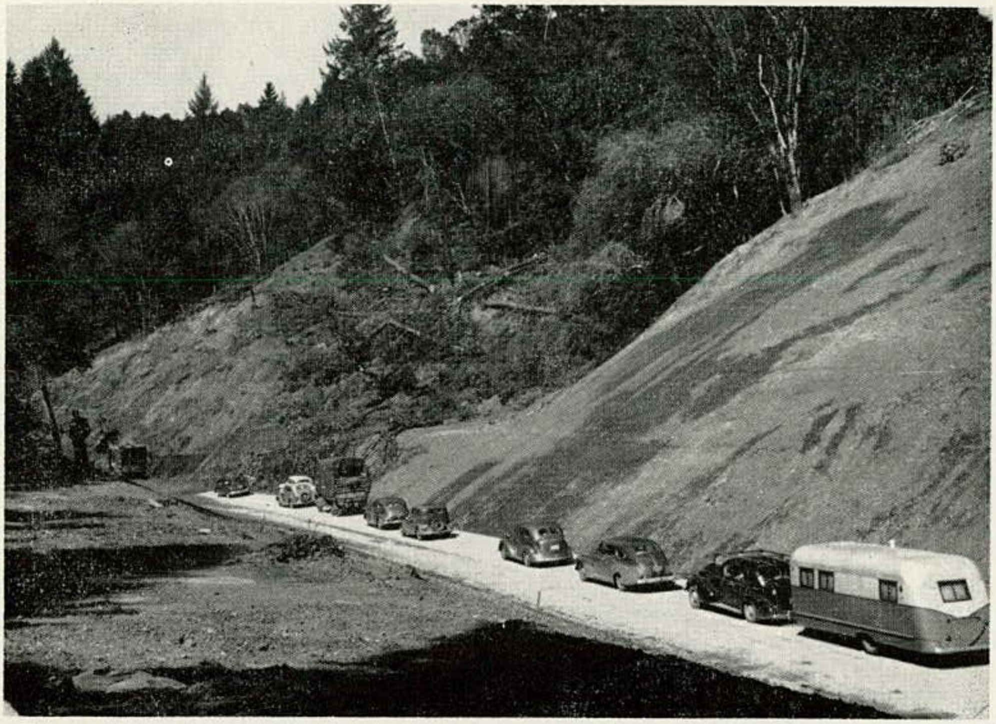


Figure 1. What price landslides? Traffic on Redwood Highway, Calif., obstructed by a relatively small landslide. (Photograph by A. D. Hirsch, courtesy of California Division of Highways)

$500,000; two, $100,000 to $250,000; three, $25,000 to $100,000; and four, less than $25,000. All roads reported certain years with expenditures in considerable excess of these "normal" annual costs. If the railroads that answered the questionnaire can be considered representative from the standpoint of landslide problems, it is easily seen that the direct costs of landslides to the United States and Canadian railroad system amount to well over $5,000,000 per year. Indirect costs, moreover, would far more than double this figure, for none of the costs previously cited include damages to equipment and lading. Between 1949 and 1956, for instance, such costs amounted to more than $1,000,000 on three railroads as a direct result of landslides. One railroad reports that a single slide caused 2,640 train-hours delay. At $20 per hour out-of-pocket labor cost, this item alone amounted to more than $52,000. Delays in lading, increased icing requirements, equipment rental and intangibles must have increased this amount appreciably.

Also not included in the railroad costs listed, but still directly chargeable to a landslide, was the $609,000 reconstruction cost of an irrigation tunnel in Colorado (Fig. 100). Also involved here was the loss of farm production due to lack of water; the same slide cost the railroad almost $93,000 in repairs, plus 960 train-hours delay.

Most highway departments have experienced small landslides for which restoration costs have exceeded $50,000. Many have had other slides whose excess maintenance or correction costs have exceeded $100,000. One slide in a railroad fill required approximately 250,000 cubic yards of earth over a 40-year period to maintain a fill that was originally constructed with 15,000 cubic yards. In another case 1,000,000 cubic

yards were required to restore, temporarily, the damage caused by earth movements during 25 seconds of earth shocks. Damage in the latter case totaled approximately $2,500,000, not including losses from interruption of traffic. Later shocks in 1954 necessitated removal of 200,000 cubic yards of material. Another slide section only 233 feet long showed excess maintenance costs totaling $2,850 per year (Johnston, 1952). Another reported by the same American Railway Engineering Association committee showed excess maintenance at a rate of 5,850 man-hours per mile per year. These figures for "excess" costs represent the difference in maintenance costs before and after successful stabilization by grouting.

The values of many human lives, if they could be assessed, should be added to the figures given at the beginning of the chapter. Loss of life from landslides is small compared to other accidental causes, but it illustrates the importance of thorough investigations of possible landslides. Ladd reports in 1935: "Within the last three years landslides have resulted in more than 3,000 deaths." In 1941 a Pennsylvania rockslide destroyed a bus and killed 22 persons. After long litigations, the damage suits arising from the slide were settled in 1948 at a cost of $500,000. More recently, a rockfall in Virginia resulted in one death and two injuries. A similar occurrence in New York injured 35 people, a slip-out under seepage in Maine caused three injuries, and a landslide in Japan caused 67 deaths in addition to great property damage.

Landslide Costs as Related to Types

There are, of course, rather direct relationships between the types and sizes of landslides and the costs of treating them. The various types of landslides are described in Chapter Three and shown on Plate 1. The relations between these types and their economics are discussed in the following in general terms. It must be remembered, however, that few slides fall into simple categories, economically or geologically; rather, most of them present a complex combination of factors and each slide requires individual study.

With some notable exceptions, slides in bedrock are less important economically than are slides in soils (unconsolidated materials). This is largely because rockfalls, block glides, and rockslides tend to occur in mountainous regions where little economic damage results except to railroads, highways and public utilities. Moreover, such slides on transportation routes are generally cleaned up easily, quickly and at relatively low cost. Much the same is true for soil falls and debris slides in unconsolidated materials. Most of the exceptions to these generalizations have to do with the comparatively rare slides that cause serious interruptions to public facilities or that dam or divert watercourses.

The slide types previously mentioned are commonly localized in extent and all of them are characterized by rapid movements. Slumps, on the other hand, may be comparatively large, slow to rapid in movement, and likely to cause greater economic losses than the other types. Some bedrock slumps are related to faults in the rock; considerable damage may often result from them. Many more slumps occur in shales of one kind or another. Shale is mapped as bedrock by the geologist, but many shales have the engineering characteristics of soils, hence landslides in them should be considered with the types that occur in unconsolidated materials.

The other types of slides in soil or unconsolidated material shown on Plate 1 produce, or may produce, great damage and resultant high costs. Failure by lateral spreading, rock fragment flows, loess flows, and earth flows are all rapid movements and have resulted in severe damage and loss of life. Thus, as described more fully in Chapter Three and in the literature cited there, the dikes of Holland have failed by lateral spreading; the catastrophic slide at Elm, Switzerland, by rock fragment flow; that of

Figure 2. Landslides in cities can wipe out property values. This slump-debris flow in Seattle, Wash., began in 1951 and was still moving in 1956. In lake-deposited silts and clays, it was caused primarily by heavy rains, but overloading of a too-steep slope whose toe had been removed by man and nature was a contributing factor. The street in foreground is demolished and the houses useless; total loss of property value is more than $100,000. (Photograph by D. R. Mullineaux, U. S. Geological Survey)

Kansu, China, by loess flow; and the Quebec Province, Canada, slides by mudflows. These types are involved in most of the catastrophic slides and involve great masses of earth, but it is doubtful that the economic cost of these are as great annually as the remaining types which, although much smaller in extent, are many times greater in frequency of occurrence.

The slumps included on Plate 1, are very numerous in soil structures, particularly in highway and railroad fills, levees and dikes. Such slumps often interrupt traffic on the highway or railroad. Their widths are commonly less than 500 feet and often less than 100 feet; the mass involved in a slump movement is usually less than 50,000 cubic yards. Movements are usually at a slow to moderate rate. For example, the Illinois Highway Department lists 59 slides, with 30 involving less than 5,000 cubic yards, 28 involving 5,000 to 50,000 cubic yards, and only one greater than 50,000 cubic yards. It is this type of slide that probably has produced the highest directly assessable damage, as attested by various highway commissions, railroads, and public utilities. This type of slide after maintenance may stabilize itself for a time but, unless weakening factors are corrected or removed, it may redevelop at intervals.

Sand runs, sand and silt flows, and certain debris flows also are usually lim-

ited in extent, but are of frequent occurrence and the damage and loss to farmlands and private property along rivers and lakeshores can be very high. This loss is tied in very closely with the total losses by erosion, so that differentiation into slide loss only is extremely difficult.

In any of the slide groups previously discussed the loss is not necessarily proportional to the volume. A slump involving 10,000 cubic yards on a highway can create losses in interruptions of traffic, through delays and accidents, as great as a slide involving 50,000 cubic yards under similar conditions. Correction or maintenance cost for the latter would, of course, be proportionally greater, but the total costs might well be comparable.

Items in the Cost of Landslides

A number of items comprise the cost of landslides. The chief factors are listed in the following in order of increasing cost, followed by a general discussion of each item. The order of magnitude as given represents the considered opinion of the committee and is not, of course, established by complete cost records.

ESTIMATED AVERAGE COST OF LANDSLIDES

Item	Percent of Total
Engineering	1
Additional right-of-way or property	1
Reconstruction	20
Maintenance	38
Traffic delays, damages and indirect costs	40

This discussion is based on procedures after slides have occurred. Preventive measures can often be the most economical of any action for the correction and elimination of landslides, but it is doubtful that full success will be obtained for reasons brought forth in other parts of this volume.

The greatest cost in the engineering for either prevention or correction of slides is that involved in the subsurface investigation and laboratory testing. Successful treatment is fully dependent on complete information and this information is costly to obtain. For some jobs the engineering costs for investigation and correction may approximate the other costs involved. The books of many organizations, however, do not show breakdowns of engineering costs by individual jobs, so that it is difficult to obtain reliable cost figures for even a few representative jobs. Moreover, a great many small landslides cost little or nothing in the way of engineering beyond the cost of a quick decision in the field as to the repairs needed. For this reason, it is felt that the estimate of one percent of the total direct cost of landslides that is chargeable to engineering is a reasonable one.

Any additional right-of-way necessitated by slide encroachment is expensive. The parcels involved are usually small, but the damage has been done and is evident, hence unit prices are likely to be high. In addition, the possibilities of further damage must be considered. In the case of slope failures, however, the cost of additional land, where available, is often more economical than the measures required for correction or prevention where additional area is not available. In cities, along rivers and lakes, and for other special conditions, acquisition of additional property may not be possible or may be prohibitive in cost. Damage will be higher and corrective measures more costly. The total cost of additional land, estimated at one percent, is minor in the total cost of landslides.

Reconstruction costs constitute an appreciable portion of the total cost of landslides. For the purpose of this chapter reconstruction is defined as the permanent restoration or repair of an installation so as to permit it to serve its original function. The term does not include structures for temporary use during repair. Reconstruction work commonly involves low volume, with proportionately high unit costs. Moreover, reconstruction commonly entails partial

Figure 3. Public and private property destruction on a large scale at Nicolet, Quebec. This slide, a failure by lateral spreading in "sensitive" marine clays, took place without warning on November 6, 1955. It cost a \$2,000,000 loss in property and took three lives. The house shown was once at the level of the unbroken ground above it. Except for the greater destruction caused by this one, it is similar to many former slides along much of the St. Lawrence valley. (Photograph by Montreal Gazette)

or complete modification of the original installation, aimed at avoidance of further failures. Such refinements, too, are costly. It is estimated that reconstruction costs will amount to about 20 percent of the total cost of landslides.

Maintenance costs constitute the largest single item for which it is possible to obtain approximate figures. As understood here, maintenance involves either the keeping of the original installation in service by means of routine work and material or the taking of minor corrective measures to improve conditions. It does not include complete reconstruction on the same or new location designed as permanent correction. It is estimated that 38 percent of the total is spent in maintenance. The railroads and highway commissions have many cases on record where \$2,000 to \$10,000 per year are spent on maintenance of single sections. A few of these are enumerated in the opening paragraphs of this chapter. Average annual railway expenditures for track laying and surfacing (routine labor account) and roadway maintenance accounts have totaled approximately \$500 million in recent years. It is estimated that 4 percent of this total is excess maintenance devoted to areas of substandard stability. Many highway commissions have similar records.

If full costs could be assigned to traffic delays, property damage, and indirect effects of landslides, the cost of this item would probably exceed any other single phase of the problem. A wreck of a single train can easily result in damage to lading and equipment of \$500,000. Destruction of a single truck with lad-

ing could approach $50,000. Delay of 100 travelers for two hours could be assessed possibly at $500, and some individual travelers count their time as more valuable than this. Delay of a large transport truck can be assessed at a minimum of $14 per hour and train delays can be estimated as entailing $20 per hour direct labor cost. In addition, damages incurred by delay of equipment and lading are appreciable.

One of the largest factors in the total cost of landslides is the destruction of lands and property by slips and slides along watercourses and shores of lakes and oceans. A single slide along Lake Roosevelt in the State of Washington destroyed more than 800 acres (Figs. 31 and 32). Large areas of farmlands have been destroyed by the slides in Quebec and Switzerland that are previously mentioned. Twenty-four houses were removed from a slide area in an Oregon city. A single location along Lake Michigan caused direct monetary damage to a railroad of $250,000, including $118,000 for reconstruction.

No estimate of the yearly loss from such causes is available from the questionnaires, but observations along any watercourse will indicate the seriousness of the damage. As an example, one railroad on a 60-mile section paralleling a river has records on more than 20 slide sections that are affected annually or semiannually by fluctuating river stages. These items and many others make up the item for which the cost is estimated at 40 percent of the total.

RELATIVE COSTS OF PREVENTION, CORRECTION AND MAINTENANCE

Decisions as between preventive, corrective or maintenance methods call for the utmost judgment of the engineer; at times they call also for diplomatic skill of a high order. A few generalizations are given here; many of the details as to the place of economics in engineering decisions as to choice of method appear in Chapters Seven and Eight.

Preventive measures may be the most economical ones to take in many instances, but no one ever received credit for preventing a slide that never occurred. For this reason, perhaps, it may be difficult for an engineer to convince his superiors that preventive measures are justified, particularly if they involve large expenditures. Where safety of human life is at stake, of course, it is common practice to provide the funds required for adequate protection, regardless of the cost. Thus, rock scaling to eliminate danger of falls is practiced generally by railroads and highways. This is so, even though the cost of rock scaling may be many times that required for cleanup operations if falls were allowed to occur.

The greater the construction cost on new work the more justified are additional expenditures for prevention of slides. This holds particularly true for any installation, such as a dam, where a slide would destroy the usefulness of the structure completely. For many soil structures that are at or near critical heights in fill, however, the failures may occur during construction and can at that time be repaired at reasonable cost without interruption of service. It is not usually economical on such new construction to design slopes to insure stability over the whole project for the worst possible case to be encountered. If embankments are properly designed and constructed, however, embankment failures should be infrequent. This statement applies principally to potential failures in the fills themselves and does not apply generally to foundation failures or combinations; these are separate problems. The fills themselves tend to become stronger with age and service if original construction is adequate.

Much more study of the periodicity of slope failures in various soils and rocks is required before a full analysis of the economics of cut slopes can be made. Generally speaking, and for rock or soil cuts greater than 20 feet in depth, it is usually considered more economical to construct the slopes at angles that will be reasonably safe under most condi-

Figure 4. Shoreline erosion on Lake Michigan, U. S. Highway 12, south of St. Joseph, Mich. Shoreline property is valuable; the soilfalls shown here, caused by toe erosion during high lake levels, destroyed costly homes and threatened a major highway. Temporary (?) correction shown consists of slope-flattening, protective sand blanket on slope, and construction of groins to build up beach sand deposits at toe. (Photograph courtesy of Michigan State Highway Department)

tions rather than to design them with the expectation that no resloping will be required in the future. It must be remembered, too, that some slopes may be stable when constructed but may fail in later years through changes in soil strength.

The generalizations just stated must, of course, be applied with caution to any specific job. If local conditions are such that a cut slope will inevitably cause a slide, the engineer would be foolish indeed to design his slopes for anything less than the worst possible conditions.

Decisions between continued routine maintenance and corrective measures can be made for any installation for which there are good cost records. Briefly, it is believed that continued maintenance is justified if its annual cost is less than 5 percent of the estimated cost of any corrective measures that are expected to last for 20 years or more.

The Law on Landslides

Few legal precedents have been established to guide the courts in determining responsibility for landslides or in assessing the damages caused by them. This dearth of specific laws and legal decisions is perhaps due to two main factors — many, if not most, cases that involve private companies are settled out of court; most cases against State or Federal agencies are settled out of court or the public agency exercises its sovereign right of refusal to consent to be sued.

The following paragraphs summarize the facts on the legal situation as reported in the questionnaire by various state highway organizations and rail-

roads. They are necessarily incomplete and disconnected, but they serve to give some idea of the law and its application in various typical situations. It must be remembered, however, that just as each landslide problem must be considered on its own merits from the engineering standpoint, so must each case concerning damage from landslides be considered on its own legal merits.

Railroads

As a matter of general policy, and on the theory that the payment of fees for transportation of goods and persons implies safe transportation, the railroads generally settle claims without recourse to litigation.

State Highways

Many of the state highway departments reported no special legal problems connected with landslides. Montana and Pennsylvania, however, stated in their questionnaires that they rely on their sovereign rights, which absolve the state of all responsibility unless consent to sue is granted.

West Virginia reports a large number of claims against the highway department for removal of lateral support from private property during construction and for movements of highway embankments that led to encroachment on private property by the embankment toes. Most such claims have been settled without resort to court proceedings.

The North Carolina Highway Department reports that until recently there has been no forum for tort action. In the single case heard since the creation of such a court, it was held that no negligence was attributable to a highway employee.

Claims against it for destruction or damage of private roads, houses and other property, and for blocking of railroads, are reported by the Oregon Highway Department. None of these cases went through litigation; they were settled out of court on the basis of individual situations.

Ohio reports that its chief legal problems have to do with damage to private property that abuts the highways; where there is reasonable evidence that work on the highway has caused damages, the State commonly settles claims out of court. This appears to be the general policy throughout the country.

Toll Roads

The legal situation with respect to modern toll roads appears to have been untested up till now. As quasi-governmental organizations, the toll road commissions would appear to fall in the legal category of the state highway departments. Because they charge fees for travel, however, it may be that they will prove subject to the same legal considerations, in part, as are railroads and other private carriers.

Warning Signs

Several state highway organizations report that the state probably has no legal liability in any event for injuries to persons. The posting of warning signs may or may not absolve the state of responsibility, depending in part on local laws but in greater part on the finding of facts in each individual case. Thus, Kentucky reports that warning signs on highways are required when it may be reasonably assumed that the traveling public is confronted with a dangerous situation. Even here, however, the determination of liability depends on the factors in each case. A suit pending in Kentucky at the time this volume was written involved a claim against the highway department for failure to remove a tree from a slipping bank, the tree having fallen on a vehicle. In Ohio there are no specific laws concerning danger warnings, but roadway signs do not necessarily relieve the State of responsibility.

The Illinois Highway Department, on the other hand, reports that the Court

of Claims has held the State free from negligence where adequate warning signs have been placed. No specific law is involved, but signs warning the public of falling rocks would probably relieve the State of liability. Even here it appears that facts and proofs would govern each case in the future. New Hampshire goes even further — its highway department reports that the State is not liable for damages on any state highway; the establishment of warning signs further relieves the State of liability, but apparently such signs are not necessary under the law.

CITIES

Very few data are available as to the law concerning damage to urban property by landslides. In some cases sales of property have been voided by the courts when it was shown that the sale proceedings involved concealment of knowledge of landslide conditions. In Astoria, Ore., and probably elsewhere, certain areas have been withdrawn voluntarily from sale rather than take the risk of later damage claims due to slides.

The City of Los Angeles, Calif., has perhaps had more than its share of landslide problems; out of these troubles has come enlightened legislative action that might well be adopted elsewhere. The following five paragraphs, contributed by John T. McGill of the U. S. Geological Survey, summarize the situation in Los Angeles.

In January 1952 heavy rains resulted in millions of dollars of flood damage to private and public property in hillside areas of Los Angeles. The principal causes of damage were failure and erosion of slopes that had been graded for residential sites and subdivisions during the preceding six years. Because the cut and fill slopes were excessively steep and largely barren, it was not surprising that protective devices, many of which were improperly designed, had either failed completely or proved woefully inadequate.

Before the following winter the City of Los Angeles enacted amendments to the building code providing for the correction of already existing dangerous conditions insofar as practicable and for regulation and control of all new grading in designated hillside areas. Close supervision of new construction unfortunately has left little time for enforcement of the retroactive provisions of the amendments, and it is estimated that there are still about 10,000 hazardous cuts and fills within the city. Control of new grading through a system of mandatory permits, inspections, and certifications has proved very effective, however. The practice of constructing nearly vertical cuts and unusually high fills has been virtually eliminated. The code decrees that slopes of exposed surfaces of cuts and fills shall be no steeper than 1:1 and 1½:1, respectively, although deviations from these standard values are wisely required or permitted as local conditions warrant. Drainage from individual lots must be conducted to streets and away from cut and fill slopes. Erosion protection devices and/or erosion planting must be incorporated in all grading plans before permits will be granted. The City Department of Building and Safety has instigated a policy requiring inspection of sites prior to issuance of building permits in the hillside areas. It has been found in these inspections that on 40 percent of the building sites hazardous conditions already existed or, according to plans submitted, would have been created during construction of the proposed buildings.

For sites located in or adjacent to potential slide areas, building permits are issued only after approval by the Department of a report by a licensed civil or soil engineer giving results of detailed surface and subsurface investigations and recommendations for the design of foundations and control of drainage. The engineer, commonly in collaboration with an engineering geologist, must also locate the border of the area of stability and analyze the effects of possible sliding upon the proposed structure.

The problem of the uncompleted sub-

division has been solved by another recent amendment to the building code requiring that the developer post a bond insuring completion of grading work and pertinent improvements within a reasonable length of time and in accord with approved plans or in a manner that will not constitute a hazard.

In view of the cyclic recurrence in southern California of even greater storms than that of 1952, recommendations were being made in 1955 to the City Council for additional legislation to further eliminate and control hazardous conditions. The major provision of such legislation would authorize preparation by a competent engineering firm of a master plan for safe integrated development of the remaining hillside areas within the city.

Seattle, Wash., is another city that is plagued by landslides. During 1933 and 1934 alone, for instance, a total of 116 claim cases involving landslide damage were filed against the city. The following paragraphs, compiled by D. R. Mullineaux of the U. S. Geological Survey, summarize the legal situation in Seattle.

The city is not responsible for protecting the citizens from a landslide unless the slide is caused by some act of the city. Most suits brought against the city claim that a slide has been caused by street excavation, derangement of drainage, or a broken sewer. The city is responsible for slides due to broken sewers, even if they have been broken by an "act of God", because the sewers were originally put in place by the city.

The city is not responsible for warning citizens about slide areas, or danger from landslides. However, it does attempt to warn persons when they apply for a building permit. The city engineer's office maintains a map showing all recorded slides, and the areas of the slides are marked on the plat sheets held in the office which issues building permits. All building permit requests are checked against these sheets; if the location is in a slide area, the applicant must sign a statement which puts him on record as knowing it is a slide area and agreeing to take precautions. The building code does not prohibit building in a landslide area; the only stipulation is that the footings must reach to "solid ground." A reliable foundation engineer must be retained by the builder to determine what is solid ground.

Responsibility of the Individual

The practice followed by the Los Angeles and Seattle city governments of filing data on landslides and of requiring examinations by engineers calls attention to another problem. This is the question of the legal responsibility of the engineer or geologist who maps or predicts slides, thereby causing lower property values, or the one who has given his professional blessing to an area on which a slide has subsequently developed. Unfortunately, the committee has no direct information as to the legal situation. Judging by the Seattle and Los Angeles building codes previously mentioned, as well as those of some other cities, it appears to be proper for governmental bodies to maintain maps of unstable ground and to make them available for inspection to interested parties. Common sense also would make it appear safe for an individual or an agency to publish such maps if — and only if — it can be shown that the areas mapped as slides are indeed unstable. As a practical matter, however, extreme caution is advised, for "loss of property value" suits may well follow publication of maps or other predictions of future landslides.

Court Decisions

The following summaries of actual court decisions have resulted from a review of law reports. For further information the reader is referred to American Law Reports, Volume 107, pages 591 to 598, and to the American Law Reports Blue Books of supplemental decisions.

Most court cases involve claims for personal property damage or personal injury; no cases were found that concerned damage to land by landslide en-

croachment or by interference with drainage. The question of liability is often a difficult one to determine, because the damage results from the forces of nature. Some of these results are predictable, but more often they are unpredictable, at least in a legal sense. Such so-called "acts of God" generally excuse liability in the absence of proof of negligence in construction or maintenance.

Some railroads, however, have made out-of-court settlements for such "acts of God" as hurricanes, even though no negligence could have been legally ascribed to the carrier. Apparently the whole question of negligence in landslide cases — and even whether a specific landslide is to be considered an "act of God"— is moot in the courts.

Engineering skill and judgment are important factors in many court cases, and perhaps the determining ones in some decisions. Two examples serve to illustrate this point.

In the case of *Boskovich* versus *King County* (Wash.) (188 Wash. 63), it was held that the motorist was not entitled to recover for injuries sustained when a landslide broke loose from a steep hillside bordering the highway and struck the automobile, because there was no proof that negligence in construction or maintenance of the highway was the cause of the landslide.

In an earlier case (*Fisher* vs. *Chesapeake & Ohio Railway,* 104 Va. 635) different reasoning was followed and the railroad was held liable for injuries to one of its own employees. It was reasoned that where ordinary skill would enable engineers to foresee results and guard against them it was the railroad's duty to protect its tracks from landslides. This was based on the premise that cut as well as fill embankment for the roadbed is made by the railroad and not by natural forces; therefore, the railroad is responsible for its care and maintenance and for providing a safe place for its employees to work.

Although reaching different results, these two cases are cited because they point up the necessity for the application of engineering knowledge and skill in construction, careful maintenance, and continuous inspection.

With the exception of the Canadian Pacific Railway, the railroads reported no court actions other than the one previously cited. The following is a resumé of the findings on the Canadian Pacific case, on appeal to the Privy Council in London, as found in the reports of the Judicial Committee of the Privy Council (Law Reports Appeal Cases, 1899).

On appeal from the Supreme Court of British Columbia, the council "held, reversing the judgment of the Court below that in the absence of provisions showing an intention on the part of the Legislature to take away the appellants' right to protect their property from invasion, they were entitled to an injunction to prevent the respondents, users of the water, in disregard of their common law obligation to do no damage to the appellants' land."

This decision was rendered on appeal to the Privy Council from a decree of the Supreme Court of British Columbia October 16, 1897, dismissing an appeal from a decree of Drake, J., January 29, 1897, Supreme Court of British Columbia. The decision continues:

"At trial the judge submitted these two questions to the jury: '(1) Is the water brought by the defendants upon their land for the purpose of irrigation, the sole cause of the damage done to the plaintiff's line of railway by the slide in question? (2) Is the water brought by the defendants on their land for the purpose of irrigation the substantial cause of the damage done to the plaintiff's line of railway by the slide in question?'" The jury answered the first question in the negative and the second in the affirmative.

The trial judge, however, held that: "Irrigating the surface of his land by bringing to and passing upon it foreign water which immediately percolated to the substratum of silt, with which it mingled and then escaped from his land as liquid mud, and seriously damaged the adjoining land, was the necessary

consequence of his exercising his statutory right and did not constitute negligence or afford the owner of the adjoining land any cause for action." This decision was reversed, as previously noted.

SUMMARY

In summary, the committee cannot do better than to quote the following concluding paragraphs from Belser's excellent report (1948):

> It has been said that God gave monkeys tails, but that men had to draw their own conclusions. But one conclusion can be drawn from the state of the law with respect to the financial responsibility of traffic agencies today.
>
> What immunity from liability for inadequate traffic-devices and for improper practices traffic agencies possess exists at the sufferance of legislatures. Tort law has developed to the point where the financial responsibility of a public agency can be readily established. Social consciousness has developed to the point that the people are ready to impose liability on their governing bodies.
>
> The courts have long been straining at the bonds of precedent. Dissatisfaction with the restraining doctrines of sovereign immunity and its little half-brother, governmental functions of municipal corporations, has long been expressed by the influential text writers of our time, and by the courts themselves, even when they felt themselves bound to follow precedent on the mandates of higher courts. When those barriers are removed, those traffic agencies who have not mended their ways will be engulfed in a flood tide of pent-up litigation.
>
> Under the impact of the automobile and the increased use of the highways, through which the lifeblood of the nation runs, the states have begun to retreat from the bastion of sovereign immunity. The states have been operating highways since 1789, or since they have become states, yet it is only within the last quarter of a century that they began to make themselves liable for defects and negligence. The legislation pertaining to counties and municipalities has had a somewhat longer history, and has gone much further in the same direction.
>
> In 1946 the United States Government, the "grandpappy" of them all, passed the Federal Tort Claims Act. "The ancient principle of sovereign immunity from suit, long abandoned by the United States in the field of Contract, has been further undermined by passage of the Federal Tort Claims Act which grants to the Federal courts jurisdiction over actions against the Government for the negligence of its employees. The doctrine of immunity, inherited by this country from eighteenth century English law has been frequently attacked as an anachronism unsuited to democratic society because of the unfairness to individuals with just claims against the government."
>
> While it is not believed that the Federal government will find itself involved in many suits for traffic control deficiencies, this recognition of the social undesirability of the doctrine of sovereign immunity passed upon by the greatest law-making body of our time, representative of all the people in the nation, can be nothing if not significant of things to come. Coming events cast their shadows before.
>
> The purely verbal distinctions and logical horrors that exist in the extensive ramifications of legal doctrines thriving in the field of municipal liability and parading through the reports under the labels "governmental" and "proprietary" functions have been the subject of much comment. "A relentless barrage of unsympathetic criticism has been directed against the concept upon which the structure of the tort law of municipal corporations has been built. . . . Although critical comment appeared before 1900 widespread interest in the problem among legal commentators seems first to have been stimulated by a notable series of articles by Professor Edwin M. Borchard of the Yale University School of Law. Since that date there have appeared in the law reviews alone over two hundred leading articles and student comment on pertinent judicial decisions."

The trend is to the extension of liability. "The current of criticism has been that it is better that losses due to tort-constituting conduct shall fall upon the municipality rather than on the injured individual; and that the torts of public employees are properly to be regarded, as in other cases of vicarious liability, as a cost of administration of government, which should be distributed by taxes to the government.

"Whether as a result of this criticism or not, there is a noticeable trend in the direction of an extension of municipal tort liability, either by finding that the particular activity is not a 'governmental' one; or by discovering special reasons to take it out of the rule." And again, "The modern tendency is to restrict rather than extend the doctrine of municipal immunity. The courts and law writers are coming more and more to feel the injustice of the entire doctrine. And the tendency of courts, revolted by the hardships resulting from this doctrine in individual cases, is to introduce fictions and artificial distinctions in order to avoid the full rigor of the doctrine."

The revolt of the courts is nowhere better expressed than by Justice McGeehan in *Shaw* v. *City of New York:*

"The courts will be loath to grant immunity to a city that flagrantly flaunts scientific safeguards and experiments with untried devices of untrained, unskilled and unqualified men in this field."

References

Belser, C. H., "The Legal Responsibilities of Traffic Agencies." The Eno Foundation for Highway Traffic Control, Saugatuck, Conn., p. 62-64, 1948.

Johnston, W. T., et al., "Report of Subcommittee on Roadbed Stabilization." *Am. Ry. Eng. Assoc. Proc.*, v. 53, Table 1, p. 742, 1952.

Jones, F. O., et al., "Landslides Along the Columbia River Valley in Northeastern Washington." U. S. Geol. Survey Prof. Paper, in preparation.

Ladd, G. E., "Landslides, Subsidences and Rockfalls as Problems for the Railroad Engineer." *Am. Ry. Eng. Assoc. Proc.*, v. 36, p. 1091-1162, 1935.

Mineral Information Service, "Landslides in Ventura Avenue Oil Field." Mineral Information Service, California Div. of Mines, v. 7, no. 5, p. 1, 1954.

Chapter Three

Landslide Types and Processes

David J. Varnes[3]

It is the purpose of this chapter to review the whole range of earth movements that may properly be regarded as landslides and to classify these movements according to factors that have some bearing on prevention or control.

As defined for use in this volume the term "landslide" denotes downward and outward movement of slope-forming materials composed of natural rock, soils, artificial fills, or combinations of these materials. The moving mass may proceed by any one of three principal types of movement: falling, sliding, or flowing, or by their combinations. Parts of a landslide may move upward while other parts move downward. The lower limit of the rate of movement of landslide material is restricted in this book by the economic aspect to that actual or potential rate of movement which provokes correction or maintenance. Normal surficial creep is excluded. Also, most types of movement due to freezing and thawing (solifluction), together with avalanches that are composed mostly of snow and ice, are not considered as landslides in the sense here intended, although they often pose serious problems to the highway engineer. Such movements are not discussed because they appear to depend on factors of weather, ice physics, and thermodynamics, rather than on principles of soil mechanics or geology, and hence lie outside the province of the committee.

[3] Publication authorized by the Director, U. S. Geological Survey.

Types of Landslides

Classification

Many classifications have already been proposed for earth movements, based variously on the kind of material, type of movement, causes, and many other factors. There are, in fact, so many such schemes embedded with varying degrees of firmness in geological and engineering literature that the committee has approached the question of a "new" classification with considerable misgivings. As Terzaghi has stated (1950, p. 88), "A phenomenon involving such a multitude of combinations between materials and disturbing agents opens unlimited vistas for the classification enthusiast. The result of the classification depends quite obviously on the classifier's opinion regarding the relative importance of the many different aspects of the classified phenomenon." Each classification, including the one proposed in this volume, is best adapted to a particular mode of investigation, and each has its inherent advantages and disadvantages. However, as pointed out by Ward (1945, p. 172), "A classification of the types of failure is necessary to the engineer to enable him to distinguish and recognize the different phenomena for purposes of design and also to enable him to take the

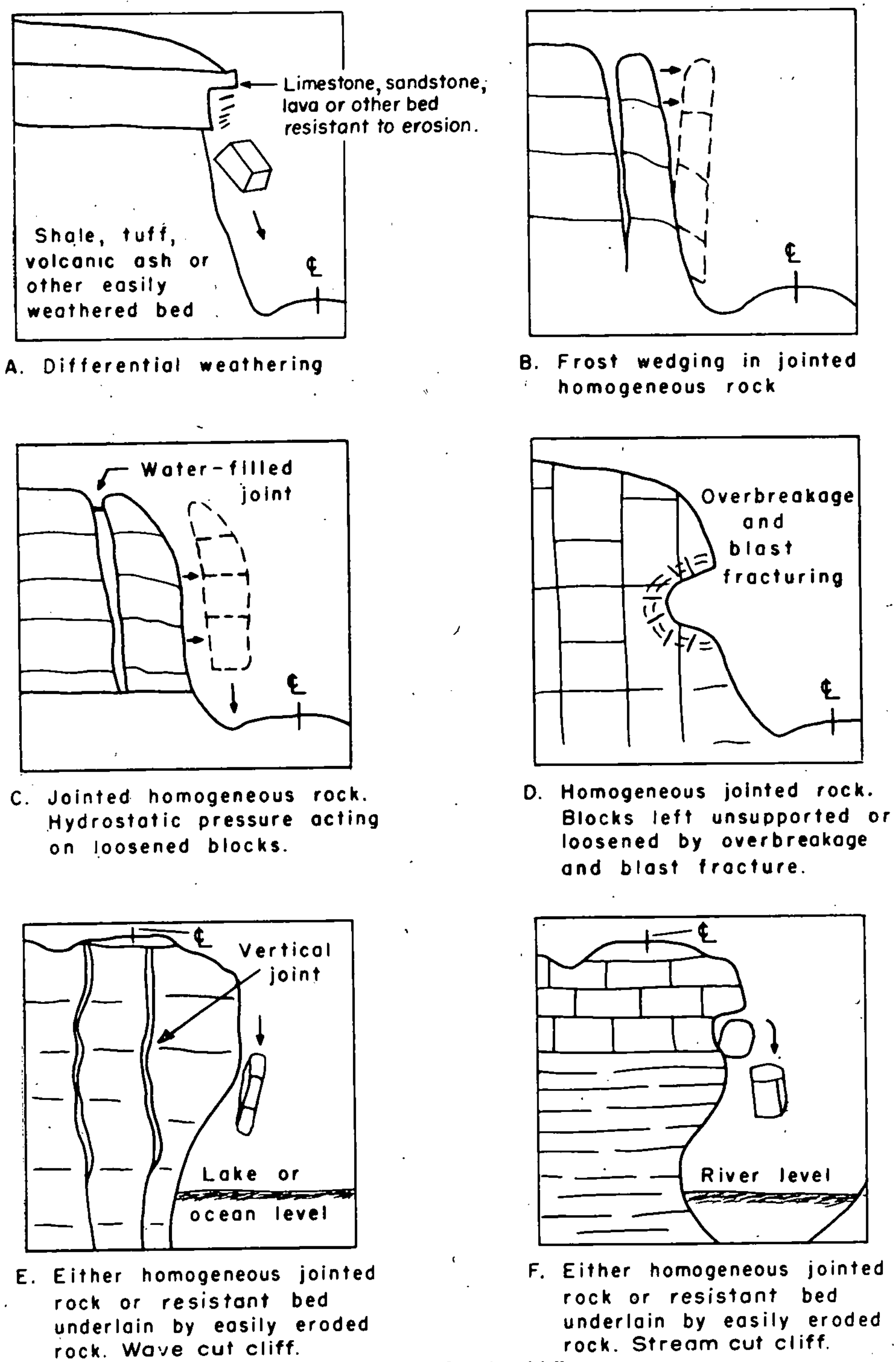
appropriate remedial or safety measures where necessary. The geographer and geologist need a classification so that they may interpret the past and predict the present trends of topography as revealed by their observations."

The classification adopted here is shown in Plate 1 and is further described in the following paragraphs. Definitions of the parts of a landslide appear in Plate 1-t. An abbreviated version, without diagrams and explanatory text, is shown in Figure 5. In preparing this classification of landslides, a deliberate effort has been made to set it up according to features that may be observed at once or with a minimum of investigation, and without reference to the causes of the slides. Two main variables are considered: (a) the type of material involved, which usually is apparent on inspection or preliminary boring; and (b) the type of movement, which usually may be determined by a short period of observation or by the shape of the slide and arrangement of debris. In its emphasis on type of movement the classification resembles, more than any other, that proposed by Sharpe (1938) for landslides and other related movements.

The chart (Pl. 1) shows examples of slides by small drawings. The type of material involved is indicated by the horizontal position of the drawing within the chart; the type of movement is indicated by the vertical position of the drawing. Water content of flow-type landslides is indicated by the relative vertical position of the drawing within the flow group. Each drawing also has a note giving the general range of velocity of movement of the landslide type, according to the scale of velocities at the bottom of the chart (Pl. 1-u).

<table>
<tr><th colspan="2" rowspan="2">TYPE OF MOVEMENT</th><th colspan="4">TYPE OF MATERIAL</th></tr>
<tr><th colspan="2">BEDROCK</th><th colspan="2">SOILS</th></tr>
<tr><td colspan="2">FALLS</td><td colspan="2">ROCKFALL</td><td colspan="2">SOILFALL</td></tr>
<tr><td rowspan="2">SLIDES</td><td>FEW UNITS</td><td>ROTATIONAL
SLUMP</td><td>PLANAR
BLOCK GLIDE</td><td>PLANAR
BLOCK GLIDE</td><td>ROTATIONAL
BLOCK SLUMP</td></tr>
<tr><td>MANY UNITS</td><td></td><td>ROCKSLIDE</td><td>DEBRIS SLIDE</td><td>FAILURE BY LATERAL SPREADING</td></tr>
<tr><td rowspan="5">FLOWS</td><td></td><td rowspan="2">ROCK FRAGMENTS</td><td colspan="3">ALL UNCONSOLIDATED</td></tr>
<tr><td></td><td>SAND OR SILT</td><td>MIXED</td><td>MOSTLY PLASTIC</td></tr>
<tr><td>DRY</td><td>ROCK FRAGMENT FLOW</td><td>SAND RUN   LOESS FLOW</td><td></td><td></td></tr>
<tr><td></td><td></td><td>RAPID EARTHFLOW</td><td>DEBRIS AVALANCHE</td><td>SLOW EARTHFLOW</td></tr>
<tr><td>WET</td><td></td><td>SAND OR SILT FLOW</td><td>DEBRIS FLOW</td><td>MUDFLOW</td></tr>
<tr><td colspan="2">COMPLEX</td><td colspan="4">COMBINATIONS OF MATERIALS OR TYPE OF MOVEMENT</td></tr>
</table>

Figure 5. Classification of landslides, abbreviated version (see plate 1 for complete chart with drawings and explanatory text).



A. Differential weathering

B. Frost wedging in jointed homogeneous rock

C. Jointed homogeneous rock. Hydrostatic pressure acting on loosened blocks.

D. Homogeneous jointed rock. Blocks left unsupported or loosened by overbreakage and blast fracture.

E. Either homogeneous jointed rock or resistant bed underlain by easily eroded rock. Wave cut cliff.

F. Either homogeneous jointed rock or resistant bed underlain by easily eroded rock. Stream cut cliff.

Figure 6. Examples of rockfalls.

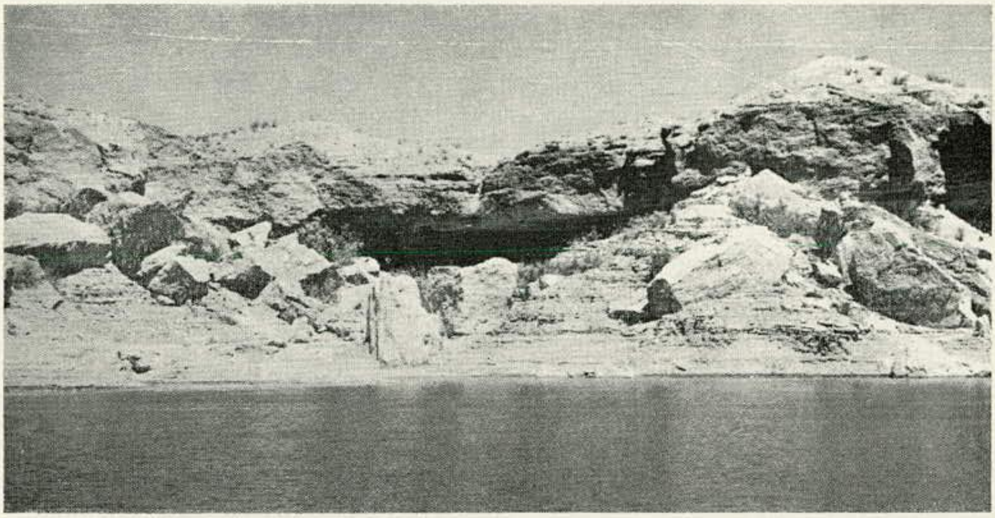


Figure 7. Rockfall due to undercutting along shore of Las Vegas Bay, Lake Mead, Nev. The rock is the Muddy Creek formation (Pliocene?) consisting here of siltstone overlain by indurated breccia. The movement is straight down by gravity, in contrast to rockslide, which slides on a sloping surface. (Photograph by U. S. Bureau of Reclamation, February 24, 1949)

Materials are classed, for falls and slides, into bedrock and soils. The term "soils" is used in the engineering sense and includes clastic material, rock fragments, sheared or weathered bedrock, and organic matter. Falls and slides involving bedrock are shown in the upper left part of the chart; those involving soils are shown in the upper right part of the chart. The materials of flows are grouped into various categories. Material is classified according to its state prior to initial movement, or, if the type of movement changes, according to its state at the time of the change to the new type of movement.

Types of movement are divided into three principal groups — falls, slides, and flows. A fourth group, complex slides, is a combination of any or all of the other three types of movement. There may be, of course, variations in the type of movement and in the materials from place to place, or from time to time, in an actual landslide, so that a rigid classification is neither practical nor desirable.

Type I — Falls

Falls are very common. In rockfall and soilfall, the moving mass travels mostly through the air by free fall, leaping, bounding, or rolling, with little or no interaction between one moving unit and another (Pl. 1-a, b). Movements are very rapid to extremely rapid (see rate of movement scale, Pl. 1-u) and may or may not be preceded by minor movements. Several varieties of rockfall are illustrated in Figures 6 and 7.

Type II — Slides

In true slides, the movement results from shear failure along one or several surfaces, which are either visible or may reasonably be inferred. Two subgroups of slides may be distinguished according to the mechanics of movement — those in which the moving mass is not greatly deformed (Type IIA), and those which are greatly deformed or consist of many small units (Type IIB). The Type IIA group includes the familiar slump

or rotational shear types of slides; it includes also undeformed slides along more or less planar surfaces, for which the term "block glides" is here proposed. Type IIB slides include most rockslides, debris slides, and failures by lateral spreading.

A — Relatively Undeformed Material

Type IIA slides are made up of one or a few moving units. The maximum dimension of the units is greater than the relative displacement between units and is comparable to or greater than the displacement of the center of gravity of the whole mass. Movement may be structurally controlled by surfaces of weakness, such as faults, bedding planes, or joints.

Slumps. — The commonest examples of Type IIA or undeformed slides are slumps. Slumps, and slumps combined with other types of movement, make up a high proportion of the landslide problem facing the highway engineer. The movement in slumps takes place only along internal slip surfaces. The exposed cracks are concentric and concave toward the direction of movement. In many slumps the underlying surface of rupture, together with the exposed scarps, is spoon-shaped (see Fig. 8). If the slide

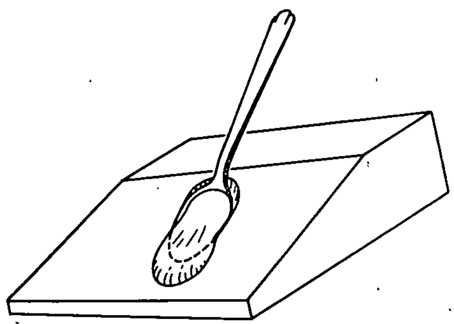


Figure 8. Spoon-shaped slope failure. Slope failures are often spoon-shaped, as in this sketch, or cylindrical as shown in Figure 9.

extends for a considerable distance along the slope perpendicular to the direction of movement, much of the rupture surface may approach the shape of a sector of a cylinder whose axis is parallel to the slope (see Fig. 9). In slumps the movement is more or less rotary about an axis that is parallel to the slope. The top sur-

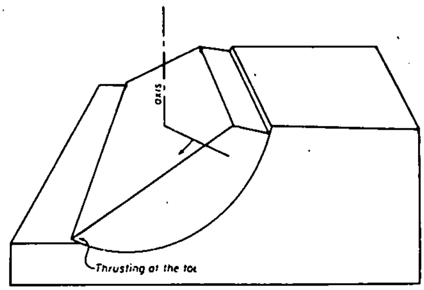


Figure 9. Rotational shear on cylindrical surface.

face of each unit tilts backward toward the slope (see Figs. 9, 10, 11, and 12, and Pl. 1-c and 1-h).

Figure 10 illustrates some of the commoner varieties of slump failure in various kinds of material. Figure 12 shows the backward tilting of strata exposed in a longitudinal section through a small slump in lake beds. Although the surface of rupture of slumps is a concave-upward curve, it is seldom a circular arc of uniform curvature. Often the shape of the curve is greatly influenced by faults, joints, bedding, or other pre-existing discontinuities in the material. The influence of such structures must be considered very carefully when the engineer makes a slope stability analysis that assumes a certain configuration for the surface of rupture. Figures 12 and 13 illustrate how the surface of rupture may follow bedding planes for a considerable part of its length. Upward thrusting and slickensides along the lateral margin of the toe of a slump are shown in Figure 14.

The scarp at the head of a slump may be almost vertical. If the main mass of the slide moves down very far, the steep scarp is left unsupported and the stage is set for a new failure at the crown of the slide similar to the original slump. Occasionally the scarps along the lateral margins of the upper part of the slide

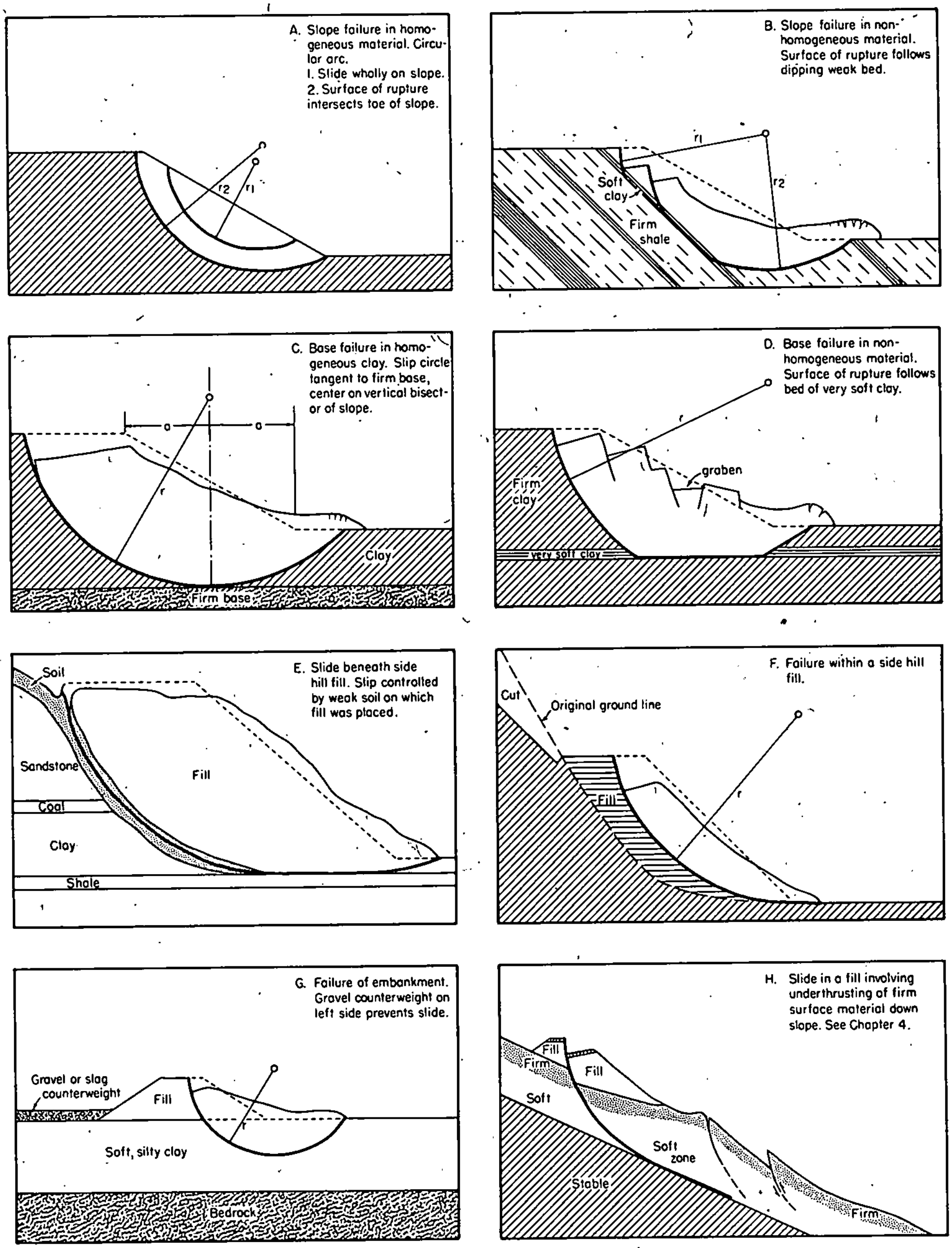


Figure 10. Some varieties of slump.

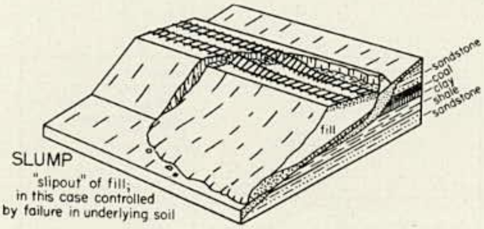


Figure 11. Slump. "Slipout" of fill; in this case controlled by failure in underlying soil.

may also be so high and steep that slump blocks break off along the sides and move downward and inward toward the middle of the main slide. Figure 15 shows, in plan, such an unusual type of slump units along the upper margins of a slide; the longest dimensions of these units are parallel with, rather than perpendicular to, the direction of movement of the main slide.

Any water that finds its way into the head of a slump may be ponded by the backward tilt of the unit blocks or by other irregularities in topography, so that the slide is kept wet constantly. By the successive creation of steep scarps and trapping of water, slumps often become self-perpetuating areas of instability and may continue to move and enlarge intermittently until a stable slope of very low gradient is attained. Material in the lower parts of slumps may become so greatly broken or churned up that the toe advances as an earthflow or debris slide with a type of motion distinct from slumping at the head. The combination of slump and earthflow, as illustrated in Figures 16, 28, and 47, and Plate 1-h, occurs frequently. Slumping movement does not generally proceed with more than moderate velocity unless the toe is in water or unless flowing movements remove material as fast as it is brought down from above.

Block Glides. — Not all Type IIA slides have the characteristic form and

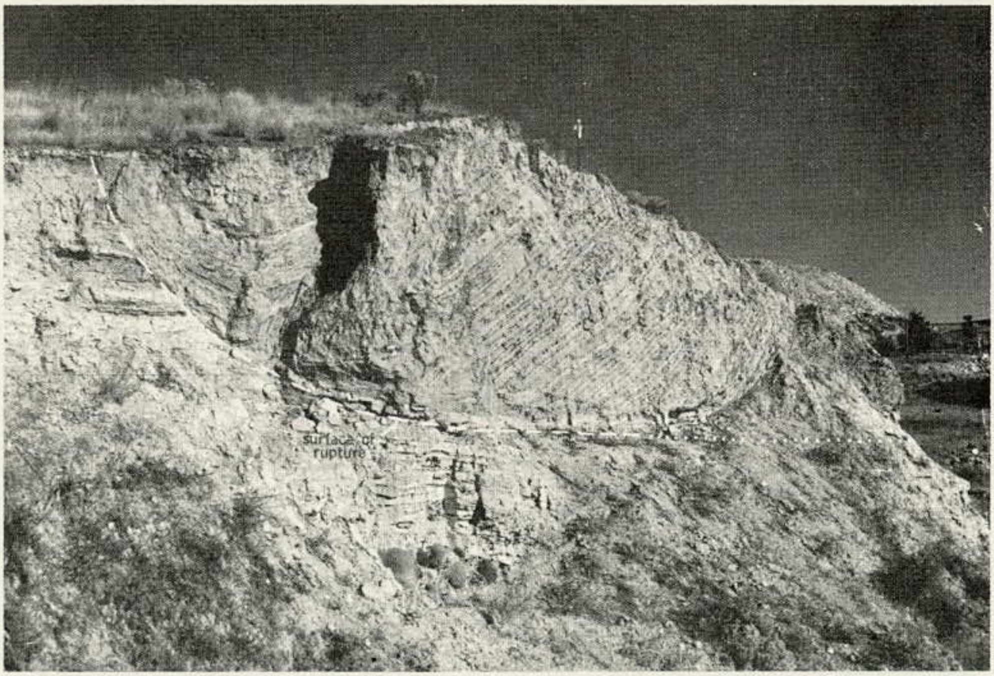


Figure 12. Slump in thinly bedded lake deposits of silt and clay in the Columbia River valley. Note backward tilting of beds above surface of rupture. (Photograph by F. O. Jones, U. S. Geological Survey)

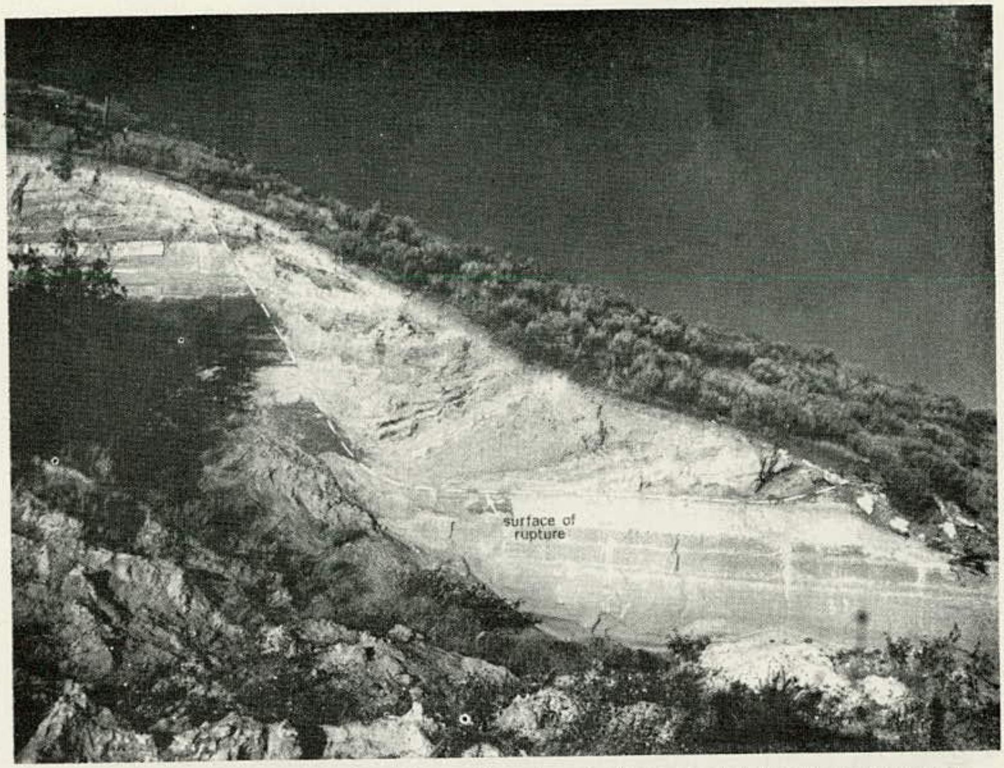


Figure 13. Slump in bedded lake deposits similar to those shown in Figure 12. Note how the surface of rupture follows a horizontal bedding plane for part of its length. (Photograph by F. O. Jones, U. S. Geological Survey)

rotary movement of a slump. In some, the mass progresses out, or down and out, as a unit along a more or less planar surface, without the rotary movement and backward tilting characteristic of slump. The moving mass may even slide out on the original ground surface. The term "block glide" is here applied to Type IIA slides of this kind. The need for distinguishing this type of slide from slump arises partly from restriction of slump to movement that is only along internal slip surfaces that are generally concave upward. But the distinction is useful also in planning control measures. The rotary movement of a slump, if the surface of rupture dips into the hill at the foot of the slide, tends to restore equilibrium in the unstable mass; the driving moment, therefore, decreases and the slide stops moving. A block glide, however, may progress indefinitely if the surface on which it rests is sufficiently inclined and as long as the shear resistance along this surface remains lower than the more or less constant driving force. Several examples of block glides are illustrated in Figures 17, 18, 19, and 38, and Plate 1-d, e, f, and g. Plate 1-g is an example suggested by the Iowa State Highway Commission. Block glides, alone or in combination with other types of movement, are probably quite common, although they seem to have attracted little attention in the literature.

B — Greatly Deformed Material

Type IIB landslides comprise those in which the movement is by sliding but the material is deformed or breaks up

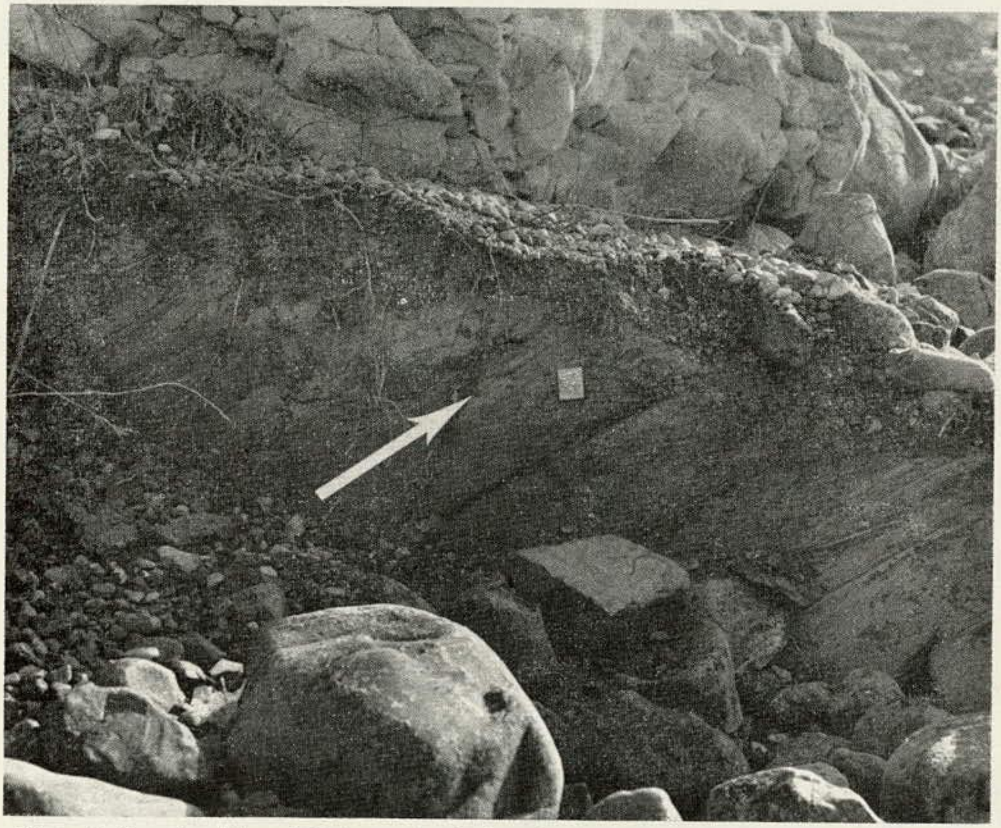


Figure 14. Upward and forward displacement at the lateral margin of the toe of a slump, right bank of the Columbia River downstream from Grand Coulee Dam, Wash. Slickensides are on the active toe, which consists of clay and silt overlain by river gravel. Arrow shows movement of toe relative to stable area in foreground. (Photograph by U. S. Bureau of Reclamation)

into many more or less independent units. With continued deformation and disintegration, especially if the water content or velocity — or both — increases, a Type IIB slide may change to a Type III flow. All gradations exist. The maximum dimension of the units is comparable to or less than the relative displacement between them, and generally much smaller than the displacement of the center of gravity of the whole mass. Movement is controlled, perhaps more frequently than in slumps, by pre-existing structural features, such as faults, joints, bedding planes, or variations in shear strength between layers of detritus. Movement often progresses beyond the limits of the original surface of rupture, so that parts of the mass may slide out over the original ground surface. Rate of movement ranges within wide limits among the several varieties of Type IIB slides and may vary greatly from one time to another in the development of a single slide.

Rockslides and Debris Slides. — Loose rockslides are a common variety of Type IIB slide consisting of many units (see Figs. 20, 41, and 94, and Pl. 1-i). Various kinds of slides involving natural soil, unconsolidated sedimentary material, and rock detritus are included as debris slides under Type IIB. Examples of these are illustrated in Figures 21, 22, and 23, and Plate 1-j. These slides are often limited by the contact between loose

Zone A. Movement chiefly by large-scale slumping along slip surfaces.

a, a', a" Principal slump units.

b, b' Narrow slump units with axes perpendicular to axes of main slump units and parallel with the length of the main slide.

c "Island" remaining after downward movement of unit d from area e.

Zone B. Zone of earthflow. Movement chiefly by flowage.

Zone C. Toe of slide area. Original form altered by railroad reconstruction work.

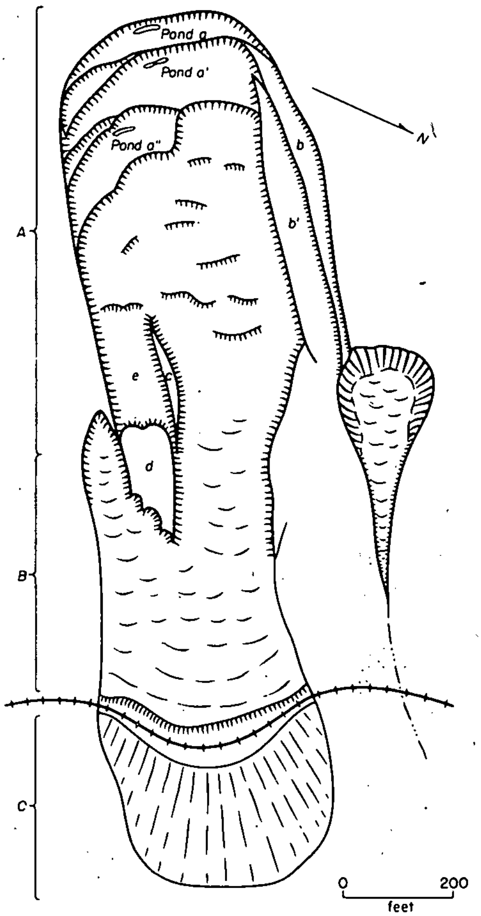


Figure 15. Ames slide near Telluride, Colo. This slump-earthflow landslide occurred in glacial till that overlies Mancos shale. Repeated slumping took place along the upper margins after the main body of material had moved down. Note that the long axes of slump blocks b and b' are parallel with rather than perpendicular to the direction of movement of the main part of the slide. Blocks b and b', however moved toward the left, rather than toward the observer. (See Varnes, Helen D., 1949)

material and underlying firm bedrock. With increase in water content or with increasing velocity, debris slides grade into the flowing movement of debris avalanches.

Failures by Lateral Spreading. — The slide shown in Plate 1-k is due to lateral spreading of soft clay from beneath firmer material. Related types of failure are described by Newland (1916) and by Terzaghi and Peck (1948, pp. 368-369, 401-404). In most places the failures take place along zones of high pore-water pressure in homogeneous clay or along partings of sand or silt in clay. The movement in these types of slides is usually complex, involving translation, breaking up of the material, some slumping, and some liquefaction and flow. These failures are arbitrarily classed with deformed slides rather than with flows because the material in motion generally slides out on a more or less planar surface, and in doing so it may break up into a number of semi-inde-

Figure 16. Aerial view of the Cedar Creek slide near Montrose, Colo. The landslide in the foreground is moving to the right and consists of slumps with earthflows at the toe. The material is Mancos shale overlain by 10 to 20 feet of gravel, which caps the mesa on the left. The original railroad alignment is completely destroyed and the new alignment is being covered by earthflows. (See Varnes, Helen D., 1949) (Photograph by R. W. Fender, Montrose, Colo.)

pendent units. The dominant movement is translation rather than rotation. If the underlying mobile zone is thick, the blocks at the head may sink downward

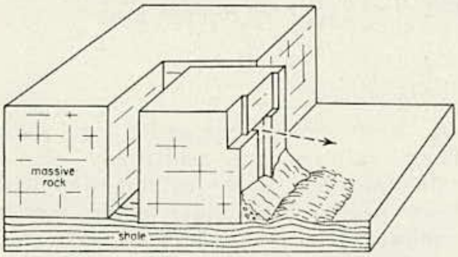


Figure 17. Block glide. Slide at a quarry face.

as grabens, not necessarily with backward rotation, and there may be upward and outward extrusion and flow at the toe. Movement generally begins suddenly, without appreciable warning, and proceeds with a rapid to very rapid velocity; but there are also some cases of slow movement (see Fig. 115), or of slow movement preceding sudden failure.

These kinds of slides appear to be members of a gradational series of landslide types in surficial materials extending from block glides at one extreme, in which the zone of flowage beneath the sliding mass may be very thin, to earthflows or completely liquefied mudflows at the other extreme, in which the zone of

flowage includes the whole mass. The form taken depends upon local factors. Most of the larger landslides in glacial sediments of northern North America and Scandinavia lie somewhere within this series (see Fig. 3).

The large and sometimes disastrous landslides in Sweden and Norway have stimulated much excellent study. In a recent summary, W. Kjellman (1955) states that lateral spreading, although possible, has not been proved for any Swedish landslide. The slides are successive, however, in that they grow rapidly while moving. If the slide grows in the direction of its own motion it is termed "progressive." One that grows in the opposite direction (headward) is called "retrogressive." Kjellman gives a step-by-step analysis of progressive failure. He discounts the statements that quick or sensitive clay — that is, a clay which loses practically all its shear strength if disturbed — is a main cause.

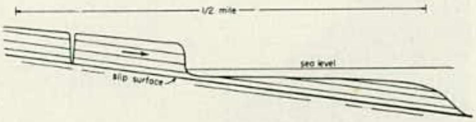


Figure 18. Block glide. Body of sliding bedrock at Point Fermin, Calif. (see also Figure 19). Maximum average rate of movement 0.1 foot per week. (From Miller, 1931)

Figure 19. Block glide at Point Fermin, near Los Angeles, Calif. The photograph indicates minor slumping into the gap at the rear of the main mass and imminent rockfalls at the sea-cliff. The principal motion, however, is by gliding along gently seaward dipping strata. (Photograph by Spence Air Photos)

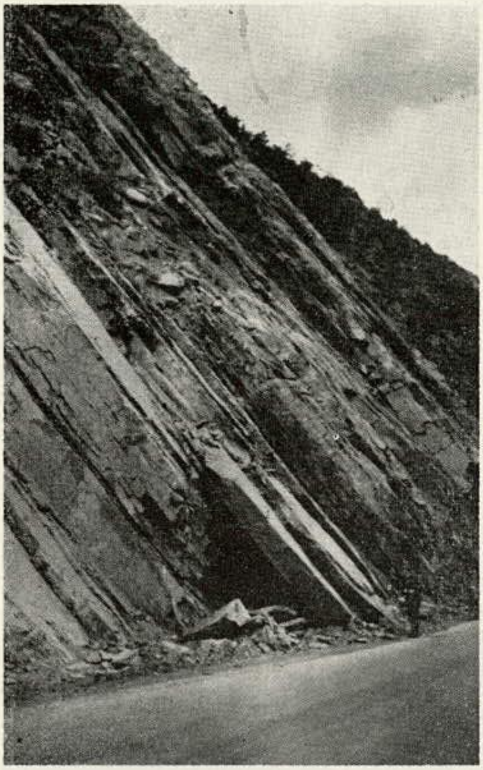


Figure 20. Small rockslide on dipping sandstone strata near Glenwood Springs, Colo. Slide controlled primarily by dip of beds toward road. (Photograph by D. J. Varnes, U. S. Geological Survey)

Odenstad (1951) gives an analysis of retrogressive failure in the landslide at Sköttorp on the Lidan River (see Fig. 24).

In a summary of Norwegian investigations, L. Bjerrum (1955) re-emphasizes the importance of sensitivity in leached marine clay and concludes that failure is not successive but instantaneous over the whole sliding surface.

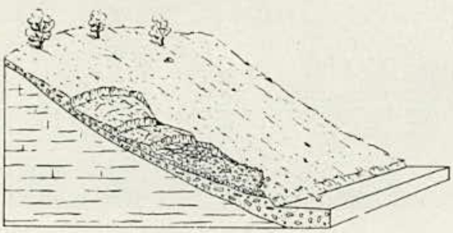


Figure 21. Debris slide of the soil disintegrating slip variety. (After Kesseli, 1943)

All investigators would agree that failures in glacial and marine sediments of Pleistocene age present some common and characteristic features. Among these are: sliding, which often exists for no apparent external reason; generally sudden failure (see Fig. 3); instability of very gentle slopes; dominant movement by translation; and importance of pore-water pressure in creating instability. All degrees of disturbance of the masses have been observed; some slides consist almost entirely of one large slab or "flake," others liquefy almost entirely to small chunks or mud.

Type III — Flows

In flows, the movement within the displaced mass is such that the form taken by the moving material or the apparent distribution of velocities and displacements resembles those of viscous fluids. Slip surfaces within the moving mass are usually not visible or are short-lived, and the boundary between moving and stationary material may be sharp or it may be a zone of plastic flow. The material is, by necessity, unconsolidated at the time of flow but may consist of rock fragments, fine granular material, mixed debris and water, or plastic clay. As indicated in Plate 1, there is a continuous sequence from debris slide through debris avalanche to debris flow as solid material composed of mixed rock, soil, or detritus takes on more water. Earthflows in plastic or predominantly fine-grained material become mudflows at higher water content.

Dry Flows. — The word "flow" naturally brings water to mind, and some content of water is necessary for most types of flow movement. But there have been a surprising number of large and catastrophic landslides, which flowed according to the foregoing definition yet were nearly or quite dry. Therefore, the classification of flows on the chart indicates the complete range of water content from dry at the top to liquid at the bot-

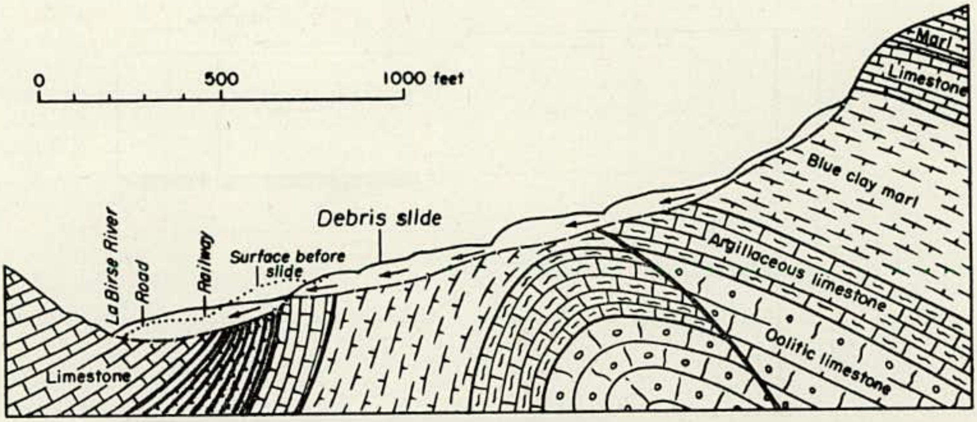


Figure 22. Debris slide, Moutier Court Gorges, Switzerland. Slide is composed of weathered blue clay marl, talus, and older slide material. (After Buxtorf and Vonderschmitt in Peter, A., 1938)

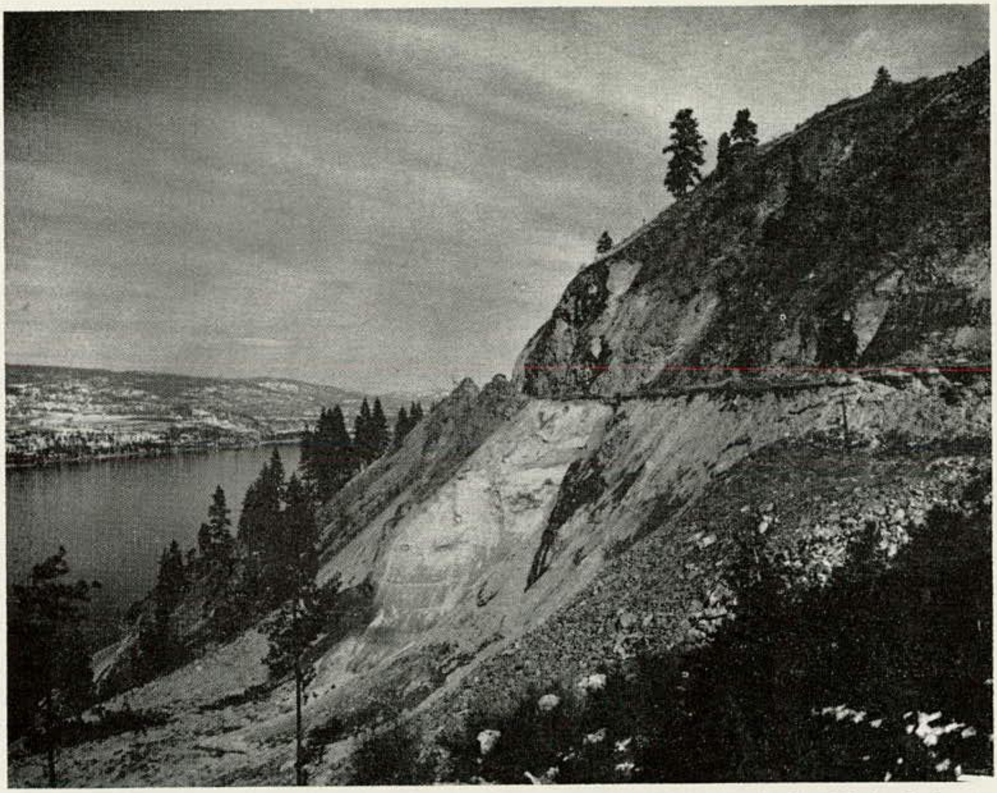


Figure 23. Debris slide along the Great Northern Railway near Kettle Falls, Wash. The slide involved unconsolidated sediments and talus and was limited by the contact between light-colored materials, exposed in the scarp at center of photo, and darker firm bedrock. The slide passed over the highway at the base of the slope and into Lake Roosevelt, creating a destructive wave. The slide has been corrected, at least temporarily, by clearing the roadway of fallen material; that is, partial excavation of the toe. (Photograph by F. O. Jones, U. S. Geological Survey)

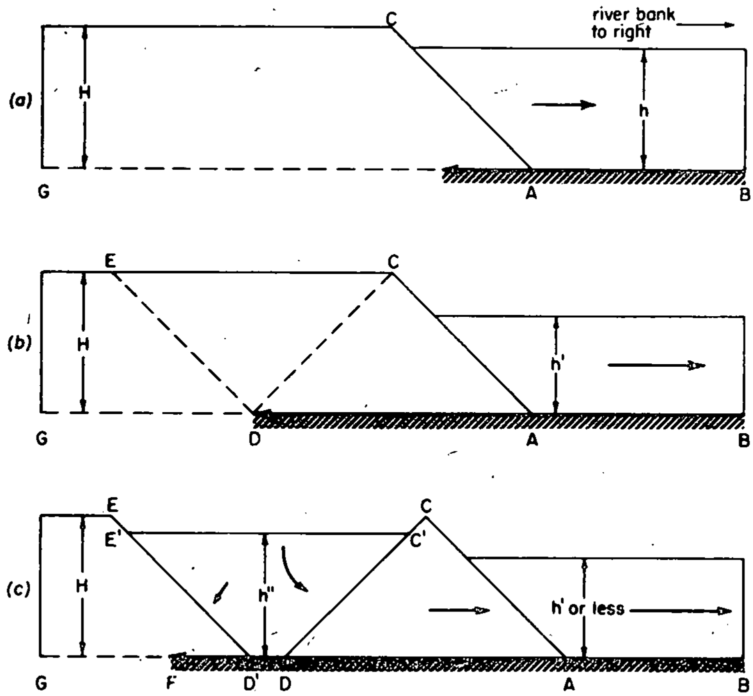


Figure 24. Retrogressive failure, landslide at Skottorp, Sweden, according to Odenstad (1951). Failure in sensitive clay began at the river bank and spread landward along a particularly weak surface BG at a depth H below ground surface. At the stage shown in drawing (a), a secondary slip surface has developed along AC, but the block to the left of A is still stable, being supported in part by the material to the right of A. Height h decreases as the material to the right of A moves out; also the failure surface continues to spread to the left, as in drawing (b). When height h has decreased to a critical value h′, complementary slip surfaces develop along CD and ED and wedge CDA moves to the right, drawing (c). Wedge E′C′DD′ deforms and moves down and to the right. The process is repeated when the height h″ of wedge E′C′DD″ decreases to the critical value h′.

tom. The horizontal position within the chart indicates the type of unconsolidated material, whether it is mostly rock fragments, sand, silt, or nonplastic material, mixed rock and soil, or mostly plastic. Blank spaces within this part of the chart indicate incompatible combinations, such as dry plastic material, or combinations for which there are no known examples of flows.

Dry flows that consist predominantly of rock fragments are here termed rock fragment flows. They may originate in two ways — by volcanic explosion, or by a large rockslide or rockfall turning into a flow. The latter two varieties are termed rockslide avalanche and rockfall avalanche, respectively. Clear-cut examples of rock fragment flows resulting from volcanic explosion are not known in North America. The "glowing cloud" or "nuée ardente" eruptions of very hot ash are not regarded as landslides. The remarkable flow at Bandaisan, Japan (Sekiya and Kikuchi, 1889, p. 109), appears to be, however, a true example of a volcanic rock fragment flow. The landslide that occurred in 1925 along the Gros Ventre Valley in Wyoming (Alden, 1928) is an example of a rockslide that turned into a flow.

Rockfall avalanches are most common in rugged mountainous regions. The disaster at Elm, Switzerland (Heim, 1932, pp. 84, 109-112), which took 115 lives, started with small rockslides at

Figure 25. Dry flow of silt. Material is lake bed silt of Pleistocene age from a high bluff on the right bank of the Columbia River, 2 1/2 miles downstream from Belvedere, Wash. Flow was not observed while in motion, but is believed to result from blocks of silt falling down slope, disintegrating, forming a single high-density solid-in-air suspension, and flowing out from the base of the cliff. (Photograph by F. O. Jones, U. S. Geological Survey)

each side of a quarry on the mountainside. A few minutes later the whole mass of rock above the quarry crashed down and shot across the valley. The movement of the rock fragments, which had to this moment been that of rockslide and rockfall, now took on the character of a flow. The mass rushed up the other side of the small valley, turned and streamed into the main valley and flowed for nearly a mile at high velocity before stopping (see Pl. 1-l). About 13,000,000 cubic yards of rock descended an average of 1,450 feet vertically, in a total elapsed time of about 55 seconds. The kinetic energy involved must have been enormous. The flowing motion can perhaps be explained by assuming much internal interaction between the rock fragments and between them and entrapped highly compressed air, so that the whole mass became a density current of high gravity and unusual velocity. A similar and even larger rockfall avalanche occurred at Frank, Alberta, in 1903, also with great loss of life and property (McConnell and Brock, 1904). Such flows probably cannot be produced by a few thousand or a few hundred thousand cubic yards of material. Many millions of tons are required, and when that much material is set in motion, perhaps even slowly, predictions of behavior based on past experience with small failures become very questionable. Perhaps the best way to study such

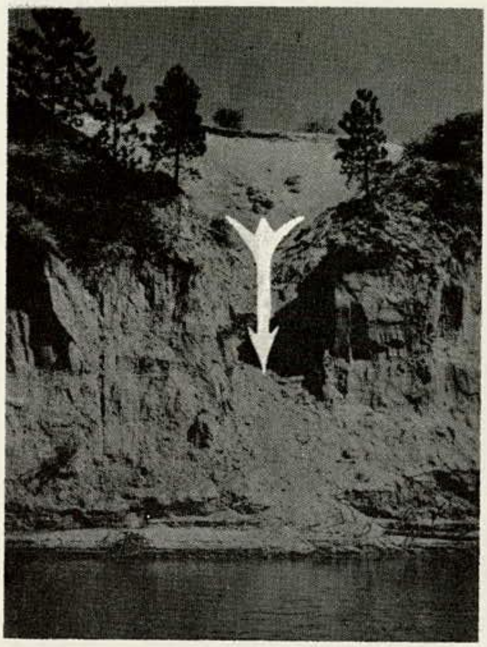


Figure. 26. Sand run. Material is sand over lake bed silt, Columbia River valley. Dry sand from upper part of terrace flowed like a liquid through notch in more compact sand and silt below. (Photograph by F. O. Jones, U. S. Geological Survey)

failures is by models, which are small enough to comprehend with the eye and mind, and constructed with due regard for the great decrease in the strength and other physical properties of the materials as required by scale factors determined through dimensional analysis.

From the meager accounts available, somewhat the same mechanism as operated at Elm produced the loess flows that followed the 1920 earthquake in Kansu Province, China (Close and McCormick, 1922), shown in Plate 1-n. Apparently, the normal fairly coherent internal structure of the porous silt was destroyed by earthquake shock, so that, for all practical purposes, the loess became a fluid suspension of silt in air and flowed down into the valleys, filling them and overwhelming villages. Small flows of dry silt, powdered by impact on falling from a cliff, have been recognized; but as far as is known, none have been studied in detail (see Fig. 25). The well-known fluidlike motion of dry sand, as illustrated in Plate 1-m and Figure 26, needs no comment.

Wet Flows. — Other types of flows, shown in Plate 1-o to s, require water in various proportions. The gradations between debris slide and debris flow reflect very largely the differences in water content, although material of a given water content may slide on a gentle slope but flow on a steeper slope. Debris slides and, less commonly, debris avalanches may have slump blocks at their heads. In debris slides, the moving mass breaks up into smaller and smaller parts as it advances toward the foot, and the movement is usually slow. In debris avalanches, progressive failure is more rap-

Figure 27. Debris avalanche or debris flow, Franconia Notch, N. H. This landslide occurred June 24, 1948, after several days of heavy rainfall. Only soil mantle, 10 to 15 feet thick, which lay over bedrock on a slope of about 1:1, was involved. The slide scar is about 1,500 feet long. Note natural levees along sides of flow. U. S. Route 3 is in the foreground. (Photograph courtesy of New Hampshire Department of Public Works and Highways)

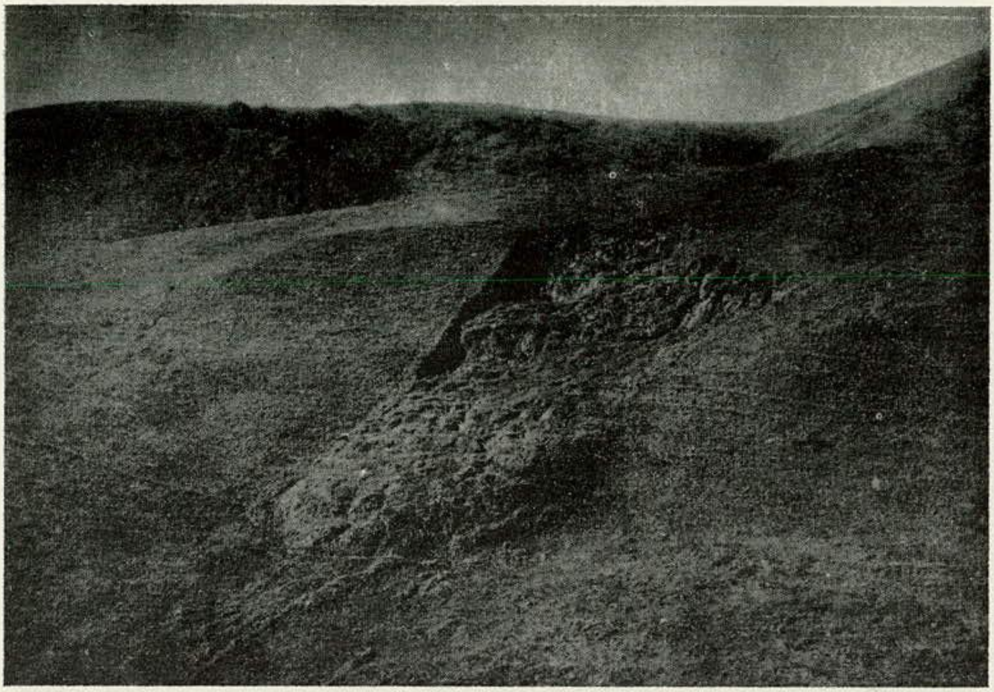


Figure 28. Earthflow developing from slump near Berkeley, Calif. (Photograph by G. K. Gilbert, U. S. Geological Survey)

id and the whole mass, either because it is quite wet or is on a steep slope, flows and tumbles downward, commonly along a stream channel, and advances well beyond the foot of the slope. Debris avalanches are generally long and narrow and often leave a serrate or V-shaped scar tapering uphill at the head, as shown in Figure 27, in contrast to the horseshoe-shaped scarp of a slump (see Fig. 104).

Debris flows, called mudflows in some other classifications, are here distinguished from the latter on the basis of particle size. That is, the term "debris flow" as used here denotes material that contains a relatively high percentage of coarse fragments, whereas the term "mudflow" is reserved for material with at least 50 percent sand, silt, and clay-size particles. Debris flows almost invariably result from unusually heavy precipitation or from sudden thaw of frozen soil. The kind of flow shown in Plate 1-r often occurs during torrential runoff following cloudbursts. It is favored by the presence of deep soil on mountain slopes from which the vegetative cover has been removed by fire or other means; but the absence of vegetation is not a necessary prerequisite. Once in motion, a small stream of water heavily laden with soil has transporting power out of all proportion to its size; and as more material is added to the stream by sloughing its size and power increase. These flows commonly follow pre-existing drainage ways, incorporating trees and bushes, and removing everything in their paths. Such flows are of high density, perhaps 60 to 70 percent solids by weight, so that boulders as big as an automobile may be rolled along. If such a flow starts on an unbroken hillside it will quickly cut a V-shaped channel. Some of the coarser material will be heaped at the side to form a natural levee, while the more fluid part moves

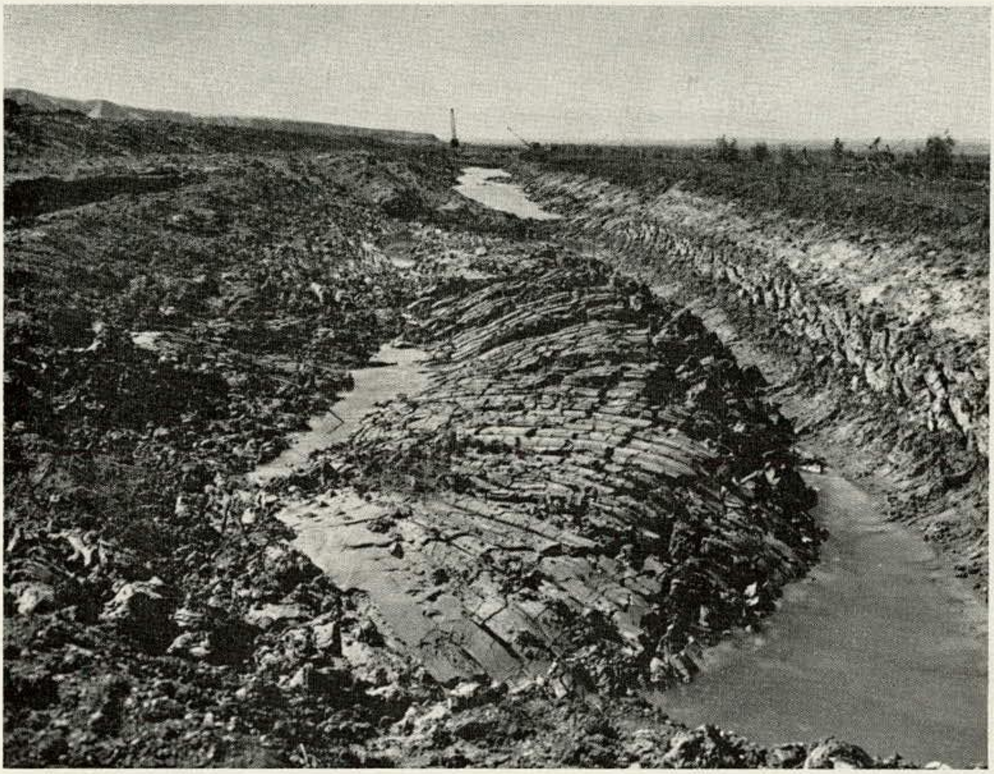


Figure 29. Upthrust toe of a slump-earthflow resulting from failure of a canal levee on Middle Rio Grande Project, N. Mex. The raised toe is about 5 feet high and 200 feet long. (Photograph by U. S. Bureau of Reclamation)

down the channel (see Fig. 27). Flows may extend many miles, until they drop their loads in a valley of lower gradient or at the base of a mountain front. Some debris flows and mudflows have been reported to proceed by a series of pulses in their lower parts; these pulses presumably are caused, in part, by periodic damming and release of debris.

An earthflow is a flow of slow to very rapid velocity involving mostly plastic or fine-grained nonplastic material. The slow earthflow shown in Figure 28 and Plate 1-p may be regarded as typical of an earthflow resulting from failure of a slope or embankment. The failure follows saturation and the building up of pore-water pressure so that part of the weight of the material is supported by interstitial water, with consequent decrease in shearing resistance. If relatively wet, the front of the mass bulges and advances either in more or less fluid tongues or, if less wet, by a gradual tumbling or rolling-over motion under the steady pressure of material behind and above. Many slowly moving earthflows form the bulbous or spreading toe of slump slides (see Fig. 16 and Pl. 1-h). Figure 29 shows the spreading, bulbous, upheaved toe of a slump-earthflow resulting from failure in a canal embankment.

Earthflows may continue to move slowly for many years under apparently small gravitational forces, until stability is reached at nearly flat slopes. At a higher water content the movement is faster, and what are here considered to be true mudflows are the liquid "end member"

of the slump-earthflow series in dominantly fine-grained material.

The rapid type of earthflow illustrated in Figure 30 and Plate 1-q, called earthflow by Sharpe (1938, p. 50) and clay flow by Terzaghi and Peck (1948, p. 362), is different from the foregoing and is not easily classified because it shows some similarity to failure by lateral spreading. These flows usually take place in sensitive materials; that is, in those materials whose shear strength is decreased to a very small fraction of its former value on remolding at constant water content. Terzaghi and Peck state (1948, p. 361):

> During a slide in such a clay the moving mass breaks up into chunks that are lubricated by the remolded portion of the clay. The mixture of chunks and matrix is so mobile that it may flow like a stream for hundreds or even thousands of feet on an almost horizontal surface. . . . During the flow [at Riviere Blanche, Quebec] a roughly rectangular area having a length of 1,700 feet parallel to the river and 3,000 feet perpendicular to the river subsided 15 to 30 feet. Within several hours, 3,500,000 cubic yards of the underlying silty clay moved into the river channel through a gap 200 feet wide. The channel was blocked for over two miles, and the upstream water level was raised 25 feet.

Similar flows have occurred in other parts of Canada, in the state of Maine, and in the Scandinavian countries. The index properties of the soils which flow in this manner are not yet reliably known. The few data which are available indicate that the soils are either very fine rock flours or very silty clays of glacial origin with a natural water content high

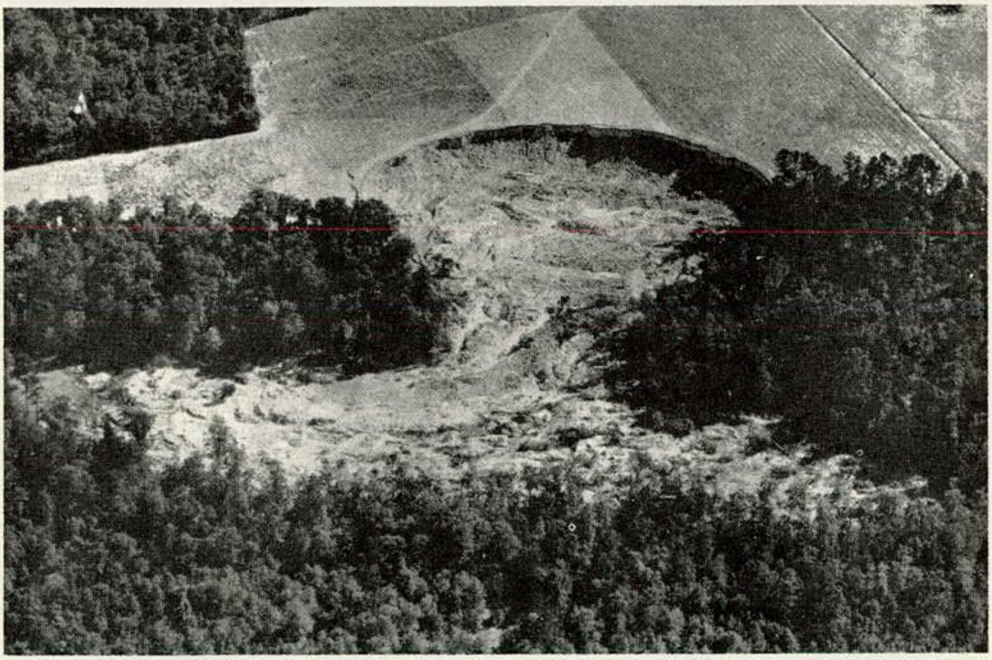


Figure 30. Earthflow near Greensboro, Fla. Material is flat-lying partly indurated clayey sand of the Hawthorn formation (Miocene). The length of the slide is 900 feet from scarp to edge of trees in foreground. Vertical distance from top to base of scarp is 45 feet and from top of scarp to toe is 60 feet. The slide occurred in April 1948 after a year of unusually heavy rainfall, including 16 inches during the 30 days preceding the slide. (Photograph from R. H. Jordan, 1949)

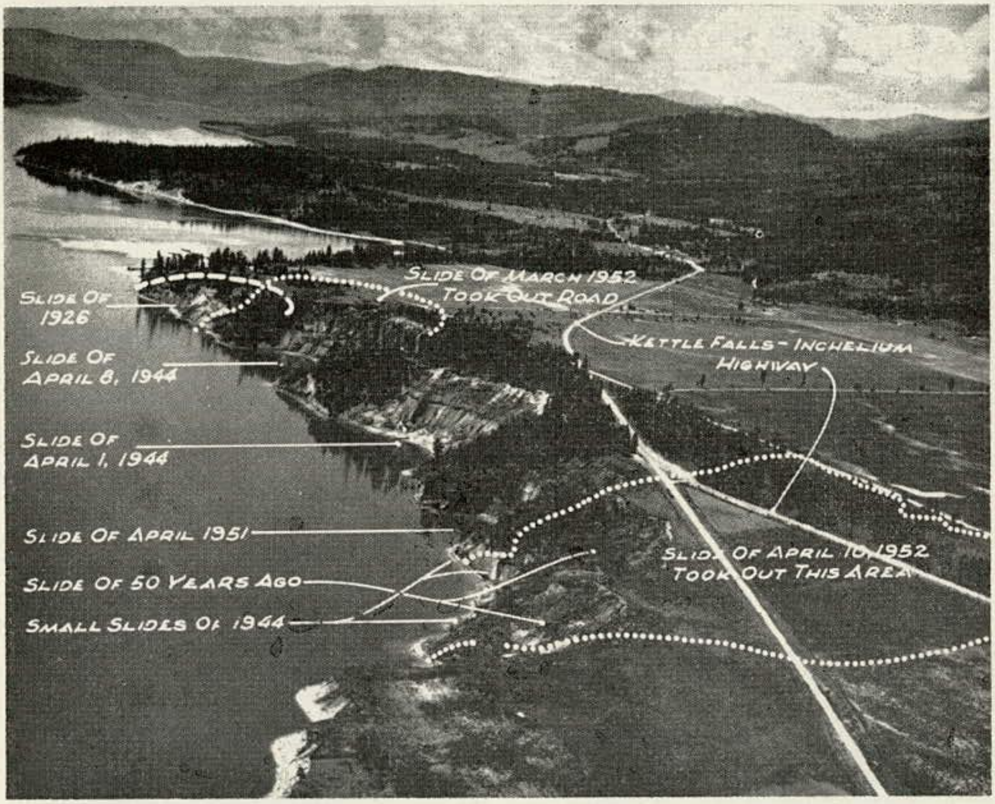


Figure 31. Reed Terrace area, right bank of Lake Roosevelt reservoir on Columbia River, near Kettle Falls, Wash., on May 15, 1951. The slide of April 10, 1952, involving about 15,000,000 cubic yards, took place by progressive slumping, liquefaction, and flowing out of glacio-fluvial sediments through a narrow orifice into the bottom of the reservoir. (Photograph by F. O. Jones, U. S. Geological Survey)

> above the liquid limit. . . . The excessive water content, which seems to constitute a prerequisite, indicates a very high degree of sensitivity and possibly a well-developed skeleton structure.

The large slide on the Reed Terrace near Kettle Falls, Wash., shown in Figures 31 and 32, resembles in some respects the earthflow at Riviere Blanche shown in Plate 1-q. The lower part of the exposed section of the Reed Terrace slide is composed of laminated silty clays, similar to those described by Terzaghi and Peck in the foregoing. The terrace is capped by sand and gravel. The slide of April 10, 1952, involved about 15,000,000 cubic yards. According to F. O. Jones of the U. S. Geological Survey, who has made a study of slides along the Columbia River Valley,[4] it seems likely that the initial failure took place by lateral spreading of the fine-grained saturated sediments below water level. The sliding that followed the initial failure, however, was similar to slump-earthflow (Pl. 1-h), earthflow, and mudflow. Repeated sliding developed a group of interlocking alcoves, enlarging the slide laterally and landward and severing three roads. The slide had cut back 2,000 feet from the original shore by

[4] Jones, F. O., written communication.

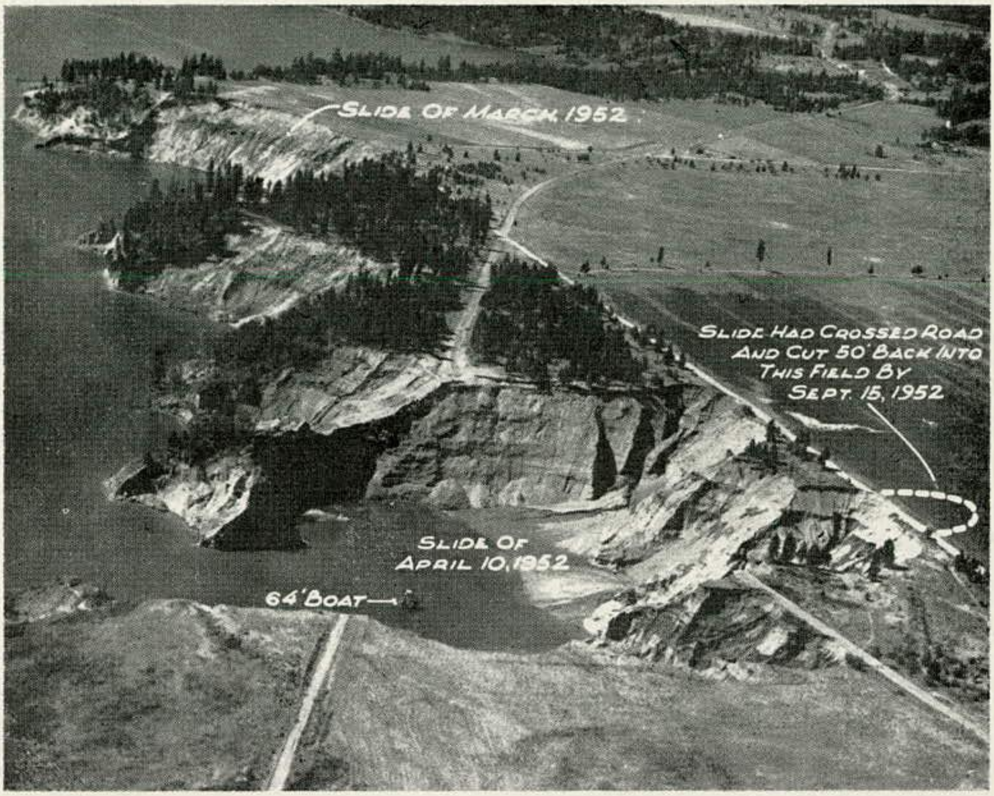


Figure 32. Reed Terrace area, Lake Roosevelt, Wash., after slide of April 10, 1952. (Photograph taken August 1, 1952, by F. O. Jones, U. S. Geological Survey)

April 13. A notable feature is the narrow orifice, which during the major movement was only 75 yards wide, and through which the slide material flowed out under water along the reservoir bottom.

Liquid sand or silt flows, such as illustrated in Plate 1-s, occur mostly along banks of noncohesive clean sand or silt. They are especially common along tidal estuaries in the coastal provinces of Holland, where banks of sand are subject to scour and to repeated fluctuations in pore-water pressure due to rise and fall of the tide (Koppejan, Van Wamelon, and Weinberg, 1948; and Müller, 1898). When the structure of the loose sand breaks down along a section of the bank, the sand flows out rapidly upon the bottom, and, by repeated sloughing, the slide eats into the bank and enlarges the cavity. Sometimes the scarp produced is an arc, concave toward the water; sometimes it enlarges greatly, retaining a narrow neck or nozzle through which the sand flows.

Type IV — Complex Landslides

More often than not, any one landslide shows several types of movement within its various parts or at different times in its development. Most slides are therefore complex. Several shown on the chart, for example those drawn largely from actual slides (Pl. 1-h, k, and l), are complex, but each illustrates a dominant and characteristic type of movement and

so can be fitted into the classification without too much difficulty.

Because the purpose of classifying landslides is to provide better data for use in controlling or avoiding them, it is of the greatest importance that for complex slides the classification be made at the time control or preventive measures are to be taken.

Landslide Processes

The process of landsliding is essentially a continuous series of events from cause to effect. An engineer faced with a landslide is primarily interested in curing the harmful effects of the slide. In many instances the principal cause of the slide cannot be removed, so it may be more economical to alleviate the effects continually or intermittently without attempting to remove the cause. Some slides occur in a unique environment and may be over and done with in a few seconds. The damage can be repaired, and the cause may be of only academic interest unless legal actions are to be taken. More often, however, landslides take place under the influence of geologic, topographic, or climatic factors that are common to large areas. These factors, these causes, must then be understood if other similar slides are to be avoided or controlled.

Very seldom, if ever, can a slide be attributed to a single definite cause. The process leading to the development of the slide has its beginning with the formation of the rock itself, when its basic physical properties are determined, and includes all the subsequent events of crustal movements, erosion, and weathering, until some action, perhaps trivial, sets a mass of it in motion downhill. The last action cannot be regarded as the one and only cause, even though it was necessary in the chain of events. As Sowers and Sowers (1951, p. 228) point out, "In most cases a number of causes exist simultaneously, and so attempting to decide which one finally produced failure is not only difficult but also incorrect. Often the final factor is nothing more than a trigger that set in motion an earth mass that was already on the verge of failure. Calling the final factor *the cause* is like calling the match that lit the fuse that detonated the dynamite that destroyed the building *the* cause of the disaster." In this connection, however, the determination of all the geologic causes of a landslide should not be confused with determination of legal responsibility (see Chapter Two).

The interrelations of landslide causes are very lucidly and graphically presented by Terzaghi (1950, p. 105-110). His work and that of Sharpe (1938, p. 83-87), Ladd (1935, p. 14-18), Bendel (1948, p. 268-337), and many others referred to elsewhere have been used extensively in the preparation of this section (see also Varnes, 1950).

All true slides (excluding falls) involve the failure of earth materials under shear stress. The initiation of the process can therefore be reviewed according to (a) the factors that contribute to high shear stress and (b) the factors that contribute to low shear strength. A single action, such as addition of water to a slope, may contribute both to an increase in stress and to decrease in strength. But it is helpful to separate mentally the various physical results of such an action.

The principal factors contributing to the instability of earth materials are outlined in the following. The operation of many factors is self-evident and needs no lengthy description; some factors are briefly discussed or reference is made to literature that gives examples or treats the subject in detail.

Factors That Contribute to High Shear Stress

A. Removal of lateral support

This is the commonest of all factors leading to instability and includes the actions of:

1. Erosion by:
    a. Streams and rivers in the production of most natural slopes. The literature on this subject

is vast. For introduction see Terzaghi (1936, 1950), Terzaghi and Peck (1948), Taylor (1948), Ladd (1935), Sharpe (1938), Ward (1945); bibliographies in these references and in Tompkin and Britt (1951).
   b. Glacier ice. Many valleys in mountainous regions were deeply cut by glaciers; when the ice retreated, landslides occurred on a large scale. See Howe (1909).
   c. Waves, and longshore or tidal currents. See the following: slides along coast of England, Ward (1945, 1948); coastal bluff at Santa Monica, Calif., Hill (1934); flow slides in Holland, Koppejan et al. (1948) and Müller (1898); along Mississippi River, Fisk (1944), Senour and Turnbull (1948).
   d. Subaerial weathering, wetting and drying, and frost action.
2. Creation of new slope by previous rockfall, slide, subsidence, or large-scale faulting.
3. Human agencies:
   a. Cuts, quarries, pits, and canals. Panama Canal, Binger (1948); MacDonald (1942?); National Academy of Sciences (1924); Wolf and Holtz (1948).
   b. Removal of retaining walls, sheet piling, etc.
   c. Draining of lakes or drawdown of reservoirs. See also seepage pressure under "Factors Contributing to Low Shear Strength."

B. Surcharge
1. Natural agencies:
   a. Weight of rain, hail, snow, and water from springs.
   b. Accumulation of talus overriding landslide material.
2. Human agencies:
   a. Construction of fill.
   b. Stockpiles of ore or rock. Hudson Valley, Terzaghi (1950, p. 105); Skempton and Golder (1948).
   c. Wastepiles. From strip mining, Savage (1950).
   d. Weight of buildings and other structures and trains.
   e. Weight of water from leaking pipelines, sewers, canals, reservoirs, etc.

C. Transitory earth stresses

Earthquakes have triggered a great many landslides, both small and very large and disastrous. Their action is complex, involving both increase in shear stress, and, in some examples, decrease in shear strength. They produce horizontal accelerations that may greatly modify the state of stress within slope-forming material. In the case of potential circular-arc failure, horizontal acceleration causes a moment about the center of the arc (Terzaghi, 1950, p. 89-91, and Taylor, 1948, p. 452), which when directed toward the free slope adds to its instability. Vibrations from blasting, machinery, and traffic also produce transitory earth stresses.

D. Regional tilting

Progressive increase in slope angle through regional tilting has been suspected as a contributing cause to some landslides (Terzaghi, 1950, p. 94). The slope must obviously be on the point of failure for such a small and slow-acting change to be effective.

E. Removal of underlying support
1. Undercutting of banks by rivers and waves.
2. Subaerial weathering, wetting and drying, and frost action.
3. Subterranean erosion.
   a. Removal of soluble material such as carbonates, salt, or gypsum; collapse of caverns. See Messines (1948), Buisson (1952).
   b. Washing out of granular material beneath firmer material. Terzaghi (1931), Ward (1945, p. 189-191).

4. Human agencies, such as mining.
5. Loss of strength in underlying material.
   a. Large masses of limestone over shale. At Frank, Alberta, and at Pulverhörndl in the Alps. Terzaghi (1950, p. 95-96).
   b. Compact till over clay. Terzaghi (1950, p. 96-97).
   c. Failure by lateral spreading. Newland (1916), Odenstad (1951), Ackermann (1948).

F. Lateral pressure due to
1. Water in cracks and caverns.
2. Freezing of water in cracks.
3. Swelling.
   a. Hydration of clay.
   b. Hydration of anhydrite. See also Messines (1948).

Factors That Contribute to Low Shear Strength

The factors that contribute to low shear strength of rock or soil may be divided into two groups. The first group includes factors deriving from the initial state or inherent characteristics of the material. They are part of the geologic setting that may be favorable to landsliding, and they change little or not at all during the useful life of a structure. They may exist for a long period without failure occurring. The second group (B, C, and D hereafter) includes the changing or variable factors that tend to lower shear strength of the material.

A. The initial state
1. Composition.
   Inherently weak materials, or those which may become weak upon change in water content or other changes as described in B, C, and D. Included especially are sedimentary clays and shales; decomposed rocks; rocks composed of volcanic tuff, which may weather to clayey material; materials composed dominantly of soft, platy minerals, such as mica, schist, talc, or serpentine; organic material.
2. Texture
   a. "Loose" arrangement of individual particles in sensitive clays, marl (von Moos and Rutsch, 1944), loess, sands of low density, and porous organic matter.
   b. Roundness of grains. See Chen (1948) on increase in compressibility and internal friction with increase in angularity.
3. Gross structure
   a. Discontinuities such as faults, bedding planes, foliation in schist, cleavage, joints, and brecciated zones. The effect of joints in rock is self-evident; the mechanism of progressive softening of stiff fissured clays is well described by Skempton (1948).
   b. Massive beds over weak or plastic materials.
   c. Strata inclined toward free face.
   d. Alternation of permeable beds, such as sandstone, and weak impermeable beds, such as shale or clay.

B. Changes due to weathering and other physico-chemical reactions
1. Physical disintegration of granular rocks such as granite or sandstone under action of frost, thermal expansion, etc. Decrease of cohesion.
2. Hydration of clay minerals. Absorption of water by clay minerals and decrease of cohesion of all clayey soils at high water contents. Swelling and loss of cohesion of montmorillonitic clays. Marked consolidation of loess upon saturation due to destruction of clay bond between silt particles (see American Society for Testing Materials, 1951, p. 9-34).
3. Base exchange in clays. Influence of exchangeable ions on physical properties of clays. See Grim (1949), Rosenqvist (1953), Proix-Noe

(1946), Tchourinov (1945), and American Society for Testing Materials (1952).

4. Drying of clays. Results in cracks and loss of cohesion and allows water to seep in.
5. Drying of shales. Creates cracks on bedding and shear planes. Reduces shale to chips, granules, or smaller particles.
6. Removal of cement by solution. Removal of cement from sandstone reduces internal friction.

C. Changes in intergranular forces due to pore water (see especially Taylor, 1948, Chap. 16)

1. Buoyancy in saturated state decreases effective intergranular pressure and friction.
2. Intergranular pressure due to capillary tension in moist soil is destroyed upon saturation.
3. Seepage pressures of percolating ground water result from viscous drag between liquid and solid grains.

D. Changes in structure

1. Fissuring of preconsolidated clays due to release of lateral restraint in a cut (Skempton, 1948).
2. Effect of disturbance or remolding on sensitive materials such as loess and dry or saturated loose sand. The great loss of shear strength of sensitive clays has been tentatively attributed to breakdown of a loose structure (Rosenqvist, 1953), but this has not been demonstrated. See also Skempton and Northey (1952).

References

Ackermann, E., "Quickerden und Fleissbewegungen bei Erdrutschen." Zeits. der Deutschen Geolog. Gesellschaft, B. 100, s. 427-466, 1948.

Alden, W. C., "Landslide and Flood at Gros Ventre, Wyoming." Am. Inst. Min. and Met. Eng., Tech. Pub. No. 140; Trans., v. 76, p. 347-361, 1928.

American Society for Testing Materials, "Symposium on Consolidation Testing of Soils." Am. Soc. Testing Materials Spec. Tech. Pub. 126, 1951.

American Society for Testing Materials, "Symposium on Exchange Phenomena in Soils." Am. Soc. Testing Materials Spec. Tech. Pub. 142, 1952.

Bendel, Ludwig, "Ingenieur-geologie, ein Handbuch für Studium und Praxis." v. 2, Springer-Verlag, Vienna, 1948.

Binger, W. V., "Analytical Studies of Panama Canal Slides." Proc. of 2d Internat. Conf. on Soil Mech. and Found. Eng., Rotterdam, v. 2, p. 54-60, 1948.

Bjerrum, L., "Stability of Natural Slopes in Quick Clay." *Geotechnique*, v. 5, no. 1, p. 101-119, 1955.

Buisson, M. M., "Les Glissements de la Falaise de Sainte-Adresse." *Annales de L'Institut Technique du Batiment et des Travaux Publics*, no. 59, p. 1131-1146, 1952. Translation No. 33 by Séverine Britt available from U. S. Geol. Survey.

Chen, Liang-Sheng, "An Investigation of Stress-Strain and Strength Characteristics of Cohesionless Soils by Triaxial Compression Tests." Proc. of 2d Internat. Conf. on Soil Mech. and Found. Eng., Rotterdam, v. 5, p. 43, 1948.

Close, U., and McCormick, E., "Where the Mountains Walked." *Nat. Geog. Mag.*, v. 41, p. 445-464, 1922.

Fisk, H. N., "Geological Investigation of the Alluvial Valley of the Lower Mississippi River." Mississippi River Comm., Corps of Eng., U. S. Army, Vicksburg, Miss., 1944.

Grim, R. E., "Mineralogical Composition in Relation to the Properties of Certain Soils." *Geotechnique*, v. 1, no. 3, p. 139-147, 1949.

Heim, A., "Bergsturz und Menschenleben." Fretz and Wasmuth, Zurich, 1932.

Hill, R. A., "Clay Stratum Dried Out to Prevent Landslips." *Civil Eng.*, v. 4, p. 403-407, 1934.

Howe, Ernest, "Landslides in the San Juan Mountains, Colorado." U. S. Geol. Survey Prof. Paper 67, 1909.

Jordan, R. H., "A Florida Landslide." *Jour. Geology*, v. 57, no. 4, p. 418-419, 1949.

Kesseli, J. E., "Disintegrating Soil Slips of the Coast Ranges of Central California." *Jour. Geology*, v. 51, no. 5, p. 342-352, 1943.

Kjellman, W., "Mechanics of Large Swed-

ish Landslips." *Geotechnique,* v. 5, no. 1, p. 74-78, 1955.

Koppejan, A. W., Van Wamelon, B. M., and Weinberg, L. J. H., "Coastal Flow Slides in the Dutch Province of Zeeland." Proc. of 2d Internat. Conf. on Soil Mech. and Found. Eng., Rotterdam, v. 5, p. 89-96, 1948.

Ladd, G. E., "Landslides, Subsidences and Rockfalls." *Am. Ry. Eng. Assoc. Proc.,* v. 36, p. 1091-1162, 1935.

MacDonald, D. F., "The Third Locks Project, Panama Canal Slides." The Panama Canal, Special Engineering Division, Balboa Heights, Canal Zone, 1942 (?).

McConnell, R. G., and Brock, R. W., "Report on the Great Landslide at Frank, Alberta." App. to Rept. Supt. Mines, Canada Dept. Interior, Ann. Rept. 1902-1903, Pt. 8, 17 p., 1904.

Messines, Jean, "Les Éboulements dans les Gypses: Inefficacité des Travaux de Reboisement (Landslides in Gypsum: Ineffectiveness of Reforestation)." *Comptes Rendus des Séances de L'Academie des Sciences,* Paris, tome 226, no. 16, p. 295-296, 1948. Translation No. 14 by Séverine Britt available from U. S. Geol. Survey.

Miller, W. J., "The Landslide at Point Fermin, California." *Sci. Monthly,* v. 32, p. 464-469, 1931.

Moos, A. von, and Rutsch, R. F., "Über Einen durch Gefügestörung Verursachten Seeufereinbruch (Gerzensee, Kt. Berne)." *Eclogae Geologicae Helvetia,* Zurich, v. 37, no. 2, p. 385-400, 1944. Translation No. 17 by Séverine Britt available from U. S. Geol. Survey.

Müller, Fr., "Das Wasserwesen der Niederlandischen Provinz Zeeland." Ernst and Son, Berlin, 1898.

National Academy of Sciences, "Report of the Committee of the National Academy of Sciences on Panama Canal Slides." *Nat. Acad. Sci. Mem.,* v. 18, 84 p., 51 pls., 19 figs., 1924.

Newland, D. H., "Landslides in Unconsolidated Sediments; with Description of Some Occurrences in the Hudson Valley." New York State Mus. Bull. 187, p. 79-105, 1916.

Odenstad, Sten, "The Landslide at Sköttorp on the Lidan River." Royal Swedish Geotechnical Inst. Proc., no. 4, 1951.

Peter, A., "Glissement de Terrain dans les Gorges de Moutier-Court." *La Route et la Circulation Routiere,* no. 10, p. 165-172, 206-208, Solothurn, Switzerland, 1938.

Proix-Noe, Marthe, "Etude d'un Glissement de Terrain dû à la Présence de Glauconie." *Comptes Rendus Hebdomadaires des Séances de l'Académie des Sciences,* v. 222, no. 7, p. 403-405, 1946. Translation No. 6 by Séverine Britt available from U. S. Geol. Survey.

Rosenqvist, I. Th., "Considerations on the Sensitivity of Norwegian Quick-Clays." *Geotechnique,* v. 3, no. 5, p. 195-200, 1953.

Savage, C. N., "Earthflow Associated with Strip Mining." *Mining Eng.,* v. 187, no. 3, p. 337-339, 1950.

Sekiya, S., and Kikuchi, Y., "The Eruption of Bandaisan." *Jour. Coll. Sci. Imp. Univ. Tokyo,* v. 3, pt. 2, p. 91-172, 1889.

Senour, Charles, and Turnbull, W. J., "A Study of Foundation Failures at a River Bank Revetment." Proc. of 2d Internat. Conf. on Soil Mech. and Found. Eng., Rotterdam, v. 7, p. 117-121, 1948.

Sharpe, C. F. S., "Landslides and Related Phenomena." Columbia University Press, New York, N. Y., 1938.

Skempton, A. W., "The Rate of Softening in Stiff Fissured Clays, with Special Reference to London Clay." Proc. of 2d Internat. Conf. on Soil Mech. and Found. Eng., Rotterdam, v. 2, p. 50-53, 1948.

Skempton, A. W., and Golder, H. Q., "Practical Examples of the $\phi = 0$ Analysis of Stability of Clays." Proc. of 2d Internat. Conf. on Soil Mech. and Found. Eng., Rotterdam, v. 2, p. 63-70, 1948.

Skempton, A. W., and Northey, F. D., "The Sensitivity of Clays." *Geotechnique,* v. 3, no. 1, p. 30-53, 1952.

Sowers, G. B., and Sowers, G. F., "Introductory Soil Mechanics and Foundations." The Macmillan Co., New York, N. Y., 1951.

Taylor, D. W., "Fundamentals of Soil Mechanics." John Wiley & Sons, New York, N. Y., 1948.

Tchourinov, M. P., "Modifications par Altération dans la Composition, la Structure et les Propriétés des Argiles du Crétacé Inférieur." *Comptes Rendus (Doklady) de l'Académie des Sciences de l'URSS,* v. 49, no. 5, p.

364-368, 1945. Translation No. 15 by Séverine Britt available from the U. S. Geol. Survey.

Terzaghi, Karl, "Earth Slips and Subsidences from Underground Erosion." *Eng. News-Record,* v. 107, p. 90-92, 1931.

Terzaghi, Karl, "Stability of Slopes of Natural Clay." Proc. of 1st Internat. Conf. on Soil Mech. and Found. Eng., Harvard University, v. 1, p. 161-165, 1936.

Terzaghi, Karl, "Mechanism of Landslides." In "Application of Geology to Engineering Practice." Berkey Volume, Sidney Paige, Chairman; Geol. Soc. America, p. 83-123, 1950.

Terzaghi, Karl, and Peck, R. B., "Soil Mechanics in Engineering Practice." John Wiley & Sons, New York, N. Y., 1948.

Tompkin, J. M., and Britt, S. H., "Landslides." A selected annotated bibliography. Highway Research Board Bibliography 10, Washington, D. C., 1951.

Varnes, D. J., "Relation of Landslides to Sedimentary Features." In "Applied Sedimentation," P. D. Trask, editor, p. 229-246, John Wiley & Sons, New York, N. Y., 1950.

Varnes, H. D., "Landslide Problems of Southwestern Colorado." U. S. Geological Survey Circ. 31, 1949.

Ward, W. H., "The Stability of Natural Slopes." *Geog. Jour.*, London. v. 105, no. 5-6, p. 170-197, 1945.

Ward, W. H., "A Coastal Landslip." Proc. of 2d Internat. Conf. on Soil Mech. and Found. Eng., Rotterdam, v. 2, p. 33-38, 1948.

Wolf, W. H., and Holtz, W. G., "Slope Stability Studies for the Delta-Mindota Intake Canal." Proc. of the 2d Internat. Conf. on Soil Mech. and Found. Eng., Rotterdam, v. 3, p. 268-274, 1948.

Chapter Four

Recognition and Identification of Landslides

Arthur M. Ritchie

The problems of recognition of landslides and identification of landslide types are as complex as are the materials and processes that cause them. As treated more fully in Chapter Three, the basic conditions that favor slides depend to a large extent on the character, stratigraphy and structure of the underlying rocks and soils, on the topography, climate and vegetation, and on surface and underground waters. All of these factors vary widely from place to place; their variations are reflected in differences in the rate and kind of landslide movements that result from their interaction.

Because so many variables are involved in the production of landslides, it is natural that many tools, theoretical and practical, must be used in recognizing and classifying them. The selection of methods and criteria for use in solution of a given problem is made with appreciation of the local conditions. Obviously, not all the available techniques are used on every job. One single factor may call for entirely different methods in determining the danger of landslides. Nevertheless, it is important to have all the methods in mind in case they are needed.

It is the purpose of this chapter to describe some of the techniques used in recognition and classification and to indicate their possible applications. One important method — photointerpretation — is treated separately in Chapter Five. The other approaches can be logically treated in two groups — the means of determining whether or not landslide movements have actually taken place or are likely to do so in the future, and the means of identifying the various types of landslides and their constituent parts. The application of these methods and criteria to the solution of landslide problems appears in succeeding chapters, particularly in Chapter Six.

Evidence for Actual or Potential Landslides

Environmental Factors

A knowledge of the general setting is essential in the recognition of either potential or actual landslides. By setting is meant all the factors that make up the physical environment — geology, soils, topography, climate — for a difference in any single factor, such as climate, can have pronounced effect on the other factors, hence on the probability of slides.

Similar geologic and soil conditions tend to give rise to similar landslides — and recognition features — but only under constant climatic conditions. For example, in an arid region a mature topography developed on old inactive slumps is characterized by smooth, low rounded surfaces, known collectively as hummocky ground (see Fig. 33). Even though modified, the slump blocks are still discernible; the depressions between them are disconnected and there is no well-defined drainage system. In regions of heavy rainfall, however, and in the same amount of elapsed time, the topographic expression of identical geologic conditions is entirely different. The

Figure 33. Hummocky ground, one of the most easily applied criteria for recognizing landslides. In this slide, one mile south of Springdale, Utah, the hummocks are larger and more irregular than they are on many slides. (Photograph by H. E. Gregory, U. S. Geological Survey)

slopes of the hummocky ground are flatter and the depressions are filled with swamp mud and water, with some semblance of an integrated drainage system. This difference is, of course, due to the fact that landslide topography reaches maturity more rapidly in humid regions than it does in arid ones.

Knowledge of the general setting is best had by means of thorough personal acquaintance with the area and long-continued careful observations and analysis. Failing this method, and even accompanying it, much of the necessary background knowledge can be gained through study of available aerial photographs (see Chapter Five) and of topographic, geologic and soil maps. With these facilities the trained observer can obtain a great deal of information on the character of the slopes, of surface and subsurface drainage, and on the character and distribution of the different kinds of rocks and soils that cover them.

Only rarely do geologic or soil maps show landslides as such or describe their causes, nor do most of them show features in the detail that is necessary to answer specific problems. Such maps, however, as well as air photos, are of great assistance in giving background knowledge that is needed as a setting for detailed studies. Some of the useful facts that can be gleaned from maps of one kind or another, or from air photos, are as follows:

1. Rock and soil units and their characteristics.
2. Areal distribution of rock and soil units.
3. Sequence of rock and soil units. For example, a weak unit that could cause failure may not be exposed at the surface but may be plainly shown on a geologic cross-section or on a soil profile.

4. Character and distribution of folds, faults and joints in bedrock, all of which may seriously affect its susceptibility to sliding.
5. Location of volcanic cinder cones and similar features that offer special problems.
6. Drainage pattern — streams, lakes and swamps, all of which give indications of relative permeability of underlying materials.
7. Bowl-shaped headwater regions of creeks, which suggest landslide origins.
8. Terraces, slopes, and depressions.
9. Abnormally steep slopes, with mounds of possible landslide origin at their bases.
10. Scalloped escarpments that suggest landslide origins.
11. Anomalous constrictions in canyons, quite possibly caused by landslides.

Aerial photographs and geologic maps must be used with caution; they can be of great assistance in developing the setting for more detailed studies, but they seldom contain the detail that is required to answer specific questions. Many geologic formations throughout the country are labeled as troublemakers because they are susceptible to landsliding, yet no formation forms slides throughout the entire extent of its outcrop area. It is, thus, incorrect to give the entire formation a bad name. Slight differences in environmental factors or in facies within the formation can and do lead to stability as well as to instability. Moreover, slight differences within a formation may give rise to entirely different types of landslide. The Astoria formation of Oregon and Washington is a good case in point. It is composed of sandstone and siltstone that grade imperceptibly eastward from finer to coarser grain sizes. In the western portion of its area of outcrop the Astoria produces many deep-seated slump landslides; many of the higher hills are nothing more than remnants of slump blocks. Eastward, however, where the materials are somewhat coarser grained, the formation is characterized by debris slides that grade downward into flows. The practical difference to the engineer is that the fine-grained slump-forming materials will not stand up in fills, thus requiring road construction to be made entirely within cuts. On the other hand, the coarser materials that form debris slides farther east can be used successfully in fills if special precautions are taken in beginning them.

The example just given illustrates the fact that generalized geologic maps and descriptions have definite limitations, but this does not imply any lack of faith in their value as a tool for coping with landslide problems. Indeed, as the boundaries of the Astoria formation — and of other geologic units that present comparable problems — become better known, and the facies characteristics are mapped in greater detail, it is increasingly feasible to predict the kind and character of landslides that are likely to occur in any part of the formation.

The study of the environment of the area should give at least a general answer to a fundamental question: "Do landslides already exist along the proposed location?" If the answer is no, it may be taken as an indication that there are no regional factors that are themselves conducive toward landslides. If the answer is yes, it is probable that there are two or more regional factors which, acting together, may lead to landslides. For example, in the north-central part of Washington there is a great and widespread deposit of silt, called the Nespelem formation. When dry, as in small isolated hills or terrace remnants, this silt stands well in natural or artificial cuts. When wet by spring waters, however, the silt is very likely to slide. The conjunction of the two factors — silts and spring horizons — should be a danger signal to the engineer who is planning a highway or other structure in the area.

Studies of all existing landslides in the region are thus warranted in order to determine their geologic settings and their causes. If similar conditions are present along the proposed location it

can be assumed that slides will occur there too.

POTENTIAL SLIDES

Even if the preliminary examination of the general environment has indicated that no landslide movements have yet taken place, it is still incumbent on the investigator to determine whether the ground to be disturbed by the proposed construction will prove reasonably stable. Man is not capable, nor is money available, to study in detail and to guarantee the stability of all the slopes along most proposed highways. As a general rule, the amount of investigation that is warranted is a function of the landslide susceptibility of the surrounding country. Too, it is a function of the degree of damage that might be expected to occur to persons or installations if a slide should occur. In other words, the more serious a landslide might be, the more detailed should be the search for potential slides.

After a knowledge of the general environment has been obtained, either by firsthand observation or by study of existing maps and air photos, the next essential step is to visit the site itself and examine its physical conditions. The whole site should first be studied from a distance, for a forest is more easily recognized than are the trees. Special attention should be given to the slopes, changes in slope, and their relationship to the different materials involved. Cracks and other evidences of motion, as well as all sources of water, should be noted. The structure of the underlying bedrock, as well as the depth of overburden, should be determined carefully.

Evidence of soil creep and of "stretching" of the ground surface, should also be sought. Stretching is here distinguished from soil creep because it indicates comparatively deep-seated movement, whereas soil creep is of superficial origin. The phenomenon of stretching is most commonly observed in noncohesive materials that do not form or retain minor cracks readily. The best evidence of stretching consists of small cracks that surround or touch some rigid body, such as a root or boulder, in otherwise homogeneous material; these cracks form because the tensional forces tend to concentrate at or near the rigid bodies.

For recognition of a potential landslide condition where bedrock is hidden, a preliminary but adequate field investigation of the soil, coupled with shear measurements in the laboratory, is perhaps the best means available. Such a combined field and laboratory investigation, backed by at least general knowledge of the underlying rocks, should reveal the soil profile and ground water conditions along a proposed route even where surface features alone do not provide sufficient warning. It must be remembered, however, that there are some rather severe limitations on the applicability of shear measurements to landslide problems; these are further discussed in Chapter Nine.

Potential slides of the rockfall and soilfall type can commonly be foreseen simply by recognizing geologic conditions that are likely to produce overhanging or oversteepened cliffs. Some of the geologic settings that fall in this category are as follows:

1. Massive lava flow underlain by strongly fractured flow or by poorly consolidated volcanic tuffs.
2. Lava flow underlain by easily erodible sandstone.
3. Sandstone or limestone underlain by coal seams or by relatively soft shale.
4. Cliff subject to erosion by waves or running water at its base.
5. Frozen ground or rock subject to local thawing by lake or running water.
6. Firm cohesive or partly consolidated soil underlain by noncohesive soil or fine sand that will be easily eroded by wind or water, by excessive drying, or by seepage pressures from within the slope if it is exposed during construction.

All of the foregoing geologic situations involve a stronger unit over a

weaker one. Too often the weak unit is more or less completely obscured by talus or other debris from the stronger layer. This is a point which serves to emphasize the need for thorough field examination.

Effect of Proposed Construction

Many landslides are caused by man's upsetting the natural causes of erosion. This is another way of saying that consideration of the effect of proposed or future construction itself cannot be neglected in the search for dangers of potential landslides. Any cut or fill will change the local stress conditions; it is, therefore, necessary to analyze the possible effects of the stress readjustments to future cuts or fills, whether natural or manmade, and to evaluate the effect of modifying the erosional process that was in operation.

For instance, construction of jetties or groins along Lake Michigan and many other lake and ocean shores disturbs the normal processes of beach erosion and formation, especially in places where there is an interruption of littoral drift of beach sand that protects adjacent bluffs. The location of a scenic highway skirting such a bluff is, then, contingent on future plans for jetty construction along the beaches (see Fig. 4).

A similar problem is encountered when a highway is constructed in a region where adjacent lands may be irrigated in the future. In such a case, it is well to consider the probable results of a rising water table, for such a change in ground water conditions may detrimentally affect a cut or fill section of a highway or of a bluff above the new road. As a concrete example, the Washington State Highway Commission in 1955 was confronted with a $600,000 relocation job, plus purchase of water rights, because the initial instability of the region below Grand Coulee Dam had been aggravated by local irrigation projects.

The effect of water on silts in reservoir banks is well-known, as is the effect of drainage of silts during reservoir drawdown. Dams also affect the regions below them, and can set up potential landslide conditions there. If the materials in the valley walls below the dam are at the critical point of stability a change in the regional water table may trigger the sensitive materials. A dam also retards the normal movement of the river's load; the river bed downstream becomes starved for material, which leads to excessive scour, deepening of the channel and undermining of the banks.

If the engineer keeps proposed or future construction in mind, and evaluates the effect which this construction may have on the soil profile, the underlying rock, and ground water conditions, he will go a long way toward recognizing a potential landslide problem and will be able to make plans to avoid or to stabilize the sensitive mass. Typical situations that should be looked for in this connection are as follows:

1. Restriction of ground water flow by sidehill fill.
2. Overloading of relatively weak underlying soil layer by fill.
3. Overloading of sloping bedding planes by heavy sidehill fill.
4. Oversteepening of cuts in unstable rock or soil.
5. Removal, by cut, of thick mantle of pervious soil if the latter is a natural restraining blanket over a softer core.
6. Increase in seepage pressure by cut or fill that changes direction or character of ground water flow.
7. Exposure, by cut, of stiff fissured clay that may soften when exposed to surface water.
8. Removal of mantle of wet soil by sidehill cut; such a cut may remove toe support, causing soil above cut to slide along its contact with stable bedrock.
9. Increase in hydrostatic head below surface of a cut in silt or permeable clay if surface is allowed to freeze or to become covered with impervious slough material.

Figure 34. Early signs of impending debris slide, highway along Clear Creek, Colo. Displacement of fence, upbulge of pavement, and distress in bridge abutments (see also Figure 35) all gave early indications of movement at the toe of an incipient slide. In several places, now covered by patching, the centerline stripe was offset along cracks. (Photograph by D. J. Varnes, U. S. Geological Survey)

Actual Slides

The term landslide, by definition, implies that movement has taken place, hence an analysis of the kind and amount of movement becomes a key to the nature of an active slide. Similarly, prediction of the kind of movement that may take place in the future is prerequisite to the analysis of potential slides.

Quite commonly the first visible sign of ground movement is recorded by settlement of the roadway or, depending on the road's location within the moving mass, an upbulge of the pavement. In some cases it is possible to find evidence of landslide movement that has not yet affected the highway but that may do so in time. Thus, minor failure in an embankment, material that falls on the roadway from an upper slope, or even the progressive failure of the region below a fill may well presage a larger landslide that will endanger the road itself. Other evidences of movement are to be found in broken pipe or power lines, spalling or other signs of distress in concrete structures, closure of expansion joints in bridge plates or rigid pavements (see Figs. 34 and 35), or loss of alignment of building foundations. In many cases, arcuate cracks and minor scarps in the soil give advance notice of serious failure (see Fig. 38).

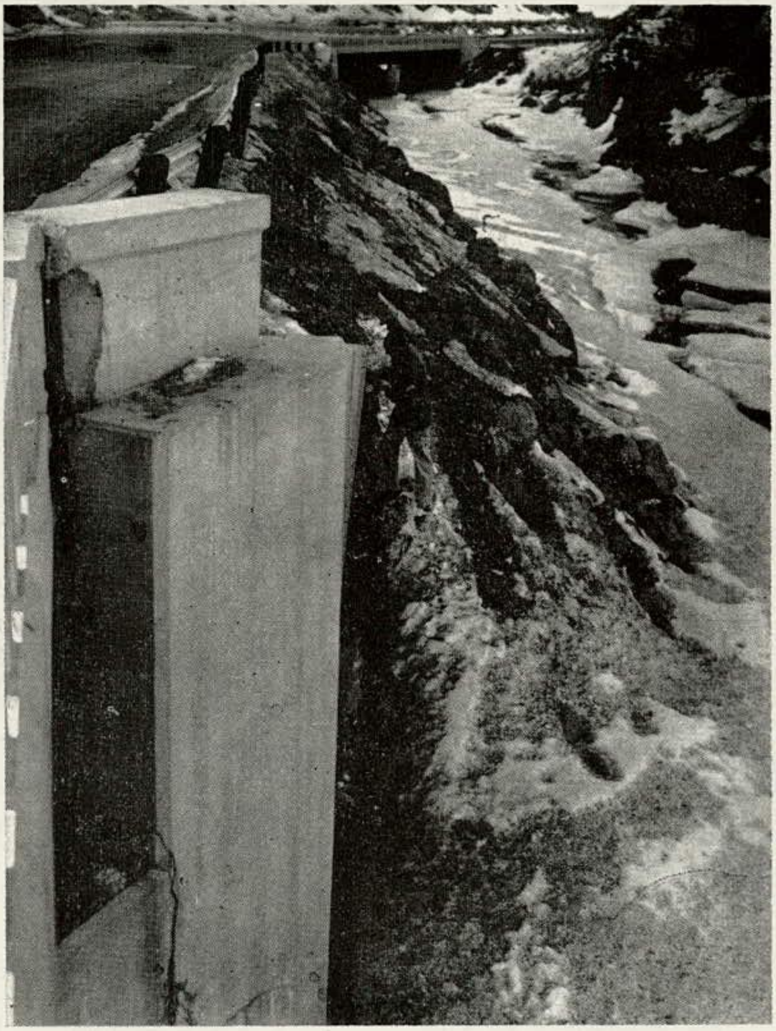


Figure 35. Distress in bridge abutment indicates incipient slide. Right-hand wing wall of bridge shown in lower center of Figure 34. In addition to offset of the wing wall shown here, rockers beneath bridge girders were tipped. (Photograph by D. J. Varnes, U. S. Geological Survey)

The chief evidences of movement in the various parts of each type of landslide are summarized in Table 1 and in the section on "Identification of Landslide Types." Most of these evidences are self-explanatory and require no further explanation. The facts that can be deduced from a study of surface cracks are so important in recognizing and classifying slides that they deserve a few words separately.

Significance of Cracks

The ability to recognize small cracks and displacements in the surface soils and to understand their meaning is one that deserves cultivation because it can

produce accurate knowledge of the cause and character of movement that is prerequisite to correction. The significance of tiny cracks around boulders or roots as evidence of "stretching" of the ground surface has already been mentioned.

Surface cracks are not, as is commonly assumed by some, necessarily normal to the direction of ground movement. For example, cracks near the head of a slump are indeed normal to the direction of horizontal movement, but the cracks along its flank are nearly parallel to it.

Small *en echelon* cracks commonly develop in the surface soil before other signs of rupture take place; they are, thus, particularly valuable tools in the recognition of potential or incipient slides. They result from a force couple in which the angle between the direction of motion and that of the cracks is a function of the location within the landslide area. It follows that for many cases a map of the *en echelon* cracks will delineate the slide accurately, even though no other visible movement has taken place (see Fig. 36). Figure 37 shows an actual set of *en echelon* cracks.

In addition to indicating incipient or actual movement, cracks in the surface soils are locally useful in helping to determine the type of slide with which one is dealing. For example, in a slump the walls of cracks are slightly curved in the vertical plane and are concave toward the direction of movement; if the rotating slump block has an appreciable vertical offset the curved cracks wedge shut in depth. In block glides, on the other hand, the cracks are nearly equal in width from top to bottom and do not wedge out in depth. This is because failure in a block glide begins with tension at the base of the block and progresses upward toward the surface. Block glide can be distinguished from lateral spreading by the presence of a few major breaks in the upper parts of a block glide (see Fig. 38) whereas lateral spreading is characterized by a maze of intersecting cracks.

Cracks in block glides of cohesive soils are commonly almost vertical, regardless of the dip of the slip-plane, whereas in block glides of rock the inclination of the cracks depends on the joint systems in the rock.

One of the most helpful applications of a study of cracks lies in the distinction between incipient block glides and slumps. If the outline of the crack pattern is horseshoe-shaped in plan, with or

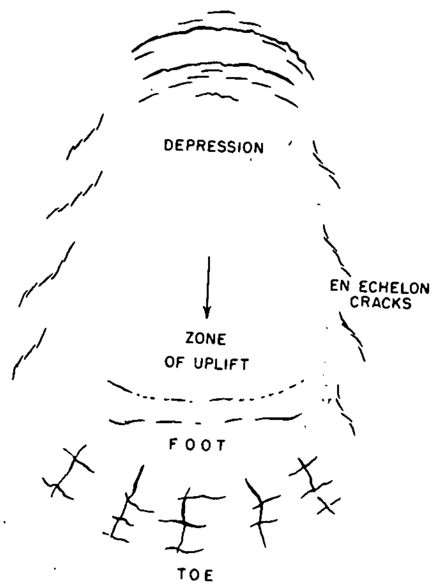


Figure 36. Tension cracks as typically developed in a slump slide in cohesive materials. (Based on Terzaghi and Peck, Fig. 151, 1948)

without concentric cracks within it, a slump is almost certainly indicated. If, on the other hand, most of the surface cracks are essentially parallel to the slope or cliff face, a block glide is probably in the making. In either case, additional cracks may develop as major movement gets under way, but these will generally conform to the earlier crack pattern (see Figs. 18 and 19).

Hidden Landslides

Among the most difficult kinds of slides to recognize and guard against are old landslides that have been cov-

Kind of Material	Type of Motion	Stable Parts Surrounding the Slide		
		Crown	Main Scarp	Flanks
Falling: Rockfall	Rock	Loose rock; probable cracks behind scarp; irregular shape controlled by local joint system	Usually almost vertical; irregular, bare, fresh. Usually consists of joint or fault surfaces	Mostly bare edges of rock
Soilfall	Soil	Cracks behind scarp	Nearly vertical, fresh, active, spalling on surface	Often nearly vertical
Sliding: Slump	Soil	Numerous cracks, most of them curved concave toward slide	Steep, bare, concave toward slide, commonly high. May show striae and furrows on surface running from crown to head. Upper part of scarp may be vertical	Striae on flank scarps ha strong vertical compon near head, strong horizon component near foot. Hei of flank scarp decrea toward foot. Flank of sl may be higher than origi ground surface between f and toe. *En echelon* cra outline slide in early stag
	Rock	Cracks tend to follow fracture pattern in original rock	As above	As above
Block glide	Rock or Soil	Most cracks are nearly vertical and tend to follow contour of slope	Nearly vertical in upper part, nearly plane and gently to steeply inclined in lower part	Flank scarps very low, crac vertical. Flank cracks usua diverge downhill
Rockslide	Rock	Loose rock, cracks between blocks	Usually stepped according to the spacing of joints or bedding planes. Surface irregular in upper part, and gently to steeply inclined in lower part; may be nearly planar or composed of rock chutes	Irregular
Flowing: Dry Rock fragment flow	Rock	Same as rockfall	Same as rockfall	Same as rockfall
Sand run	Soil	No cracks	Funnel-shaped at angle of repose	Continuous curve into ma scarp
Wet Debris avalanche Debris flow	Soil	Few cracks	Upper part typically serrate or V-shaped. Long and narrow Bare, commonly striated	Steep, irregular in upper pa Levees may be built up alo lower parts of flanks
Earthflow	Soil	May be a few cracks	Concave toward slide. In some types scarp is nearly circular, slide issuing through a narrow orifice	Curved, steep sides
Sand or silt flow	Soil	Few cracks	Steep, concave toward slide, may be variety of shapes in outline — nearly straight, gentle arc, circular, or bottle-shaped	Commonly flanks converge direction of movement

Parts That Have Moved			
Head	Body	Foot	Toe
lly no well-defined head. Fallen terial forms a heap of rock xt to scarp.	Irregular surface of jumbled rock, sloping away from scarp. If very large, and if trees or material of contrasting color are included, the material may show direction of movement radial from scarp. May contain depressions	Foot commonly buried. If visible, the foot generally shows evidence of reason for failure, such as underlying weak rock or banks undercut by water	Irregular pile of debris or talus if small. If the rockfall is large, the toe may have a rounded outline and consist of a broad, curved transverse ridge
lly no well-defined head. Fallen terial forms a heap next to rp	Irregular	As above	Irregular
nants of land surface flatter an original slope or even tilted o hill creating depressions at ot of main scarp in which rimeter ponds form. Transrse cracks, minor scarps, grans, and fault blocks. Attitude bedding differs from surround- area. Trees lean uphill.	Original slump blocks generally broken into smaller masses; longitudinal cracks, pressure ridges, occasional overthrusting. Commonly develops a small pond just above foot	Transverse pressure ridges and cracks commonly develop over the foot; zone of uplift, absence of large individual blocks, trees lean downhill	Often a zone of earthflow, lobate form, material rolled over and buried; trees lie flat or at various angles mixed into toe material
bove	As above, but material does not break up as much or deform plastically	As above	Little or no earthflow. Toe often nearly straight and close to foot. Toe may have steep front
tively undisturbed. No rotan	Body usually composed of a single or a few units, undisturbed except for common tension cracks. Cracks show little or no vertical displacement	No foot, no zone of uplift	Plowing or overriding of ground surface
y blocks of rock	Rough surface of many blocks. Some blocks may be in approximately their original attitude; but lower down, if movement was slow translation	Usually no true foot	Accumulation of rock fragments
head	Irregular surface of jumbled rock fragments sloping down from source region and generally extending far out on valley floor. Shows lobate transverse ridges and valleys	No foot	Composed of tongues. May override low ridges in valley
lly no head	Conical heap of sand, equal in volume to head region	No foot	
be no head	Wet to very wet. Large blocks may be pushed along in a matrix of finer material. Flow lines. Follows drainage lines and can make sharp turns. Very long compared to breadth	Foot absent or buried in debris	Spreads laterally in lobes. Dry toe may have a steep front a few feet high.
monly consists of a slump ock	Broken into many small pieces. Wet. Shows flow structure	No foot	Spreading, lobate. See above under "slump"
erally under water	Spreads out on underwater floor	No foot	Spreading, lobate

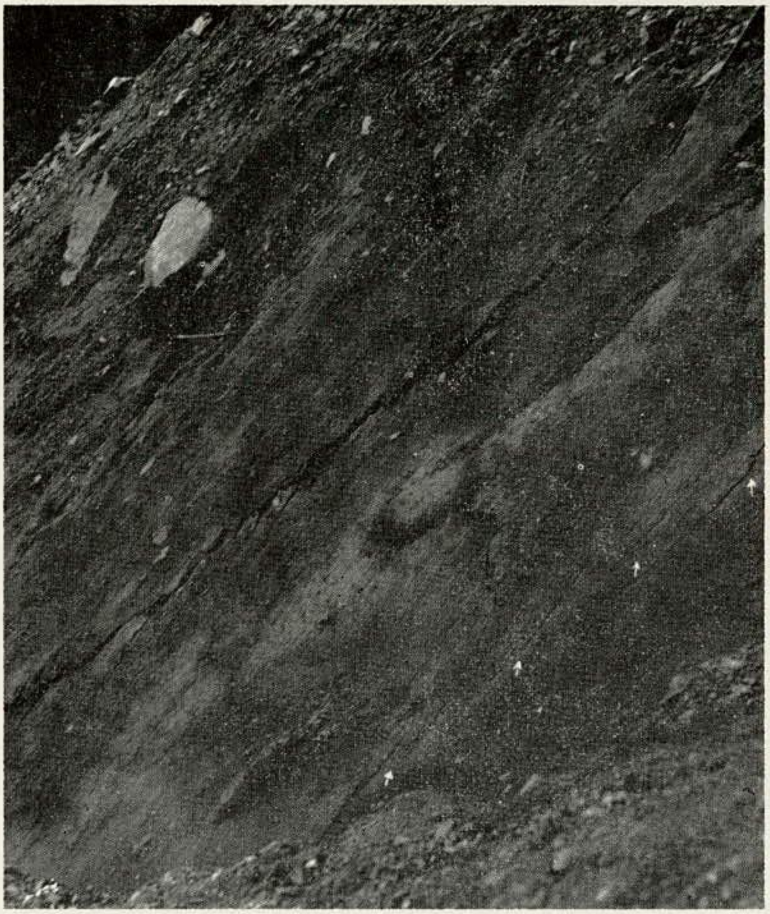


Figure 37. Minor en echelon cracks in soil reflect nearby landslide of major proportions, White Pass, Wash. The soil, only about one foot thick, covers a contact between diorite above and shale beneath. The first system of en echelon cracks has sheared through to form a single irregular fracture; a second line of en echelon cracks (indicated by white arrows) is developing below the first. (Courtesy Washington Department of Highways)

ered by glacial till or other more recent sediments. In such cases as the two examples mentioned hereafter it is probably impossible to predict, in detail, the existence of old buried slides or the effect that they may have on new construction work that happens to expose them. One who knows the recent geologic history of the region intimately, however, may well be able to make some controlled guesses as to the probable existence of such slides, and even as to where they are most likely to be found.

One example is shown in Figure 39. The unstable body of soft shale and chalk shown is clearly related to a fault in the bedrock, but it also has the characteristics of a surface landslide. Since this slide took place, the area was covered by two or more layers of loess, which completely obscured the evidence of the landslide until the rising waters of the lake undercut the bank and exposed the old slide.

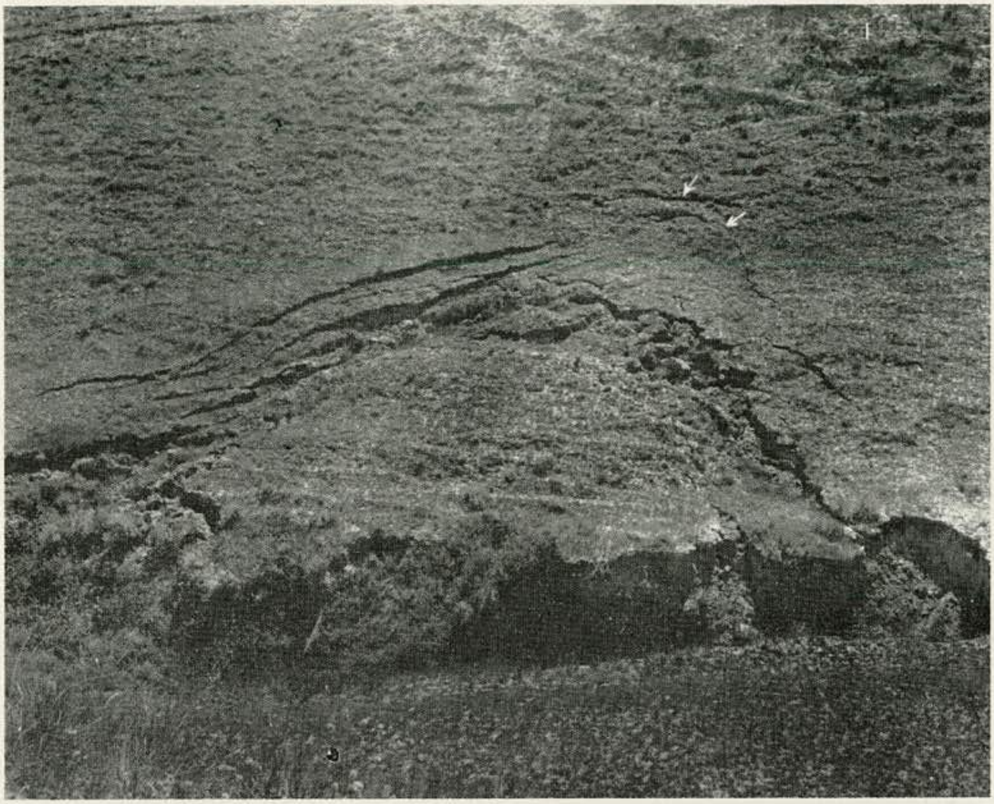


Figure 38. Block glide in cohesive materials, near Portage, Mont. The slide, in alluvium, colluvium and a little wind-deposited silt, is moving out over the surface of an alluvium-filled stream channel with little or no rotation of the block and without developing a zone of uplift at the foot. Note the parallel step scarps along the main scarp and the drag effects along the flanks, both characteristic of block glides. The arrows indicate overbreak cracks that develop after the main scarp is formed; possibly because of the abrupt break in slope, these are more sharply curved than are those above most slump slides. (Photograph by E. K. Maughan, U. S. Geological Survey)

A second example is shown in Figure 40. This landslide, which blocked a major highway near Snoqualmie Pass in the Cascade Mountains, was one of the most disastrous that has ever occurred in Washington. The valley wall, composed of strongly fractured graywacke, was cut to a steep angle by a valley glacier. Retreat of the ice removed lateral support from the rock and resulted in a slump failure that sheared through the fractured graywacke along a typical slump circular arc. Later, readvance of the ice removed projecting material along the valley wall and covered the landslide remnants with a 10-foot thick plaster of glacial till. The resulting slope appeared harmless enough before construction, but soon after the till and some of the rock had been removed it was found that major movement was taking place. Examination showed that the excavation was in the foot region of the old slump. Unloading of the foot caused one-half million yards of rock to cascade down the slope.

Identification of Landslide Types

Once it has been established that land movement has taken place, or is still going on, the next essential step is to

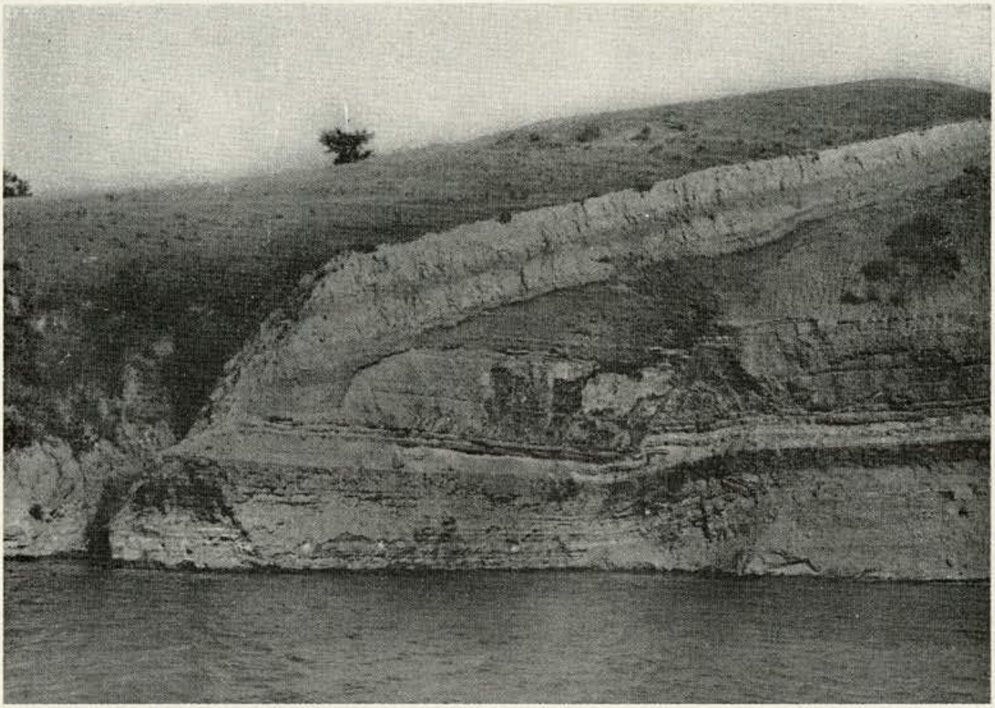


Figure 39. Hidden landslide exposed when overburden of loess was removed by bank-cutting along lake shore, Fort Randall Reservoir, S. Dak. The buried soil profiles indicate two periods of loess deposition after faulting and landsliding took place in the underlying shale and chalk of Cretaceous age. All surface evidence of the landsliding was obliterated by the loess until the lake waters cut a new face. (Photograph by C. F. Erskine, U. S. Geological Survey)

identify the type of landslide. One would not apply the same corrective procedure to a rockfall as to a block glide, nor to a flow and a slump. If maximum benefit is to be had from the preventive or corrective measures finally employed, therefore, it is imperative to learn to recognize the kind of slide that exists. Table 1 summarizes the surface features of various parts of active slides as they aid in identification of the different types. Further generalizations are given in the following paragraphs.

It is important to observe that landslides may change in character and that they are usually complex, frequently changing their physical characteristics, as well as their marks of identification as time goes on. For instance, a landslide examined a year after its occurrence may have changed remarkably from the conditions immediately following the original movement. Certainly, if a landslide developed as a slump slide and over a period of time turned into a flow, the original report on the nature of the slide would be invalid as a basis for planning a correction of the slide at the later date. The identification of the type of slide should be made at the same time it is to be corrected. Even if the landslide that started as a slump but later changed to a flow is corrected as a flow, this does not necessarily mean that the adjoining area, which may be still a slump block, or yet another area that has not moved at all, should require the same kind of correction. Each slide must be classified according to its own characteristics at the time it is to be corrected. If this is not done, time may destroy the value of the identification

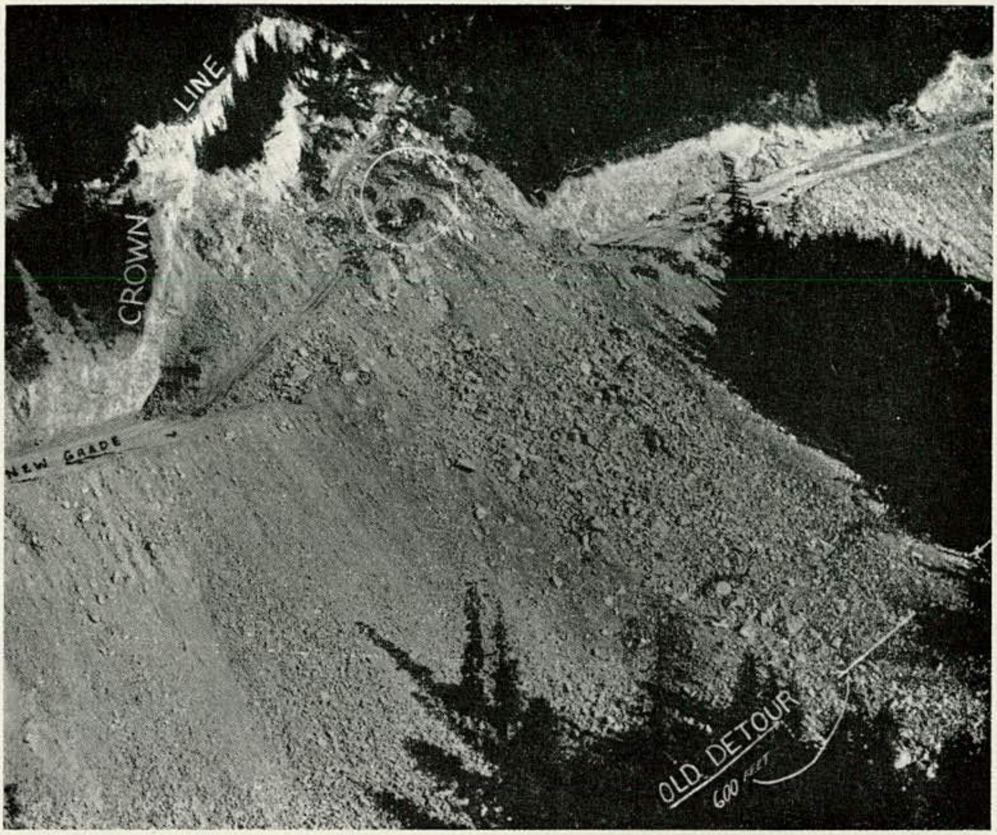


Figure 40. Originally hidden slump block after reactivation by construction; Snoqualmie Pass, Wash., August 13, 1953. After the slide began, and to facilitate removal of unstable material, further movement was deliberately induced by pumping water into the mass. Because the cut slope was more than 160 feet high, the front of the slide moved as a rockfall avalanche, the complete slide taking place in a few seconds. For scale, note 2 1/2-yard power shovel and two bulldozers in circled area. (Photograph courtesy of Pacific Builders and Engineers, Inc.)

work and a corrective procedure based on the previous characteristics of the slide is likely to be the wrong one.

FALLS

Rockfalls and soilfalls are best recognized by the accumulation of material that is not derived from the underlying slope and that is foreign to normal processes of erosion. In most cases this material consists of blocks of rock or earth scattered over the surface or forming a talus slope. If undercutting by lake or stream waters has caused the fall, the rate of failure is proportional to the ability of the water to remove the fallen material. Thus a fast-moving stream may remove material almost as fast as it falls, thus removing the evidence but encouraging continuous further falls. On the other hand a lake, or some parts of an ocean shore, must depend only on wave action to disintegrate and remove the fallen material, hence the evidence tends to remain in sight but continued falls tend to be inhibited.

Most of the material yielded by a rockfall is necessarily close to the steep slopes from which it came, but some may

bound down the slope and come to rest far from its present source.

If the rockfall or soilfall is active or very recent its parent cliff is commonly marked by a fresh irregular scar. This scar lacks the horseshoe shape that is characteristic of slumps; instead, the irregularity of its surface is controlled by the joints and bedding planes of the parent material (see Figs. 41 and 42).

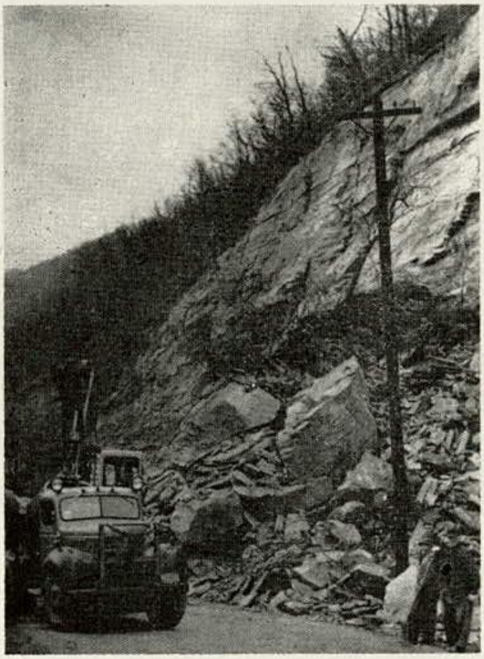


Figure 41. Rockslide-rockfall along Route 105 in Pennsylvania. Beds nearly flat; slide controlled by joints dipping at steep angle toward road. (Photograph by Pennsylvania Department of Highways)

Some idea of the intensity and state of activity of a rockfall can be inferred from the presence or absence of vegetation on the scarp and by the damage done to trees by the falling rocks. In active areas, the trees are scarred and debarked or there is evidence of healed wounds. If the rockfall is severe or long-continued, conifers and other long-lived trees are absent; their places may or may not be taken by aspen or other fast-growing species. Many rockfalls follow chutes or dry canyons that can usually be differentiated from normal watercourses or paths cut by snow avalanches (see Fig. 42).

Some soilfalls exhibit most of the characteristics of rockfalls; others proceed by mere spalling of the surface; but even this activity, if long continued, can lead to removal of considerable quantities of material. Any rockfall or earthfall may, of course, presage major landslide movement in the near future.

Slides

Slides, as distinct from falls and flows, are characterized by a host of features that are observable at the surface. These features are related to the kind of material in which the slide occurs, as well as to the amount and direction of motion.

Slumps are characterized by rotation of the block or blocks of which they are composed, whereas block glides are marked by lateral separation with but little vertical displacement and by vertical, rather than concave, cracks. Lateral spreading with few if any cracks, on the other hand, is characteristic of earthflows. One form of lateral spreading, in which a plastic layer is squeezed out by the weight of an overlying rigid layer, is here termed a "piston slide." In this type, a part of the upper layer may drop vertically, without rotation, into the space left by removal of the plastic layer.

Rockslides are generally easy to recognize because they are composed wholly of rock, boulders, or rock fragments. Individual fragments may be very large and may move great distances from their source. Most rockslides are controlled by the spacing of joints and bedding planes in the original rock. There is particular danger of forming a rockslide of serious proportions if construction is undertaken in an area marked by a system of strongly developed joints or by bedding planes that dip steeply outward toward the natural slope. This is especially so if the natural slope angle is steeper than

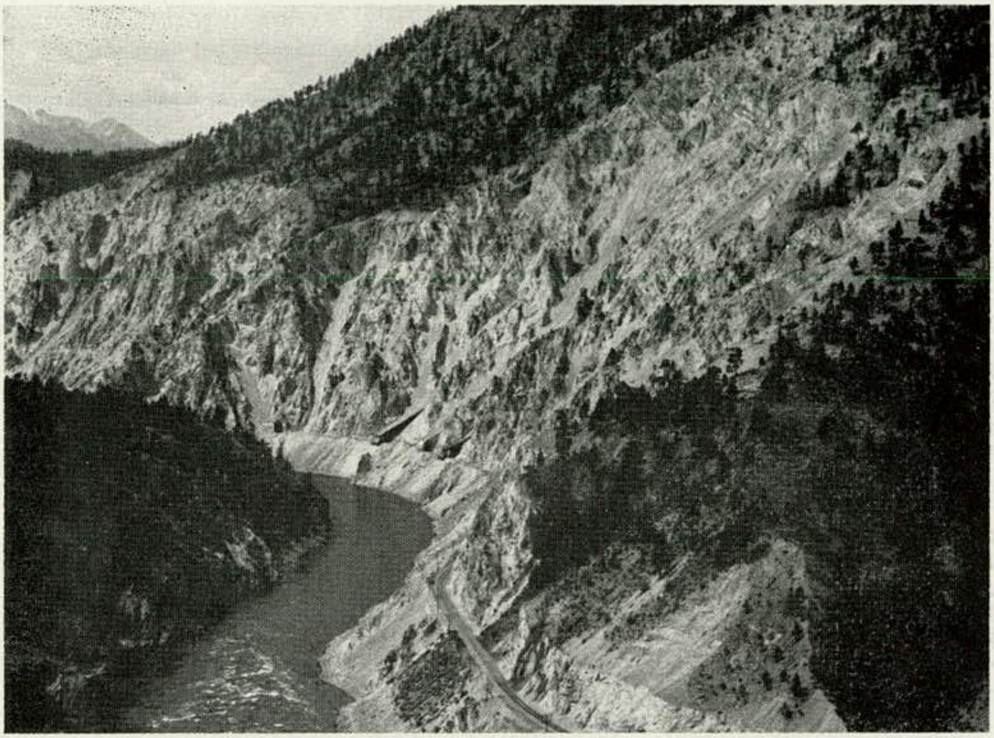


Figure 42. Rockfall and rockslide near Skihist, British Columbia, on Canadian National Railroad. Note the bare active slopes, the closely spaced jointing of the rocks, the rock chutes, and the absence of water. This picture also shows one method of protection against landslides — falling debris from above is by-passed over the tracks by means of wooden and concrete sheds. (Photograph by F. O. Jones, U. S. Geological Survey)

the angle of repose of the broken rock. Water is seldom an important factor in causing rockslides (see Fig. 42), although in some instances it helps to weaken bedding or joint planes that would otherwise offer high frictional resistance. Any seepage that is apparent after a rockslide has taken place is most likely to be seen in the scarp region or, perhaps, in the slide material itself.

Slumps rarely form from solid, hard rocks, although special combinations of factors have been known to produce them. Slumps are widespread, however, in sands, silts and clays and in the weaker bedded rocks. There they can be readily identified from surface indications, though only after considerable movement has taken place.

The head region of a slump is characterized by steep escarpments and by visible offsets between separate blocks of material (Fig. 16). The highest escarpment is commonly just below the crown; because the crests of the flanks are lower than the crown they can be recognized as flanks even if the escarpment on one of them happens to be larger than the one below the crown. If the landslide is active, or has been active recently, the scarp is bare of vegetation and may be marked by striations or grooves that indicate the direction of movement that has taken place. At the head, the striations tend to reflect downward movement, whereas striations along the flanks may be nearly horizontal. If the slump is compound, its several horseshoe-shaped scarps will appear as a scalloped edge in plan view (see Fig. 32).

Undrained depressions and perimeter lakes, bounded upward by the main scarp, characterize the head regions of many slumps; even if internal drainage prevents such ponds from holding water for long periods their depressions may be evident. In humid regions the head area may remain greener than surrounding areas because of the swampy conditions. In the San Juan region of southwestern Colorado, for instance, groves of aspen trees are commonly good indications of wet ground conditions, hence of slides and unstable ground. In northern West Virginia the swampy areas in the head remain green during the winter, whereas the well-drained toe areas are brown.

The tension cracks near the head of a slump are generally concentric and parallel to the main scarp. Many such cracks are obscured by rubble or other noncohesive materials, but even so they may be indicated by evidence of surface "stretching," by lines of rock fragments that have been displaced, or even by blades of grass that have been pulled down into the cracks by sand or other loose surface material as it sifted into them.

The head region of a slump, or of a block glide with surficial slump characteristics, may also be recognized by the presence of slump grabens which have experienced some rotation (Fig. 43). These are depressed fault blocks of soil or rock, caused in part by decrease in curvature of the shear plane. This produces tension and ultimate failure in the main slump block because of lack of support on its uphill side.

The amount and direction of rotation that any slump block has undergone can usually be ascertained by determining the slope of its surface as compared with that of the original slope. Comparison of the dip and strike of bedded material within the slump block with the original attitude of the unslumped material is an even more exact and more foolproof method of determining the amount of rotation, because erosional alteration of the surfaces may give an erroneous impression. In a general way the amount of rotation is a measure of the amount of displacement.

The part just above the foot of a slump is a zone of compression. The slumped material is confined by the foot and by the flanks of the main scarp so that it is compressed by the load upslide into the bottom part of the bowl-shaped surface of rupture. In this region, therefore, there are no open cracks.

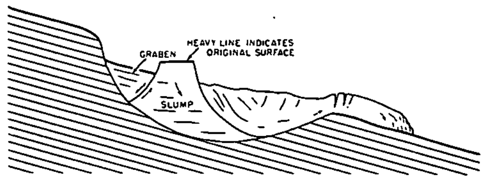


Figure 43. Graben on a slump slide. The rotation of a slump block is uphill, resulting in a flattening of the original ground surface; whereas the graben block, which breaks off from the slump block, rotates downhill. Grabens do not form if the slump block has sheared on a surface that approaches the arc of a circle; instead, they form on slump blocks that slide over a principal surface of rupture having a marked decrease in its curvature, causing a greater horizontal movement for each foot of vertical offset in the center portion of the slide than in the head region.

The foot region is marked by a zone of tension and uplift because the slumped material is required to stretch over the foot before it passes further downhill. This stretching destroys any remaining slump blocks because of the change in direction of forces. The small blocks and fragments that result tend to weather to rounded forms, producing hummocky ground. It is also in the foot zone that pavement uplifts and cracks, so disconcerting to the motorist and highway engineer, are most likely to take place.

Seeps, springs, and marshy conditions commonly mark the foot and toe of a slump. Moreover, trees tend to be tilted downhill, rather than uphill as they near the head (see Figs. 44 and 46). This is because there is a tendency for the surface material to roll over (see Figs. 29 and 43) as it moves downhill on the original slope surface, tilting trees and

grass and even burying trees and other material that fall before it.

The approximate age of some slumps can be inferred by study of the bent trees. If older trees are bent but younger ones are straight, for instance, it is probable that the slide has not moved during the life of the younger trees. On the other hand, the sizes of the tree trunks at the points where the bends occur give a running history of the rotation.

Just below the foot of a slump the ground is commonly marked by long transverse ridges, separated from one another by open tension cracks. These cracks seldom remain open for long, and they do not form scarps or other evidence of displacement, for the material is no longer confined but spreads out laterally and develops radial cracks at the toe.

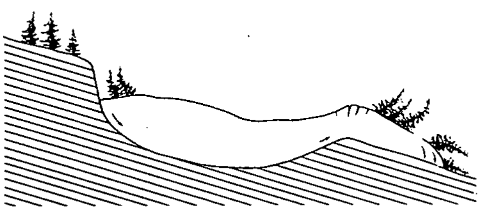


Figure 44. General orientation of trees on a slump landslide. Because of rotation, the trees on the blocks are bowed uphill, a result of the tree tops tending to grow vertically while the stump portion changes with the rotating land surface. Contrast the head and the toe region, and compare with Figure 46.

Rockslides. — Rockslides can be distinguished from block glides and slumps by their size, shape and makeup. They commonly occur only on steep slopes and most of them are single, rather than multiple. They are composed of numerous small block units with random rotation, mixed in a matrix of finer-grained material. Most of them are wet, and large rock fragments tend to float on or in the matrix. Rockslides have no definite surface of rupture that is concave upward, as do slumps, and they do not move as unrotated multiple units like block glides. Many rockslides are thinner than either slumps or block glides because they are commonly restricted to the weathered zone in bedrock or to surficial talus. The shape of its shear zone, therefore, conforms with the unweathered bedrock surface and is not controlled to any large extent by joints or bedding planes in the bedrock. Many talus slopes produce rockslides by failure within the body of talus.

Flows

Dry flows are not difficult to recognize after they have taken place, but it is virtually impossible to predict them in advance. They are commonly very rapid and short-lived. Dry flows are rarely composed of rock fragments, more commonly of uniformly sized silt or sand. They exhibit no cracks above the main scarp and flow lines in them are poorly developed or nonexistent. Except for sand runs, they have no well-defined foot.

If rock fragments are set in motion by free fall, their inertia may cause them to act like a fluid and to flow a mile or more out into a valley. Dry loess may be set in motion by earthquake or other external vibrations, become fluid, and flow down a slope. Sand runs also behave somewhat as fluids, but in the latter part of their course the sand particles are more likely to slide than to flow.

Wet flows occur when fine-grained soils, with or without coarser debris, become mobilized by an excess of water. Most of them behave like wet concrete in a chute, with differences due to water content, but the flow of some wet silts and fine sands is triggered by shocks. "Quick" or sensitive clays may be liquefied by the leaching of salts or by other causes that are not completely understood.

Wet flows are generally characterized by greath length, by their generally even gradient and surface, by the absence of tension cracks, and by the lack of blocky units and minor scarps. If tension cracks are present, they are bowed in the direction of movement (see Fig. 45), showing the effect of movement of wetter material in depth beneath the drier crust. An older flow that has had time to dry

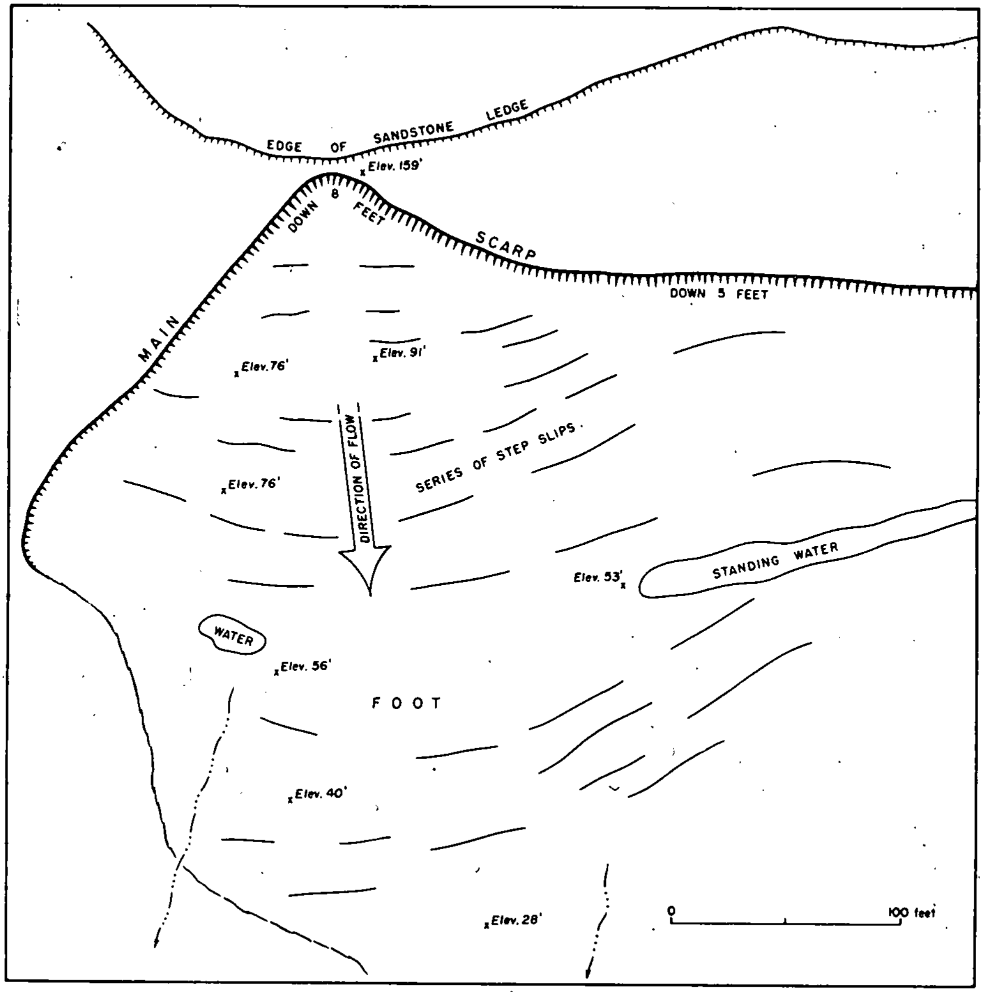


Figure 45. Crack pattern in slump that indicates flowage in depth beneath harder material at surface. Broken pipes from reservoir at top of hill dumped a large amount of water into an old slide and reactivated it. Horseshoe-shaped scarp is imperfect, differentiating it from that of a true slump. The greatest movement is near the center of the slide, as indicated by arrangement of cracks and of standing water. The fact that cracks are convex outward is indicative of flow movement in depth. South side of Reservoir Hill, Dunbar, W. Va. (From drawings supplied by Robert C. Lafferty, Consulting Geologist)

may show large shrinkage cracks or flow lines. In many cases the main scarp area is emptied by removal of all flow material and resembles a glacial cirque in some degree. In other cases there may be imperceptible gradation downward from soil creep to mudflow.

The rate of flow is dependent on the total amount of material that feeds the slide; the accumulated debris imposes a hydrostatic head on the entire mass below it and tends to maintain a constant rate of movement. The flow is under pressure everywhere from the material above it; consequently, the mass shows few if any cracks over the foot. Flows can and do make sharp turns and move around any firmly fixed obstacles that

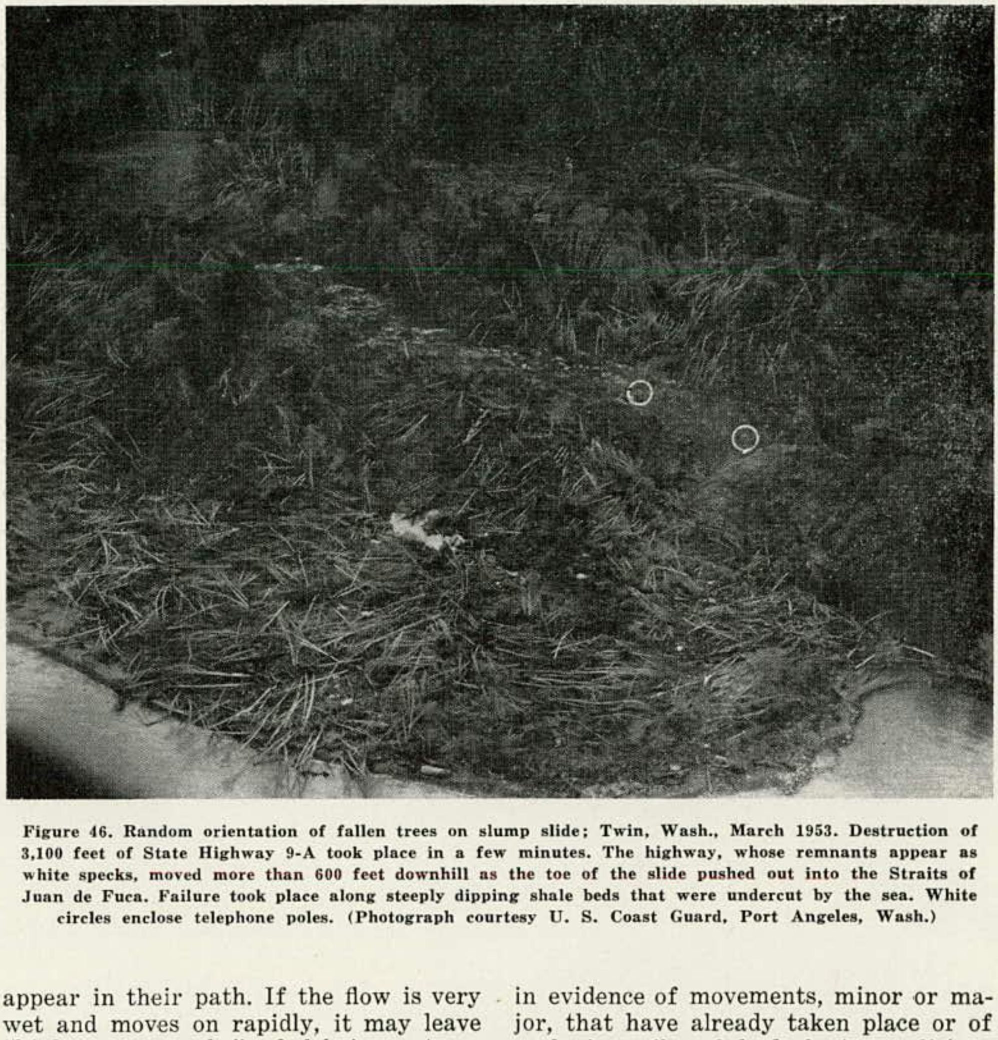


Figure 46. Random orientation of fallen trees on slump slide; Twin, Wash., March 1953. Destruction of 3,100 feet of State Highway 9-A took place in a few minutes. The highway, whose remnants appear as white specks, moved more than 600 feet downhill as the toe of the slide pushed out into the Straits of Juan de Fuca. Failure took place along steeply dipping shale beds that were undercut by the sea. White circles enclose telephone poles. (Photograph courtesy U. S. Coast Guard, Port Angeles, Wash.)

appear in their path. If the flow is very wet and moves on rapidly, it may leave "high water marks" of debris on trees or other objects; along its sides it may leave ridges of debris called torrent levees.

Conclusion

All landslide investigations must start with recognition of a distressed condition in the natural or artificial slope or of the dangers that are involved in readjustment of those stress conditions by construction work. The evidence for distressed conditions that may be present, or that may be induced, lies chiefly in evidence of movements, minor or major, that have already taken place or of geologic, soil and hydrologic conditions that are likely to cause movement in the future.

Once the fact of land movement, actual or potential, has been established, the next essential step is to identify the type of landslide. One would not apply the same corrective procedure to a rockfall as to a block glide, any more than one would attempt to prevent a slide without knowing the kind of slide he expects. If maximum benefit is to be had from the preventive or corrective measures finally employed, therefore, it is imperative to learn to recognize the kind

of slide that exists or that is to be expected.

This chapter attempts to isolate certain specific characteristics that will prove that there has been, or will be, movement and that will help identify the type of landslide that is involved. Landslides are not simple, however, and more than one kind of motion is involved in many of them. Despite the complexity of a given slide or its associated geologic conditions, most of the facts in the foregoing paragraphs can be applied beneficially. The knowledge so gained will help limit the problem, serve as a guide to the drilling program, and restrict the choice of preventive or corrective procedures that may be applied.

It must be recognized, of course, that most of the criteria mentioned throughout this chapter must be qualified. Compare, for instance, the statement that "the trees in the head region of a slump slide lean uphill whereas those near the toe lean downhill or lie flat" with the situation shown in Figure 46. There, thousands of trees, which were felled by a slump slide in a matter of minutes, lean in all directions. Even here a careful frequency count would show the foregoing statement to be true; but without such a count the leaning trees provide no evidence as to their places on the slide or their relations to the individual slump blocks.

Reference

Terzaghi, Karl, and Peck, R. B., "Soil Mechanics in Engineering Practice." John Wiley & Sons, New York, N. Y., fig. 151, p. 354, 1948.

Chapter Five

Airphoto Interpretation

Ta Liang and Donald J. Belcher

Airphoto interpretation, one of the many tools for recognition of actual or potential landslides mentioned in the preceding chapter, warrants treatment in a separate chapter. This is because the interpretation of aerial photographs for engineering purposes is a relatively new and growing field. It is one, moreover, whose techniques and possibilities are perhaps less known to most than are most other engineering and geologic techniques.

Highway engineers have long been familiar with the use of topographic maps, both in planning and in ground reconnaissance. It was natural, therefore, that when aerial photographs became available, the engineer should make use of them as an additional tool. Aerial photographs present a complete map, as well as a three-dimensional model of the area covered. When properly interpreted, they reveal not only the topography but also considerable information concerning soil, geology, and other natural, as well as manmade, features.

Use of airphoto interpretation in various phases of highway engineering has increased rapidly during recent years. The fact that almost all of the United States and a good part of the world is already covered by aerial photography of suitable scales is an important stimulant. New photography is being added rapidly.[5] In addition, new techniques in production and interpretation processes have continued to extend the advantages of aerial photography.

Advantages

The advantages of using airphotos in the investigation of landslides are summarized as follows:

1. Airphotos present an over-all perspective of a large area. When examined with a pocket or mirror stereoscope, overlapping airphotos give a three-dimensional view.
2. Boundaries of existing slides can be readily delineated on airphotos.
3. Surface and near-surface drainage channels can be traced.
4. Important relationships in drainage, topography, and other natural and manmade elements that seldom are correlated properly on the ground become obvious in airphotos.
5. A moderate vegetative cover seldom blankets details to the photointerpreter as it does to the ground observer.
6. Soil and rock formations can be seen and evaluated in their "undisturbed" state.
7. Continuity or repetitions of features are emphasized.
8. Routes for field investigations and program for surface and subsurface ex-

[5] Detailed information as to availability of existing airphotos may be obtained from: Map Information Service, U. S. Geological Survey, Washington 25, D. C. Prevailing scales of photographs: 1:15,000 to 1:30,000. Price for each photograph, covering 6 to 9 square miles: \$0.50 to \$0.65. Airphotos taken specifically for highway projects are usually of much larger scale and may be procured through the highway authority concerned.

ploration can be effectively planned.

9. Recent photographs can be compared with old ones to examine the progressive development of slides.

10. Airphotos can be studied at any time, in any place, and by any person.

11. Through airphotos, information about slides can be transmitted to others with a minimum of ambiguous description.

Limitations

Although aerial photography proves a very useful tool for the study of both existing and potential landslides, the highway engineer should be aware of its limitations. Some of these follow.

Personal Experience. — The usefulness of airphotos increases with the individual's experience in interpretation and with his knowledge concerning the area under study. An inexperienced interpreter should be particularly careful in a new, complex area in which he has little background knowledge.

Scale. — The scale of ordinary existing photography (1:15,000 to 1:30,000) is adequate for the study of most terrain and slide problems. However, in geologically complex areas or in areas where landslides are rather small, a scale of 1:5,000 to 1:10,000 would be desirable. Pictures within this range of scale are commonly available when the route has been photographed for photogrammetric mapping purposes. Photography of scales even larger than this is good for detailed examination, but the area covered in each photograph is then limited and, therefore, the over-all perspective is more difficult to grasp.

City Development. — In well built-up areas, natural conditions are altered or concealed by human activities. There, air photography may have special merits in city planning and related purposes, but its usefulness in landslide investigation is greatly handicapped, especially when the landslides are small.

Ground Investigation. — It should be emphasized that the use of airphotos cannot and should not replace ground investigation entirely. Through careful planning with airphotos, however, the surface and subsurface exploration necessary for a landslide study can be profitably reduced to a minimum.

Principles of Airphoto Interpretation

The interpretation of airphotos includes three major steps: (a) examination of airphotos to get a three-dimensional perception, (b) identification of ground conditions by observing certain elements appearing in the photographs, and (c) interpretation of photographs with respect to specific problems by association of ground conditions with one's background experiences. The quality and reliability of any interpretation is, of course, enhanced in direct ratio to the interpreter's knowledge of the soils and geology of the area under study. The acquirement of such knowledge, either by field examination or by study of available maps and reports, should, therefore, be considered an essential part of any photointerpretation job.

Three-dimensional perception can be acquired with a little practice by any person having normal vision. Ability in the identification of ground conditions and the interpretation of them in terms of specific engineering problems grows with one's experience in the use of aerial photographs and in his specific field.

There are several major elements that can be seen in air photographs that indicate ground conditions accurately. They are: landform, drainage and erosion, vegetation, soil tones, and man-made features. These features are discussed briefly hereafter; more thorough treatments appear in the papers of Belcher (1943, 1946) and Liang (1952). A bibliography on airphoto interpretation in general was compiled by Colwell (1952) and should be consulted.

Landform

The term landform as used by photo-interpreters indicates a mappable unit

of the earth's surface that appears on the aerial photograph to be made up essentially of a single kind of geologic material, which together with similarity in overburden and in topographic expression give a recognizable homogeneity to the unit. Because the underlying geology tends to be the key factor in determining the appearance of a unit in aerial photographs, most of the landforms described in this chapter are given geologic terms.

Certain landforms are more susceptible to landsliding than are others, hence the identification of landform is highly important. By observing the topographic expression and the boundary of a unit area, and by comparing it with known sample photographs, a landform can often be identified on airphotos. For areas where geologic or soil maps are available, such identification can, of course, be checked against the facts shown on those maps.

The following major landforms (in the airphoto-interpretation sense) are classified according to differences in their physical composition: consolidated sedimentary rocks, intrusive and extrusive igneous rocks, metamorphic rocks, glacial deposits, unconsolidated sedimentary deposits, and windlaid materials. Each of these groups, together with the normal weathering products of each one, poses relatively distinct problems for the engineer, particularly from the standpoint of landslide susceptibility. Each one, moreover, can be more or less easily identified on aerial photographs. Numerous examples of each of the foregoing landforms, and of subtypes of each, are described and illustrated in the references previously cited.

Drainage and Erosion

The density and pattern of drainage channels in a given area reflect directly the nature of the underlying soil and rock. The drainage pattern is obvious in some cases, but more often it is necessary to trace the channels on a separate sheet of paper in order to study the pattern successfully.

Under otherwise comparable conditions, a closely spaced drainage system denotes relatively impervious underlying materials; widely spaced drainage, on the other hand, indicates that the underlying materials are pervious. Generally speaking, a treelike drainage pattern develops in flat-lying beds and relatively uniform material; a parallel stream pattern indicates the presence of a regional slope; rectangular and vinelike patterns, composed of many angular drainageways, are evidence of control by underlying bedrock, and a disordered pattern, interrupted by haphazard deposits, is characteristic of most glaciated areas. Indeed, disordered pattern of a much smaller scale is common in landslide deposits. There are other patterns developed in response to special circumstances. A radial pattern, for instance, is found in areas where there is a domal structure in the rocks, and a featherlike pattern is common in areas where there is severe erosion in rather uniform material, such as loess.

The shape of gullies appearing in airphotos gives valuable information regarding the characteristics of surface and near-surface materials. Thus, long, smoothly rounded gullies should indicate clays, U-shaped gullies indicate silts, and short, V-shaped gullies indicate sands and gravels.

Soil Tones

Soil tones are recognizable in photographs unless there is a very heavy vegetative cover. Black-and-white, rather than color, photography is commonly used in present-day engineering projects. Thus, the color tones examined are merely different shades of gray, ranging from black to white. Because gray tones are highly respondent to soil-moisture conditions on ground, they are an important airphoto element in landslide investigations.

A soil having high moisture content normally registers a dark tone and low moisture a light tone. The moisture con-

dition is a result of the physical properties of the soil or the topographic position of the ground, or both. The degree of sharpness of the tonal boundary between dark and light soils aids in the determination of soil properties. Well-drained coarse-textured soils show distinct tonal boundaries whereas poorly-drained fine-textured soils show irregular, fuzzy boundaries between tones.

Vegetation

Vegetative patterns reflect both regional and local climatic conditions. The patterns in different temperature and rainfall regions can be recognized in airphotos. Locally, a small difference in soil moisture condition is often detected by a corresponding change of vegetation. A detailed study of such local changes is very helpful in landslide investigations. For instance, wet vegetation, represented by dark spots or "tails," is a clue to seepage in slopes. Cultivated fields, as well as natural growths, are good indicators of local soil conditions. Thus, an orchard is often found on well-drained soils; the sparseness of vegetation in nonproductive serpentine soils, where landslides are common, is very conspicuous and revealing.

Manmade Features

The identification of manmade features such as highway, railroad, and airport locations; dams, canals, and irrigation systems; sand and gravel pits, stone quarries, mining and other industrial operations, is obviously important in the investigation of landslides. With a little practice, an engineer who is familiar with these items on the ground should have no difficulty in recognizing them in airphotos. Some old, overgrown manmade features are actually easier to see in photos than on the ground.

Interpretation of Landslides in Airphotos

Having obtained a general understanding of a given area through airphoto examination of the major elements discussed in the preceding section, the engineer may proceed to a study of the specific features that are related to landslides.

Landslide Indications

An engineer already familiar with the appearance of landslides on the ground should orient himself to the airphoto view of landslides by examining photographs of some known examples. The difference between an air view and a ground view results chiefly from the fact that the former gives a three-dimensional perspective of the entire slide area, but at a rather small scale. Ground photos, on the other hand, show only two dimensions but on a larger scale. The indications of a landslide in airphotos are: the sharp line of break at the scarp; the hummocky topography of the sliding mass below it; the elongated, undrained depressions in the mass; and the abrupt differences in vegetative and tonal characteristics between the landslide and the adjoining stable slopes. Inclined position of trees in landslides is often observable in photographs.

Where a highway is built on unstable soil, the irregular outline and nonuniform tonal pattern of broken or patched pavement are often visible, even in relatively small-scale photography. Failures due to improper fill or inherently weak soil are also registered.

Vulnerable Locations

Many slides are too small to be readily detected in small-scale photography. In addition, the highway engineer often must cover an extended territory. Consequently, it is very important for him to locate and to examine closely all of the areas where the visible signs of slides may not be apparent, but in which

there are special conditions that are conducive to slides. Typical vulnerable spots are as follows:

Cliffs or Banks Undercut by Streams. — Banks that are subject to attack by streams commonly fail by sliding. Where the banks are made up of soil or other unconsolidated material the weakest, hence most favorable slide position, is often located at the point of maximum curvature of the stream, where the bank receives the greatest impact from the water. In areas of rock outcrops, on the other hand, the section at and near the point of maximum stream curvature is often occupied by hard rock and the weak spots are to be found on both sides adjacent to that section.

Steep Slopes. — In stereo-examination of airphotos, it is reasonably easy to observe and compare the different hill slopes within a land unit. In a potentially dangerous area, large earth masses standing on the steepest slope are naturally the most vulnerable to landslides and should be examined closely. Comparison of slopes for this purpose should, of course, be confined to slopes of similar materials. Thus, a slope cut in earth or talus should not be compared with a rock cliff in an adjacent land unit.

Contributing Drainage. — Water contributes greatly to many slides. Careful examination of existing slide scars often indicates that a line connecting the scars points to some drainage channels on higher ground. Such drainage may appear on the surface or go underground and reappear as seepage water causing the damage. This drainage-slide relationship can frequently be detected in airphotos.

Seepage Zones. — Seepage is likely to occur in areas below ponded depressions, reservoirs, irrigation canals, and diverted surface channels. Such circumstances are sometimes overlooked on the ground because the water sources may be far above the landslide itself, but they become obvious in airphotos. The importance of recognizing the potential danger in areas below diverted surface drainage, especially in jointed and fractured rocks, needs particular emphasis. It has been proven repeatedly, through extensive field experience, that within an unstable area one of the most dangerous sections is the lower part of an interstream divide through which surface water seeps from the higher stream bed to the lower one. The recognition of seepage is sometimes aided by the identification of near-surface channels (appearing in airphotos as faint, dark lines), wet, tall vegetation on the slope (shown as dark dots or "tails"), and displaced or broken roads adjacent to the slope.

Old Landslides

An investigation of existing landslides in any area gives an excellent basis for evaluating the possibility of future landslides (see Fig. 47). The indications of an old slide are similar to those of new slides except that they are not as fresh or as striking. Thus, the scarp may not appear sharp; the hummocky ground surface, although still present, may be subdued topographically; drainage and vegetation may have become established on the mass; and the change of gray tones between the landslide mass and the adjacent areas may be gradational rather than abrupt. As a matter of fact, the degree to which the vegetation and drainage are established on the mass helps determine the relative age and stability of the moved land.

Once an old landslide is found on the photographs it serves as a warning that the general area has been unstable in the past and that new disturbances may start new slides. However, such a warning should not discourage construction unconditionally. The unstable condition of the past does not necessarily exist today. In some western states, for example, railroads built in extensive old landslide areas have been stable for a long time.

In addition to the registration of unstable slopes, the airphoto also furnishes an excellent reference for the engineer to judge the attitude of slopes that are

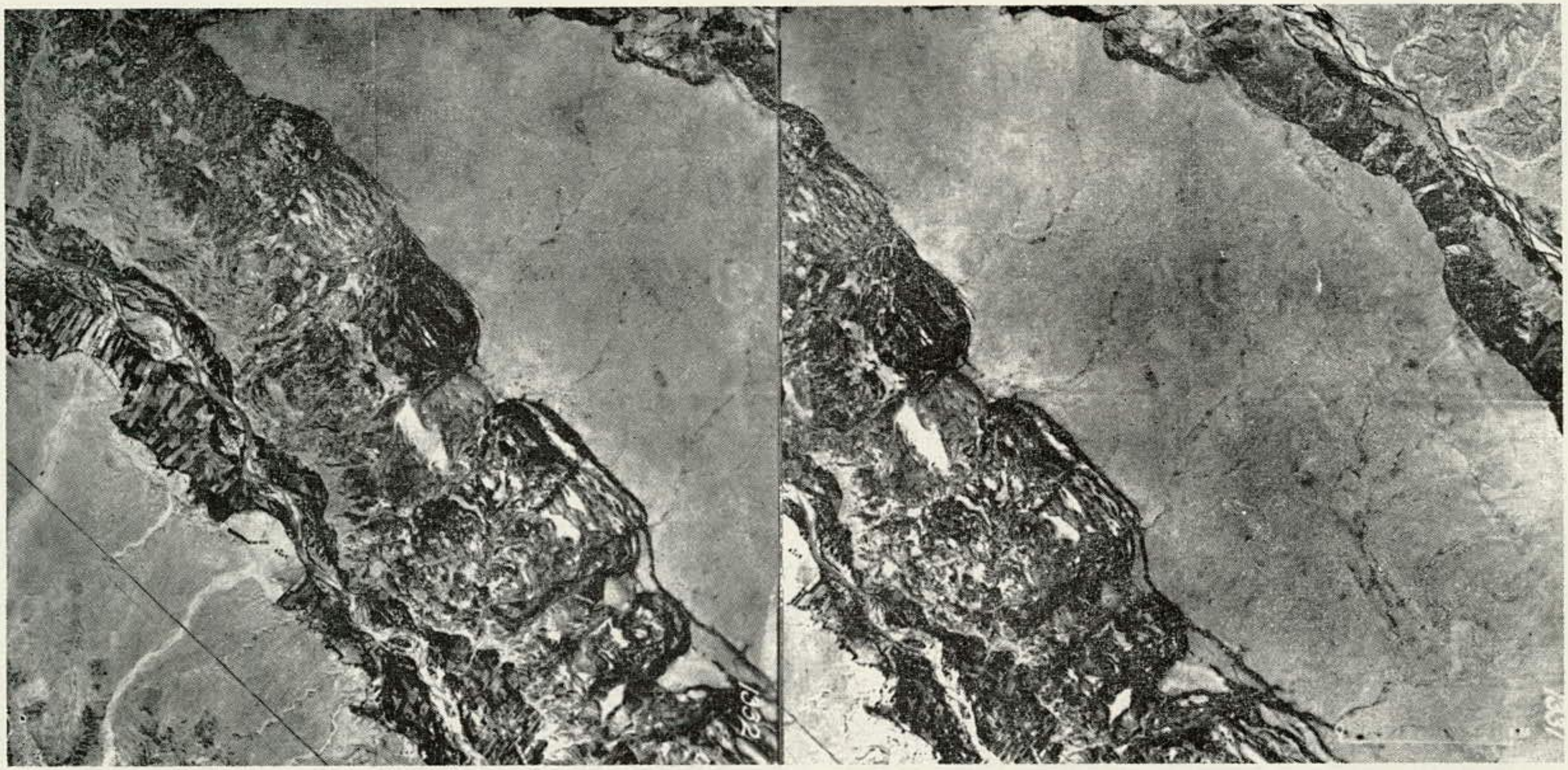


Figure 47. Old landslide, Rio Arriba County, N. Mex. This is one of the largest slide areas in the country. The slide is of such magnitude that it can be readily spotted even in the photo-index sheet. The characteristic sharp cliff at the scarp, hummocky surface and ponded depressions are well illustrated. Slides which the engineer ordinarily encounters are generally of much smaller magnitude, although they may assume similar forms.

The combination of basalt and the underlying sediments provided a favorable condition for the slide in this area; the once actively downcutting and laterally eroding river precipitated the movement. The well-established vegetation (shown in dark gray tones) and drainageways in the moved mass indicate that the general area is now stabilized. The currently critical spots are (a) where the river or artificial construction has cut into the toe of the lower slopes; (b) areas immediately below ponded depressions; and (c) areas along the cliff where imminent rockfall is indicated by breaking marks. The linear cliff above the slide indicates that the fracture pattern of the caprock is in coincidence with the horizontal axis of the slide. (Aerial photograph by U. S. Department of Agriculture)

generally stable. Within the photo coverage, there is always a wide choice of combinations of circumstances, such as drainage, topographic position, and association with a gully or stream. For guidance in the design of new slopes the engineer often can find some existing slopes having conditions similar to the ones he is to build.

Landforms Susceptible to Landslides

Landslides are rare in some landforms and common in others. Most of the forms susceptible to landslides are readily recognizable in airphotos. The identifying elements and significant facts about them are summarized and illustrated in the following sections.

It should be noted that the order of presentation hereinafter follows a sequence based on origin and character of the materials rather than on the order of their importance in landslide occurrence. In general, the forms most susceptible to landslides are basaltic lava flows, serpentine, clay shale, and tilted sedimentary rocks; other forms are susceptible occasionally, depending on local circumstances.

Consolidated Sedimentary Rocks and Their Residual Soils. — The discussion of rocks and their residual soils is combined in this and in the following two sections because the recognition of types of residual soils depends primarily on the recognition of the landform developed in the parent rocks. The determination of depth of residual soil requires considerable judgment. However, the engineer working constantly in his own region should have no difficulty in estimating the depth once he is familiar with local conditions.

Generally speaking, rounded topography, intricate drainage channels and heavy vegetation are indicators of probable deep soils, in contrast to the sharp, steep, resistant ridges and rock-controlled channels commonly found in areas of shallow soil. The local climatic and erosion pattern should be considered in the interpretation.

A very high percentage of all slides occurs in residual soils and weathered rocks. They are usually in the form of slumps or flows. Rockfalls and rockslides, by definition, occur only in bedrock terrain.

In horizontal positions, massive sandstone is little likely to slide. Clay shale, especially if interbedded with sandstone or limestone, is highly susceptible to landslides (Figs. 48, 49, 50, and 51). Landslides are uncommon in thickly bedded limestone unless it is interbedded with shale or other soft rocks. In steeply tilted positions, any sedimentary rock may fail by sliding (Fig. 52). Depending on the dip angle, joint system, and climate, slides may take one or a combination of the forms of rockfalls, rockslides, debris falls, debris slides, and earthflows. River undercutting and artificial excavation are important factors in initiating landslides in both horizontal and tilted rocks.

Methods of identification of sedimentary rocks in airphotos are well established. Hard sandstones are noted for their high relief, massive hills, angular drainage, and light tones; clay shales are noted for their low rounded hills and well-integrated treelike drainage system; and soluble limestones are characterized by their sinkhole development in temperate humid areas and by rugged karst topography in some tropical regions. Interbedded sedimentary rocks show a combination of the characteristics of their component beds. When horizontally bedded, they are recognized by their uniformly dissected topography, contour-like stratification lines, and treelike drainage; when tilted, the parallel ridge-and-valley topography, the inclined but parallel stratification lines, and the trellis drainage are evident.

The identification of landform as a means of detecting associated landslides is important in the flat-lying sedimentary group because the slides there are often small and, therefore, not very obvious in the photographs. This is particularly

Figure 48. Clay shale, Monongalia County, W. Va. This stereo pair shows an area where clay shales predominate and landslides are active. There are very few competent beds in the general area as evidenced by the rounded, soft slopes and dull, uniform, gray tones. Minor irregularities as signs of movement are seen in most of the steep slopes. Even without artificial disturbances, nature is actively reducing the relief of the area by creeps, flows, and slides. At area (A), both the railroad and highway have experienced continuous landslide troubles. The irregular outline of the bank along the river and the patchwork on the road pavement are clearly seen in the photos. The steep slope and active attack by the river provides a favorable condition for landslides. Furthermore, surface drainage in the back of the slope is blocked by a hill and water is seeping through the hill toward the river. Such a circumstance is conducive to slides. (Aerial photograph by U. S. Department of Agriculture)

Figure 49. Shale in same general area as shown in Figure 48, Monongalia County, W. Va. Area (B) shows one of the most unstable slopes in the area. The disordered, hummocky forms on the hillside indicate that flows and slides are active. The irregular outlines of the road is a sign of continuous refilling and repatching because of slides. Slide scars are also prominent on the opposite side of the valley, particularly on the steeper slopes. (Aerial photograph by U. S. Department of Agriculture)

Figure 59. Flat-lying sedimentary rocks, Mesa Verde National Park, Montezuma County, Colo. The interbedded competent sandstones and soft shales are nearly horizontal, as indicated by the contour-like stratification lines on hills. The numerous landslides and erosional scars, seen as white patches, are striking throughout the area. Serious slides are marked (A), (B), (C), (D), and (E). (C) indicates where caprock fails, (D) indicates slumps where shale is primarily involved. Drainage condition in back of (C) helped to promote the mass movement.

It is difficult to maintain the highway on the shale slopes because they are already oversteepened and do not provide a good foundation; further disturbance would hasten the slide. Because of the difficulty in maintaining the roads on the steep slopes, several routes (X) have been abandoned in the general area. A plan of relocating the scenic highway that passes the hazardous area (C) and (D) is now under consideration. The new route will follow the valleys and go through a 1,400-foot tunnel (F). (Aerial photograph by U. S. Department of Agriculture)

Figure 51. Ground view of rockslides and rockfalls in shale and sandstone, shown in Figure 50. (Photograph by National Park Service)

true for slides in colluvial deposits at the base of flat-lying beds. Furthermore, sedimentary rocks are the most widespread of all surface rocks and their conditions are to be met everywhere.

Intrusive and Extrusive Igneous Rocks and Their Residual Soils. — *Basaltic lava flows* are one of the most common representatives of the extrusive igneous rocks. They are readily identifiable in airphotos. Basalt is highly susceptible to different types of landslides (Fig. 53). Basalts often form the caprock in a plateau, with sharp, jagged cliff lines clearly visible in photographs. Surface irregularities or flow marks, sparseness of surface drainage, and dark tones are confirming airphoto characteristics.

If a basaltic flow is underlain by or interbedded with soft layers, particularly if it occupies the position of a bold escarpment, a very favorable condition for large slumps is present. The joints and the cracks in basalt give rise to springs and seepage zones and greatly facilitate movement. Rockfalls and rockslides along rim rock are usually favored by vertical jointing of basalt and by undercutting of basaltic cliffs. Talus accumulations of various magnitudes are found at the foot of cliffs. Disturbance of talus slope during road construction has caused some large slides of talus materials. Old slides and breaks indicating incipient slides often can be seen in photographs.

In areas of relatively deep weathering the landscape is somewhat modified. A more rounded topography and heavier vegetation develops, although dark tones still predominate. Slumps of both large and small size are common in basaltic soils.

Granite and related rocks are the most widely occurring intrusive igneous rock types. The landslide potential of granitic rocks varies widely, depending on

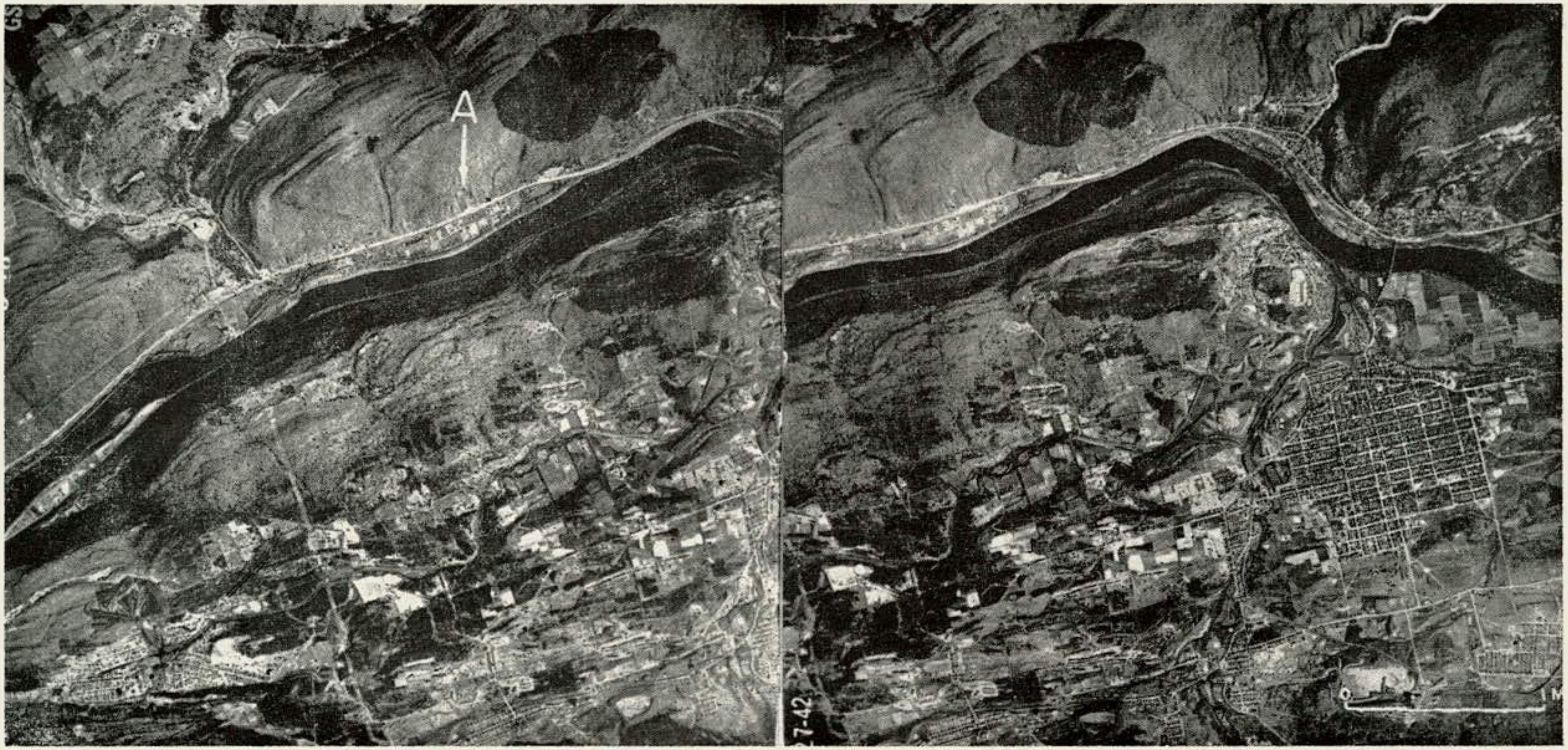


Figure 52. Tilted sedimentary rocks, Luzerne County, Pa. This stereo pair shows how an airphoto interpreter might predict the exact location and magnitude of a future slide. That this is an area of dangerously dipping sedimentary rocks is self-evident. Along the major highway, the most critical spot is at (A), where there is a clearly defined breaking line. Such an incipient break, although striking in the photo, is not obvious on the ground. Five years after this photograph was taken, when the highway below the break was being widened, the whole block of 400,000 cubic yards came down during an unusually heavy rain. (Aerial photograph by U. S. Geological Survey)

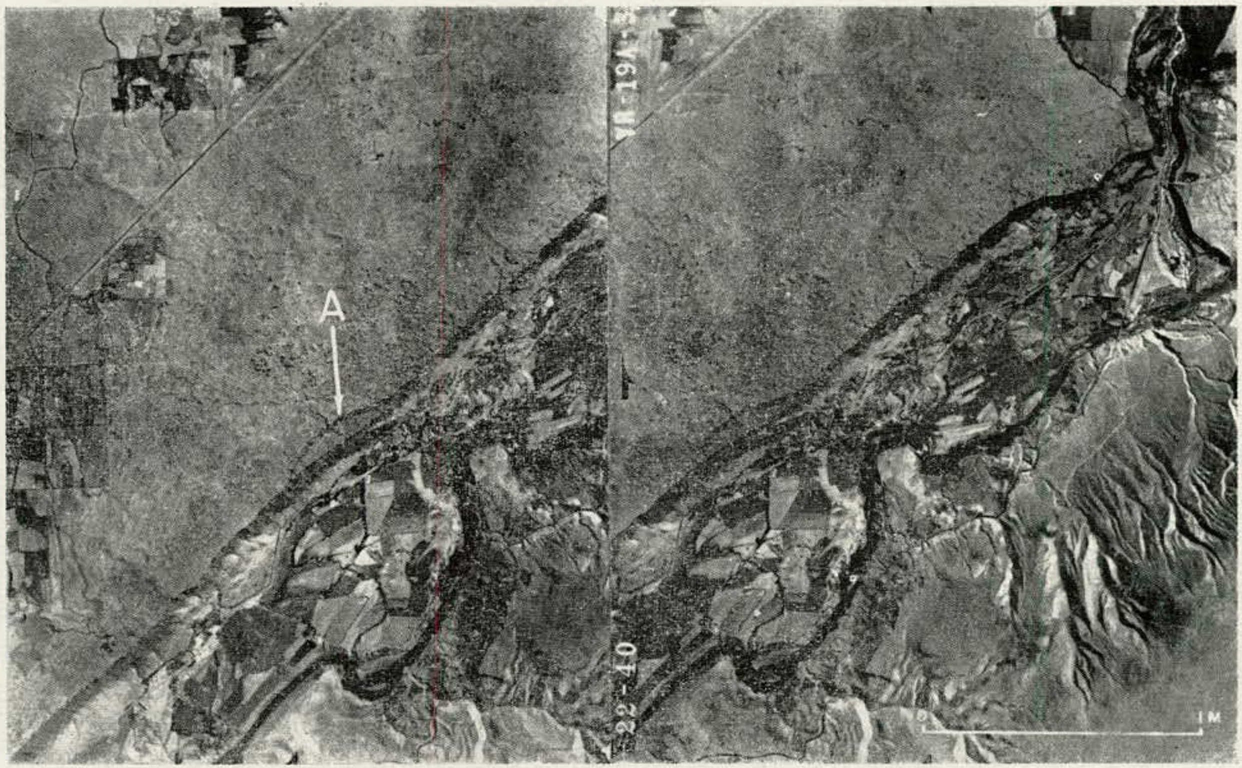


Figure 53. Basalt flow, Gooding County, Idaho. The basaltic plateau on the far side of the river is recognizable in the photo by its sharp cliff, minor surface marks, and dark tones. It is underlain by beds of tuff and clay, creating a favorable situation for landslides. There is a belt of talus accumulation and landslide deposits along almost the entire bottom of the cliff. Landslides are distinguished from talus slopes by the presence of a sharp break on the upland and the hummocky topography of the mass. An incipient slide is indicated at (A). Here, the partial breaking of a block of basalt from the mass is clearly shown; the slide can be precipitated by a slight disturbance. Smaller and less distinct breaks often appear in basalts along the cliff edge and can be detected by a careful inspection of airphotos. (Aerial photograph by U. S. Department of Agriculture)

the composition of the rock and its fracture pattern, the topography, and the moisture conditions. In granites that are highly resistant to weathering or of low relief, there is generally no slide problem. In hilly country where the granite is deeply weathered, slumps in cut slopes, as well as in natural slopes, are common. Fractures in the rock and high moisture condition undoubtedly are favorable factors in producing landslides.

Granitic masses are identified in airphotos by the rounded (old) to A-shaped (young), massive, uniform hills, and by the integrated treelike drainage pattern with characteristic curved branches. The presence of fractures and the absence of stratification and foliation aid to confirm the material.

Metamorphic Rocks and Their Residual Soils. — Landslides in metamorphic rocks vary greatly. The interpretation problem is rendered even more difficult because the criteria for identification of different metamorphic rocks in airphotos are not well established. Although the airphoto characteristics of major types of gneiss, schist, slate, and serpentine have been worked out, these rocks do not often have exposures of sufficient extent to be recognized by their topographic expression.

Within the metamorphic group, many slides are associated with *serpentine.* Serpentine areas are identified in airphotos by their sinuous ridge, smoothly rounded surface, short steep gullies, very poor vegetative cover, and dull gray tones.

There are, however, many serpentine areas where stable slopes prevail. Low relief and low rainfall are among the factors responsible for the stability of some of those serpentine slopes. A close examination of airphotos to detect existing scars is necessary before the instability of a serpentine area can be concluded. Within a general area, local conditions, such as vegetation, moisture, and slope, may create special, favorable circumstances for landslides.

Glacial Deposits. — Landslides are common in some glacial and glacio-fluvial deposits. Although most of the distinct glacial forms are easily identified in airphotos, there are complex areas which require a high degree of skill for their identification.

Moraines are found in nearly all glaciated areas. They are identified in airphotos by their jumbled, strongly rolling to hilly terrain. In moraines, particularly in the semiarid areas, there is a large proportion of waste, untilled land. Disordered drainage pattern, irregular fields, and winding roads are confirming clues.

Minor slumps, debris slides, and earthflows are common in cut slopes in moraines as the result of the presence of undrained depressions and seepage zones in the mass. Because morainic hills are usually small, these slides are not very extensive. They are, nevertheless, large enough to cause continuous trouble to many highway maintenance engineers (Figs. 54 and 55).

Slides in shallow *glacial mantle* overlying bedrock often take the form of slumps, debris slides, and debris falls, and often contribute to failures in artificial fill. They usually occur along valley walls that have been oversteepened by glaciation. The topography of such areas is basically that of the underlying bedrock with slight local modifications, depending on the thickness of the mantle. These cases are commonly found in the northern and northeastern United States where sedimentary beds predominate (Fig. 56). Slides seldom occur in other kinds of glacial deposits, such as kames, eskers, outwash plains, and till plains.

Unconsolidated Sedimentary Deposits. — Within this group, which includes such diverse forms as flood plains, alluvial fans, beach ridges, and swamps, most landslide problems are associated with dissected coastal plain deposits, river terraces, and lake beds.

Coastal plains are among the well-established forms that can be definitely recognized in airphotos. An undissected coastal plain is identified by its low, flat topography; its association with tidal

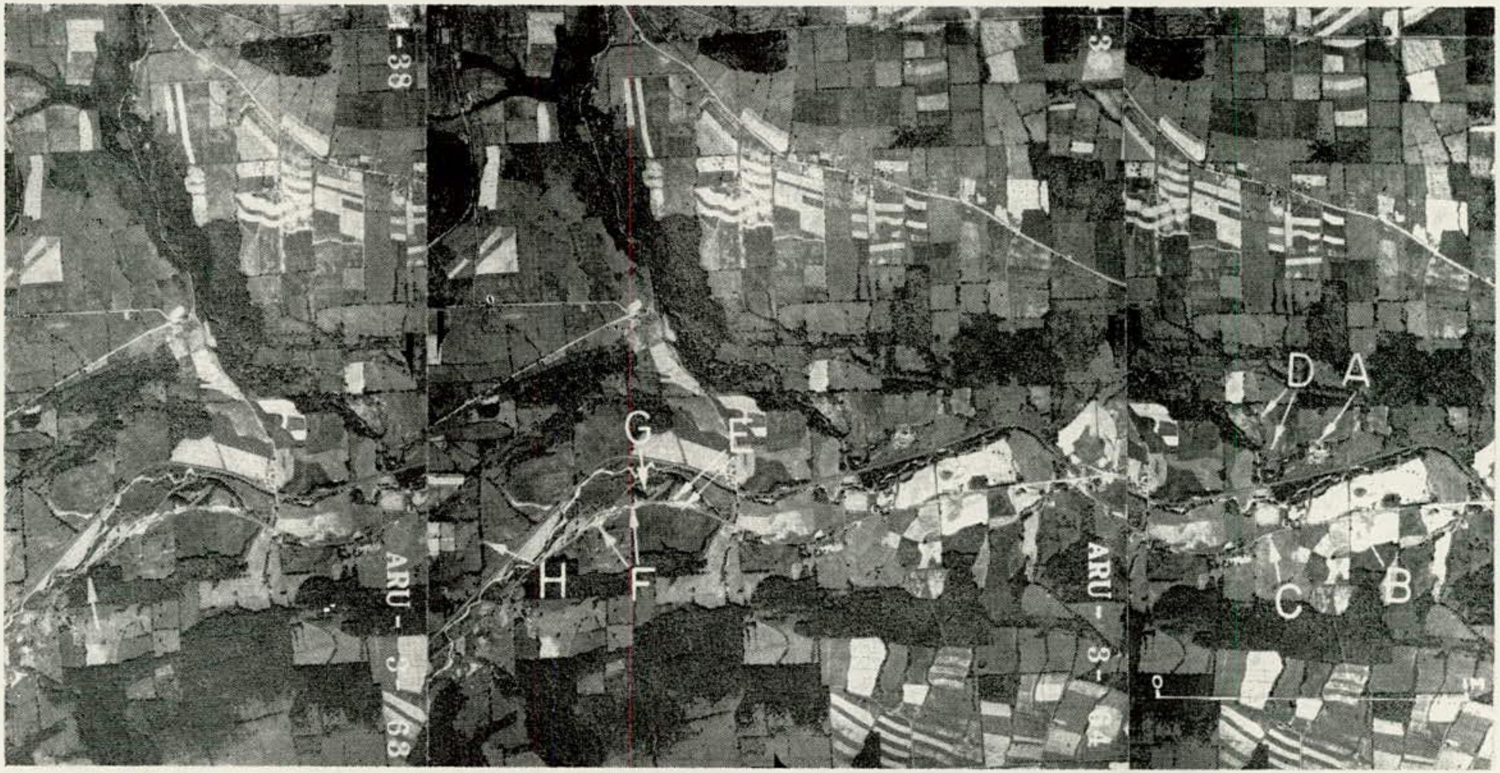


Figure 54. Moraine and lake bed, Tompkins County, N. Y. New slumps and debris slides, as well as old slide scars, are common occurrences in this glaciated valley where post-glacial erosion has dissected the moraines and the overlying lake deposits. Morainal areas are recognized by their hummocky topography. Lake deposits are usually distinguished by their flat, horizontal disposition. However, when lake clays are dissected, such a criterion no longer holds. Rather, the clues for clay identification, such as the characteristic smooth slopes, high degree of dissection, and gradual change of color tones, are more applicable.

A close inspection of the photo reveals that there are many old landslides, (A) being a prominent example. Other old slides, such as (D), (G), and (H), are common throughout the valley. All of them have been more or less stabilized, as indicated by the established vegetative pattern. Most highway cuts of moderate depth have experienced landslides, as in (B), (C), (E), and (F). Although deep-seated and large-scale slides are not likely to occur in such an area, continuous maintenance work in clearing the sliding material and in protecting slopes from erosion is necessary. (Aerial photograph by U. S. Department of Agriculture)

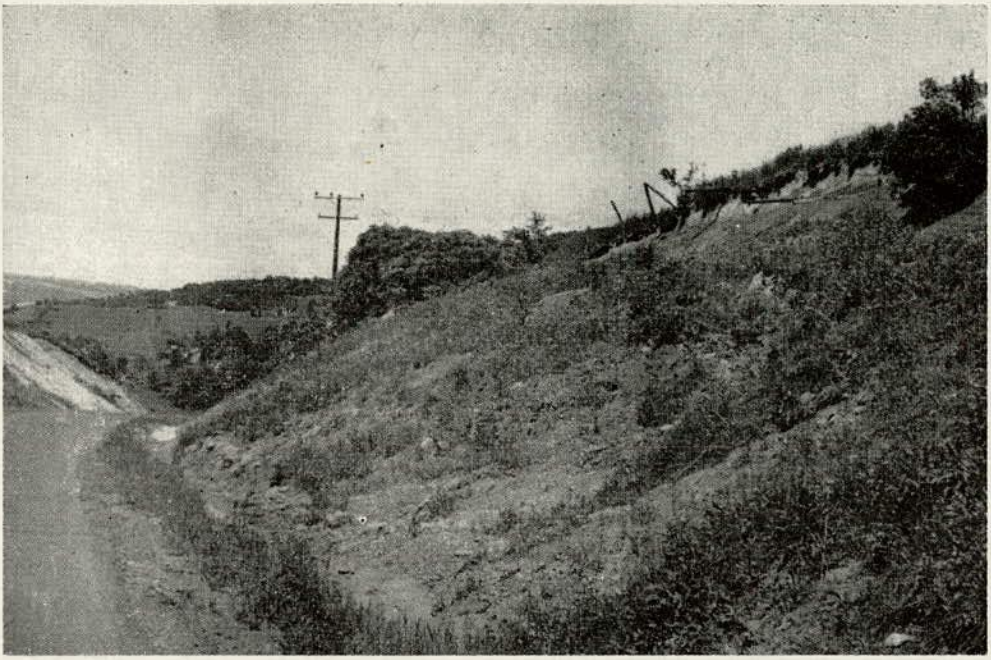


Figure 55. Ground view of a typical slide on a cut slope shown in Figure 54. (Photograph by Donald J. Belcher)

flats, marshes, and swamps; and the presence of broad, shallow, tidal stream channels. The dissected coastal plain is identified by its rolling to rugged topography and integrated drainage system. It is also associated with coastal features and appears on airphotos to be somewhat similar to areas underlain by consolidated sedimentary rocks.

In undissected plains, landslides offer a problem only in the construction of canals or similar structures that require deep excavation in flat lands. In dissected plains, however, slumps in natural hill slopes, as well as in road cuts, are common (Figs. 57 and 58). The stratified and unconsolidated nature of the sands, silts, and clays that characterize most coastal plains have provided a favorable situation for landslides.

Terraces are easily recognized in airphotos as elevated flat land along major or minor valleys. Terraces of gravel and sand are usually stable, maintaining a clean slope on the face. However, where terraces are composed of interbedded silts and clays, or where the natural equilibrium is disturbed by artificial installations, slumps will occur. Slumps in terraces naturally start on the unsupported slope facing the low land. The presence of slide scars along the terrace front is a reliable indicator of instability (Fig. 59).

Lake bed deposits generally display flat topography unless they are dissected. Although generally composed of clays, lake beds have little chance to slide except when exposed at valleys or at deep cuts. There have been slides of considerable magnitude in lake clays under each of the following circumstances: (a) where lake clays are interbedded with or, especially, are overlain by granular deposits, and (b) where lake clays overlie bedrock at shallow depth and the base level of erosion of the general area is greatly lowered. The former situation is common in some glaciated regions of New York. The latter combination has produced slides of extraordinary magnitude in western Canada.

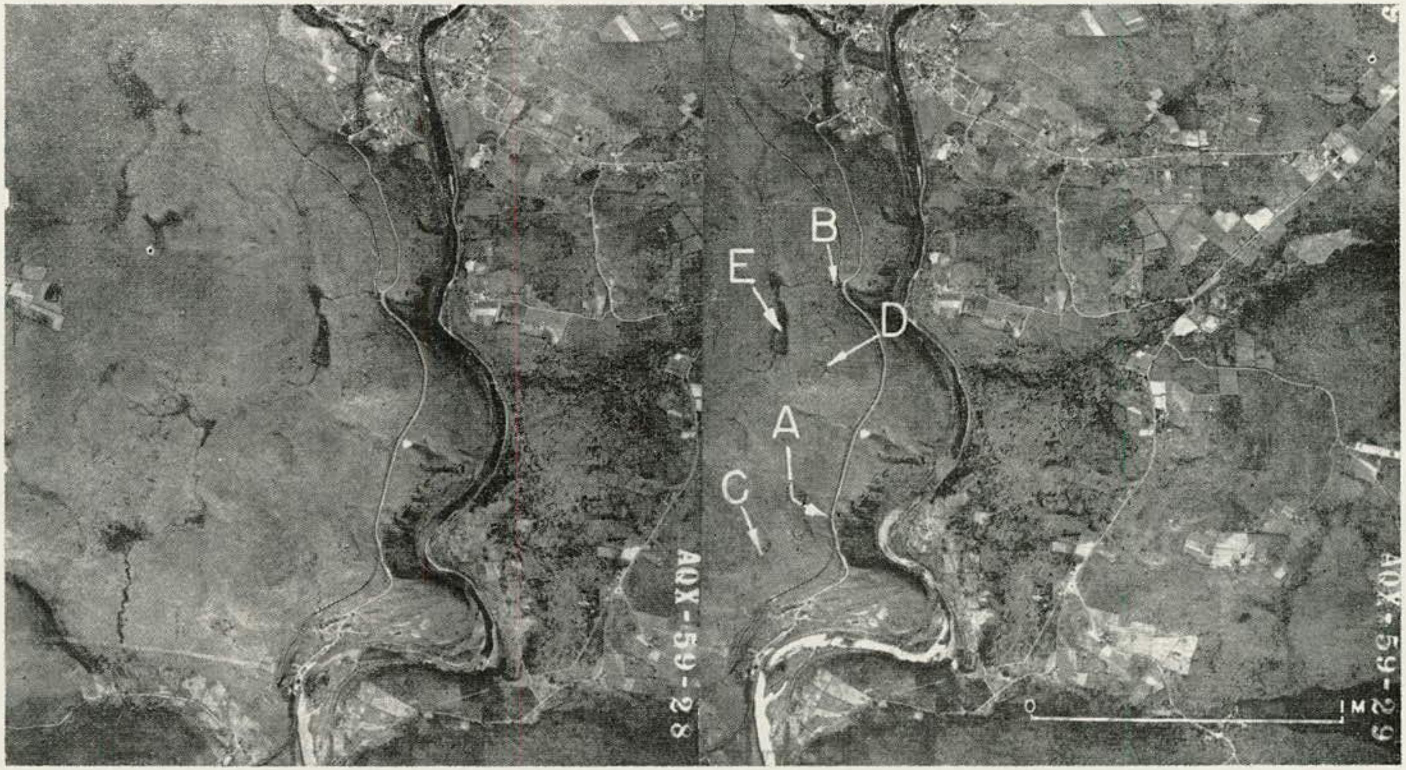


Figure 56. Glacial mantle over bedrock, Pike County, Pa. Thin deposits of unconsolidated materials on bedrock often develop landslides along stream valleys where undercutting is prevalent and the bank slopes are progressively steepened. Glacial deposits are usually unequally distributed over the bedrock — thinner on the hills and thicker over the valley. They tend to smooth the original bedrock topography. In the picture, at (A), the slide is shown to progress toward the road, threatening the road and the pipeline of a hydroelectric plant behind the road. A similar threat, though of lesser degree, exists at (B).

In examining the airphotos, it is clear that sandstone and shale outcrop at places like (C) and (D). On the basis of the general configuration of the sedimentary rock hills, the relative depths of unconsolidated deposits at various points can be estimated and a systematic, instead of haphazard, program for subsurface investigation can be planned.

Drainage conditions are clearly shown in the photographs. Ponded depressions, like (E), are obvious in the picture; but on the ground, it would take much time and effort to locate them in this tree-covered area. Drainage of such depressions would reduce the danger of impending slides below them. (Aerial photograph by U. S. Department of Agriculture)

Figure 57. Coastal plain, Prince Georges County, Md. The stereo pair shows a proposed road (white line on left photo). The dissected plain is identified by its low, soft hills and the associated tidal channels. Cut slopes steeper than the natural slopes are susceptible to slides unless adequate precautions are taken. In highway location in an area like this, it would be better to set the grade line below dangerous clay layers so that even if a slide occurs, it would not affect the foundations of the road. At (A), a road constructed after the photos were taken was located above the clay. The subsequent slide not only damaged the upper slope but took away part of the pavement as well. At (B) and (C), the road cut into the toe of the natural slopes. Since the road was located below the clay layer, slides in both places occurred on the cut slope only. The cut slope at (D) also failed, but the drainage and topographic situation was more favorable there and the slide was stabilized shortly. At (E) is an old slide that can be easily recognized in the photograph; it is hidden by vegetation when inspected on the ground. (Aerial photograph by U. S. Department of Agriculture)

Figure 58. Ground view of landslide at point (A) shown in Figure 57. Here, the gray clay layer lies underneath the pavement, which is damaged by the slide. (Photograph by Ta Liang)

Undissected lake clays are easily identified in airphotos by their characteristic broad, level tracts, dark gray tones, and artificial drainage practices. Dissected and complex lake bed areas are relatively difficult to identify, particularly for one that is not familiar with the local geologic conditions. Again, the presence of existing slides is the most reliable warning signal.

Windlaid Materials. — *Loess,* or wind-deposited silt, can be identified unmistakably in airphotos by its vertical-sided gullies, which are evenly spaced along wide, flat-bottomed tributaries to show a featherlike drainage pattern. Equal slopes on hills and valleys, an indication of uniform material, heavy vegetative cover, and soft gray tones serve to confirm the landform.

Loess is well known for its minor slumps, generally called catsteps. The catsteps are seen in airphotos as fine, roughly parallel, light tone contours (Figs. 60 and 61). On the ground, the individual steps of these small slumps are commonly 2 to 4 feet wide, and several inches to 2 or 3 feet high.

Complex Forms. — Most of the landforms previously described may be called simple forms because they consist predominantly of one type of material in each unit. In nature, however, complex or superimposed forms are numerous and of common occurrence. This is especially true in glaciated areas, as mentioned previously. They are further emphasized here because of their significance in landslide studies. Airphoto recognition of the basic simple forms is definitely helpful in the interpretation of complex forms.

A change of material vertically or horizontally in complex areas often affects the internal drainage characteristics and creates slope stability problems. The most common situation favorable to slides is when impervious formations

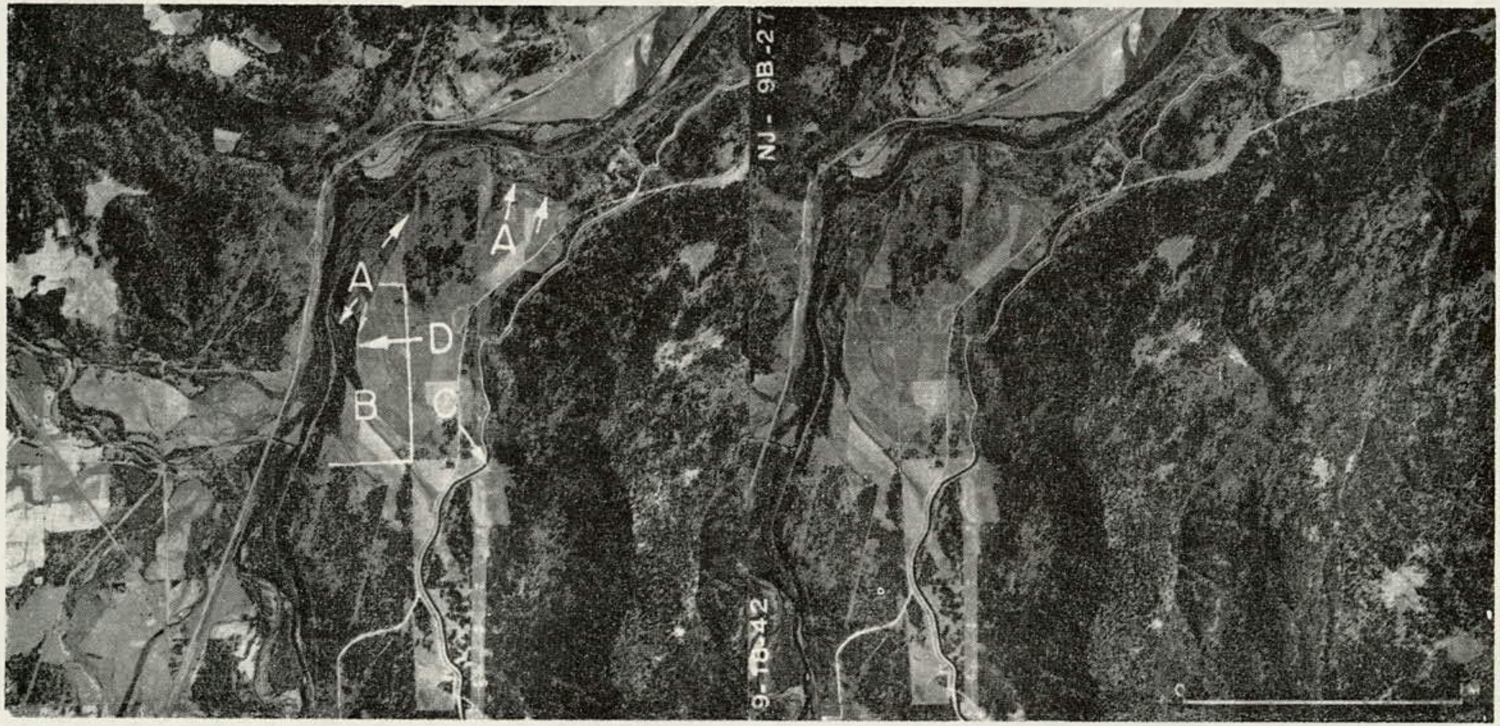


Figure 59. River Terrace, Kittitas County, Wash. The combination of pervious and impervious beds is a favorable condition for slides, often deep-seated ones. The instability of the land shown in this stereo pair is indicated by the numerous slide scars (A) along the terrace front. When the irrigation system of the farm (B) above the railroad was connected to the main canal (C) a new slide became imminent. The most probable next slide (D), which actually took place later, could be predicted in advance from airphotos as it is the steepest slope and is actively attacked by the river. (Aerial photograph by U. S. Department of Agriculture)

Figure 60. Loess, Lincoln County, Nebr. This stereo pair shows loess of great depth which is identified by steep-sided, flat-bottomed gullies, equal slopes in hills and valleys, and soft tones. The catsteps — small slumps — are seen as light fine contours all over the area. (Aerial photograph by U. S. Department of Agriculture)

Figure 61. A ground photo showing catsteps in loess, such as those shown in Figure 60. (Photograph by Ta Liang)

are underlain by relatively pervious beds.

Actual failures are commonly detectable in airphotos in the following situations:

1. Glacial outwash or delta deposit over old lake bed. Photo pattern changes from that of light-toned, well-drained outwash at high ground to poorly drained lake clays exposed on the slopes. Old landslide scars are present in the slopes.
2. Glacial drift over shale. The photo is likely to show numerous landslides along river banks composed of shallow drift.
3. Valley fill over bedrock. The photo may show the landform characteristic of bedrock, but this is modified locally by fill deposits. Slides of fill material along steep hill slopes may be observed.
4. Sand over clay. This combination is common both in glacial and coastal plain areas. Slope failures along natural or cut slopes can be seen in many photos.

Procedure for Detecting Evidence of Landslides in Airphotos

A step-by-step procedure for landslide investigation by airphotos is outlined in the following:

1. Lay out locations of road or other planned structure on photos.
2. Take a quick survey, on the photographs, of all cliffs or banks adjacent to river bends, and of all steep slopes in the photo area, to see if landslide movements are evident.
3. Outline areas along the right-of-way that show consistent characteristics of topography, drainage, and other natural elements within the same unit.
4. Evaluate the general landslide potential of the areas with the help of Table 2.
5. Make a detailed study of all cliffs or banks adjacent to river bends and all steep slopes above and below the center line of the road. It is important to com-

TABLE 2

AIRPHOTO IDENTIFICATION AND LANDSLIDE EVALUATION OF LANDFORMS

Elimination Procedure	Supporting Characteristics	Probable Landform[1]	Landslide Potential[2]
I. Level terrain			
A. Not elevated:		Flood plain, etc.	(c)
B. Elevated:			
	Uniform tones	Terrace, lake bed	(b)
	Surface irregularities, sharp cliff	Basaltic plateau	(a)
II. Hilly terrain			
A. Surface drainage not well integrated:		Limestones, etc.	(c)
B. Surface drainage well integrated:			
1. Parallel ridges			
a. Parallel drainage	Dark tones	Basaltic hills	(a)
b. Trellis drainage	Ridge-and-valley topography, banded hills	Tilted or folded sedimentary rocks	(a)
c. Featherlike drainage	Vertical-sided gullies	Loess	(b)
2. Branching ridges			
a. Featherlike drainage	Vertical-sided gullies	Loess	(b)
b. Treelike drainage			
(1) Banding on slope		Flat-lying sedimentary rocks	(b)
(2) No banding on slope			
	Moderately to highly dissected ridges, uniform slopes	Clay shale	(a)
	Low ridges, associated with coastal features	Dissected coastal plain	(a)
	Winding ridges connecting conical hills, sparse vegetation	Serpentine	(a)
3. Random ridges or hills			
a. Treelike drainage			
	Low, rounded hills, meandering stream	Clay shale	(a)
	Massive, uniform, rounded to A-shaped hills	Granitic rocks	(b)
	Bumpy topography[3]	Moraine	(b)
b. Disordered drainage			
	Disordered, overlapping hills, associated with lakes and swamps[3]	Moraine	(b)

[1] The landforms listed are the most likely ones to represent the condition listed. It must be remembered, however, that other kinds of geology and terrain can give photographic representation similar to some of those listed. Only a high degree of skill in photo interpretation or knowledge of the local geology can be regarded as certain to avoid errors.

[2] (a) susceptible to landslides; (b) susceptible to landslides under certain conditions; (c) not susceptible except in dangerous locations discussed above.

[3] Glaciated areas only.

pare slopes within the same unit area rather than of different areas. For instance, slopes in bedrock would be more stable, even though steeper, than slopes in adjacent soil areas. Realize that slides usually appear small in photos, and so look carefully, inspecting slopes in minute detail. Look especially for:

a. Existing slides. Relatively new slides appear in white tones; vegetation and drainage are not well established on them. The reverse conditions are true for old slides.
   (1) Hillside scars and hummocky topography.
   (2) Parallel moon-shaped dark patches on hillside, likely to reflect vegetation in minor depressions. Draw a line through the axis of scars or crescents in the slides. This line often points to drainageways on higher ground that contribute to the landslide movement.
   (3) Irregular outline of highways and random cracks or patches on existing pavement.

b. Potential slides
   (1) Ponded depressions and diverted drainageways.
   (2) Seepage areas suggested by faintly dark lines, which may mean near-surface channels and fanshaped dark patches, probably reflecting wet vegetation.

6. Ground check some of the landslides that are recognized in airphotos.

7. Ground check all suspected spots, using methods and criteria described in Chapter Four.

References

Belcher, D. J., "The Engineering Significance of Soil Patterns." *Highway Research Board Proceedings.* v. 23, p. 569-598, 1943.

Belcher, D. J., et al., "The Origin, Distribution, and Identification of United States Soils, with Special Reference to Airport and Highway Engineering." Civil Aeronautics Administration Technical Report No. 52, 1946.

Colwell, R. N., "Bibliography (Photographic Interpretation)." American Society of Photogrammetry, Manual of Photogrammetry, p. 600-602, 1952.

Liang, Ta, "Landslides — An Aerial Photographic Study." Cornell University Ph.D. Thesis, 274 p., 1952.

Chapter Six

Field and Laboratory Investigations

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Purpose and Scope

The scope of investigation of a landslide or a potential landslide area will depend on the importance of the slide. There will be many, many more slides given a quick cursory inspection than will be examined and investigated in complete detail. Nevertheless, in order to present the full picture of an investigation, Table 3 indicates the various possible lines of attack on each of the several kinds of landslides shown in the classification chart (Pl. 1).

There are several factors that should be ascertained in the investigation of any slide in order to assess its importance. These include:

1. Location
   a. Station, mile post, distance from well-known point
   b. Above or below grade
2. Effect on traffic
   a. Stopped
   b. Partially open
   c. Open
3. Size
   a. Surface limits — head, foot, flanks
   b. Subsurface — maximum depth to surface of rupture
4. Material
   a. Soil
   b. Rock
5. Water
6. Weather
7. Evidence of movement
8. History of slope

All of these factors have been indicated in Table 3 as factors to be investigated in connection with all slides. At this point, however, the investigation may be nothing more than a series of rapid approximations to provide a basis for an opinion as to whether or not a more formal investigation is in order. Indeed, these approximations may be sufficient in themselves to permit the selection of the appropriate method of control or correction. The factors listed will commonly suffice for the study of minor slides in the nuisance category; they can be ascertained with the expenditure of a minimum of time and money. The competent maintenance engineer can, in his own district, usually provide the answers to the questions implied by the above list. The succeeding statements are directed more to the investigation of a slide of some magnitude and importance which, although it occurs infrequently, may cause more trouble and cost more money than a hundred of the minor ones.

The purpose of a landslide investigation may be to determine the cause of the slide and to plan the appropriate measures for repair and reconstruction. On the other hand, the purpose may be to provide the necessary data for prosecution or defense of a damage suit, for a theoretical investigation, or for any number of other considerations. The following discussion endeavors to cover the pertinent aspects of an investigation which should, insofar as possible, provide the data needed for any of the fore-

TABLE 3

SUMMARY OF METHODS OF INVESTIGATION[1] OF LANDSLIDES

		Survey and Mapping																	
												Internal							
		Location			Limits							Fractures				Flow Lines		Evidence of Movement[2]	
Type of Movement	Plate Number	Geographic	Legal	Elevation	Scarp	Crown	Toe	Foot	Flanks	Slopes	Surface of Rupture	Strike	Dip	Displacements	Depth Opening	Trend	Grades	Linear	Rectangular
I. Falls:																			
Rockfall	1-a	0	x			0	0		0	0	x	x	x		x				
Soilfall	1-b	0	x			0	0		0	0	x	x	x		x				
II. Slides:																			
Relatively undeformed material																			
Bedrock																			
Slump	1-c	0	x	x	0	0	0	0	0	x	0	x	x	x	x			x	x
Block glide (La Pita)	1-d	0	x	x	0	0	0	0	x	x	0							x	x
Block glide (cone)	1-e	0	x	x			x		x	x	0							x	x
Block glide (slab)	1-f	0	x	x		0	0	0	x	x	0	0	0		0			x	x
Soil																			
Plastic																			
Slump	1-g	0	x	x	0	0	0	0	0	x	0							x	x
Slump	1-h	0	x	x	0	0	0	0	0	x	0	x	x	x	x	x	x	x	x
Greatly deformed material																			
Bedrock																			
Rockslide	1-i	0	x	x	0	0	0	0	0	x	0								
Soil																			
Granular																			
Debris slide	1-j	0	x	x	0	0	0	0	0	0	0								
Plastic																			
Failure by lateral spreading	1-k	0	x	x	0	0	0	0	0	0	0	x	x	x	x	x	x	x	x
III. Flows (all unconsolidated):																			
Dry																			
Granular																			
Rock fragment flow	1-l	0	x	x	0	0	0	0	0	x	0					x	x		
Sand run	1-m	0	x	x	0	0	0	0	0	x	0					x	x		
Loess flow	1-n	0	x	x	0	0	0	0	0	x	0					x	x		
Variable water content																			
Granular																			
Debris avalanche	1-o	0	x	x	0	0	0	0	0	x	0					x	x		
Plastic																			
Earth flow	1-p	0	x	x	0	0	0	0	0	x	0					x	x	x	x
Liquid																			
Granular																			
Sand or silt flow	1-q	0	x	x	0	0	0	0	0	0	0								
Debris flow	1-r	0	x	x	0	0	0	0	0	0	0					x	x		
Plastic																			
Earthflow-mudflow	1-s	0	x	x	0	0	0	0	0	0	0					x	x		
IV. Complex slides	1-t	0	x	x	0	0	0	0	0	0	0	x	x	x	x	x	x	x	x

[1] 0 = Basic investigation; x = Additional investigation.
[2] Offsets of elements.

TABLE 3 (Continued)

Survey and Mapping										Subsurface Investigations												
Internal and External										Drilling												
Water					Bedrock/Soil					Identification				Undisturbed Sampling			Excavation			Geophysical		
Springs, Seeps	Dye Tests	Permeable Layers	Water Table Elevation	Flow	Types and Formation	Clay Minerals	Structure	Interface Contact	Stratigraphic Relations	Standard Penetration	Augur	Casing	Core NXM	Shelby	Split Spoon	Man Size	Pit	Trench	Tunnel	Resistivity	Seismic	Gamma Ray Neutron
0		x			0	x	x		x				x									
0		x			0	x	x		x	x												
0	x	0	x	x	0	x	x	x	0				0							x		x
0		x	x		0	x	x		0	x	x		x									x
0		x	x		0	x	x		0	x	x		x									x
0		x	x		0	x	x		0				x									
0	x	x	x		0	x	x	x	x	x				x	x	x	x	x				
0	x	x	x		0	x	x		x	x	x			x	x	x		x				x
0		x	x		0	x	x		0				x									
0		x	x		0	x	x		0				x									
0	x	x	x		0	x	x	x	0	x				x	x	x	x			x		x
					0	x	x	x	x													
					0		x		x	x				x	x							
					0		x		x	x				x	x							
0					0		x		x				x									
0	x	x	x	x	0	x	x	x	x	x	x			x	x	—			—		—	x
0		x	x		0		x			x				x	x							
0		x	x		0	x	x	x	x	x	x		x		x					x		
0		x	x		0	x	x	x	x	x				x	x					x		x
0	x	x	x	x	0	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x

TABLE 3 (Continued)

Type of Movement	Plate Number	Weather: Precipitation: Long-Term	Weather: Precipitation: Short-Term	Weather: Precipitation: Type	Weather: Precipitation: Rate	Weather: Temp.: Long-Term	Weather: Temp.: Short-Term	Weather: Bar. Press.: Short-Term	History of Slope: Geological	History of Slope: Historical Changes: Erosional	History of Slope: Historical Changes: Construction	History of Slope: Historical Changes: Hydrologic	History of Slope: Rate of Movement of Slide	History of Slope: Eye Witness Description of Slide	History of Slope: Submergence
I. Falls:															
Rockfall	1-a		x	x	x		x	x	x	x	x			x	
Soilfall	1-b		x	x	x		x			x	x			x	
II. Slides:															
Relatively undeformed material															
Bedrock															
Slump	1-c	x	0	x	x	x	0	x	x	x	x		0	x	x
Block glide (La Pita)	1-d	x	x	x	x	x	x		x	x	x		0	x	
Block glide (cone)	1-e	x	x	x	x	x	x		x	x	x		0	x	
Block glide (slab)	1-f	x	0	x	x	x	0	x	x	x	x		0	x	
Soil															
Plastic															
Slump	1-g	x	0	x	x	x	0	x	x	x	x	x	0	x	x
Slump	1-h	x	0	x	x	x	0	x	x	x	x	x	0	x	x
Greatly deformed material															
Bedrock															
Rockslide	1-i	x	0	x	x	x	0		x	x	x	x	0	x	
Soil															
Granular															
Debris slide	1-j	x	x	x	x	x	x		x	x	x	x	0	x	
Plastic															
Failure by lateral spreading	1-k	x	x	x	x	x	x	x	x	x	x	x	0	x	x
III. Flows (all unconsolidated):															
Dry															
Granular															
Rock fragment flow	1-l	x	x	x	x	x	x	x	x	x	x	x	0	x	
Sand run	1-m							x	x	x	x		0	x	
Loess flow	1-n							x	x	x	x		0	x	
Variable water content															
Granular															
Debris avalanche	1-o	x	x	x	x	x	x	x	x	x	x	x	0	x	
Plastic															
Earth flow	1-p	x	0	x	x	x	x	x	x	x	x	x	0	x	x
Liquid															
Granular															
Sand or silt flow	1-q									x		x	0		x
Debris flow	1-r	x	x	x	x	x	x	x	x	x	x	x	0	x	
Plastic															
Earthflow-mudflow	1-s	x	x	x	x	x	x	x	x	x	x	x	0	x	x
IV. Complex slides	1-t	x	x	x	x	x	x	x	x	x	x	x	0	x	x

TABLE 3 (Continued)

Vibrations				Graphic Record										Laboratory Tests										
														Soils							Mineralogic			
	Trans.			Photos					Slide Motion						Atterburg Limits				Triaxial Compression					
Seismic	Railway	Highway	Machinery	Pre	Post	Detail	Cross-Sections	Maps	Precipitation	Temperature	Ground Water Flow	Construction or Erosion	Relief Model	Mechanical Analysis	Plastic	Liquid	Field Moisture Content	Density	Undisturbed	Remolded	X-Ray	Differential Thermal	Petrographic	Weathering
x	x	x	x	x	x	x	x	x																x
x	x	x	x	x	x	x	x	x																
x	x	x	x	x	x	x	x	x	x	x	x	x	x								x	x	x	x
x	x	x	x	x	x	x	x	x	x	x	x	x												
x	x	x	x	x	x	x	x	x	x	x	x	x												
x	x	x	x	x	x	x	x	x	x	x	x	x									x	x	x	x
x	x	x	x	x	x	x	x	x	x	x	x	x		x	x	x	x	x	x	x	x	x		
x	x	x	x	x	x	x	x	x	x	x	x	x		x	x	x	x	x	x	x	x	x		
x	x	x	x	x	x	x	x	x	x	x	x	x									x	x	x	x
x	x	x	x	x	x	x	x	x	x	x	x	x									x	x	x	x
x	x	x	x	x	x	x	x	x	x	x	x	x		x	x	x	x	x	x	x				
0				x	x	x	x	x													x	x	x	x
0				x	x	x	x	x						x			x							
0				x	x	x	x	x						x			x							
x	x	x		x	x	x	x	x															x	x
x	x	x	x	x	x	x	x	x	x	x	x	x		x	x	x	x	x					x	x
				x	x	x	x	x				x												
x	x	x	x	x	x	x	x	x	x	x	x	x		x	x	x	x	x			x	x	x	x
x	x	x	x	x	x	x	x	x	x	x	x	x		x	x	x	x	x	x	x				
x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x

going uses. It is somewhat more comprehensive than is required for the common slide with which the highway engineer must usually deal.

The scope of any landslide investigation depends on the property damage done or threatened, the values involved, the importance of the land area involved, the time allowable for study, the immediacy of the need for control and corrective measures. A slide in an urban area, for example, involving high property values, or utilities, and threatening the safety of the public, warrants an analysis in more detail than a slide in some isolated area that affects only mountain lands and secondary country roads.

The intensity of the investigation is, again, a function of the importance of the slide with reference to land use and other factors. The initial examination of the slide area, constituting a simple reconnaissance, may be all that is required, inasmuch as the solution of the slide problem may be obvious, and no other parties than the "owner" affected. However, should other parties than the "owner" be affected or threatened, or should damage claims be made or anticipated, a detailed slide analysis may be in order. For the purpose of assisting the engineer in an appreciation of the full scope of a landslide analysis the following procedures are outlined. It is reiterated that these may be useful or required only in part, depending on the particulars of the slide involved.

Mapping Methods

The purpose in mapping a landslide is to obtain and record in graphic form such data as may be observed in the field and from which significant inferences and facts relative to the cause, mechanics, and potentialities of movement, past, present, and future, may be drawn. The mapping procedure may be divided into two realms — general or areal, and geologic.

Location

The purpose of the general or areal mapping procedure is to fix the slide area in space so that there can be no doubt as to its geographic position. It is desirable, though not essential, to locate the slide with reference to mean sea level. If this is wanted, some point near or associated with the slide area must be referenced to an acceptable municipal, State, or Federal benchmark. These are most commonly located on bridge abutments, public building cornerstones or monuments, or on easily recognized topographic features. The benchmarks may have been established by the U. S. Geological Survey, or by the U. S. Coast and Geodetic Survey. If, as sometimes happens, the altitude is not shown on the benchmark tablet, the required information can be obtained by sending a description to the bureau that placed the marker.

Whereas benchmarks are recorded as reference points for vertical control, other features also may and should be utilized in mapping the slide location. These may be legal or geographic. Legal reference points may be considered as property lines and corners, highway or railway survey stations, or state coordinate systems. For most parts of the western United States the township, range and section system of land surveys provides accurate and legal descriptions of locations. Latitude and longitude determinations are useful and are easily understood. Geographic references may be applied to easily identified terrain and drainage features such as hills, streams or springs; such references should give exact distances and directions, rather than such vague terms as "near," "northerly," and "in the vicinity of."

Scale and Contour Interval

The map scale used is largely a function of the areal extent and economic importance of the slide in question, but

it may also depend to some extent on the use to which the map is to be put. A small slide several dozens of feet across, but involving extensive property damage or physical injury to the public, may warrant mapping on a scale such as 1 inch on the map to 5 or 10 feet on the slide area. On the other hand, a slide covering several hundreds or thousands of acres may be mapped on a scale of 1 inch on the map to 50 or 100 feet on the ground. However, some smaller portions of the same slide might be selected for mapping on a much larger and more revealing scale.

It is often desirable to map not only the areal limits of the slide and the position of significant features within it, but also the physical configuration or topography. In this case a contour map may be prepared. Here, again, the judgment of the analyst must be exercised; first to decide whether a contour map is essential, and second, in the choice of the most desirable contour interval to illustrate the surface features of the slide. Whereas a 2-foot contour interval may be required in one slide study, a 10- or 20-, or even 50-foot contour interval may be satisfactory in another.

Field Methods

The field methods employed in mapping the slide area are flexible and vary with the importance and degree of accuracy required. Where a comprehensive graphic portrayal of the slide seems desirable, both plan and profile illustrations may be prepared. The accuracy of this mapping becomes more important if continuing ground movement exists or is anticipated. In order to insure high accuracy, triangulation stations should be established on stable ground, outside of the slide area. From these a baseline may be established below or above the disturbed mass. From this baseline points in the slide area may be established and checked periodically for movement.

It may be desirable to survey and set up a grid system over the slide region. In such a case the grid squares may be on 25-foot or 100-foot centers, or any other distance that seems applicable to the problem at hand. The grid corners, once determined, may be used to check both horizontal and vertical movements. Sections, or topographic profiles, may be prepared along the grid lines, and overlays representing various time intervals often reveal striking changes in the slide surface that otherwise might go unnoticed. Regular checks may indicate changes in the movement rate such that resurgent acceleration may be indicated in time to forestall a catastrophe.

The grid or other survey method of measuring displacements can be supplemented by strain gages or other means. In at least one investigation, continuous records of the movement of certain points were obtained by an ingenious adaptation of an automatic stage recorder, such as is used for stream measurements. The recorder was placed on stable ground and a wire stretched from it to an accurately located stake set in the landslide mass. Thus any movements of the stake were automatically recorded on the meter's drum.

Either plane table or regular surveying methods can be used for preparing planimetric or contour maps and for determining the positions of reference points within and outside the slide area. Either method is applicable to mapping of small areas, but level and transit methods are perhaps to be preferred for larger ones. Accurate maps also can be made, of course, by special methods from aerial or even terrestrial photographs. It is doubtful, however, whether such methods can be applied satisfactorily and economically to most ordinary landslide problems.

Area to Be Mapped

Selection of the size of the area chosen for the slide map is important. It is obvious that the entire mass of disturbed ground should be shown; but to show only the affected ground is not enough.

Its relationship to the associated terrain and to cultural features (such as buildings and highways) is especially important. However, there is a significant limit to the area outside of the disturbed mass that should be included. The following dimensions are suggested as guides to the judgment of the analyst. In general, along or parallel to the contours, the map should extend about twice the width of the slide on each side of the slide. It should be borne in mind, however, that topographic features may indicate modifications to this general axiom.

Across the contours, or up and down the slide, the following principles may be applied. The minimum distance upward should be at least to the first sharp break in slope above the slide crown. The maximum distance needed would be to the top of the slope. Intermediate distances can be chosen, depending on the physical features of the terrain and the judgment of the analyst. The minimum downward distance that the map should illustrate is to the first sharp break in slope below the slide toe. The maximum downward distance is the bottom of the slope. Again, intermediate distances depend on the terrain and the analyst's judgment.

Mapping the Slide

Limits of Slide

The final map should show the slide proper, associated water conditions, and its geologic framework. The limits of the slide should be mapped first, so as to depict its shape and size. The limits listed hereinafter may be observed graphically on the "classification of landslides" chart (Pl. 1-t). The upper part of the slide is the *crown,* or that point where the slide mass breaks away from the original ground slope. The cliff-like face below the crown is the *main scarp.* The contact of the mass of slide debris with the main scarp is the *head* of the slide. These together mark the upper limit of the slide. The lower limit of the slide is the *toe,* which is the margin of the disturbed material most distant from the main scarp. The *tip* is that point on the toe most distant from the crown of the slide. The lateral limits of the slide are called the *sides* or the *flanks.* Displacement of the slide mass with reference to the crown and flanks should be mapped. Displacement at the toe may not be measurable because the *foot* (the line of intersection between the lower part of the surface of rupture and the original ground surface) may be buried. This displacement at the toe, however, may be inferred by interpolation and projection. Slopes on the main scarp below the crown and on the flanks should be determined, because they may aid in determining the depth and character of the slide mass. The surface of separation is the basal limit, or *surface of rupture.*

Surface of Rupture

The surface of rupture is easily recognizable at the crown and on the flanks, where it is the limit of displacement and where it may, in fact, be marked by a cliff or scarp. Underground, however, where it forms the bottom of the slide, no such striking expression calls attention to its presence and it can only be determined by means of subsurface exploration. In dealing with slides in which the slide mass and frame are composed of the same homogeneous materials, the recognition of the slide plane or surface of rupture may have to be based only on the striations or slickensides developed by motion of the slide mass on the "plane of failure." Commonly, this plane of failure is a series of closely spaced subparallel surfaces in and between which detrital rock fragments, if present, will be oriented in parallel with the plane of failure. The materials in this zone of failure — for it really is a zone and rarely a single plane — are usually softer than in the overlying slide mass or underlying stable ground. The water content of the material in this zone is

generally higher than in the disturbed and undisturbed material above and below, because of higher permeability of the fractured material and, in many cases, of a rather direct connection to a source of moisture. The higher water content and lesser resistance to penetration may often be more reliable indications of the location of the surface of rupture in borings and samples than are slickensides and striations. In many cases the surface of rupture is determined finally by correlating zones of high water content and low penetration resistance in several borings.

Estimating Depth of Slump Slides

An early estimate of the maximum depth of a slide — from the ground surface to the surface of rupture — is of great value as a guide to the magnitude of the slide and to the critical depths to which subsurface exploration will have to be carried. The following quick method, devised by Arthur M. Ritchie, author of Chapter Four, permits reliable estimates for these preliminary purposes and is based on a minimum of required observational data. It is directly applicable only to slides of the slump type.

Slip Circle Method. — Consider a slump block, indicated by Figure 62A. The only field measurements required are the positions of the crown, A, and foot, B, and the profile of the ground surface between them. In case the position of the foot is obscured by toe material that has overridden it, its position must be estimated, possibly by reference to the point of maximum uplift, measurement of the amount of vertical uplift, or the location of long transverse tension cracks; a detailed profile of the entire slide also may aid in indicating the position of the foot. Plot points A and B and the ground line on a graph. Locate point O at the intersection of the perpendicular bisector OC of line AB and the horizontal line OA. (Point O is placed on the horizontal from point A on the premise that the tangent of the line through A to the slip circle is never steeper than vertical.) Scale the distance OA and scribe an arc; this defines the maximum depth of the slide material at point D.

Concentric Circle Method. — If there has been appreciable offset of some positive reference point, such as the edge of a pavement, the approximate position of the center of rotation, hence the maximum depth of the slide, can be found

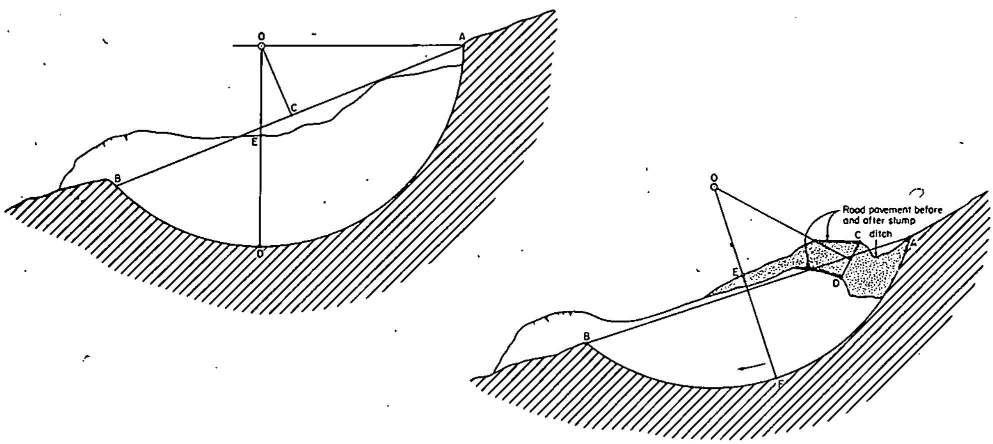


Figure 62. Quick methods for estimating depth of a slump slide. A, slip circle method; B, concentric circle method.

by the concentric circle method. This is shown in Figure 62B. The positions of points C and D can be accurately determined, as can the position of the crown, A. The foot, B, can be determined or estimated. Plot points A, B, C, and D on a graph. Draw lines AB and CD and bisect each line. The bisectors will intersect at point O, the center of rotation of a unit slump block, because the rotational paths of most of the segments in a slump are concentric about a common center. The maximum thickness of the slide, EF, can be scaled directly from the graph.

Either of the two methods just described can be applied to a slump made up of several individual blocks by analyzing the geometry of the lowest block in the series. This is possible only because in most cases the rupture surfaces of individual slump blocks of a multiple block slide tend to lie tangent to a common shear plane.

Internal Structure

Internal, mappable structural features should be recorded. As described more fully in Chapter Four, these include fractures, flow lines, and displacements of surficial features. A noteworthy description of the internal structural features of a slide is given by Krauskopf, Feither and Griggs (1939); their discussion is limited to the geologic interpretation of the features they mapped and does not lead to consideration of control or correction measures. The characteristics of fractures and cracks to be noted are as follows:

1. Strike.
2. Dip.
3. Elevations.
4. Displacements — vertical, horizontal, and components of the same, including rotational movements.
5. Depths of openings. These will vary from one to another fracture, as well as individually, with time, according to adjustments within the slide mass.

The significance of mapping fractures and of studying the meaning of openings along them is described more fully in Chapter Four, on recognition and identification. In slides in the more plastic materials, flow features may take the place of, or blend with, the fractures and cracks that are more commonly associated with the drier types of material. The trends of flow paths involving direction and pattern should be shown, as should the grades or gradients of soil flows in the slide mass or in associated terrain. Surficial features may have been deformed on or within the slide mass. They are easily spotted and should be shown on the map. These consist of offsets of linear elements such as fences, vegetation lines, leaning trees, ditches, roads, railroads, pipelines, walls, utility lines, and the like. To be included with these features are rectangular elements which may have been deformed, such as houses, buildings, other structures, and fields.

Sources of Water

The mapping should show all sources of water in and adjacent to the slide area, such as springs, seeps or permeable layers. These features may be found in the main scarp or outside of the slide perimeter, depending on the terrain and the physical characteristics of the subsurface materials. Water may enter the slide area from other sources which should also be shown, such as ditches, canals, drainage lines, pipelines, and sewer lines.

Slide Materials

Materials within the slide should be mapped and the following characteristics determined, as well as the distribution and thickness of each type of material. Soils are mentioned first because of the great prevalence of slides in soil. It is necessary to know the engineering soil type in terms of standard classification, such as that of the American Association of State Highway Officials (1955) or the

unified soil classification of the Corps of Engineers (U. S. Waterways Experiment Station, 1953). The agricultural soil type, if readily determinable, may be as helpful as geologic classification. The structure of the soil (such as prismatic, dense, granular) should be recorded. Relative permeability, dip of the bedding, and mineralogy may be required information. The location of large inclusions of bedrock or of boulders may prove of value. The bedrock should be mapped according to normal geologic methods to show the type of rock and its structure, including bedding, schistosity, cleavage, joints, and faults. The superposition of softer over harder beds and vice versa are important data which may affect the rate of weathering, the location of permeable beds, and the tendency toward further landslide movement.

Mapping the Frame

To appreciate the mechanics involved in the slide movement, and to plan appropriate control or corrective measures, it is necessary to obtain some knowledge of the soil and bedrock that form the frame of the slide. The geologic and soils data to be mapped within the slide should likewise be mapped in the frame.

Hydrology

Most slides are intimately related to hydrologic conditions. Often a slight variation in the normal climate may be sufficient to upset the terrain stability and initiate slide movement. Such data as may be obtainable from the local weather bureau, utility companies, colleges and other organizations that record weather data regularly should be obtained and carefully analyzed. Applicable data that should be studied include the records of rainfall immediately before and during the slide, as well as long-term rainfall records. Not only should the data for the past month or even year be analyzed, but also the cyclical data for periods of years. The concentration or intensity of the precipitation may be, and often is, important. The type of precipitation, such as rain or snow, cannot be omitted.

Temperature data, if freezing is involved, may be every bit as significant as the precipitation data and should include the periods immediately before, after, and during the slide, as well as the long-term record of temperature for correlation with the long-term precipitation record.

Ground water data, if obtainable, provide a basis upon which to draw conclusions as to pore water and hydrostatic pressures. Water table records are desirable for the periods immediately before, during and after the slide, although they are often difficult or impossible to get. Long-term records of water table fluctuations may be of much use. Records of ground water flow, both of immediate and long duration, are often unobtainable, but are very helpful if available.

Barometric pressures may seem unimportant, but they may be the triggering effect to set a slide in motion. These barometric records are obtainable from the weather bureau and from utility companies.

The hydrologic data should be plotted against the rate of movement of the slide or slides; rate of movement is commonly plotted against precipitation. The facts needed for such plots are not as unobtainable as they may seem at first glance; they have often proved of value in presenting testimony in courts of law.

Subsurface Investigations

Subsurface investigations are made for the purpose of determining the physical, geologic, and mineralogic characteristics of the slide and of the underlying and adjacent stable bedrock or soil "frame" materials, the location of the surface of rupture, and ground water conditions. Some or all of these facts can,

of course, be determined from surface mapping alone, but subsurface investigations are desirable, if not essential, to yield more precise data. In cases involving considerable property damage and subsequent litigation, subsurface investigations are very important. Not all types of such investigations will be used on a single slide, hence the judgment and the applicability of the methods used are a real responsibility of the analyst. It goes without saying that in many very active slides no such investigations may be feasible.

Layout

The layout of the subsurface investigations is based on the requirements of the particular questions that must be answered. In general, the basic question of size of slide or quantity of material in the slide mass will require that borings be made in the slide mass first, before proceeding to the investigation of the frame, in which the cause or causes of the slide may be found. It is good practice to develop a profile of borings along the center line of the slide, with the first boring placed above the midpoint of the slide but well below its head; this profile should seek to find the area of possible maximum depth of the plane of failure. The next most important area to be explored is the foot of the slide area, where the lower limit of the surface of rupture intersects the preslide ground surface. The location of the foot determines the downhill limit of the broken slope beneath the slide mass. Determination of this point may indicate a change in methods of correction for structures lying uphill or downhill from it. Other borings may be distributed within the slide and in the surrounding frame as may best fit the case to develop such data as appear necessary.

Methods

Drill holes, test pits and test trenches are the most commonly used methods for subsurface exploration; choice between these will vary with the existing conditions. Borings to identify the materials are known as identification borings and include standard penetration borings, auger borings, and core borings. Any or all of these may be cased.

Undisturbed samplings for use in soil slides, or in overburden, include Shelby tube and split spoon techniques. Shelby tube sample methods use a thin-wall tube wherein the sample is taken intact; the tube is sealed and submitted to the laboratory for opening and study. Split spoon samples are removed from the sampler in the field and either examined there or placed in sample jars for subsequent study. Test pits, trenches and tunnels are commonly limited in depth and slow to dig, but they have the advantage of permitting visual examination of the undisturbed soil in place. Such examination may be the only means of fixing definitely the location and slope of the surface of rupture.

Large diameter borings, made with calyx drills or large earth augers, have been used recently in slide investigations. Calyx drills may be used in rock, whereas augers may be used only in soil and soft shale. The large earth auger may drill a hole 21 ft deep and 36 in. in diameter in a matter of minutes, in contrast to a test pit that may require one or two days or longer. In a slowly moving slide an earth auger hole may provide for visual examination where a test pit would not stay open long enough to be completed and examined.

Ground water investigations may be concerned either with the movement of ground water or with the ground water level and hydrostatic pressures. Flow or seepage tests using dyes, such as household bluing, or fluorescein in neutral waters and uranine in acid waters, may be very helpful in tracing the movement of waters under or through the slide mass, and in locating the surface of separation.

Observation wells may be used to measure the water table level, and piezometers will supply information on hy-

drostatic pressures. If exploratory borings are made it often is advisable to install an observation well or a piezometer in at least one of the bore holes so that future observations of the ground water conditions will be possible. An inexpensive device that has operated successfully as either an observation well or a piezometer is the porous tube piezometer (Casagrande, 1949). One man with no special tools or equipment can easily install it in any bore hole with a diameter greater than 1½ in.

Geophysical studies may be useful for preliminary subsurface exploration. The methods are relatively inexpensive and rapid, and serve to indicate the number and thicknesses of soil layers and the approximate depths to firm bedrock. Refraction seismic traverses, electrical resistivity traverses, and gamma ray neutron logs are most applicable to landslide studies. The selection of a method and the interpretation of the data should be made by a specialist who is familiar with the local geologic features and who understands the limitations of each method.

Identification borings are frequently necessary to interpret the geophysical data in terms of the physical natures and depths of soil and rock units. One or two borings generally validate a large number of geophysical tests. The borings should include enough rock core to prove the depth to firm bedrock.

Geophysical methods are not a substitute for drive-sample and core borings at sites where detailed specific data regarding the character of bedrock or overburden are required.

Refraction seismic traverses detect wave disturbances produced by detonating explosive charges at depths of 4 to 6 ft below original ground. Ordinarily ½ to 2 lb of 60 percent gelatine-type dynamite is used; occasionally greater charges are needed. The rate at which these wave disturbances are propagated varies widely according to the physical properties of the medium. Granular and plastic materials (such as sand, clay, gravel, and glacial till) are characterized by velocities of 600 to 6,000 ft per second. Rigid materials (such as shale, sandstone, and the crystalline and metamorphic rocks) are characterized by velocities of approximately 7,000 to 20,000 ft per second. The velocity of wave transmission through any material is approximately equal to the square root of the appropriate elastic constant divided by the density of the material; hence its rigidity and elasticity can be interpreted to some degree. The velocities are controlled by variables of texture, moisture content, degree of compaction, degree of weathering, attitude of bedding or schistosity, and the frequency and distribution of jointing. The seismic refraction method is most successful in areas of simple geology having wide contrasts in velocities of soil and bedrock. For landslide studies it is probable that only the seismic method would commonly be employed. The method is used to:

1. Compute depths to firm bedrock.
2. Detect the number of layers of soil units overlying bedrock.
3. Determine data for preparation of an approximate subsurface contour map of the concealed bedrock between and at shot points, thus supplying better average depth data than by spot-sampling with bore holes.
4. Detect the thickness of weathered rock overlying firm bedrock when other methods have indicated the presence of a prominent weathered zone.
5. Differentiate recent alluvium from underlying older and more compact soils.
6. Determine the altitude of the water table in coarse unconsolidated material.
7. Determine, qualitatively, the identification, water content, degree of compaction, and relative permeability of soils when the interpreter is familiar with the local geologic setting.
8. Determine the strike of the foliation in buried metamorphic rocks, even though the site area lacks outcrops.

9. Determine the position of the concealed surface of rupture of a slide; that is, whether the rupture occurs wholly in soil, wholly in bedrock, or at the soil-bedrock contact.

The resistivity of an earth material varies approximately as the reciprocal of the total amount of ionized salts in the pore fluid. Apparent resistivity is obtained by measuring the change in electrical potential between one pair of electrodes when a current is introduced into the ground through another, outer, pair of electrodes. The equipment is more portable and can be operated by a smaller crew than that required by the seismic method. Moreover, the possibility that the explosive energy used in seismic traverses could "trigger" the slide is absent. However, interpretation of resistivity data is exceedingly difficult and is largely empirical because no simple mathematical relation exists between resistivity values and the depths to boundaries between zones of different resistivity. One of the better methods is to conduct the test alongside a well whose log is known. This establishes a standard for interpretation of the local area. The method is most useful for determining horizontal variations in overburden, and hence for locating buried sand lenses in glacial till, locating boundaries between outwash sands and glacial till, and determining thicknesses of top strata overlying pervious sands and gravels.

Gamma ray neutron logs are a composite of two measurements. Since gamma rays penetrate considerable thicknesses of iron and of concrete the method can be used in cased holes. Gamma radiation of a particular stratum is essentially constant over wide areas, thus measurement of gamma rays provides a guide to lithology. The magnitude of neutrons (secondary gamma rays) depends on the quantity of hydrogen ions in a stratum which may occur in oil, water, or the rock itself. They are, thus, a measure of porosity in saturated soil. The two logs have been used extensively for stratigraphic correlation by petroleum companies, but they seem to have little application in landslide investigation.

Cost

In order that the cost element in landslide investigations may be included, the costs of some of the standard drilling methods are stated in the following. To be sure, the cost of subsurface investigations will vary with the materials to be drilled, the size of job, the accessibility of the site of operations, the distance to water, and other factors, but in general the ratio of costs of different types of operations will be similar from place to place. The following prices are suggested as probable commercial prices in areas that are reasonably accessible to wheel-mounted equipment:

Type of Drilling	Price per Foot
Standard penetration test	\$ 4.00- 4.50
2-in. Shelby tube samples	6.00- 7.00
4-in. power augers	1.50- 2.00
36-in. power augers	10.00-12.00
36-in. calyx hole in rock	50.00-85.00
2⅛-in. core boring in rock (NXM)	4.50- 6.00
6-in. cased horizontal auger holes	6.00

If geophysical methods are to be considered, it is worth noting that a resistivity determination of depth to bedrock at a depth of about 20 ft, if performed as part of a series of determinations, may be estimated to total \$4 to \$5. If the depth were on the order of 100 ft, the cost of each determination might total \$20 to \$25. These costs should be compared only with the costs of smaller diameter augers, as no samples are provided by the resistivity measurements and those provided by the augers are worthless except for very qualitative use. The cost of the auger determination of top of rock would be \$30, as compared to \$4 for resistivity, for a 20-ft depth. These unit prices indicate that subsurface investigations are not inexpensive;

but there is still no substitute for the essential facts in dealing with landslides, and the cost of the subsurface investigations should not be allowed to deter or prevent one from making a sound estimate of the situation based on required and necessary information. One cannot work without the necessary tools.

History of Slope

Determination of the history of the slope is perhaps one of the most important phases of landslide study and analysis. An understanding of the slope history is most beneficial in an interpretation of the causes and the mechanics of the movement. The geologic history and mode of development of the slope may provide the key to the analysis, as in the case of a slide on the weathered trace of a fire clay in colluvial soil. Insofar as they can be determined. both natural and artificial changes in the slope must be analyzed. Among these changes are: (a) construction changes, including those that involve cutting into the slope and those involving the imposition of surcharges on the slope; and (b) hydrologic changes due to seasonal or cyclical variations in temperature and precipitation, with resultant variation in hydrostatic and pore pressures, or to changes in the position and rate of movement of the ground water. Search should be made for evidence of previous movements of the slope and of nearby slopes. Eyewitness descriptions of the slide, if available, should be included in the slope history.

Vibrations may provide a "triggering" effect to initiate slide movement. These may occur as natural phenomena or may be artificially induced. Seismic vibrations may come from earthquake shocks or similar natural phenomena, or from the use of explosives. Vibrations may also be caused by passage of trains or trucks on railroads or highways. Other types of vibrations may be attributed to operating machinery, such as crushing plants and stamp mills.

Photographs

The importance of photographs, both terrestrial and air, cannot be overestimated in an analytical study. They are invaluable in presenting evidence in a damage suit or academic study. Preslide photographs may be impossible to obtain in most cases, but when available they are most convincing in showing "before and after" effects. Post-slide photographs are of two types: general, illustrating the over-all picture of the slide; and specific, illustrating details of particular slide features. Stereoscopic photographs in color are very helpful in preserving detailed information for office study by the analyst and for use in explanation to those who have not had an opportunity to examine the slide at firsthand.

Laboratory Tests

A knowledge of the physical properties of the soils and rocks involved in landslide areas, and the critical points where stability is affected, may be very beneficial in determining effective control, corrective, or preventive measures. Obviously, the nature of the laboratory tests to be made will depend on the problem involved. These tests may be routine identification tests, they may be shear tests, or they may involve mineralogic or weathering tests.

There are two principal groups of tests: (a) those classified as *soils tests*, and (b) those classified as *mineralogic*. These are briefly summarized in the following; detailed descriptions of all of them are to be found in standard text books.

Soil Tests

To utilize fully the field soils data that are obtained, supplementary laboratory tests should be performed. The tests which may be made fall into two broad categories — routine identification tests and shear tests. The types of test to be

made will depend on the problem at hand. Experience alone will often supply all the information which is necessary. However, when quantitative results are desired, it is desirable to perform at least a nominal number of tests.

Identification tests should include determinations of the Atterburg limits as well as the field moisture content of the soil. In addition, grain size tests, or mechanical analyses, may be in order.

Inasmuch as the Atterburg limit tests are made on disturbed samples of soil, their use is limited in connection with landslide problems. However, the purpose of making these tests is only to identify the soil and to assign to the soil a quantitative designation that will aid the engineer in estimating its probable behavior in the field. It is common practice in some localities merely to affix a visual description which will tell in broad terms its behavior. Unfortunately, the same visual description may mean different things to different individuals. The assignment of a standard test value to a soil will eliminate this difficulty and permit engineers from widely separate locations to speak in common terms.

The limit tests have come into such common usage that they have become routine for most laboratories and are thus relatively inexpensive to perform. Many useful correlations have been found between these test values and the potential behavior of the soil.

Field moisture contents can be conveniently made at the time the Atterburg limit tests are made. The strength of a soil is dependent on many variables, including density, moisture content, structure, texture, geological history, and many others. Generalizations are difficult, at best, regarding the interrelationship of these variables. Identification alone will not take into account all of these variables, but will yield data which will be most helpful.

The sensitivity of clays is most important. Sensitivity has been defined by Terzaghi and Peck (1948) as the ratio of the soil's unconfined strength in an undisturbed condition to that after remolding. This ratio increases as the sensitivity increases.

Laboratory tests for estimating the shearing resistance of soils are either of the direct shear, the triaxial compression, or the unconfined compression types. The direct shear test has en utilized by many engineers for a great number of years. There are several inherent disadvantages to this test, however, and the triaxial test is preferred. The fundamental assumptions and laws governing the strength of materials at failure are the same for both types of tests. The direct shear test consists of applying a shearing force on a soil sample encased in a split box. The shearing resistance is then measured on a plane between the upper and lower frames of the box. Among the disadvantages of this type of test are change in cross-sectional area during shear and rapid changes in moisture content as shearing progresses.

In contrast, the triaxial test is a compression test made under conditions of constant lateral pressure. Testing technique is very critical when considering this type of test. Certain fundamental considerations must be given to rate of loading, drainage during testing, and confining pressures which are used. For instance, a soil will exhibit high test values if it is permitted to drain during the testing period. Likewise, the time permitted for drainage will greatly affect the test results.

Mineralogic Tests

All landslides, as well as the rocks or soils from which they are derived, are made up of minerals. Because each mineral and rock has its own physical and chemical properties, it often is important to know the mineralogy and petrology, not only for scientific reasons, but also because it may have a bearing on the treatment method to be chosen. For example, fine-grained materials that contain a large proportion of sodium- or

potassium-bearing clay minerals may be reasonably stable, whereas if the sodium or potassium is replaced by calcium ions, as by the percolation of ground water through a marine clay, the same material may become extremely sensitive. Again, fine-grained materials composed of rock flour, as are some glacial sediments, have entirely different properties from those composed of clay minerals. Because of differences such as these, and because of the useful inferences that can be drawn from them, laboratory studies of the mineralogy and petrology are needed in many thorough-going studies of landslides. Among the most useful methods are the following:

1. *X-ray diffraction* — This test is made on fine-grained rock or clay-mineral soils when the fine grain size or lack of identification features makes usual methods of identification impossible. The diffraction pattern of the unknown mineral is compared with the patterns of known minerals until a perfect match is made and identification assured.
2. *Differential thermal analysis* — A test devised for mineral identification utilizing the thermal properties of minerals when heated at a uniform rate from room temperature to temperature near or at 1,000 C. Exothermic and endothermic reactions in a given mineral take place at typical temperatures and with typical magnitudes, thus permitting identification of the sample.
3. *Petrographic* — Identification tests utilizing the petrographic microscope, when the mineral grains are sufficiently large, employing the use of thin sections (impregnated or not as required) and polished sections. Here the intrinsic physical properties are determined optically and identification of the mineral grains and their relationship to each other is made.

Weathering Tests

Tests can simulate weathering by employing wetting and drying, and freezing and thawing techniques. When properly evaluated they may be a direct test for the determination of factors such as volume change, elasticity, and porosity.

Synthesis of Data

The preceding steps may be likened to finding the pieces of a puzzle; the succeeding steps are akin to assembly of those pieces. The data derived from the investigations previously outlined are of little value until they are interrelated in an understandable whole. These data present a group of facts, and possibly inferences, out of which the true picture of the event should emerge. The selection of any random section of the data may provide a completely erroneous conclusion as to the mass of the slide, its rate of movement, or its cause. If one is to determine the full implications of the event, care should be exercised to restrain the desire for hasty judgment and the acceptance of the apparent cause or causes until all of the evidence is in.

This is not to suggest that all of the data previously mentioned would be necessary to the full analysis of every landslide, because that is not the case. It is rather to emphasize the necessity to follow an investigation through to a logical conclusion; apparently sound conclusions as to cause arise rapidly in the early stages of landslide investigations only to be almost as rapidly disproved as additional data are developed.

Judgment during the period of investigation will permit the extension of certain lines of attack or the restriction of others. It is clear, for instance, that laboratory testing can be reduced to a minimum in dealing with a series of rockfalls, whereas a construction slide in a clay embankment may require elaborate laboratory tests as the only means of determining the critical factors. Similarly, during the synthesis of the data it may be apparent after preliminary examination that certain facets of the data are of no particular importance.

The field data comprise the observable and measurable facts in and adjacent to the slide. These data may well be more easily treated in graphic form than in any other manner. The presentation should be reduced to the simplest and most understandable forms consistent with the problem.

The basic "paper" may consist of cross-sections of the slide area, but in general the first "paper" should be a suitably referenced map on which are plotted all of the surficial data and the locations of all subsurface explorations. From this map may be drawn cross-sections on which the subsurface data may be plotted. The subsurface materials should be correlated from boring to boring or pit to pit to provide a realistic concept of the underground conditions. The water table may be indicated on the sections, but not without proper identification as to date. These cross-sections may well carry suitably located notes on water content and other characteristics of the soil. The locations of fractures in the slide that are intersected by the sections, and the position of the main surface of rupture, are fundamental data to be shown.

Associated with such descriptive papers are the less frequently used three-dimensional models. These are prepared for use in some court cases where simplicity and clarity in presentation of data to laymen on juries, as well as to the court and the opposing attorneys, are most important. Both models and cross-sections are important technically from the standpoint of assurance that the investigator's interpretation has been successfully drawn. If it proves difficult or impossible to show the interpretation on paper or by means of a model, it is highly probable that the interpretation itself is incorrect or incomplete.

The associated meteorological data, not derived directly from the study of the slide but obtained from other sources, must be correlated with the occurrence of the slide. Such data are usually in the form of tables of numbers, which are generally difficult of quick comprehension, and which should, therefore, be treated graphically. It is common practice to plot the precipitation (and temperatures, if freezing weather is involved) for the season during which the slide occurred and for a period of several years prior to the slide in order to note the effect of excessive precipitation. If the period of motion continues over a relatively long period of time, precipitation may be plotted against movement. Similarly, precipitation may be plotted against ground water levels in boreholes in the slide or against water levels in nearby observation wells.

The photographic record of the slide is most important in properly documenting the successive stages in development of the slide. Adequately dated, located and oriented photographs present essentially irrefutable testimony as to conditions, and may be the only basis for explanation and interpretation in court if the basic maps and sections are thrown out on some legal technicality.

Time in connection with landslides is important, not only from the standpoint of rate of movement but also from the standpoint of chronological sequence. The history of the slope on which the landslide occurred may furnish the clue to cause of the slide. It is well, therefore, to record in proper sequence all of the events which involve motion on the slope. There may be included in this sequence events in which no evidence of motion is shown, such as the date of excavation of toe material or the superposition of surcharge material, neither of which may have produced any motion *at that time.* The dates of change in condition of vibration, loading, water content, or restraint are always important.

The minimum summation of the data should include a base map showing the structure and distribution of the materials in and adjacent to the slide, a cross-section showing the relation of the materials in the slide to those in adja-

cent stable ground and the location of the water surface, suitable charts expressing the precipitation and temperature records during and preceding the motion, and a chronological record of the events associated with the motion. The mathematical examination of conditions, where applicable, should provide an indication of the magnitude of forces involved in the failure.

Disposition of the Report

Upon the completion of the field and laboratory reports and investigations, to whom should the reports go, and what restrictions, if any, should be placed on their distribution? This question might appear to be answered very simply, but such is not always the case. Obviously the reports are prepared for the "owner" or employer, whether he be a private individual; a construction or engineering firm; or a municipal, State, or Federal agency or commission. In most instances the reports are prepared for the chief engineer or his duly authorized representative. In some cases such a report is prepared for a lawyer or law firm, the court, an insurance company, a bank, or some similar interested party.

In many of these cases the reports are treated as confidential matter and the distribution is limited and restricted; as, for example, by the plaintiff or defendant in a court case, by an engineering firm relative to the development of a housing area, or by a bank considering investment in the area involved. In areas of interest to the military services such a report may be restricted because of the effect its release might have on a national defense program, or on military operations.

When such reports are made for municipal, State, or Federal agencies, having been paid for out of public monies they are expected to be available, if desired, for public scrutiny. On the other hand, a fine line may be drawn when the report is made for a private or semiprivate utility, such as a railroad or turnpike commission. Where a report might adversely affect revenue or undermine the faith of the public in such a utility, it often is wise to limit its distribution and attendant publicity, provided, of course, such action would not unnecessarily expose the public to unreasonable danger.

In most cases it is anticipated that the recipient of the report will act upon it in a way consistent with the safety of the public and/or the reasonable economic treatment indicated by the scope and magnitude of the slide problem. These actions may call for control and treatment of the slide, or even abandonment of the site area. In every case the economics involved will control the measures taken in relationship to any slide problem.

References

American Association of State Highway Officials, "Standard Recommended Practice for the Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes." In "Standard Specifications for Highway Materials and Methods of Sampling and Testing," p. 45-51, Washington, D. C., 1955.

Casagrande, Arthur, "A Nonmetallic Piezometer for Measuring Pore Pressures in Clay." Appendix in "Soil Mechanics in the Design and Construction of the Logan Airport." *Jour. Boston Soc. Civil Eng.*, v. 36, no. 2, p. 214-221, 1949.

Heiland, C. A., "Geophysical Exploration." Prentice-Hall, Inc., New York, N. Y., 1946.

U. S. Waterways Experiment Station, "Unified Soil Classification System." Technical Memorandum 3-357, 3 v., Vicksburg, Miss., 1953.

Wisconsin Geological Survey, "Geophysical Methods Applied to Geologic Problems in Wisconsin." Bull. No. 78, Scientific Series No. 15, 1954.

Part II

Solution of the Problem

Chapter Seven

Prevention of Landslides

Arthur W. Root

Preceding chapters have been devoted to the nature, classification, recognition, and investigation of landslides, all of which are of only academic interest unless they are utilized in the prevention or correction of landslides. On the other hand, a knowledge and understanding of these subjects will be of invaluable assistance in the selection and design of the most economical and effective methods of preventing or correcting landslides, which should be the ultimate objective of the reader for whom this book is primarily intended.

There is no sharp line of demarcation between prevention and control or correction of landslides; the basic principles governing them are the same, and many of the general methods of treatment are similar. However, there are significant differences which justify separate chapters on the two phases of slide treatment, even though this results in some duplication or repetition.

The treatment of potential landslides, where there is no evidence of any previous slide movement, would clearly be preventive in nature. Likewise, there would be little doubt that treatment of landslides developing during or subsequent to construction should be classified as corrective in nature. Where old landslides are involved, however, treatment might be considered as either preventive or correctional — if the landslide is geologically old and has been quiescent for centuries, treatment could scarcely be classed as correctional; on the other hand, treatment of old landslides which apparently have been inactive for a number of years, rather than of centuries, might be considered as either preventive or correctional.

Any attempt to classify an existing slide according to age or degree of quiescence would be confusing. Accordingly, prevention of landslides as discussed in this chapter will apply not only to unstable areas and potential landslides, but also will include all existing landslides which might be disturbed or reactivated by proposed construction, either by imposing additional load or by excavation. The category of slide correction, treated in Chapter Eight, then includes all landslides which develop during or subsequent to construction.

As would be expected, most of the treatment methods for the prevention of landslides are also used for correction or control purposes. On the other hand, some of the corrective measures are seldom if ever applied as preventive treatment. Table 4 is a summary of the more common methods of treatment for both correction and prevention of landslides. For convenience of reference the numerous methods of treatment have been listed under four general types, with a fifth category for miscellaneous methods, most of which are used less frequently. In this table there is no reference to the cause of the landslides. This omission is feasible only because it is not always essential to know the cause or causes of a landslide as such in order to prescribe treatment. Frequently there is no one single cause for a land movement, but a combination of two or several contributing factors.

TABLE 4

SUMMARY OF METHODS FOR PREVENTION AND CORRECTION OF LANDSLIDES

Effect on Stability of Landslide	Method of Treatment	General Use		Frequency of Successful-Use[1]			Position of Treatment on Landslide[2]	Best Applications and Limitations
		Prevention	Correction	Fall	Slide	Flow		
Not affected	I. Avoidance methods:							
	A. Relocation	x	x	2	2	2	Outside slide limits	Most positive method if alternate location economical
	B. Bridging	x	x	3	3	3	Outside slide limits	Primary highway applications for steep, hillside locations affecting short sections (parallel to c/L)
Reduces shearing stresses	II. Excavation:[3]							
	A. Removal of head	x	x	N	1	N	Top and head	Deep masses of cohesive material
	B. Flattening of slopes	x	x	1	1	1	Above road or structure	Bedrock; also extensive masses of cohesive material where little material is removed at toe
	C. Benching of slopes	x	x	1	1	1	Above road or structure	
	D. Removal of all unstable material	x	x	2	2	2	Entire slide	Relatively small shallow masses of moving material
Reduces shearing stresses and increases shear resistance	III. Drainage:							
	A. Surface:							
	1. Surface ditches	x	x	1	1	1	Above crown	Essential for all types
	2. Slope treatment	x	x	3	3	3	Surface of moving mass	Rock facing or pervious blanket to control seepage
	3. Regrading surface	x	x	1	1	1	Surface of moving mass	Beneficial for all types
	4. Sealing cracks	x	x	2	2	2	Entire, crown to toe	Beneficial for all types
	5. Sealing joint planes and fissures	x	x	3	3	N	Entire, crown to toe	Applicable to rock formations
	B. Subdrainage:							
	1. Horizontal drains	x	x	N	2	2	Located to intercept and remove subsurface water	Deep extensive soil mass where ground water exists
	2. Drainage trenches	x	x	N	1	3		Relatively shallow soil mass with ground water present
	3. Tunnels	x	x	N	3	N		Deep extensive soil mass with some permeability
	4. Vertical drain wells	x	x	N	3	3		Deep slide mass, ground water in various strata or lenses
	5. Continuous siphon	x	x	N	2	3		Used principally as outlet for trenches or drain wells

Increases shearing resistance	IV. Restraining Structures:							
	A. Buttresses at foot:							
	1. Rock fill	x	x	N	1	1	Toe and foot	Bedrock or firm soil at reasonable depth
	2. Earth fill	x	x	N	1	1	Toe and foot	Counterweight at toe provides additional resistance
	B. Cribs or retaining walls	x	x	3	3	3	Foot	Relatively small moving mass or where removal of support is negligible
	C. Piling:							
	1. Fixed at slip surface		x	N	3	N	Foot	Shearing resistance at slip surface increased by force required to shear or bend piles
	2. Not fixed at slip surface		x	N	3	N	Foot	
	D. Dowels in rock	x	x	3	3	N	Above road or structure	Rock layers fixed together with dowels
	E. Tie-rodding slopes	x	x	3	3	N	Above road or structure	Weak slope retained by barrier, which in turn is anchored to solid formation
Primarily increases shearing resistance	V. Miscellaneous Methods:							
	A. Hardening of slide mass:							
	1. Cementation or chemical treatment							
	(a) At foot		x	3	3	3	Toe and foot	Non-cohesive soils
	(b) Entire slide mass		x	N	3	N	Entire slide mass	Non-cohesive soils
	2. Freezing	x		N	3	3	Entire	To prevent movement temporarily in relatively large moving mass
	3. Electro-osmosis	x		N	3	3	Entire	Effects hardening of soil by reducing moisture content
	B. Blasting		x	N	3	N	Lower half of landslide	Relatively shallow cohesive mass underlain by bedrock Slip surface disrupted; blasting may also permit water to drain out of slide mass
	C. Partial removal of slide at toe	-	-	N	N	N	Foot and toe	Temporary expedient only; usually decreases stability of slide

[1] 1 = frequently; 2 = occasionally; 3 = rarely; N = not considered applicable.
[2] Relative to moving or potentially moving mass.
[3] Exclusive of drainage methods.

Moreover, the methods listed in Table 4 and described in this and the following chapter can only be applied successfully if the nature and history of the slide are thoroughly understood; whether or not such understanding is translated back into terms of the causes of the slide is immaterial to solution of the problem. All of the landslide treatments which improve the stability of an active or potential landslide mass do so either by reducing the activating forces which tend to induce the movement, or by increasing the shearing resistance or other forces that resist the movement. It is apparent, therefore, that any treatment which accomplishes either of these two effects will be of some benefit in preventing or minimizing landslide movement. For any particular landslide, however, not all types of treatment will be equally effective or economical. The selection of the best method of treatment is an engineering problem, requiring the evaluation of many factors which will be discussed later in this chapter.

The *prevention* of landslides is, in many respects, more difficult than *correction*, from the standpoint of both analysis and design. The limits, type and depth of an existing active slide can usually be determined by exploration and investigation; in contrast, the prevention of an incipient or potential landslide requires: first, recognition of the hazard, which may not be at all evident from superficial examination; second, anticipation of the character and magnitude of movement which may occur; and third, design of suitable treatment which will prevent any land movement during or following the proposed construction. Perhaps a fourth requirement should be added — decision by those in control that the hazard is sufficiently real to justify the expense of treatment.

One type of landslide, because it is so prevalent and costly to correct, is particularly troublesome to highway engineers; this is the roadway "slipout," a landslide which occurs at or below roadway grade, with a portion or all of the roadbed moving downward and outward. Such slipouts usually occur where the roadbed is partially on embankment, and typically do not extend above roadway grade. However, if the surface of rupture is deep and the highway is on sidehill, cut and fill section, the head of the slipout may be within the cut slope above the road.

In the entire field of landslide prevention and control, no other type of landslide presents such a challenge to the soil engineer and geologist, or affords such an opportunity for effecting savings in cost. Even though much less spectacular than the large landslides in slopes above roadway grade, the slipout of a large embankment is difficult and costly to correct. Often a nominal expenditure for treatment during construction would prevent the subsequent occurrence of a slipout which might seriously impair the usefulness of the highway and cost tens of thousands of dollars to correct.

This chapter considers only those embankment slipouts in which the surface of rupture is wholly or partially in original ground beneath the fill. Embankments may fail within themselves due to improper slope design, poor compaction, or similar causes. Although these failures are true landslides, according to the definition used in this book, embankment slope failures above natural ground are not discussed here. Similarly, embankments placed on level terrain — that fail solely because of displacement of weak foundation soil — are not treated here.

It will be noted that throughout this book the landslides most frequently cited or discussed are on highways. This is appropriate, not for the reason that the writers are principally highway engineers, but because the field of highway engineering will derive the greatest benefit from the application of sound engineering to the problem of landslide prevention and correction. Most of the references to highways would apply also to railroads; however, the mileage of new railroad construction is negligible compared to roads, and the problem of the railroads is primarily control and

correction of landslides on existing roadbeds. In the design and construction of dams and similar large structures thorough investigation of landslides is more common practice than is the case with highway construction, hence it can be presumed that most of the facts contained here are well known to the engineers in those fields.

Recognition of Existing Landslides

Recognition and investigation of unstable areas and the design of preventive treatment should be considered as essential phases of the preliminary planning and design of any project on which the proposed construction might induce land movements. It must be remembered that landslides may be caused by two general types of construction activities: (a) the imposing of additional load, such as by embankments, dams, or other structures; and (b) the changing of existing ground slope by excavation, erosion, or other causes. It is true that landslides may occur where the existing ground is undisturbed by man; this is evidenced by the numerous landslides which occur in many areas remote from any construction activity. The possibility of such landslides affecting proposed facilities should not be overlooked, particularly if the new facility is located on or crosses an old landslide. There have been many instances of residential developments on old quiescent landslides, where various conditions, perhaps unrelated to the residential construction, have caused the landslide to become active, resulting in damage to buildings and structures within the slide area. Development and construction over a large area may obliterate all evidence of the original landslide, leaving the purchaser blissfully unaware of any hazard.

In highway construction the proposed location may cross an old inactive landslide of such areal extent that it is overlooked in the usual routine soil survey. Or, the engineer may recognize and treat small local unstable areas without realizing that they are merely manifestations of large-scale land movement. It is seldom that any large-scale landslide, however old, does not leave some telltale evidence which can be detected by an engineer trained to look for the proper features. Many old landslides can be most readily recognized by the proper interpretation of aerial photographs, as described in Chapter Five. Once the slide area is identified by this or other means, a detailed ground study will commonly reveal further evidence of previous land movement. Field methods for recognizing and identifying old landslides are described in Chapter Four.

Having identified an existing landslide, active or latent, the engineer can then determine whether avoidance is economically practicable; if the slide cannot be avoided the necessary investigation can be made to determine the extent and nature of preventive treatment required.

Investigation

Recognition, classification and investigation of landslides have been discussed in previous chapters. All of the previously described techniques for detecting old landslides should be utilized during the reconnaissance or preliminary stages of a project in order to recognize and identify any old landslide, whether active or quiescent.

Recognition of existing landslides, although important, is not sufficient. A geologically ancient landslide may now be quite stable, so far as being affected by proposed construction. On the other hand, the excavation or loading involved in the construction may induce land movement even where there is no evidence of previous landslides. Where preventive measures are to be applied the investigation would, in general, be similar to that described in Chapter Six. Investigation in connection with landslide prevention does, however, differ in some respects from that to be applied to an inactive landslide: unstable areas must be explored, even though no prior slide movement is suspected, and a study made

of the possible effects of the proposed construction. If the proposed highway or structure will be located upon or across, or may be affected by an old landslide, an analysis must be made to determine whether the slide area will be stable under the conditions which will be imposed by the construction. In both cases the limits of the potential or incipient landslide are necessarily unknown, in contrast to slide correction investigations in which an active slide of definite extent already exists.

In any area of inherently low stability, especially where slides are known to be prevalent, the design of any major structure should be preceded by thorough investigation. Particularly in the construction of embankments on steep slopes in localities of questionable stability, each such site should be viewed with suspicion and thoroughly explored during preliminary stages of the project. Similarly, the design of cut slopes in such areas should be carefully scrutinized; in regions where instability is evidenced by land movement on existing routes, exploration of all proposed major excavation may be required. One error which is all too prevalent in the investigation of potential landslides is that of basing the analysis and design on data derived from shallow borings or test pits. All too often the material recorded in shallow borings as "solid formation" or "bedrock" may be merely float rock, boulders, or a thin layer of hard material underlain by a dangerously weak horizon. In excavation areas the borings should extend below the proposed grade; the foundation for embankments or other structures should be explored to whatever depth might be affected by the proposed loading.

A common justification for failure to make thorough exploration of potential landslides is that such exploration is too costly and would result in curtailed highway construction. It may be true that detailed soil surveys and landslide investigations are not economically practicable on certain unimportant roads, but for the freeways, toll roads, and other high-standard highways which comprise the major portion of the current highway programs, the engineer can afford only the best available practice in detecting and preventing landslides. The cost of controlling one preventable slipout or major landslide on a project will often more than offset the cost of a proper preliminary investigation on the entire project. It may not be possible or economical to design a highway to preclude the possibility of an occasional landslide, but this fact should merely emphasize rather than minimize the need for better engineering in the recognition and treatment of unstable areas.

The investigation aimed at slide prevention should be a cooperative project by the geologist and the soil engineer, or be made under the jurisdiction of an engineer who is thoroughly familiar with both of these phases of engineering.

To be of greatest value, the geologic and soils studies should be both general and specific. That is, a general or regional knowledge of the soils and geology in the area under investigation will be helpful in estimating the landslide potentials on a specific project, but such knowledge cannot take the place of detailed studies along the proposed right-of-way.

Much of the desirable general information on geology and soils can be obtained from available maps or from geologists or soil specialists who are familiar with the area. In a given geographic region, landslides tend to be more prevalent in one particular geologic formation or soil type than in others. Many of these "bad actors" are known in various parts of the country. The questionnaire upon which parts of this volume are based elicited a long list of geologic formations that are known to be landslide-susceptible. The list is not reproduced here because it is known to be incomplete for some parts of the country, a situation that could lead to a false sense of security in places. Moreover, it is not safe to label any entire geologic formation as landslide-susceptible; landslides generally result from a combination of

conditions or "causes" and stratigraphic sequence or rock type alone do not necessarily presage land movement. Nevertheless, if the engineer is familiar with the geologic formations and soils in his region that are especially susceptible to sliding, investigations can be made to determine the presence or absence of other contributing factors.

Methods of making detailed studies of the geology and soils of the construction site itself are described in Chapter Six. Both surficial and bedrock geology should be studied, for many of the most troublesome slides are confined to the mantle soil or weathered zone. A geologist trained in the study of surficial materials — and in the practical application of his results — can aid the soil engineer in his application of the theories of soil mechanics to the analysis of actual or potential slides in earthy materials. In the case of bedrock or rocky formations, of course, the geologic survey may provide the most useful information obtainable.

Analysis

Regardless of how comprehensive or thorough the investigation and exploration may be, the utilization of the data thus obtained depends on proper interpretation and analysis of those data. Methods of analyzing landslides and determining the effect of control treatments are described in detail in Chapter Nine. Practical applications of analytical methods have also been described by Baker (1952). In many cases the area being investigated is not amenable to the classical, theoretical methods of analysis; nevertheless, application of the principles of soil mechanics usually makes possible a rational comparison of various treatments, even though the absolute stability cannot be accurately computed.

Correct interpretation of the geological, geophysical, boring and test data derived from the investigation of a potential or actual slide area constitutes the most difficult phase of the engineering pertaining to landslide prevention or correction. Familiarity with local conditions and a background of experience in landslide work will, of course, assist the engineer in exercising the sound judgment which is essential to the solution of the specific problem involved. Soils are characteristically nonuniform; the stability of an embankment or excavation is influenced by a great many factors; and the geology and subsurface water conditions are often complex. Because of these conditions, the analysis of landslides and the design of control treatment cannot be standardized or routine; however, an understanding of the types, causes, and mechanics of landslides will make possible the application of certain basic principles having general validity.

Prevention of all types of landslides may be accomplished by one or more of the following methods: (a) reduction of activating forces, (b) increasing the forces resisting movement, and (c) avoidance or elimination of the slide.

Reduction in the activating forces can be accomplished by two general methods — removal of material from the portion of the slide which provides the driving force tending to cause movement, and subdrainage to eliminate hydrostatic pressure and/or to diminish the weight of the soil mass by reducing moisture content. However, the stabilizing effect of subdrainage is generally due primarily to increasing the shear resistance rather than by reduction of the motivating forces.

There are a great many methods for increasing the forces resisting slide movement, including the following: subdrainage, in order to increase the shear resistance of the soil; elimination of weak zones or potential surfaces of rupture by stripping or by breaking up or benching of smooth sloping surfaces; construction of restraining structures such as piles, walls, cribs, or toe support fills; and solidification of loose granular material by chemical treatment.

The most obvious and sometimes the most economical, but often overlooked, method of preventing landslides is by

avoidance. Methods by which this may be accomplished include: relocation of the proposed highway or structure to avoid unstable terrain, complete removal of an existing slide; or bridging the unstable area.

Evaluation and Comparison of Various Treatments

Avoidance of Potential Slides

It is not often feasible to avoid a potential slide by changing the location of a proposed highway or structure, but the possibility should not be overlooked. In some cases the highway can be shifted into stable ground by a slight change in alignment. Even though it may not be feasible to avoid an old landslide or an unstable area completely, it may be possible to so locate the highway that the slide area is crossed at the safest location, and where the construction would be least likely to induce further slide movement.

Where the proposed excavation will cross formations that are susceptible to bedding plane slides, the slide hazard can sometimes be reduced by adjusting the alignment so that the cut slopes will intercept the beds at a more favorable angle to the bedding planes. In some places, for instance, it may be possible to choose the opposite side of a valley or hill, where the bedding planes of the rock will dip into the cut slope rather than dipping toward the roadway.

In Chapter Four, there are listed nine ways in which proposed construction of cuts or fills may induce landslides. These factors are not repeated here, but they should be kept in mind when evaluating the probable effects of the new construction; recognition and consideration of these factors are essential in determining whether an attempt should be made to forestall a potential landslide by avoidance.

Prevention of landslides by avoidance does not necessarily require a change in alignment or location of a highway or structure. In some cases a revision in the grade line of a proposed highway may be equally effective in preventing slide movement. For example, where the most desirable grade line for new highway construction requires excavation and undercutting of an unstable slope, it may be possible to adjust the profile grade of the road so as to avoid any excavation at the toe of the hill, and instead to provide additional support by construction of an embankment which will act as a toe support or "strut."

If there is no way to avoid a potential slide and if preventive treatment will not assure stability, it is sometimes necessary to construct a bridge across the unstable area. The cost of a bridge is usually prohibitive, however, and extreme care must be exercised to design a structure which will not itself be damaged by moderate slide movement.

Bridging may be done in conjunction with the prevention method listed under II-D in Table 4, "Removal of all unstable material." The removal of all or a portion of the unstable material may be necessary to protect the structure from damage should slide movement occur. Figure 63 shows a bridge constructed across an active landslide which would not support an embankment and which could not be avoided by change of alignment of the highway. The bridge was so constructed that the superstructure could be shifted laterally on extended pile caps, in order that the alignment of the bridge could be maintained as the substructure moved with the landslide. By periodically sluicing out the slide material above the bridge, the slide movement at the bridge site has been held to a minimum, and the proper position of the bridge superstructure has been maintained.

A more common application of the bridging method of slide prevention is the sidehill viaduct. On a sidehill cut and fill section with a steep transverse slope the terrain below the highway grade may be too unstable to support the heavy embankment required. In such a case it is sometimes more economical to

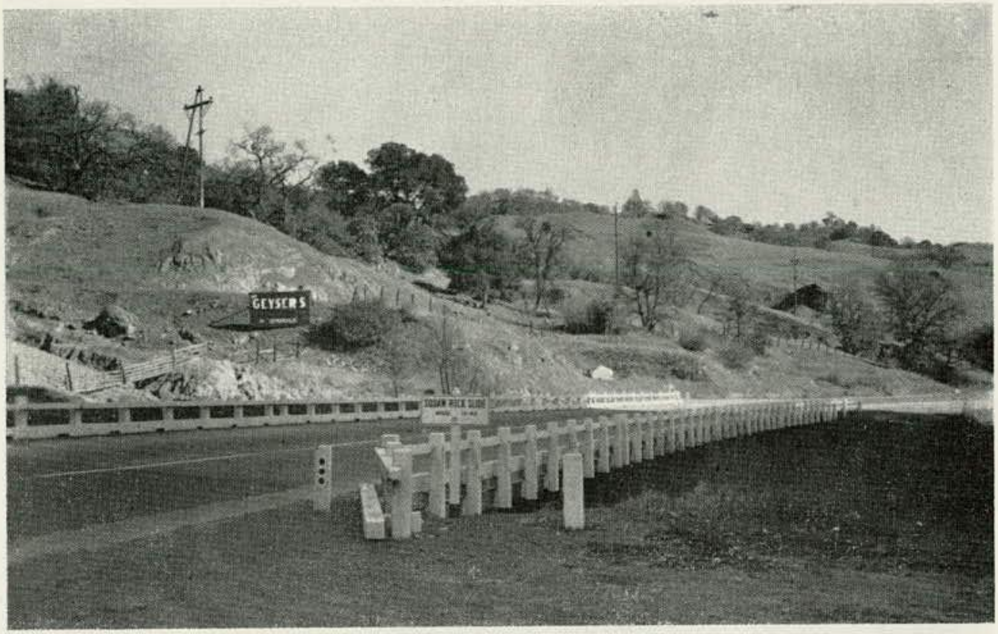


Figure 63. Landslide avoidance by bridging. Bridge on piling constructed across foot of active landslide near Hopland, Calif. Bridge incorporates provision for realigning superstructure if further sliding should cause shifting of piles. (Photograph courtesy of California Division of Highways)

support the outer half of the roadway on a viaduct, rather than to stabilize the foundation to support an embankment. Figure 64 illustrates the sidehill viaduct type of bridging.

In addition to such actual avoidance methods, many precautions may be observed which will minimize the possibility of land movement as a result of the proposed construction. Many of these precautionary measures are phases of design which should be considered whenever a structure is proposed in a location where ground movement might occur. Use of lightweight embankment material might be mentioned as an example of a design method to prevent landslides. If tests of the foundation soil in a proposed embankment area indicate that the soil will not support the load with the desired factor of safety, it is sometimes possible to substitute lightweight embankment material (such as cinders, volcanic tuff, or similar material), thereby reducing the embankment load sufficiently to provide a satisfactory factor of safety against sliding of the embankment. If the engineer has an understanding of the nature and mechanics of landslides, there will be less likelihood of necessary design considerations being overlooked.

In deciding whether to avoid an unstable area or to adopt preventive treatment, an economic comparison of the alternate locations will often supply the answer. Such cost comparisons should, however, consider the total cost rather than the cost of construction only. Proper consideration should be given to such factors as maintenance costs, probable service efficiency of the facility, and possible interruption in service or structural damage by land movement. It is true that an accurate appraisal of the last factor may be difficult; nevertheless, a rational comparison of alternates is impossible without consideration of the

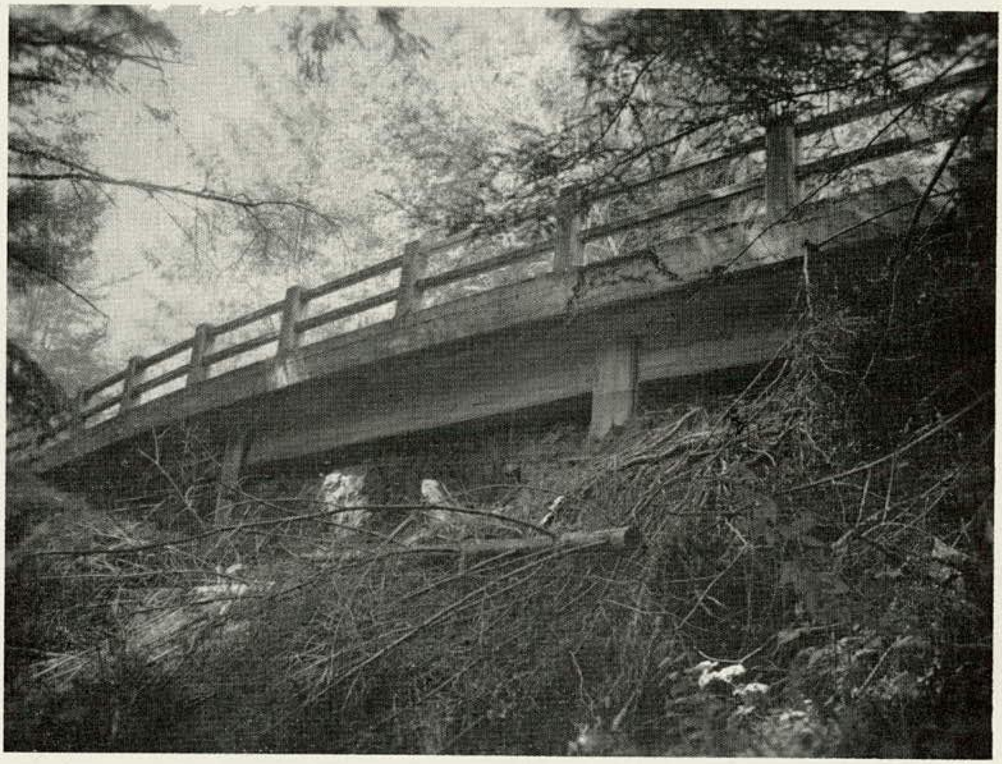


Figure 64. Landslide avoidance by bridging near Santa Cruz, Calif. Sidehill viaduct constructed across short unstable area (Photograph by Bruce Utt, courtesy of California Division of Highways)

risks involved. A choice between alternate locations based only on construction costs may be fallacious; frequently, a large initial investment may be most economical if the total costs for the life of the structure are considered. In practice, however, the method of financing a project may be such that immediate availability of funds will control the design. As a result, the selection of the most economical alternate may not be feasible, and the adoption of a less safe or less economical design with lower initial cost must be accepted as a compromise.

In such a case, the engineer in charge would probably be well-advised to see that a complete record of the investigation and predictions, as well as the reasons for the compromise, be placed on permanent file.

Excavation

Preventive measures in excavation areas consist primarily of proper slope design and drainage. It is usually more economical to design the excavation with slopes which will minimize sliding, rather than to excavate steep slopes and then flatten the slopes after sliding has occurred. This is especially true if the slides are of the slump type; for the reworked soil may have only a fraction of the strength of the in-place soil. For example, a material which would be stable if excavated on 2:1 (2 horizontal: 1 vertical) slopes may, after sliding has occurred on a steeper slope, require 4:1 slopes to prevent further sliding.

In dealing with a homogeneous soil, the strength of which can be determined

Figure 65. Multiple benching of cut slope to prevent landslides by unloading, by providing catchment areas for debris and by surface drainage. Inclined benches are used here to provide roadways for construction and maintenance equipment. (Photograph courtesy of Pennsylvania Department of Highways)

with reasonable reliability by laboratory tests, the slopes required for stable cuts can be computed with considerable accuracy by applying the theories of soil mechanics (Chapter Nine). It is seldom, however, that these ideal conditions prevail — homogeneity of large soil masses is rare, and the effective average strength of rocks can seldom be determined by borings and laboratory tests. Moreover, it is almost impossible to predict accurately what the hydrostatic pressures will be in the future or at time of failure. Nevertheless, a knowledge of the character of the soil, and of subsurface water conditions, is no less important because it cannot always be applied directly. Proper slope design requires that this information be as complete as possible. With this knowledge, and by applying the methods of stability analysis outlined in Chapter Nine, the soils engineer can estimate the improvement in stability effected by flattening the proposed cut slope or by removing material which might induce slide movement. Such analyses are helpful, even where the true shear strengths of the soil cannot be accurately determined by laboratory tests. Obviously, these analyses are impossible without a knowledge of the character and strength of the soil throughout the cut area. Preferably such information is obtained from borings, as well as from geologic or geophysical data.

A study of existing cut slopes of similar material in the region is helpful, but extreme care is required in comparing existing and proposed cuts. Existing cuts commonly are much shallower than the excavation on an improved location; for example, 1:1 slopes in a given formation might be stable for a height of 50 ft, but

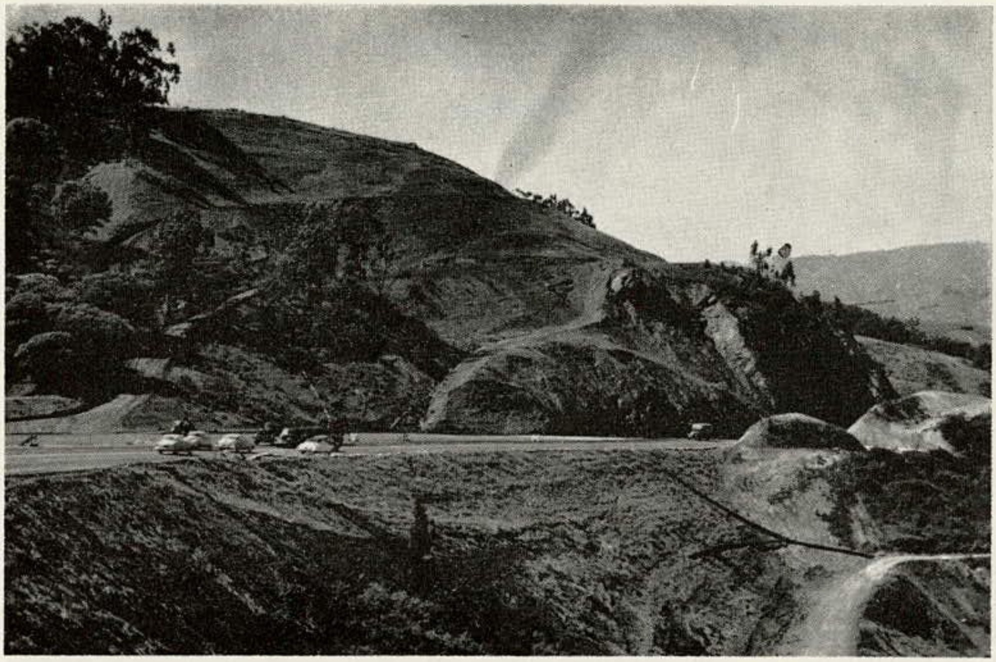


Figure 66. Prevention of landslides by flattening cut slopes near Waldo, Calif. Top of flat cut slope is at skyline in left background. Note bench construction of high embankment in foreground. (Photograph by Bruce Utt, courtesy of California Division of Highways)

the same slope might be much too steep for a 150-ft cut. Another pitfall to be avoided in basing slope design on existing slopes is the assumption that the soils and rocks, as well as the ground water conditions, will be identical in the proposed cut area as in an existing cut. Even though the distance between the two is small, conditions may be quite dissimilar. With proper consideration of these factors, the study of existing slopes can be a valuable guide, but slope design should be the responsibility of the soil engineer or geologist rather than the locating engineer. A background of experience in the same region and familiarity with local conditions are always helpful to the geologist or soil engineer.

In general, cut slopes constructed with benches or "berms" are considered preferable to equivalent uniform straight slopes. The benches should be constructed with a V or gutter section, with a longitudinal drainage grade, and with suitable catch basins and flumes or pipes to carry the water down the slopes. Paving of the gutters or ditches may be necessary to reduce erosion or to prevent percolation of water into pervious areas on the benches. The benches serve two purposes: to intercept and remove surface water or seepage from the cut face; and to prevent rocks, debris or sloughed material from falling on the roadway. The benches should be so constructed that they are accessible to maintenance equipment subsequent to construction, in order that any small slides may be removed and the drainage system may be properly maintained. Figure 65 illustrates the construction of benched cut slopes; Figures 66 and 104 illustrate slope flattening.

The first type of excavation listed in Table 4 — "A. Removal of head" — applies only to treatment of an existing

landslide; the fourth one — "D. Removal of all unstable material" — would usually be practicable only in the case of an existing slide. However, these two techniques have also been applied in some parts of the country for controlling potential slides in talus material. In the case of an existing landslide, removal of material from the head of the slide reduces the activating force, and thus has a stabilizing effect. But such unloading is usually proposed only where the slide is to be undercut at or near the toe. The amount of unloading at the head should be sufficient to compensate for any reduction in support caused by excavation elsewhere. Here again, the application of the principles of soil mechanics, as described in Chapter Nine, will enable the engineer to estimate the effects of the proposed excavation, and to design the unloading and cut slopes to provide the required stability. In cut areas the removal of all unstable material is usually not necessary, and is seldom economical except for very small masses. Even this type of treatment requires sufficient investigation to determine the depth and areal extent of the weak material. If the yardage involved is small it may be desirable to remove all of the weak material; otherwise, the strength of the weak soil should be determined, and the cut slopes designed to provide stable excavation.

Where embankment is to be constructed over an old landslide area or other material having inadequate strength to support the proposed loading, removal of the weak material should be considered. If the weak soil layer is only a few feet thick, stripping of the weak material is usually more economical than other methods of treatment. If there is evidence of seepage — and the unstable condition is often caused by ground water — suitable drainage should be provided before placing the embankment. A blanket of pervious material, together with necessary underdrains, may be required to prevent reduction of shear strength and/or development of pore pressure due to subsurface water.

Economic Considerations

In the design of cut slopes to prevent landslides, economic considerations cannot be disregarded. If failure of a structure might result in loss of life and irreparable damage, as in the case of a dam or bridge, a high factor of safety is warranted, and may indeed be essential. It is seldom economical to design cut slopes sufficiently flat to preclude the possibility of landslides, and it is often better to remove or correct a few slides during construction or as a maintenance operation than to design with excessively flat slopes. Many secondary highways that traverse rough terrain could not be constructed with available funds except by accepting some risk of landslides.

The fact that a calculated risk of slide movement must be accepted at times is, however, no justification for lack of thorough investigation and adoption of all economical means of slide prevention. With adequate information on geology, soils and ground water conditions in a proposed cut area, the probability of landslides can be estimated and the consequences of possible slides appraised; only after such an evaluation can the most economical design be selected. Any risk should be a calculated one, rather than a mere gamble arising from lack of investigation and analysis.

DRAINAGE

Surface Drainage

Every precaution should be taken to prevent surface runoff water from entering a potentially unstable area. Any sags, depressions or ponds above the slope line of either an enbankment or a cut should be drained to minimize the possibility of surface water percolating into a weak or unstable area. If the new construction crosses an old landslide its surface should be reshaped as necessary to provide good surface drainage, but unnecessary removal of vegetation

should be avoided lest excessive erosion may occur. Sealing of all surface cracks in any type of slide will be of benefit, both by preventing entrance of surface water into the slide mass and by reducing frost action in areas subject to freezing and thawing.

Although surface drainage alone will seldom correct an active landslide, any improvement in surface drainage will be beneficial. In the case of potential landslides, where no movement has occurred prior to construction, surface drainage may result in greater returns from the investment than any other type of preventive treatment, even though other preventive measures may be required in conjunction with the surface drainage. Surface runoff or the water flowing from springs or seeps should never be allowed to drain into or across an unstable area or potential landslide. Methods of improving surface drainage include reshaping of slopes, construction of paved ditches, installation of flumes or conduits, and paving or bituminous treatment of slopes.

Subdrainage

If the preliminary investigation reveals the presence of ground water which may induce slide movement, adequate subdrainage should be included in the plans. Such subdrainage is equally important in cut areas and under proposed embankments. The effectiveness and frequency of use of the various types of drainage treatment vary according to geologic formation and climatic conditions; they probably are influenced by local custom also. It is generally agreed, however, that for the majority of landslides ground water constitutes the most important single contributory cause; and in many areas of the country the most generally used successful methods for both prevention and correction of landslides consist entirely or partially of ground water control. This is especially true of the Pacific Coastal region.

Although most of the types of subdrainage treatment are applicable to the prevention and correction of both embankment slipouts and landslides in excavation areas, the differences in methods are considered of sufficient importance to justify separate discussion of subdrainage treatments applied to these two general types of landslides.

Drainage in Embankment Areas.— Slipouts may occur whenever the imposed embankment load results in shear stresses that exceed the shear strength of the foundation soil; or where the construction of the embankment interferes with the natural movement of ground water, and results in the development of pore pressure or hydrostatic pressures. Two factors must, therefore, be considered in the investigation of possible slipouts: weak zones in the foundation soil, which may be overstressed by the proposed embankment load, and subsurface water, which may either result in the development of hydrostatic pressure or may reduce the shear strength of the soil sufficiently to induce slide movement. Careful exploration will usually reveal these conditions before construction, but the investigator must be of a suspicious and inquisitive nature, as there may be no readily apparent surface indications of the unstable conditions. Some of the methods of preventing roadway slipouts are listed and discussed hereinafter.

As previously noted, if a surface layer of weak soil is relatively shallow and is underlain by stable rock or soil, the most economical treatment is usually that of stripping and wasting the unsuitable material, as illustrated by Figures 67 and 68. If seepage is evident after stripping or if there is a possibility that it may develop during wet cycles, a layer of pervious material should be placed before the embankment is constructed. This may consist of clean pit run gravel, free-draining sand, or other suitable local materials. If springs or concentrated flows are encountered, drain pipe may be required also.

Where subsurface water or soil of questionable strength is found at such

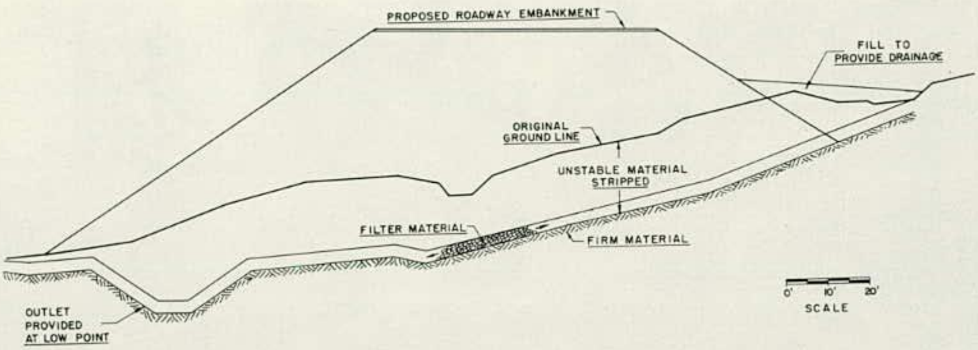


Figure 67. Stripping as a slide prevention measure. Typical cross-section of Redwood Highway in Humboldt County, Calif., showing stripping of unstable material before constructing embankment. (Drawing furnished by C. P. Sweet, courtesy of California Division of Highways)

great depths that stripping is uneconomical, deep drainage or stabilization trenches have been used successfully to prevent slipouts. Such stabilization trenches are usually excavated with power equipment with the steepest side slopes that will be stable for the minimum construction period; they should extend below any water-bearing layers and into firm material. A layer of pervious backfill material is placed on the bottom and side slopes (see Fig. 69), with an underdrain pipe in the bottom; then the trench is backfilled and the embankment constructed. Figure 70 illustrates combined use of stripping and drainage trenches.

If the unstable area is in a natural draw or depression and of limited areal extent, one trench normal to the centerline of the road may be sufficient; in the case of large areas, an extensive system of stabilization trenches may be necessary, frequently in a herringbone

Figure 68. Stripping wet unstable material before placing embankment near Orick, Calif. Blanket of pervious filter material will be spread over stripped area. (Photograph courtesy of California Division of Highways)

Figure 69. Placing filter material in deep drainage trench near Orick, Calif. Filter material being dumped over side slope and spread in bottom of drainage trench with dozer. (Photograph courtesy of California Division of Highways)

pattern. The trenches, in addition to providing subdrainage, add considerable structural strength to the foundation.

This type of treatment for prevention of slipouts has been used successfully on numerous projects. An early example was reported by Root (1938). On a recent highway construction project 4.9

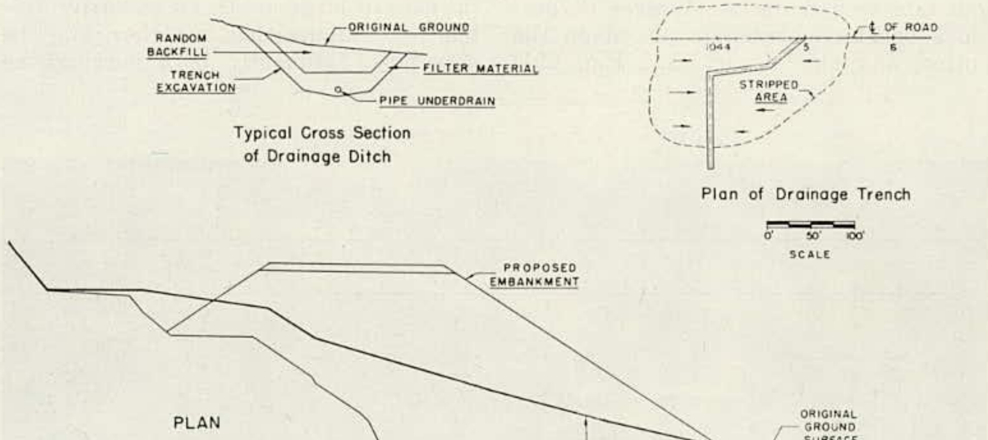


Figure 70. Slide prevention near Willits, Calif. by combination of stripping and drainage trench. Plan and cross-section of preventive treatment consisting of stripping unsuitable soil and constructing drainage trenches. (Sketch furnished by C. P. Sweet, courtesy of California Division of Highways)

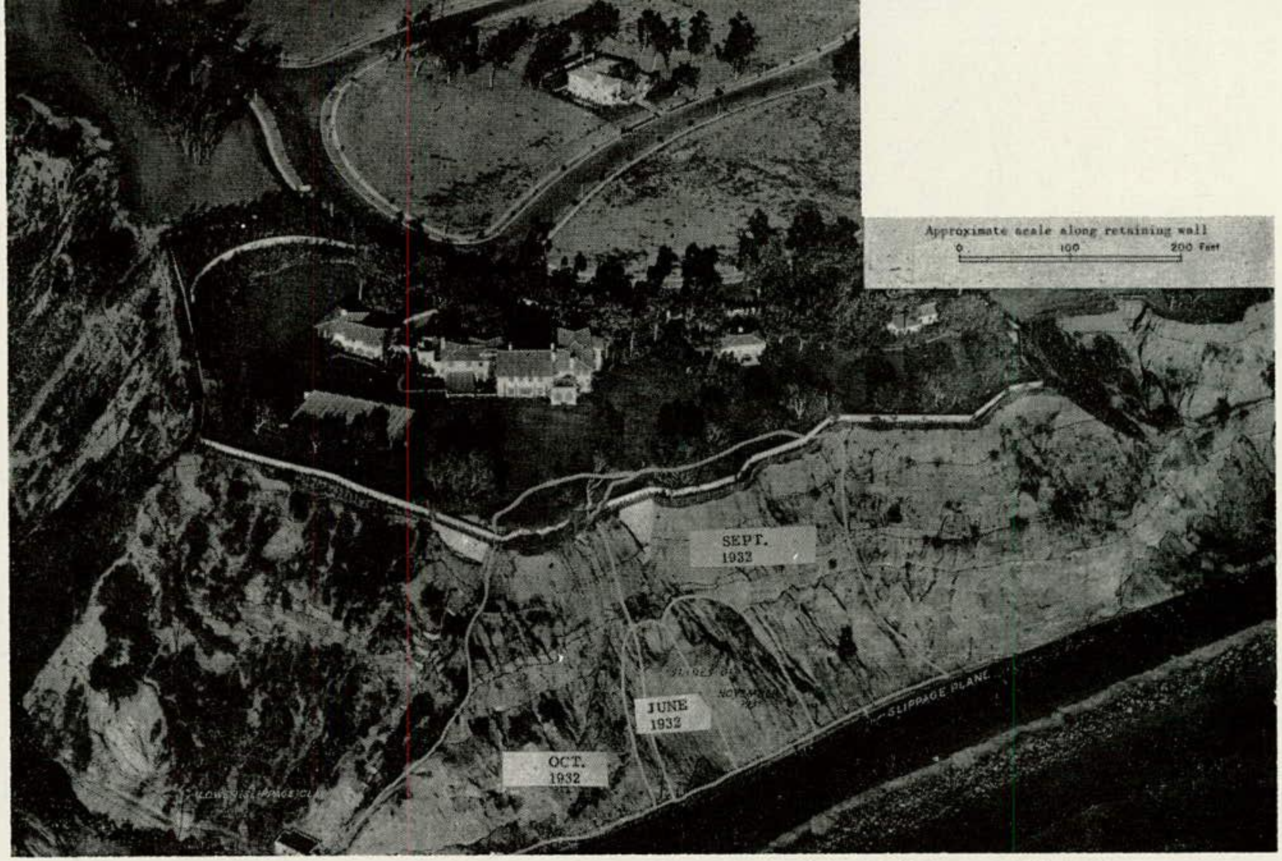


Figure 71. Large slide in the fall of 1932 northwest of Santa Monica, Calif. The highway was blocked by a sliding mass of 100,000 cubic yards, and a valuable estate was damaged through loss of approximately 100 feet by 200 feet of land. (Individual slides outlined in white, with dates.) Geologic studies indicated that movement began along slickensides in a nearly horizontal stratum of clay lying approximately 10 feet above the highway. Two exploratory tunnels were dug to drain water, believed to be lying on top of the clay stratum, and to determine the extent of the slickensides. No free water was encountered. It was decided that the most economical solution was to dry out the clay. Therefore, additional tunnels were drilled and a gas furnace was installed with blowers to circulate hot air. It was estimated that 3,000 lb of water per day were evaporated during the first six months. The furnace was in operation from August 1933 until approximately 1939, by which time movement was negligible. (Photograph by Fairchild Aerial Surveys, Inc., courtesy of Harry R. Johnson, Consulting Geologist)

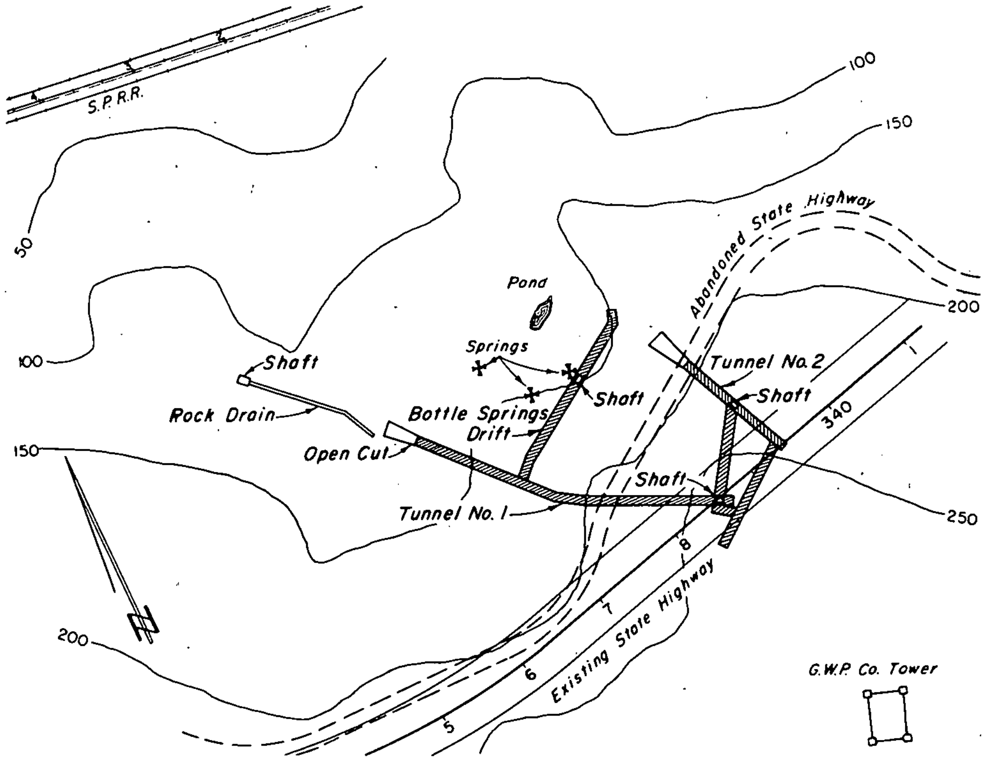


Figure 72. Drainage tunnels to prevent landslides. System of drainage tunnels installed during construction of new highway, designated as "Existing State Highway" on sketch, near Crockett, Calif. (Courtesy of California Division of Highways)

mi in length, the installation of stabilization trenches for slipout prevention required 65,000 cu yd of trench excavation; 107,000 cu yd of filter material were placed in drainage trenches and stripped areas; and more than 20,000 lin ft of perforated metal drain pipe were installed. Stabilization trenches of this kind have been constructed as deep as 40 or 50 ft. Although the cost increases rapidly with depth, this method of slipout prevention is often more economical than any other type of treatment which might be equally effective.

Where the depth to subsurface water is so great that the cost of stripping or drainage trenches becomes prohibitive, drainage tunnels are sometimes used. Although originally and more commonly used as a correctional treatment, drainage tunnels are sometimes constructed as a preventive measure. The use of drainage tunnels was fairly common at one time, both by railroads and by some highway departments; but at present this method is used rather infrequently, due largely to the relatively high cost. An elaborate installation of drainage tunnels, together with an ingenious hot-air furnace for drying out the soil, was used to control a large slide near Santa Monica, Calif. (see Fig. 71; Hill, 1934). Use of drainage tunnels in Oregon has also been described (*Roads & Streets*, 1947). These tunnels, usually about 4 ft by 6 ft in cross-section, must be excavated by manual methods; skilled tunnel workers are not normally employed on usual construction projects; and, of course, other methods

of treatment which permit the use of construction equipment are likely to be less costly than the tunnels. Figure 72 shows an installation of drainage tunnels on a highway project.

Horizontal drains have, since their development during the past few years, supplanted drainage tunnels in many cases. As was the case with drainage tunnels, they were first installed as a corrective treatment. Although they are still used principally for this purpose, they have been installed at a number of locations as a preventive treatment (see Fig. 73). Horizontal drains usually consist of perforated metal pipe, often 2 in. in diameter, forced into a predrilled hole (generally 3 to 4 in. in diameter) at a slight angle to the horizontal; the gradient of horizontal drains may range from 5 to 25 percent (see Fig. 105). The length of these drains may be as great as 200 to 300 ft or more.

The origin of the horizontal drain is somewhat obscure; however, much of the early work in developing equipment and methods was done by the California Division of Highways beginning about 1939. There are numerous installations of such drains in California, as well as in Oregon, Washington, and several other states. Equipment and techniques for installing horizontal drains have been described by Stanton (1948) and by others. An example of the extensive installation of horizontal drains in slide control work is the Ventura Avenue oil field in California, where hundreds of horizontal drains, totaling more than 40 mi. in length, have been installed in the large landslides within this oil field (Mineral Information Service, 1954).

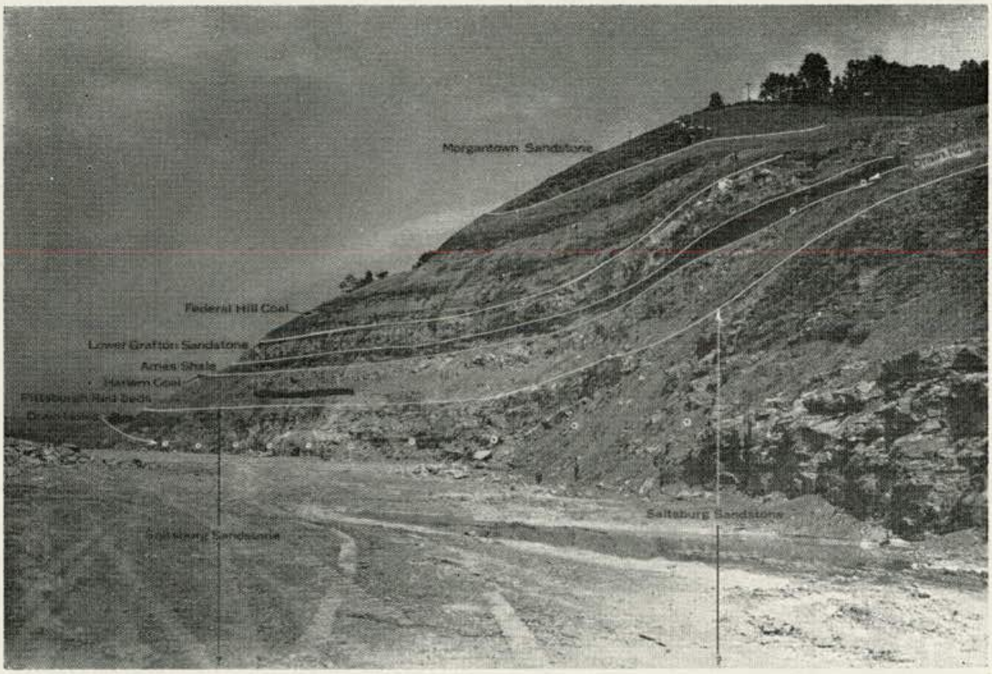


Figure 73. Horizontal drains used to stabilize cut in bedrock. The drains were placed 50 to 100 feet apart beneath permeable sandstones. One set is at the base of the Ames shale, beneath the Grafton standstone; the other at the top of a lens of indurated clay in the Saltsburg standstone. Note on skyline that cut slopes range from 1:4 (horizontal:vertical) to 1:1, depending on the character of each layer of rock. Spillway of Youghiogheny River reservoir, Pennsylvania and Maryland. (Photograph courtesy of Corps of Engineers)

Figures 76 and 77 show a system of horizontal drains that were installed as a slide prevention measure. Similar installations are frequently used as corrective treatment (Figs. 73, 104, and 106).

If wet areas or seepage zones which cannot be corrected by stripping and surface drains exist in the foundation of a proposed fill area, horizontal drains are likely to be the most effective means of removing subsurface water which might otherwise cause slipout movement. Preferably, horizontal drains should be installed in such a way that they can be inspected and maintained after the embankment is completed; this may necessitate placing a tunnel or large pipe to provide access to the drains. Thorough investigation of the unstable area, including test borings, will furnish the necessary information from which the elevation, gradient, and spacing of the horizontal drains may be determined.

Vertical drain wells for slipout prevention may be used for two purposes, as follows:

1. In conjunction with horizontal drains the vertical drain wells may provide a drainage path between lenses or strata of water-bearing material which are separated by impervious strata. If installed under an embankment, an outlet for the vertical drain well can be provided by means of a horizontal drain; such an installation is illustrated by Figure 74.

2. Vertical drain wells have also been installed under embankments to accelerate the consolidation, through removal of water, of weak compressible foundation soil. Such drains, usually 15 to 24 in. in diameter, are drilled or driven through and to the bottom of the saturated, compressible soil layers, then backfilled with coarse sand or other suitable filter material. A layer of filter material is placed over the area in which the vertical drains are installed, with outlets leading beyond the embankment slope line.

Design of this latter type of vertical drain well should be based on laboratory tests of undisturbed soil samples, from which the consolidation and strength characteristics of the soil are determined.

The continuous siphon is an ingenious method devised in the State of Washington for providing a drainage outlet for drainage wells or sumps (see Fig. 75). This siphon arrangement can be used to drain trenches, wells or sumps by siphoning instead of installing more costly tunnels, drilled-in pipes, or similar conventional outlet systems, and permits installation of subdrainage systems in areas not having readily accessible outlets. This continuous siphon method

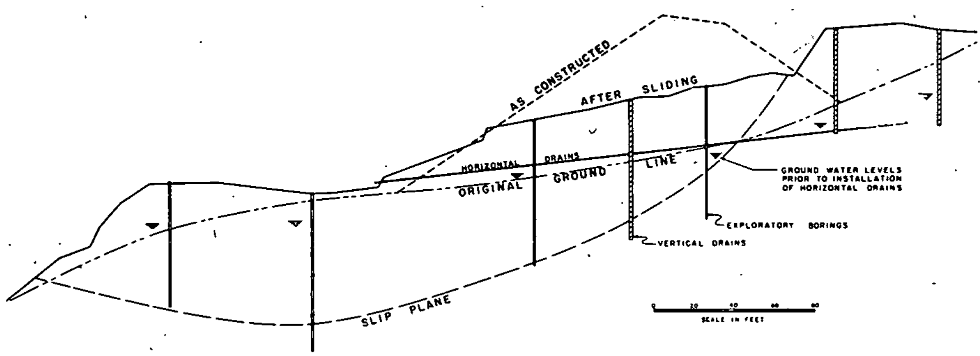


Figure 74. Slide treatment consisting of horizontal drains and vertical drain wells. This was corrective treatment of an active landslide at San Marcos Pass near Santa Barbara, Calif.; however, similar drainage treatment has been used as a preventive measure. (Courtesy of California Division of Highways)

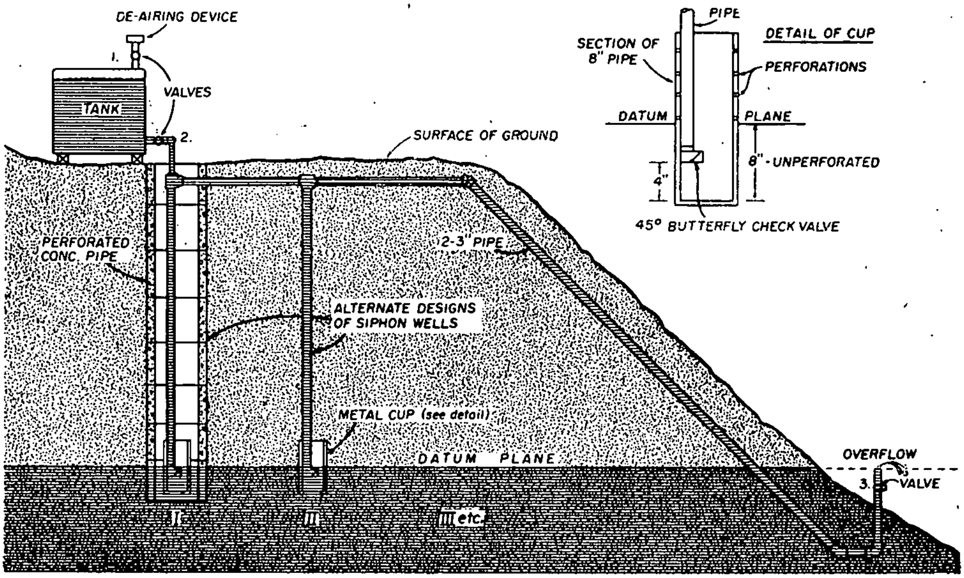


Figure 75. The Washington siphon. This system of vertical collector pipes and siphon arrangement has been successfully used by the State Highway Commission of Washington for lowering the water level and stabilizing landslides.

has the usual limitation of depth that is true of all siphons, but it is very useful where applicable.

Drainage in Excavation Areas. — All of the subdrainage methods discussed in connection with slipout prevention could as well be applied to prevention of landslides in excavation areas. Drainage trenches are sometimes installed as interceptors of subsurface water above the limits of the excavations, too often with indifferent success. There is seldom any assurance that such intercepting trenches will effectively cut off all ground water which might contribute to slope failure. If deep trenches are required the cost frequently becomes prohibitive, considering the probable effectiveness of the drainage trenches.

The most widely used successful method of subdrainage for preventing slides in cut slopes is probably the horizontal drain treatment. These horizontal drains are the same as previously described for slipout prevention. In excavation areas the drains are installed as the cut is excavated (see Figs. 76 and 77), often from one or more benches in the cut slope. Numerous cut slopes drained by this method have remained stable in spite of unfavorable soil formations and the presence of large amounts of subsurface water. It should be emphasized that if the treatment is delayed until after a landslide has developed, the cost of correcting the slide is likely to be much greater than the cost of installing drainage which would have prevented the sliding. And it is equally important to note that the need for such preventive treatment can be anticipated only if a thorough soil investigation is made before designing the project. In most cases test borings are required in addition to geologic studies or superficial inspection.

Restraining Structures

Retaining Walls and Bulkheads

Crib walls, piling, bulkheads, and other restraining devices are most commonly used as corrective measures after slide movement develops, too often with dubi-

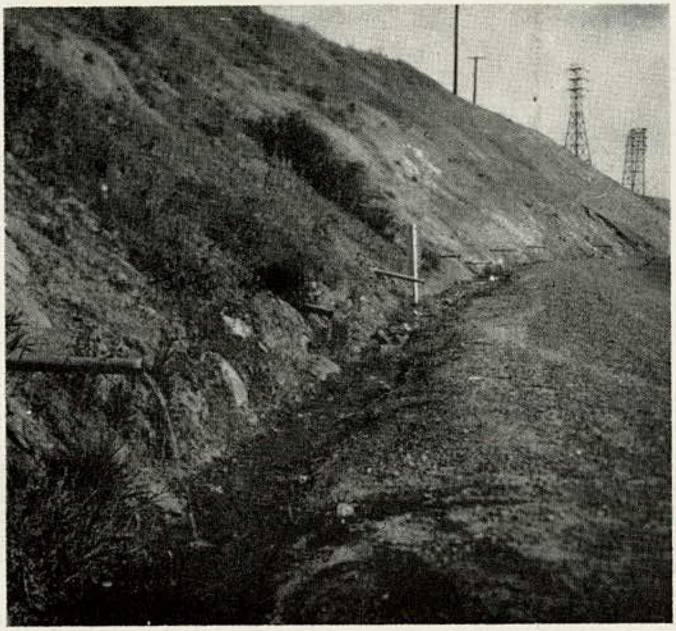


Figure 76. Horizontal drains for prevention of landslides. Horizontal drains installed from roadway grade during construction near Vallejo, Calif. Note flow of water from outlet of drain in foreground. (Photograph by A. D. Hirsch, courtesy of California Division of Highways)

ous success. These structures are more likely to be effective if installed as preventive treatment, before the soil mass has become weakened by slide action. The limitations of this type of treatment should be recognized. The increased resistance to sliding provided by any of these restraining structures is somewhat limited, and is dependent on the ability of the structure to resist (a)

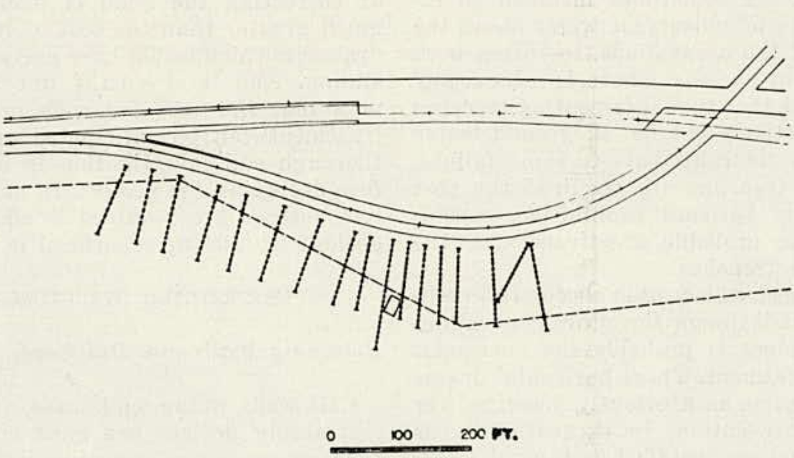


Figure 77. Plan of horizontal drains shown in Figure 76.

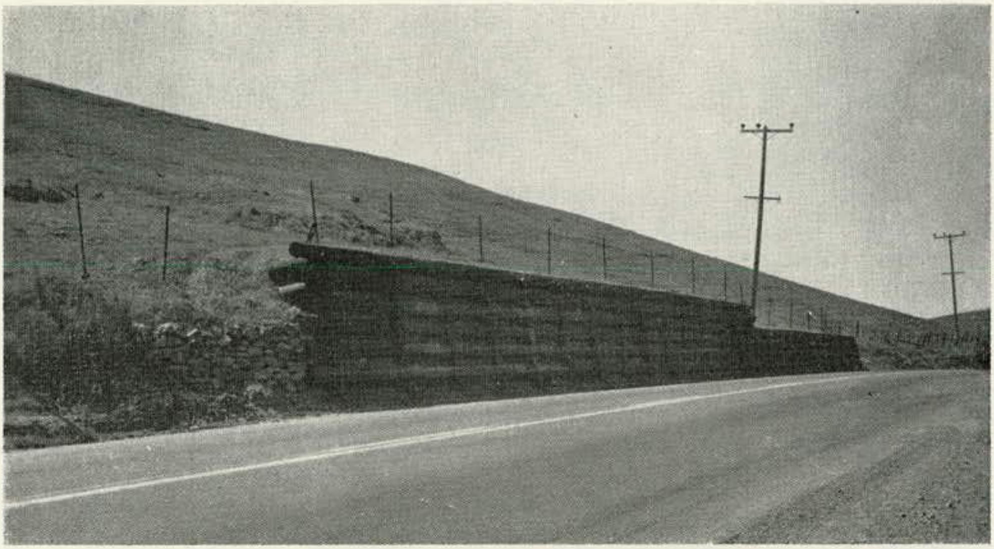


Figure 78. Log crib to prevent sliding near Napa, Calif. Note evidence of old landslides above cribbing and also near right skyline. (Photograph by Bruce Utt, courtesy of California Division of Highways)

shear action, (b) overturning, and (c) sliding on or below the base of the structure. If the forces tending to cause slide movement exceed the resisting forces by only a small amount, construction of some type of restraining structure may provide sufficient additional strength to produce stability and thus prevent slide movement.

Retaining walls and bulkheads of various types have been used: Figure 78 shows a redwood log crib; a bin-type concrete crib wall is illustrated by Figure 79; Figure 80 is an example of the metal bin-type crib wall; and Figure 81 shows a rubble masonry retaining wall installed to prevent a landslide. A unique restraining structure is shown in Figure 82, a modified slope paving as slide prevention; Figure 83 illustrates the use of coarse rock for slope paving. Figure 84 shows a masonry wall used to support overhanging rocks.

The principal use of crib walls or retaining walls is at the toe of an embankment slope where the normal fill slope would not "catch," or at the toe of a cut slope which must be undercut to provide lateral clearance for a roadbed or structure. Strictly speaking, such walls or cribs do not constitute treatment for slide prevention, but are actually a phase of the slope design. The limitations of these structures as slide preventive treatment should be recognized.

Unless the soil is free-draining, or it is known that no subsurface water will ever be present, the design and construction of any crib wall or retaining wall should include adequate provisions for drainage, including pervious backfill, drain pipes, and weep holes. The use of retaining structures is one of the earliest methods used for controlling landslides, but the results of this method, at least in the earlier attempts, were not encouraging. In 1928, Ladd reported numerous failures of retaining walls and suggested use of other methods, particularly drainage (Ladd, 1928). Figures 85 and 86 show striking examples of unsuccessful attempts to correct landslides by means of piles and bulkheads. These landslides were subsequently controlled by extensive subdrainage installations and by other means.

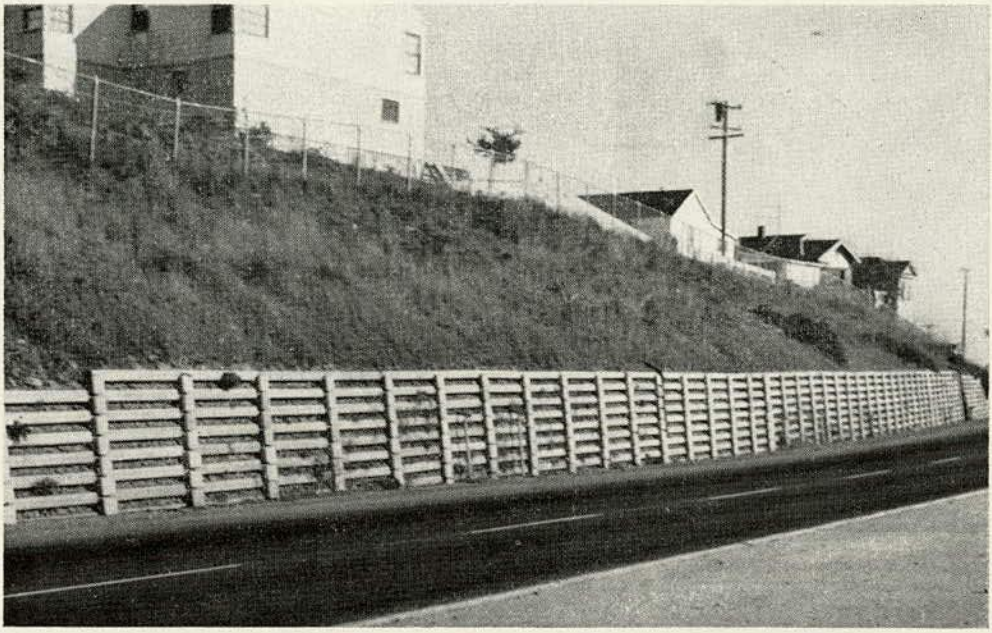


Figure 79. Restraining structure; concrete crib wall installed during construction to prevent land movement which might jeopardize dwellings located above top of cut slope at Arcata, Calif. Note gravel backfill. (Photograph by T. W. Smith, courtesy of California Division of Highways)

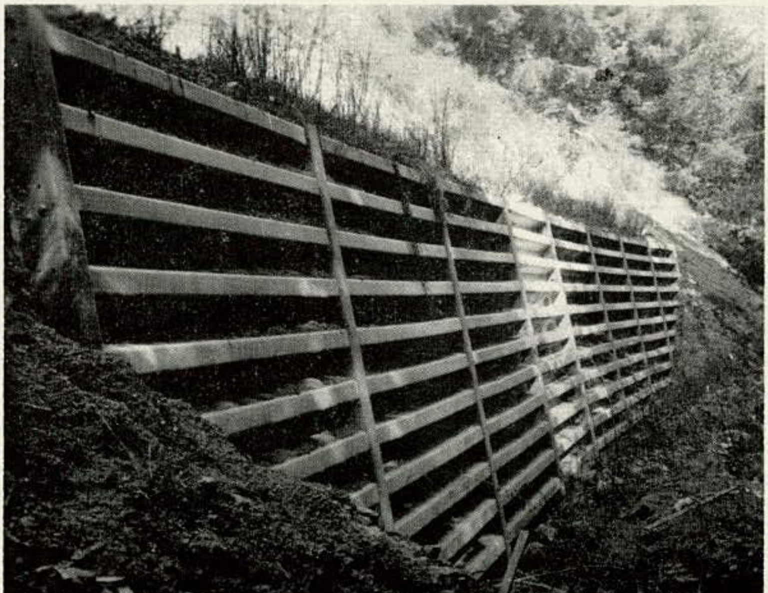


Figure 80. Restraining structure; metal crib wall near Santa Cruz, Calif. (Photograph courtesy of California Division of Highways)

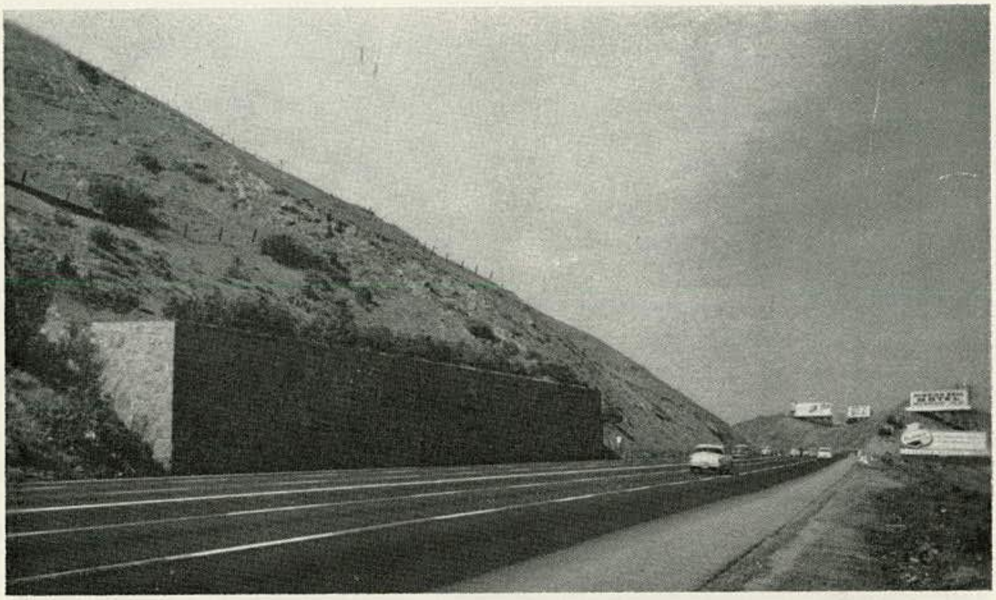


Figure 81. Rubble masonry retaining wall installed to prevent land movement in old landslide area near Brisbane, Calif. (Photograph by Bruce Utt, courtesy of California Division of Highways)

Figure 82. Restraining structure; concrete slope paving placed monolithically with an underlying grid of reinforced concrete beams, to prevent slide movement of unstable cut slope near Valona, Calif. (Photograph by E. W. Herlinger, courtesy of California Division of Highways)

Figure 83. Coarse slope paving as used along Route 2, Shelburne Falls, Mass. The bank to be retained is composed of fine sand and silt, probably 50 percent of it less than 200 mesh. The paving, 1 1/2 or more feet thick, is placed on a shaped bed. According to specifications of Massachusetts Department of Public Works, "stone for slope paving shall consist of field stones, boulders, quarry stone, or rock fragments. The stone shall have at least one reasonably flat face and a thickness perpendicular to the face of not less than 6 inches. At least 75 percent of the stone shall be 2 cubic feet or more in volume." Because rock was available from a nearby subgrade excavation the job shown here cost about $9 per square yard; usual costs are $12 to $15 per square yard. (Photograph by C. R. Tuttle, U. S. Geological Survey)

Timber bulkheads, when constructed with pervious backfill and drain pipes, have been utilized successfully to prevent the sliding of wet soil where the transverse slope is relatively flat. Most such bulkheads have little structural strength, and furnish only slightly increased restraint against sliding. Their success, in many cases, appears to be due to the drainage layer provided at the toe of the saturated slopes.

Buttresses

Buttresses at the foot of active or potential landslides are commonly used for prevention as well as correction. Such buttresses, consisting of either rockfill or earthfill, generally are used in connection with embankment construction, and seldom, if ever, to restrain slopes in excavation. The term buttress, as used here, includes earth or rock dikes installed for either of two purposes: (a) to provide weight at the toe of a landslide, such as "toe support" or "strut" fills; (b) to increase the shear strength of the soil by construction of a dike or buttress of material having substantially higher shear strength than the native soil.

In the typical slump type landslide the ground at the toe usually moves upward, forming a bulge or pushup. By adding

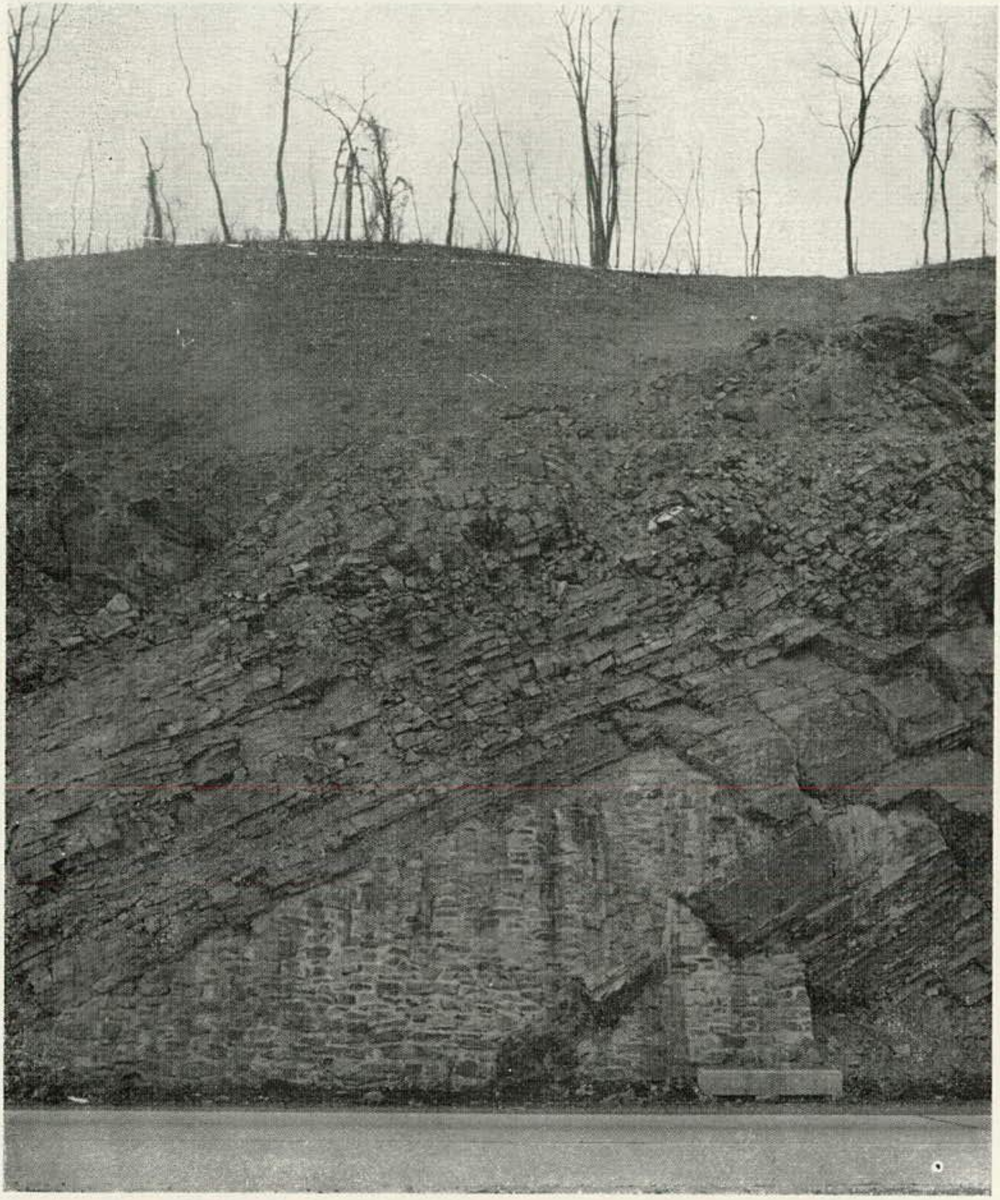


Figure 84. Masonry wall inset beneath overhanging layers of rock. This wall, plus bench on upper part of cut, serves to prevent or minimize rockfall. Note rocks in ditch, however, which represent a small but constant maintenance expense. (Photograph courtesy of Pennsylvania Department of Highways)

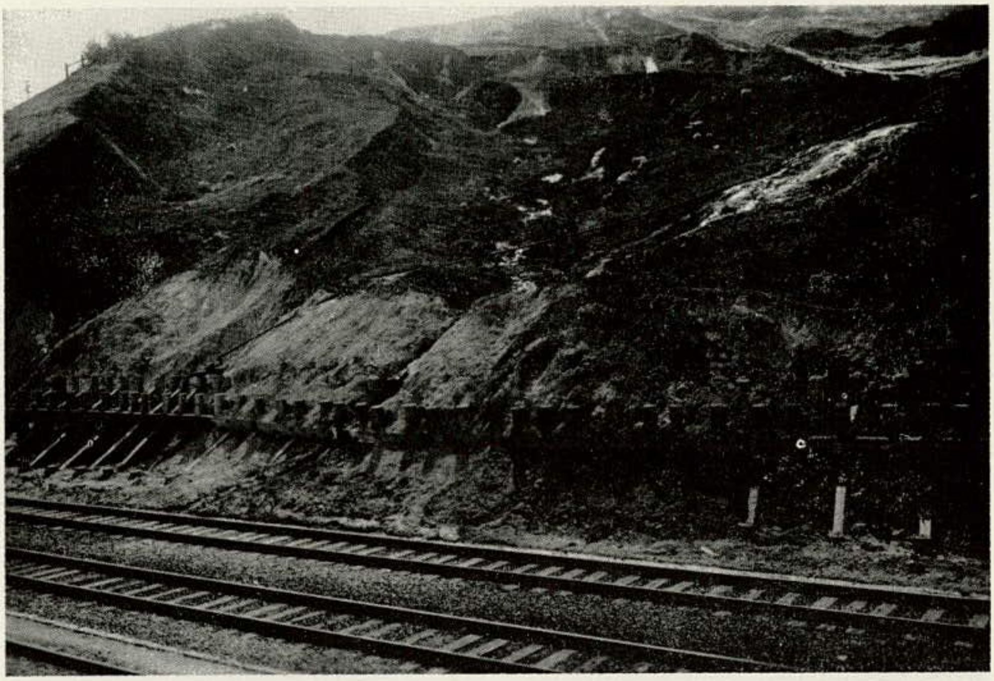


Figure 85. Failure of piles and bulkheads at Valona, Calif. Various types of piles, walls and bulkheads failed to correct this landslide, which was subsequently controlled by extensive subdrainage treatment. State highway crosses the slide near its head, at extreme top of photo. (Photograph by E. W. Herlinger, courtesy of California Division of Highways)

weight in the form of a toe support or strut fill where this upheaval would normally occur, the resistance against sliding is increased. This is one means of improving the stability of an embankment, but the strut fill must be carefully designed in order to utilize the weight most effectively and to assure that the toe support fill will not in itself be unstable. Unless a careful investigation and analysis is made there is always the danger that the additional load imposed by the toe support fill may increase the driving force rather than provide added resistance against sliding. Such fills are safest if the toe fill extends between the embankment and a natural stable bank or hill. Figure 87 illustrates this type of earth buttress. A properly designed toe support fill is more effective than merely flattening the embankment slope, because all of the added weight of the strut fill acts to resist slide movement, whereas part of the weight added in flattening the fill slope contributes to the driving force causing slide movement. Figure 88 shows a toe support embankment used as a slide preventive.

A highly specialized form of toe support to prevent slides is shown by Figure 4. Groins were built out into the water beneath an unstable slope; the groins caused shore currents to build up sand beaches at the base of the slope, thus adding weight and support to the toe. The upper slopes were also treated to retard erosion.

The rock buttress has been used as a slipout prevention measure with considerable success. If the rock buttress extends down to firm material, and is sufficiently massive, the resistance against sliding is appreciably increased by the

high shear resistance of the rock buttress. Many rock buttresses have failed because they did not extend to sufficient depth; as a result, a surface of rupture passed below the bottom of the buttress, which then moved as part of the slide. Assuming that the necessary boring and test data are available, the improvement in stability effected by construction of either type of buttress can be estimated with reasonable accuracy by application of the principles of soil mechanics, as described in Chapter Nine.

A modified application of the buttress principle has been found effective in preventing sloughing or flowing of wet cut slopes. This method, which is really a combination of drainage and buttress, consists of placing over an excavated slope a heavy blanket of clean coarse gravel or similar pervious material. If the cut slope is excavated on a 1:1 slope, for example, the gravel blanket would be placed to a $1\frac{1}{2}$:1($1\frac{1}{2}$ horizontal:1 vertical) or 2:1 slope, thus providing a wedge-shaped buttress of gravel which allows free drainage of seepage from the slope and, at the same time, furnishes

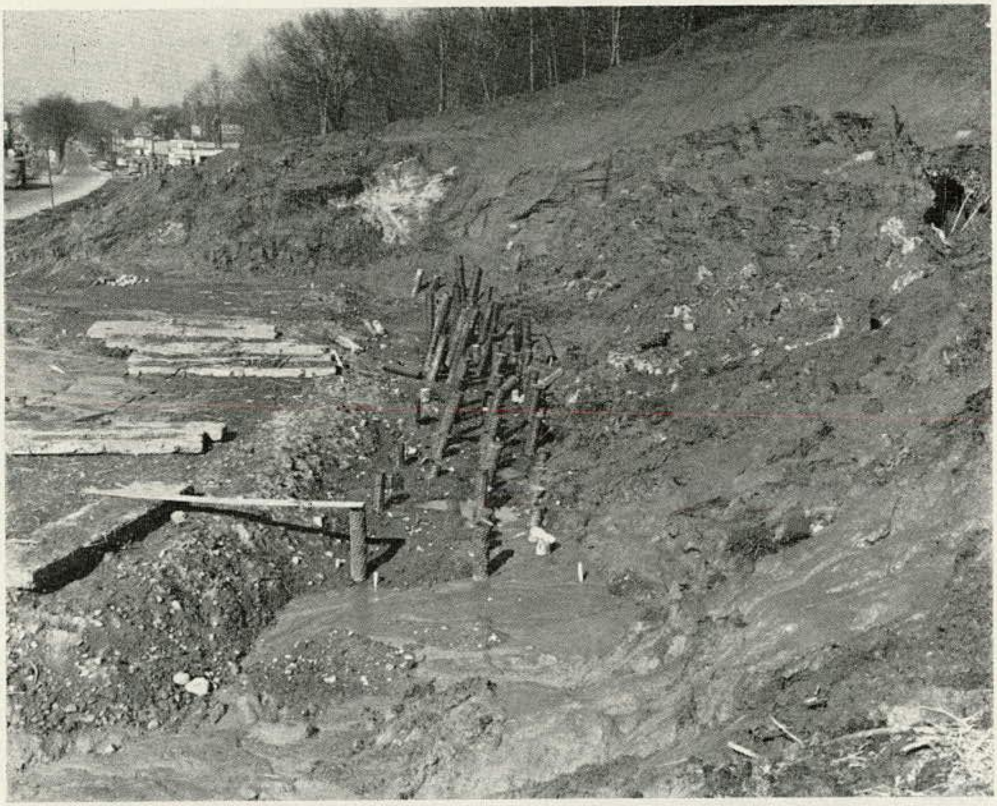


Figure 86. Failed piling in varved clay, 1 mile south of Springfield, Mass. This slump-earthflow, 300 feet wide, 75 feet high, and 60 feet deep, took place in 1954 in varved lake clays that dip 12° to 15° downslope. The summer layers are high in silt, hence provide much water for absorption by the winter layers. Slump was due to high water content, triggered by removal of toe and vibration of construction equipment. Piling having failed, the slide was corrected by a combination of rock buttress, partial removal and drainage at toe, reshaping of slope, and partial removal of head. (Photograph courtesy of Massachusetts Department of Public Works)

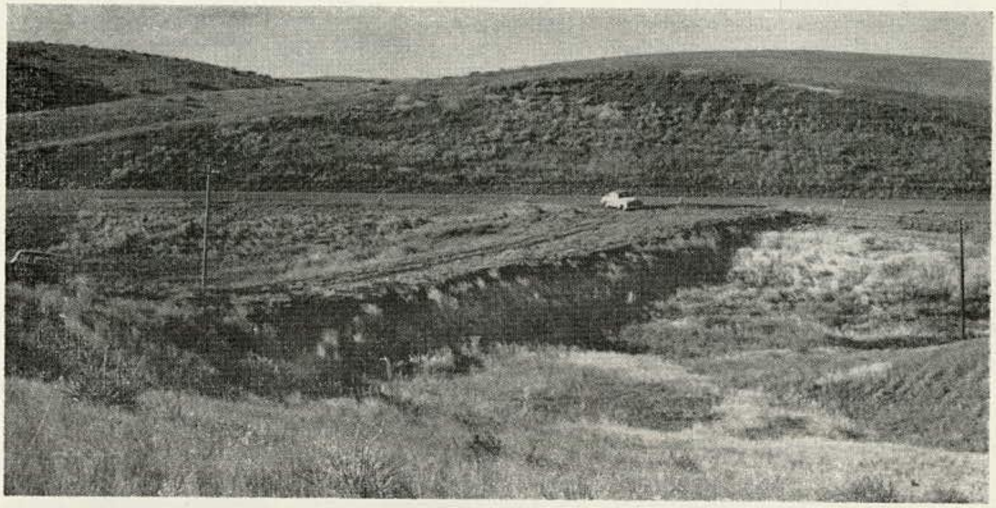


Figure 87. Earth buttress fill to prevent sliding. Earthfill constructed as buttress between highway embankment and opposite stable valley wall; culvert placed in creek channel under buttress. This particular installation (4 miles west of Pierre, S. Dak.) was for slide correction, but similar buttress fills are frequently constructed for prevention of slides. The horizontally bedded soft shale on far hill was moving down slowly and causing displacement of the highway fill toward the observer. (Photograph by D. J. Varnes, U. S. Geological Survey)

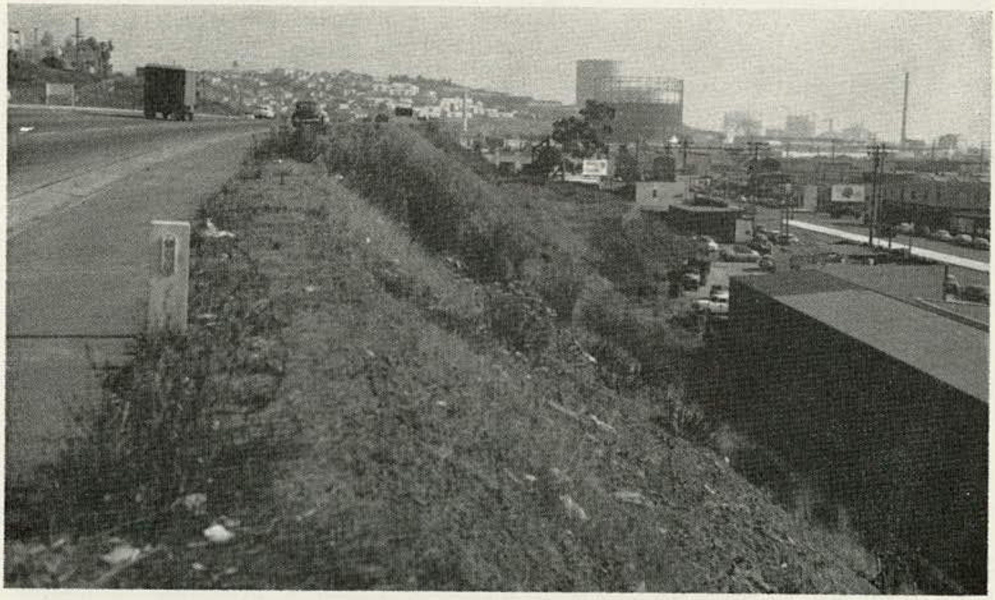


Figure 88. Restraining structure; where toe of fill extended over unstable marine clay, a toe support or earth buttress fill was constructed to prevent slipouts. Buttress fill is near middle of photo at extreme right, where right-of-way fence curves outward. Freeway near San Francisco, Calif. (Photograph by Bruce Utt, courtesy of California Division of Highways)

resistance against sliding. This type of preventive treatment is applicable only to relatively low-cut slopes, and would be economical only in locations where a plentiful supply of cheap gravel is available. The method is illustrated by the cross-section in Figure 89.

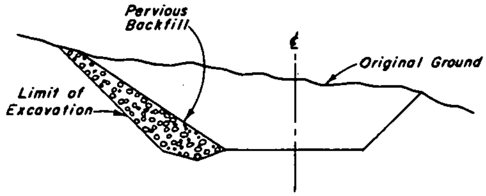


Figure 89. Modified buttress. Pervious blanket of gravel or rock prevents sliding of wet cut slope by providing drainage of slope and also by increasing strength of slope by buttress action. Typical of preventive treatment used near Boonville, Calif. (Courtesy of California Division of Highways)

Piling

Driving of piles to prevent or retard slide movement appears to have great appeal to the layman, and to some engineers, even though the majority of such installations have been far from successful. In general, the pile treatment is more likely to be effective as a preventive measure than in controlling an active landslide. The shearing resistance of a soil mass often can be considerably increased by driving piles, and the resulting increase in shear strength may be sufficient to prevent slide movement. The piles may, however, be ineffective because of (a) movement of soil between and around the piles, (b) overturning of the piles, (c) shear failure of the piles, or (d) development of a surface of rupture in the soil below the pile tips. The reasons for and means of preventing each type of failure are apparent. Too few engineers have a true conception of the magnitude of the forces which may be exerted in a landslide movement, and assume that a few piles will control or prevent sliding. In most cases a thorough analysis during the design stage would indicate such deficiencies in the design of the treatment. Of course, piles should not be driven in soils that become "quick" under vibration.

Dowels

Rockslides are not uncommon in cuts through hard durable rock, if bedding planes or joint planes are prevalent in the rock. Sliding is likely to be especially troublesome if the joint planes or bedding planes slope toward the excavation. If the cut slope is of great height, or if the rock face above the slope line is relatively steep, the sliding is likely to be progressive — movement of one block of rock removes the support from the rock in the face above, and the sliding progresses up the slope. The weakness along the joint planes may be aggravated due to percolation of water and weathering along the contact, and also by freezing in frost areas.

In many cases a small additional restraint will prevent the initial movement, although once a block of rock has started moving restraint would be most difficult. Dowels have been used successfully for preventing this type of slide. In the installation of the dowels, holes are drilled into the rock normal to and across the weak planes, then heavy steel dowels are grouted in the holes. The spacing and length of the dowels would depend, of course, on the degree of jointing and the dip of the bedding planes or joint surfaces. Control of a slide by installation of dowels has been described and illustrated by Laurence (1951). The use of dowels is seldom practicable unless the rock is durable and free from fragmentation.

Rock bolts, a modification of the dowel principle, are now used extensively to prevent movement of rock; these consist of heavy bolts with a wedge or expansion device at the lower end, which are installed in holes drilled in the rock (see Fig. 90). A large washer or plate is provided under the nut at the face of the rock. Because the tightening of the nut actuates the expansion device

Figure 90. Rock anchor bolts used to prevent slippage and fall of bedded rock in a railroad cut in northeastern Pennsylvania. The bolt consists of shank threaded at one end on which a nut and retainer plate are attached. At the end which is embedded in the rock the bolt has a forged slot. A steel wedge is forced into the slot to hold the bolt securely in the drilled hole. In some cases, slippage along very steeply inclined beds, such as those shown in Figure 20, can be prevented by means of rock bolts. (Photograph courtesy of Bethlehem Steel Company)

in the hole, no grouting is required. Such rock bolts, which have been commonly used in tunnel construction and in mines, are now frequently installed in slope faces of rock excavation, to anchor slabs or fragments of jointed rock before any movement occurs. These rock bolts, if properly placed, will often anchor key slabs of rock and thus prevent rockslides or rockfalls which might otherwise develop into large-scale movements.

Tie Rods

One of the causes listed for failure of retaining walls, cribs and piles was overturning or tilting of the retaining structure. Retaining walls frequently must be founded on such weak material that the unit pressure at the toe of the footing exceeds the bearing capacity of the foundation soil. When the restraining structure consists of piles, the material penetrated by the piles may not have sufficient shear strength to prevent tipping of the piles due to lateral thrust of the soil mass. When such conditions prevail, the use of tie rods may provide the required additional resistance against overturning. When tie rods are employed for this purpose they consist of heavy steel rods or wire rope securely fastened to rigid wales, piles, or vertical members of the restraining structure; the tie rods

are anchored to deadmen placed in the most stable accessible material back of the structure, frequently in firm ground on the upper side of the roadway. This type of treatment is best adapted to the prevention of embankment slipouts, as it is seldom feasible to install the tie rods where the restraining structure protects a cut slope. Tie rods have been used to anchor log cribs, pile and plank bulkheads, and similar restraining structures (see Fig. 91).

Miscellaneous Methods

Many other methods of slide prevention have been tried with varying degrees of success. Most of these treatments have limited application and are effective only under certain combinations of conditions. Under "Miscellaneous" in Table 4 are listed a few of these less frequently used methods of slide prevention. Because most of them are still somewhat experimental or have very limited application, they are not discussed here in detail.

Hardening of Slide Mass. — By means of artificial cementation the individual soil grains are cemented together, thus increasing the shear strength of the soil. Cementation may be accomplished by injection of chemicals or by grouting with portland cement.

Most of the chemical injection processes use sodium silicate, in combination with one or more other chemicals, which react with the sodium silicate to form a silica-gel in the interstices of the soil. Several of these injection processes, such as the Joosten and K.L.M. methods, are proprietary. The injection methods are commonly applicable only to sandy soils with an effective grain size of at least 0.1 mm. The treatment has been used successfully to effect temporary stabilization of sands during the construction period in excavation of tunnels and trenches; as a preventive treatment against large-scale slides, it has been used to a very limited extent.

Portland cement grout injections have been used successfully for cementing coarse sands and gravel, but are general-

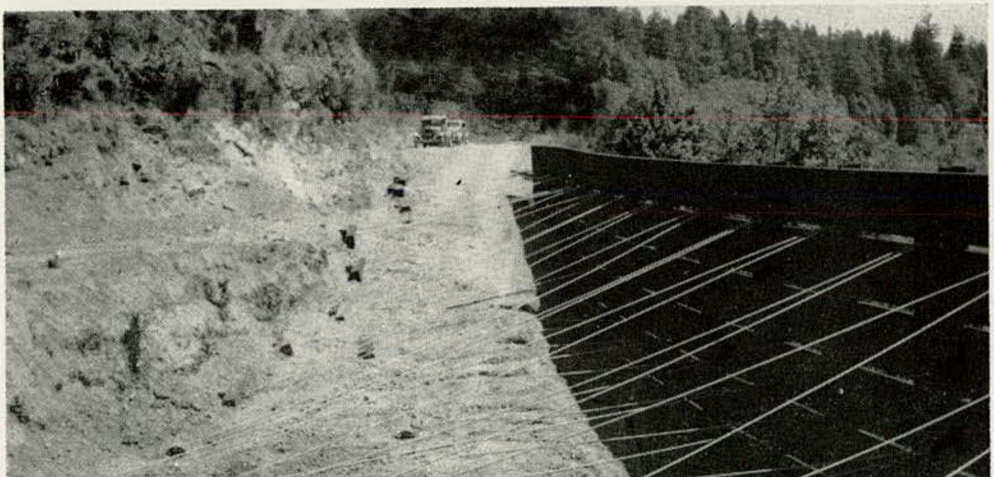


Figure 91. Timber retaining wall with tie-rods. Unstable foundation precluded use of conventional retaining wall. Lateral support provided by steel cables anchored to deadmen in firm material in slope above the wall. Embankment constructed above this wall near Guerneville, Calif., has been stable since construction in 1939. (Courtesy of California Division of Highways)

Figure 92. Fill stabilized with cement grout. Cement grouting was accomplished in 1949 in the Westgate Fill near Westgate, Va., by the Virginian Railroad. A highly micaceous soil was used in the embankment construction and compaction was very difficult. The inclined drain pipes shown at midslope were placed earlier, but were ineffective and were later abandoned. Injections were made along the slope with holes on a 10-foot gridwork. The grout was mixed to a proportion of one part cement to four parts of sand. The section was 540 feet long and the slopes were approximately 65 feet high. The cost of the correction amounted to $8,040 (Smith, 1950). (Photograph by Rockwell Smith, Association of American Railroads)

ly considered less effective with finer grained soils. However, some of the railroads have been applying grout injections as a slide corrective treatment, and report encouraging results even when the soil mass was heterogeneous or comprised largely of clayey materials (see Figs. 92 and 93). The grout was apparently dispersed through seams, cracks and fissures in the fine-grained soil, as well as into the interstices of the coarse sand and gravel. Even though the grout was not uniformly distributed throughout the soil mass, the treatment has, in many cases, effected sufficient increase in shear strength to stabilize the slide mass. The grout injection method might be less effective as a prevention treatment because of the absence of cracks and fissures, most of which tend to develop after movement occurs. Nevertheless, the method may be applicable to a greater variety of soil types than is generally believed.

Bituminous emulsions, having a lower viscosity than cement grout, will penetrate into the pore spaces of finer grained soils than will cement grout. The cost is greater than for cement grout treatment, and the emulsion is not suit-

able for use where ground water might remove the emulsion before setting occurs. There is also some question as to the permanence of the hardening, especially if ground water is present. The asphaltic emulsion treatment has not been used extensively as a landslide preventive treatment.

Freezing of soil to prevent sliding during construction is a unique method that has been used on at least one large project. A description of the operation has been published (Gordon, 1937). The freezing process is slow and relatively costly; obviously, it would be applicable only as a temporary treatment for slide prevention, and is mentioned merely as an example of an unusual means of effecting stability of a slope by an artificial hardening process.

Another method of increasing the shear resistance of a soil mass is by electro-osmosis, in which migration of water out of the soil pores is induced by causing an electric current to flow between electrodes driven into the soil. The moisture content of fine-grained soils can be reduced appreciably by the electro-osmosis process, with a concomitant increase in shear strength. The method has been used successfully on full-scale slope treatments in Europe, but most of the work done thus far in the United States has been of an experi-

Figure 93. Injection of cement grout to stabilize small fill on Southern Railway in northern Kentucky. The method used was similar to that described under Figure 92. (Photograph by Rockwell Smith)

mental nature. The process has been described by Casagrande (1948), Karpoff (1951), and others. Available information indicates that, because a rather strong electric current is required over a considerable period of time, the cost of electric power for this treatment may be prohibitive.

Another process, which is similar to electro-osmosis, is the electro-chemical hardening of clays. If aluminum electrodes are used in the electro-osmosis treatment, there is, in addition to the reduction in moisture content adjacent to the anode, a further hardening of the soil resulting from base exchange — the positive ions of the clay minerals are replaced by aluminum ions from the electrode, with a loss of metal from the electrode. The hardening effected by this process is apparently permanent. As with electro-osmosis, the power requirements are high, making the treatment costly.

Blasting. — Where a relatively shallow mass of cohesive soils is underlain by bedrock or other hard material, the contact between the two is sometimes a smooth sloping surface; such a contact plane is a potential surface of sliding, especially in the presence of subsurface water or if there is a thin layer of plastic material along the contact surface. Blasting is sometimes used to break up such a contact surface, thus providing a mechanical bond between the two surfaces. In effect the shear strength along the weak zone is increased by the shooting and breakup of the hard material. It is probable that this method has been most successful where the hard layer was underlain by a pervious formation, and the blasting provided drainage into the underlying pervious layer. The permanence of the blasting method has frequently been questioned, and there is evidence that in some cases a weak zone later developed along the original contact, due to migration of fine soil and "healing" of the fractured zone. There is, of course, the risk that the blasting, unless carefully handled, may induce slide movement during the construction treatment.

Partial Removal at Toe. — Partial removal at the toe of a landslide has been included under "Miscellaneous Methods" of slide prevention and correction in Table 4, although such excavation usually neither prevents nor corrects a landslide; on the contrary, it more often aggravates the sliding. Excavation at the toe of a landslide is sometimes necessary as an expedient to protect a structure temporarily, until the structure can be relocated or more permanent corrective treatment provided. This type of treatment is usually attempted after occurrence of a landslide, and could seldom be considered as even an attempt at prevention. The fact that a landslide sometimes remains quiescent for a considerable period of time after slide material is excavated from the toe, is merely evidence that the factors which activated the original slide were no longer present after the toe was excavated. For example, a large rockslide, involving more than a quarter of a million cubic yards of slide material, completely blocked a mountain highway a few years ago. Conditions were such that neither complete removal nor large-scale corrective treatment was feasible; sufficient material was removed at the toe of the slide, in this case at roadway grade, to permit opening the road to traffic as quickly as possible. Although no corrective measures were taken there has been no further slide movement. The slide occurred during a mild earthquake in the region, and it is probable that the slide will remain quiescent until there is another earthquake or some other changed condition reactivates the slide.

Conclusion

The number and variety of slide prevention methods discussed in the foregoing are evidence that there can be no rule-of-thumb system of prescribing treatment; and for a particular landslide or potential landslide there is seldom one and only one "correct" method of treatment. Frequently, the most economical

effective means of prevention consists of a combination of two or more of the general preventive measures described in this chapter.

For most landslides, a majority of the possible preventive treatments can be eliminated at the outset, and only a few of the many methods need be considered. Frequently, the final selection can be made only after careful comparison of two or more alternate methods. But in spite of the complexity of landslides and the wide variety of control methods, the problem of landslide prevention and correction is amenable to a rational engineering approach, by proper utilization of available knowledge on the classification, recognition, and analysis of landslides.

References

Baker, R. F., "Determining the Corrective Action for Highway Landslide Problems." Highway Research Board Bull. 49, p. 1-27, 1952.

Casagrande, L., "Electro-Osmosis." Proc. 2nd Int. Conf. on Soil Mechanics and Foundation Eng., v. 1, p. 218-223, 1948.

Gordon, Grant, "Freezing Arch Across Toe of East Forebay Slide, Grand Coulee Dam." *Reclamation Era*, v. 27, p. 12-16, 1937.

Hill, R. A., "Clay Stratum Dried Out to Prevent Landslips." *Civil Eng.*, v. 4, p. 403, 407, 1934.

Karpoff, P., "Laboratory Investigations of Electrical Drainage and Electrochemical Stabilization of Soils." Structural Research Sec. Rept., No. SP-29, U. S. Bureau of Reclamation, 1951.

Ladd, G. E., "Methods of Controlling Landslides." *Engineering and Contracting*, v. 67, p. 599-608, 1928.

Laurence, R. A., "Stabilization of Some Rockslides in Grainger County, Tennessee." *Econ. Geol.*, v. 46, p. 329-336, 1951.

Mineral Information Service, "Landslides in Ventura Avenue Oil Field." Mineral Information Service, Calif. Div. of Mines, v. 7, no. 5, p. 1, 1954.

Anon., "Curing Slides with Drainage Tunnels." *Roads and Streets*, v. 90, no. 4, p. 72, 74, 76, Apr. 1947.

Root, A. W., "Problem of Slipouts Studied by State Highway Engineers." *California Highways and Public Works*, Mar. 1938.

Stanton, T. E., "Stabilizing Earth Slopes Through Installation of Horizontal Drains." *California Highways and Public Works*, Jan.-Feb. 1948.

Chapter Eight

Control and Correction

Robert F. Baker and Harry C. Marshall

In Chapter Seven the concept of "prevention" as opposed to "correction" is developed on the basis of whether the engineering problem is in the design or the maintenance stage at the time the engineer is confronted with its study. Such a distinction is warranted, because many aspects of the analysis will depend on whether an engineering structure is already built, and must therefore be protected or repaired, or whether it is under consideration for construction, in which case future damage is the chief concern.

For both prevention and correction of landslide problems, it is well to remember that the word "problem" implies that a quantitative approach is desirable. Perhaps less obvious is the fact that an engineering problem is involved rather than an academic one. In many cases, the collection of all the data needed for a complete understanding of the movement is not justified. That is, the prevention and correction as defined herein *are related to the solution of an engineering problem — not necessarily the prevention or control of a landslide* — and a complete understanding of the slide may not be essential.

Control and correction of landslides naturally have many elements in common with prevention, particularly because both preventive and corrective methods to be used depend on differences in the same factors. Some of these factors are geology, topography, policy of the agency, experience of the investigator, type of structure involved and problems of legal liability. Prevention is more difficult than correction in respect to the identification of a landslide and estimation of its probable severity, as well as to the psychological difficulty of predicting troubles to an economy-minded agency or client. On the other hand, correction is rendered difficult because of the myriad of methods that have been used with varying success and of the fact that the agency or client may not be able to afford an expensive correction, even though some treatment is necessary. For problems in the correction stage, the limits and extent of the slide are generally well defined, and the seriousness of the problem can be assessed. Certainly the quantitative approach is more reliable when the results of the movement are already available for study. On the other hand, lack of funds and of time for study are more often controlling factors in the solution of corrective problems than they are for preventive ones.

Recognition of actual or potential slides is one of the most important and critical factors in the solution of problems by preventive methods. In correction, the nature and amount of movement are generally obvious, but even here the ability to predict and evaluate potential movement plays an important part because the stability of the area above, below, and on each side of the slide is of utmost significance. That is, it is essential to know whether the corrective measure proposed will disturb or improve the condition of equilibrium of the surrounding areas. In many cases, indeed, this factor is a controlling one in the economics concerned with the selection of the proper solution.

Previous Studies of Corrective Treatments

Much has been written on the subject of landslides, and the literature contains many descriptions of case histories throughout the world. Tompkin and Britt (1952) list 102 papers on specific landslides and 165 additional references on the subject of mass movement. Perhaps because of the inherent complexities of landslides, however, very few generalizations have been made with regard to their correction. More often than not the opinion that "landslides are individuals" has been expressed; by inference or direct statement the thought is commonly included that generalizations are not practicable. During the past few years, however, more and more study has been made of the mechanics of landslides, with a corresponding increase in knowledge of the general approach toward quantitative answers.

One of the first articles that dealt extensively with many types of corrective measures was written by Ladd in 1928. In this article and in one in 1935, Ladd discusses many of the techniques that had been used in landslide correction up to the time of his papers, with comments on the feasibility and conditions for usage. His approach is entirely empirical, with strong emphasis on treating the "cause" of the slide. Several papers by Terzaghi also deal with techniques of correcting landslides (Terzaghi, 1939, 1950; Terzaghi and Peck, 1948). No specific effort is made by Terzaghi to discuss all of the many available methods of treatment, as he concentrates primarily on the mechanics of landslides; he suggests that the principal cause of many slope failures is hydrostatic pressure (Terzaghi, 1950). Hennes (1936) considers the theoretical application and practical use of various corrective measures, and proposes a quantitative approach for determining pile spacing. He also deals at some length with proper applications of drainage techniques. California experiences with various methods of correction have been discussed by Root (1955) and others. Baker (1952) suggests a theoretical approach to the design of corrective measures based on the mechanics of movements and the importance of economic factors. Theoretical analyses of specific slides have been discussed in recent years by Krynine (1930), Palmer (1950), Larew (1952), Ireland (1953), Berger (1955), and others.

Available Methods for Control and Correction

The most commonly used methods of prevention and correction of landslides are listed in Table 4, Chapter Seven; their special applications to problems of correction are described in the latter part of this chapter. The methods are divided into five principal groups — avoidance, excavation, drainage, restraining structures, and miscellaneous — each of which has a somewhat characteristic effect on the stability of a landslide.

Avoidance techniques, as the name implies, solve the problem by completely avoiding the moving mass. Excavation treatments rely primarily on the removal of a sufficient quantity of the moving mass to reduce the motivating force, thus eliminating or ameliorating the problem. Drainage methods depend on the removal of water from the mass or on the interception of water before it enters the moving mass. Restraining structures either impose resistance in the path of the moving mass, or underpin the endangered structure. Miscellaneous methods may use any of several means for controlling the movement.

Not all of the methods that can be or have been used are listed in Table 4, or are described here. Moreover, many combinations of those methods listed have been successfully employed.

Investigations Needed

Early in the investigation a decision should be made as to whether an attempt will be made to control the move-

ment. There are numerous corrective measures which do not involve halting the movement of the landslide. Relocation of a highway is a typical example of a corrective treatment that solves the problem without necessitating control of the slide itself. It may be difficult or undesirable to be committed to such a decision early in the investigation, although time limitations may make an early decision essential. The advantages of an early decision against control of the slide lie chiefly in the savings that can be realized in the investigation. It is not unusual for a landslide investigation to cost as much or more than the correction. This expense usually can be justified on the basis of future maintenance savings or of a better position from a legal standpoint. It is important, however, to consider whether any results that could be developed by a thorough study would result in a more certain design or a less expensive correction than could be accomplished with little or no investigation.

There is no set rule as to the amount of data that should be obtained for a given landslide problem. There are, however, certain data that are required for each method of correction. The kind and amount of required detail are related also to the size and seriousness of the problem. For very small slides, the quantity of moving material may be the only fact required. For very large landslides that involve considerable expense for correction and where there is no particular urgency, complete geologic and soil analyses may be both desirable and feasible, if not, indeed, essential.

Some corrective measures are particularly suited to a mathematical approach for a quantitative design of the correction. The general principles involved and the application to various corrective measures are discussed in full in Chapter Nine. Almost without exception, complete data will be necessary if a reliable *quantitative* approach toward control of the movement is desired.

Factors in Selection of Corrective Measures

Many factors must be considered in a landslide analysis. Just as solution of any maintenance problem involves some principles that differ from those to be applied to original construction design, so are some factors peculiar to corrective treatments as distinct from preventive ones. Some of the major factors that enter into the selection of a corrective measure are described in the paragraphs that immediately follow. Some of these have to do with the geology of the slide; others are more closely related to engineering or economics. Taken together, they should serve to re-emphasize the fact that proper selection and design of a corrective or control treatment can only be based on thorough knowledge of the basic facts.

Landslide Types

The three basic types of landslide movement, as shown in the classification chart (Pl. 1), are falls, slides and flows. This classification was developed in part through recognition of the fact that three fundamentally different principles govern the movement of the various types of landslides. Falls (Figs. 41 and 94) are influenced by the laws of free-falling bodies and by the chemical and mechanical disintegration of rock; slides (Figs. 16 and 95) are failures in elastic or semi-elastic materials; and flows (Fig. 30) follow the principles of plastic flow of fluid and semi-fluid materials. This means that certain generalizations can be drawn as to the type of corrective measures to be employed for the various groups. In Table 4, Chapter Seven, the relative frequencies in application of various corrective measures to the several types of landslides are given. Detailed descriptions of the individual methods are given in succeeding pages, and the following represent general comments only.

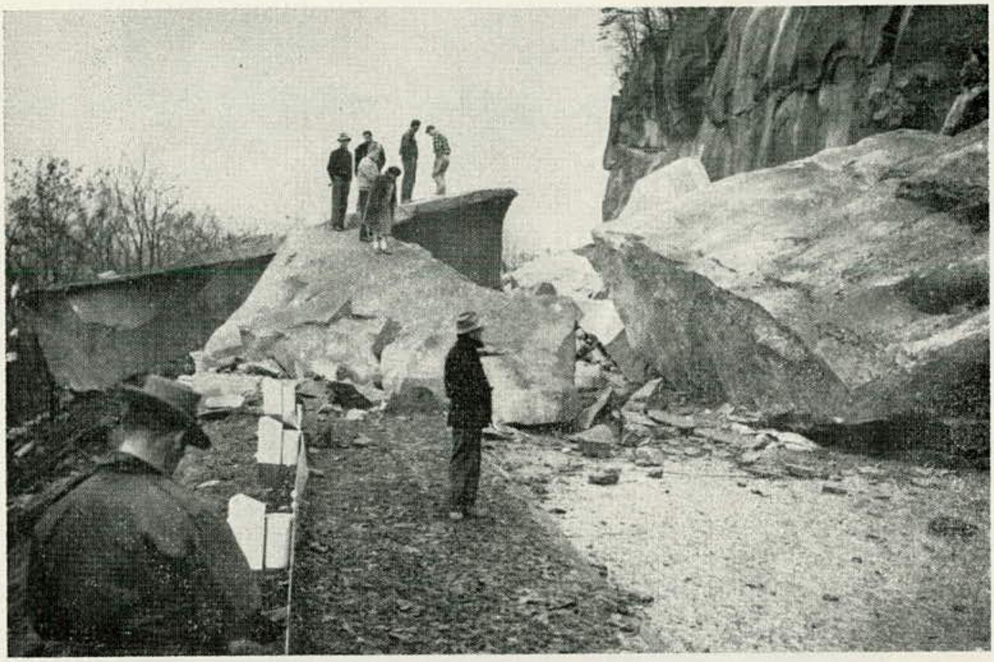


Figure 94. Rockfall on State Route 7 southwest of Marietta, Ohio. This rockfall in jointed massive sandstone of Permian age occurred after a period of intensive rainfall. It is one of several large falls that have occurred on this route since 1940. The fallen pieces came from a massive bed, the base of which lies on shale which makes up the lower 15 feet of the cut slope. (Photograph courtesy of Engineering Experiment Station News, **Ohio State University, April 1950)**

Falls

Most falls are corrected by one or a combination of the following methods: relocation, flattening the slopes, benching the slopes, and surface drainage. Other methods are also applicable to some falls. Until recent years the low cost of labor for wall construction permitted the use of bricks, stone, and thin concrete walls to insulate weak bedrock formation from the detrimental effects of weathering. Figures 96 and 97 illustrate methods for reducing damage from rockfall by protection with steel mesh and with a wire fence and concrete wall; Figure 90 shows an application of anchor bolts to this problem. Perhaps the most frequently used method for controlling rockfall is through excavation.

It will be noted that relatively few methods of correction are frequently applied to the solution of falls. It is also evident that solutions such as retaining devices and subsurface drainage are seldom as applicable to falls as they are to slides and flows. Fortunately, falls are readily differentiated from the other types of movement and the problem of preventing or adjusting for weathering failures can be recognized and treated accordingly.

Slides

The following types of corrective measures are recommended for use in the correction of slides:

1. Relocation.
2. Excavation by removal of the head, flattening of slopes, benching of slopes, or complete removal.
3. Surface drainage by open ditches, regrading the surface, or sealing of surface cracks.

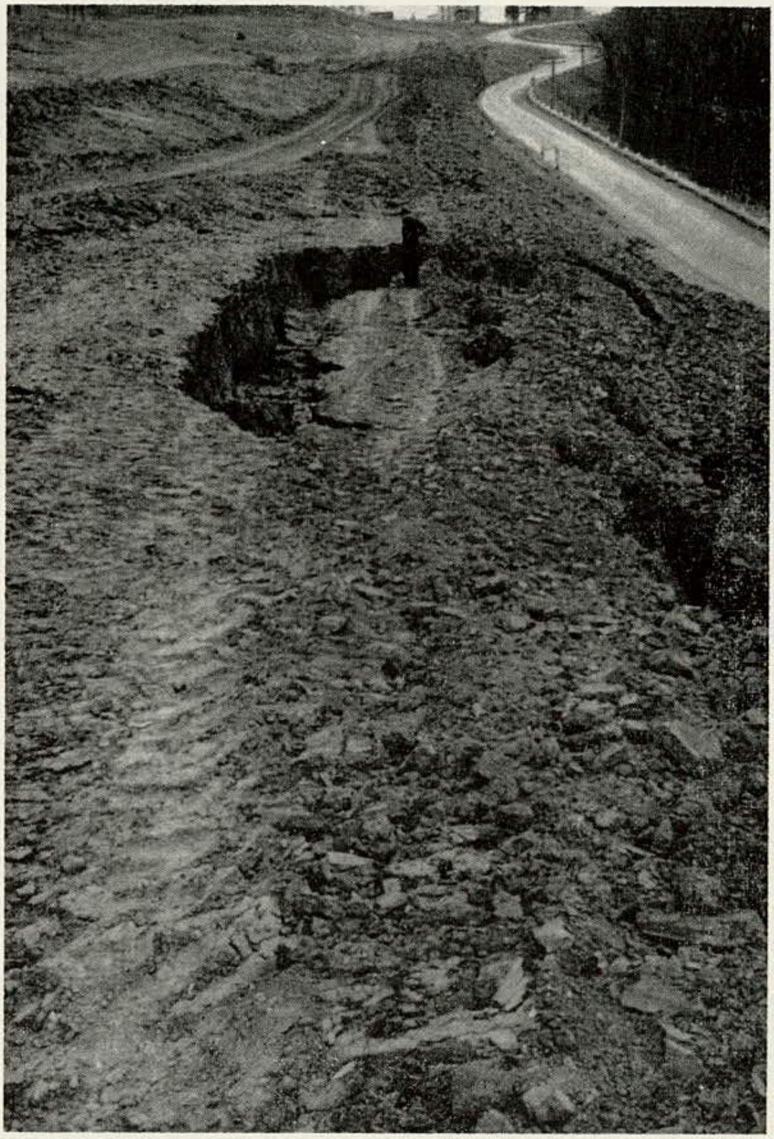


Figure 95. Landslide in newly constructed sidehill fill, U. S. Route 40, Gurnsey County, Ohio. This slide was eliminated by total excavation of the moving fill and underlying soil down to a level bench in the bedrock. After obtaining a stable foundation on bedrock and providing a drainage course up the excavated backslope, the fill was reconstructed and has shown no further movement. (Photograph courtesy of Ohio Department of Highways)

4. Subdrainage by horizontal drainage, drainage trenches, or tunnels.
5. Rock and earth buttresses at the foot.
6. Cribs and retaining walls.
7. Piling.
8. Blasting.

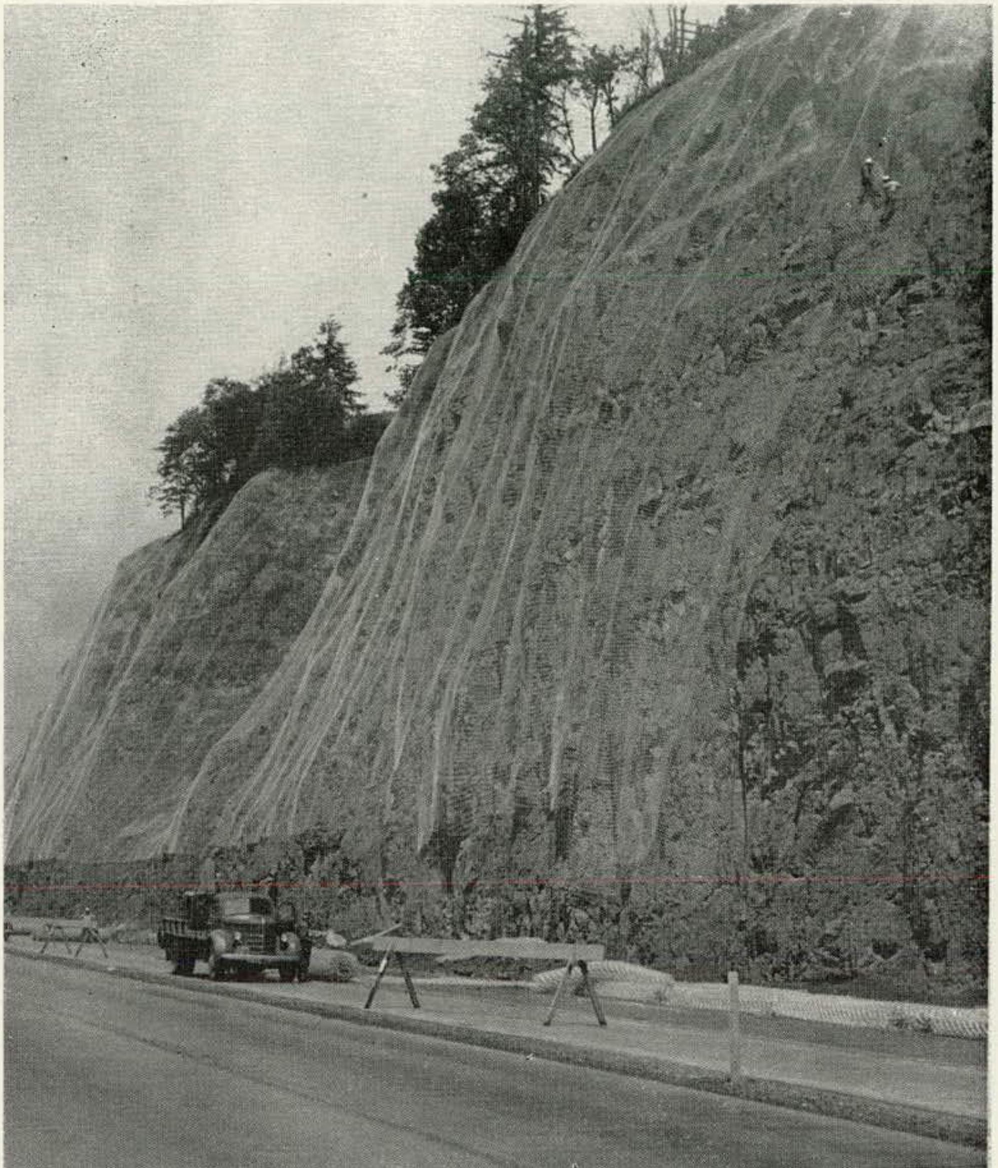


Figure 96. Wire mesh used to control rockfall near Kelso, Wash. The mesh is made of No. 9 wire and is intended to prevent rock from this 170-foot-high slope from plunging onto the highway. The rocks continue to come down, but are held against the slope and drop harmlessly into the ditch (Day, 1953). (Photograph courtesy of American Hoist and Derrick Company, St. Paul, Minn.)

All of these methods, except relocation, can be used to stop mass movement. Buttresses, cribs, retaining walls, piling and blasting methods should, however, be limited to control of small-scale slides, because they are seldom

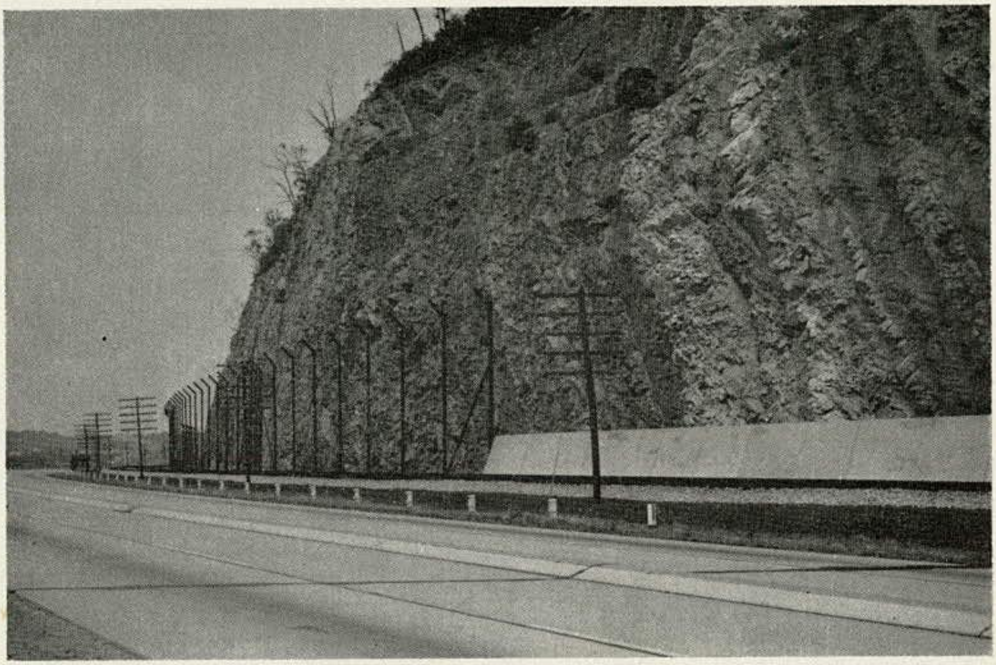


Figure 97. Wire fence and concrete wall used to protect railroad and highway from rockfall debris. Also note multiple benches for debris catchment on upper part of cut near Harrisburg, Pa. The fence is equipped with electric warning devices to warn trains of danger. (Photograph by Pennsylvania Department of Highways)

completely effective on larger ones. For those situations where there is danger to the structure through undermining, cribs, retaining walls, and piling are commonly used to provide underpinning for the foundations of the structure without regard to the size of the landslide.

Many factors in addition to the type of movement will influence the selection of the type of treatment, as is obvious from the long list of the possible methods of correction. Guides to the decision as to the technique to be used are related to experience, economics, characteristics of the treatment, etc. It is important to note, however, that there is a greater variety of potentially good methods for correction of slides than for correction of either falls or flows.

It is conceivable that the misuse of certain corrective measures is related to the failure to differentiate between slides and flows. In slides, there is significant amount of resistance to movement because of the inherent shearing resistance of the soil. This natural resistance can be improved or increased by one of several techniques. Also, the motivating forces that cause movement can be more effectively reduced in slides. This is due to the fact that curved surfaces of rupture are more common in slides than in flows; this factor permits the removal of greater bulks of the concentrated mass that is producing the force. Neither of these two advantages are commonly present in flows, for there the shearing resistance is frequently negligible and the surface of rupture is rarely curved. Because there is little inherent shearing resistance in most flows, retaining devices placed in their paths are quite likely to receive the entire force of the moving mass.

In summary, in choosing the types of corrective measures to be used in a given slide problem, consideration

should be given to the available resistance within the mass, as well as to the possibilities of effectively reducing the motivating force.

Flows

The following techniques are used most frequently for control of flow movements:

1. Relocation.
2. Excavation by flattening the slopes, benching the slopes, and complete removal.
3. Surface drainage by open ditches, regrading surface, and sealing surface cracks.
4. Subdrainage by horizontal drains and by trenches.

As is the case with slides, when the structure foundations are threatened by a shallow flow, cribs, retaining walls, and to a lesser extent piling, can be used as underpinning near the head of the flow.

Restraining structures that are designed to control the landslide are rarely to be recommended for flows. In some special situations involving small quantities of moving material, however, such structures may prove effective.

Bridging is used more frequently for flows than for either slides or falls. This is primarily because bridges are most economical on long narrow slide movements, such as characterize many flows.

Drainage, removal, and avoidance are the methods most likely to be effective for flow conditions. Drainage, in particular, should be considered. Since the moving mass is a plastic, semi-fluid, or fluid material, the removal of water should add immeasurably to the resistance to movement. Few flows originate due to hydrostatic pressures, but many slides produced by hydrostatic force develop later into flows.

Blasting is not recommended for flows except in cases where the movement is extremely slow, or less than one foot per year. Because flows do not possess significant shearing resistance, the better drainage produced by blasting would represent the entire benefit.

The conversion of slides into flows probably occurs more frequently than is generally recognized. Many clayey soils are quite "sensitive," in that a slight change in soil particle arrangement will produce a tremendous difference in the shearing resistance of the material. In such instances, correction of the problem may involve a completely different analysis and solution than would prevention. In the case of correction, a flow would be involved, whereas in the design or preventive stage the elimination of a shear failure or plastic flow could be considered.

Contributing Factors That Cause Landslides

Chapter Three summarizes the numerous contributing factors that, acting separately or together, can cause landslides, and makes clear that there is rarely a single "cause" for any given landslide.

There should be no disputing the value of determining the "principal cause" or major contributing factor, particularly for solution of legal problems. However, emphasis on search for the "cause" of movement has too frequently led to the conclusion that the movement can only be controlled by treatment of that cause. Actually, recognition of the fact that there are commonly multiple contributing factors makes it obvious that there also can be more than one solution to a given slide problem, and that treatment of any one of the causative factors will lead to a more stable condition.

The three principal reasons for determining the contributing factors that cause a landslide are as follows:

1. To aid in the determination of the most economical correction.
2. To help in the analysis of legal liability.
3. To provide guides for the preven-

tion of similar landslides in the future.

Recognition of the causes can prevent misuse of a corrective measure, but unfortunately this recognition alone cannot produce a quantitative answer.

The most serious result of failure to recognize a contributing factor that has helped cause a slide lies in the consequent inability to analyze the effect of that factor on a proposed corrective treatment. If the factor is ignored and is in reality a major or controlling influence, only temporary stability may result. This will be true regardless of whether a mathematical or empirical approach is applied to the problem. Even if avoidance methods are selected, or if the chosen treatment does not involve control of the movement, failure to recognize all of the contributive causes may mean that the correction applied may not have been the most economical solution to the problem.

In many cases water is recognized as the most important single cause of movement, and the conclusion is reached that drainage is the only answer. But what of the other contributing factors? Certainly the reduction in the effect of any single cause will produce a more stable condition.

In summary, it can be repeated here, as elsewhere throughout this volume, that the better the understanding of the history of a landslide — that is, of all the factors that caused the movement — the better and more certain will be the corrective treatment that is finally adopted.

Motivating and Resisting Forces

The control of the movement of slides and flows involves application of one or both of two basic principles: (a) reduction of the motivating force, and (b) increase of the resisting force.

The motivating force is the weight of the mass; more specifically, it is that component of the weight that parallels the surface of rupture, or slip-surface. If, as is usually the case, the slip-surface is curved, the bulk of the motivating force will come from the area that overlies the steepest portion of the slip-surface. It is important to note that there are no forces other than gravity that tend to cause movement except hydrostatic forces under certain conditions and the rather infrequent instances that involve vibration. Reduction of motivating force thus requires removal of material, and selection of the proper area from which it can be moved most advantageously.

Increase in the resisting force can be accomplished by means of retaining devices, by drainage methods, or by techniques that increase the internal shearing resistance of the moving mass itself. Actually, the principal effect of drainage may well lie in the increase of shearing resistance rather than in the nominal decrease in weight, hence of motivating force. Most of the shearing resistance within a soil or rock mass is attributable to frictional resistance and cohesion; quantitatively it depends on the component of the weight of the mass that is perpendicular to the surface of rupture. Frictional resistance is low in clayey soils, but high in other soils and in rock. Cohesion, of course, is not a factor in noncohesive granular materials, but is in clayey soils and in rocks.

Permanence of Corrective Measures

In terms of geologic time, there can be no permanent correction of a landslide — man can provide only a delaying action in the natural processes that tend to level the earth's surface. If the right conditions prevail, however, he can make an appreciable permanent change in the rate of downslope movement; even then the normal processes of erosion will continue to remove material from high places and deposit it in the low ones. From the standpoint of the engineer, however, permanence can be assessed in terms of human time; in this, the design life of the structure to be repaired or corrected is the most significant value

in estimating the permanence of a corrective measure.

Inasmuch as nature may be placing the landslide under a constantly changing set of conditions, it is difficult to determine the degree of stability unless movement has already occurred. For corrective measures, this factor is available, whereas for prevention the degree of stability may be much less evident. Even if failure has already taken place, there is no assurance that more severe conditions will not develop within the landslide after the correction has been made. Therefore, consideration must be given to the variable factors, particularly seepage and hydrostatic pressures, which will tend to change during the lifetime of the structure.

To estimate the degree of permanence, a quantitative approach is essential. Without it, the investigator must rely on experience — and no real assurance of stability can be based solely on experience without excessive over-design. As explained more fully in Chapter Nine, the degree of permanence can be expressed as a "safety factor." Naturally, the higher the safety factor the more confidence can the investigator place in continued stability of the ground he is investigating. Lack of funds may tend to prevent achievement of high safety factors in the solution of many slide problems, hence a safety factor of 1.5 or more must be considered as an indication of relative stability. Factors of 1.0 to 1.25, on the other hand, indicate that the corrective method applied is only an expedient and that maintenance or repair can be expected in the future.

Relative Position of the Structure

The position of the structure on the landslide is an important consideration, particularly for slides. Three relative locations can be considered — near the head, near the middle, or near the toe. The value of the properties above and below the landslide may be critical factors, as legal actions may result from further landslide movement.

The following is a brief summary of the effect of the relative position of the structure on the economics of the various corrective treatments:

1. Relocation — generally unaffected; if the structure is near the middle of a small slide, relocation may not be economical.
2. Removal of head — not normally practical when the structure is near the head of the slide.
3. Flattening slopes — most frequently applicable when the structure is at the toe of the slide.
4. Complete removal — in itself, only applicable when the structure is near the toe of the slide.
5. Lower grade line — only applicable when the structure is near the head of the slide.
6. Surface drainage — rarely the only corrective measure needed; always desirable in combination with other methods.
7. Horizontal drainage — applicable regardless of the position of the structure.
8. Drainage trenches — not materially affected by the position of the structure.
9. Buttresses — not materially affected by the position of the structure.
10. Cribs, retaining walls, and piling (fixed) — especially useful as underpinning when the structure is at the head of the slide and the depth of movement is shallow; otherwise not materially affected by the position of the structure.
11. Piling (not fixed) — recommended only for small, shallow slides and for structures near the head.
12. Blasting — applicable only when the structure is near the head of the slide.

Economics

Economy of time and money frequently exercises a controlling influence in the analysis of a corrective problem and in choice of treatment method. This does

not imply that incomplete or inadequate engineering and geologic studies are desirable, but rather that the economics of the situation must always play a controlling part in the investigation. After all, it is the engineer's job to accomplish any task with maximum economy and within the time and emergency conditions that exist.

If more than one method of correction is applicable to a given slide, which one should be used? The answer is, simply, the one which is most economical. There are many facets to this phase — property liability, danger to life, maintenance costs, design life — and all of the many variables in the cost picture are involved. No exact answer is possible, but none of the economic factors should be excluded from the thinking of the investigator. In some instances, agency policy may specifically define some items in the long-range economy; in others, only rough approximations may be possible. The final choice between several methods may not belong to the technical investigator, but may rest with a policy maker; he must be given a complete picture of the alternatives.

The problem of economy is a constant one in the highway engineering field, as it is in most other engineering areas. In many cases, the cost of maintaining a road affected by a landslide is less than that of a corrective treatment. Decisions on such situations may require assumptions that loss of life is not a real threat, and that driver comfort has no financial value. However, many legitimate instances will occur in which maintenance is a better solution than an inexpensive corrective measure which will not eliminate future expenditures or hazards in a permanent way.

Methods for Control and Correction

In the following pages, there are detailed descriptions of the various methods of control and correction. The generalizations given herein should be carefully evaluated for any specific problem and area with which the engineer is concerned. Blind use of the information could lead to disastrous failures.

Avoidance Methods

Relocation and bridging are the principal avoidance methods in common use. Both these techniques avoid the landslide, but in so doing they do not in themselves influence the stability of the area. In fact, if stabilization is an essential part of the problem, some additional measure will have to be combined with avoidance techniques.

The use of relocation and bridging methods is discussed in Chapter Eight. Both methods are probably more generally applicable to treatment of potential landslides than of active ones, but they can also be applied to many correction problems. They should, indeed, always be considered in the analysis of such a problem and compared with other available techniques. In many cases, particularly in mountainous terrain, the dangerous area can be avoided with a minimum of cost and an improvement in alignment. In other cases, of course, the avoidance technique may be too expensive, or alternate grade and line may be undesirable. Again, the policies of the organization concerned may place limitations on the use of avoidance methods.

The major advantage in the use of avoidance techniques is the assurance of stability. There is no other method, except complete removal, which will be as certain to correct the problem permanently. Another advantage, on occasion, is the fact that the alignment can be improved.

One disadvantage lies in the physical difficulties that are often produced by a location change or by construction of a bridge. In many cases avoidance methods will represent the most costly correction under consideration. On highways, there may also be instances where a satisfactory relocation would produce very undesirable alignment. Another factor that must be considered is that of legal liability; avoidance methods do not control the land movement, and if

further liability may develop, a stability-producing measure may be necessary simply to avoid the danger of future lawsuits.

Relocation

In comparing a proposed relocation with other possible corrective measures, it is well to keep in mind that a relocation to a firm foundation may well offer the most certain solution to the problem. This is particularly important if the engineer's recommendations must be evaluated by nontechnical supervisors or clients.

The avoidance of a landslide by relocation of the highway or other structure will be most useful in areas where firm bedrock is exposed. If the structure is near or at the head of the slide, and if bedrock is exposed or near the surface on the uphill side of the structure, immediate consideration should be given to the feasibility and economics of a relocation. In an area near Huntington, W. Va., for example, relocation was considered to be the most feasible correction, even though this would have meant excavation of the new roadway in solid rock.

If the structure is near the toe of a slide, it may be feasible to relocate it farther downhill, or below the slide. If this solution is adopted, however, one must be certain that there is no chance that renewed movement of the landslide will endanger or destroy the relocated structure.

Except for very small slides, where relocation may represent, percentage-wise, a very great cost, the magnitude of the landslide movement does not particularly affect the decision as to the use of this method. All corrections for large-scale land movements are expensive, and a relocation may very well qualify as the most economical method.

In addition to consideration of relative costs, any recommendation for a relocation must take certain other definite requirements into account. That is, the relocation must be satisfactory from a utility standpoint. For instance, the new grade line, drainage and other features of a highway must be acceptable. Again, it must be known that the ground above or below the proposed relocation will be stable in the future, hence that no new landslide problem will be precipitated. Finally, every consideration must be given to any legal or other reasons that dictate control or elimination of the landslide rather than mere avoidance of the problem.

Bridging

Bridging a landslide consists of spanning the moving mass with a structure; it is rarely practical because of the normally high costs of bridges. The permanency of the measure, as well as the opportunity that it offers to retain desired grades and alignments, renders the technique useful on occasion. Use of the technique is virtually restricted to land movements on steep slopes, and to those areas where relocation is neither feasible nor desirable (Figs. 63 and 64). For slopes flatter than 2:1, other techniques are commonly cheaper and more feasible than is bridging. Bridges are commonly applicable only to small landslides, or at least to those that are long and narrow, and perpendicular to the direction of the bridge. For slides that require bridge lengths greater than 100 to 300 ft it is doubtful if the method will compare economically with other possible corrections.

The feasibility of building a bridge to span a slide may also be affected by the depth and quantity of the moving mass itself. This is true if the length of the landslide, parallel to bridge centerline, is sufficiently great to require one or more center piers. Piers can, of course, be placed within the moving mass, but only if the overburden is shallow (less than 10 ft) and if the moving material cannot produce excessive lateral thrust against the piers.

Particular care must be taken to avoid placing the bridge abutments on material that may subsequently give way due to

undermining caused by further movement of the landslide, by weathering of exposed bedrock, or by stream erosion or other causes.

EXCAVATION METHODS

Excavation methods are designed to increase the stability of the landslide mass by reducing the forces that cause movement. As shown in Table 4, Chapter Seven, the chief excavation methods used for prevention or correction of landslides are (a) removal of head, (b) flattening of slopes, (c) benching of slopes, and (d) complete removal of all unstable material. A fifth possible method, that of lowering the grade line, can be considered as a subtype of the head-removal method, although it may amount to relocation in some instances.

Adoption of an excavation method can and should result in a relatively permanent solution to a given landslide problem. This permanency will only be attained, however, if the investigator has evaluated the probable improvement in stability accurately and if the excavation is properly designed and carried out. Generally speaking, excavation methods are more applicable to prevention of slides than to correction, because unit costs for the relatively large amounts of earthwork required are generally lower on new construction projects than they are on repair jobs. Some landslides, however, threaten existing structures and require excavation or removal. In such cases, removal of toe material to protect the structure may require removal of additional mass higher on the slide in order to reduce the stresses.

Excavation of part of a landslide may also permit better use of land, for landscaping or other purposes, that was hitherto worthless because of the hummocky and swampy character of the landslide body. Properly done, excavation should also lead to improvement in surface drainage. Moreover, subsurface drainage of many slides is feasible after removal of part of the slide material (Figs. 104 and 107).

The chief disadvantage in the use of excavation methods lies in the cost of correcting large slides. Property rights also pose economic and legal problems, for landslides do not limit themselves to property lines. Most of the excavated material must be wasted; this may be a large factor in the costs in areas where waste disposal sites are rare. Another disadvantage to this method lies in the fact that for most slides excavation must start at the top and work downward. Such procedure almost inevitably means increased unit costs.

Excavation techniques are frequently used for the control of all the classes of landslides. These methods, however, are best adapted to slides that are moving downslope toward a manmade structure; they are rarely effective for slides that threaten the installation by undermining of material on its downslope side. It must be remembered that excavation procedures may reduce resistance to movement at the same time that they reduce movement-causing stresses. Net benefit can thus be realized only by removal of that part of the moving mass that produces more stress than resistance.

The size of the slide also affects the applicability of this general method. Complete removal can, of course, be applied only to relatively small slides. The term "small" is relative, however; some organizations consider that quantities of 10,000 to 30,000 cu yd are insignificant, whereas others consider such quantities as large-scale operations.

Decision as to whether or not to adopt an excavation method can be based, at least in part, on economics. By determining the quantity of material that must be moved to reduce the stress to a safe level, the cost can be compared to other available techniques (Peck and Ireland, 1953). Generally, excavation methods may prove to be most economical for slides that involve anywhere from 20,000 to 2,000,000 cu yd of moving material. For very large slides, however, any excavation technique may be prohibitive in cost. In some such cases the flattening or benching of a newly

cut slope may produce stability at the toe and prevent upward migration of the movement. In other cases, however, it is more economical and safer to unload the upper part of a slide, even if it is very large. The Cameo slide on the Denver & Rio Grande Railroad above the Colorado River is an example of this kind (Fig. 100).

In considering the use of excavation methods, it is important to know whether the landslide should be classed as a fall, a slide, or a flow, and whether the slip-surface is curved or straight. It is also essential to know whether the failure developed at the toe of an excavation, thence proceeded upslope, or movement developed simultaneously throughout the slide area. These points are further explained in the paragraphs that follow.

Rock Slopes

The use of excavation methods in the correction of rockfalls, rockslides, and related types of movements in bedrock is rather widespread. In addition to removal of broken material that has actually fallen on the structure or that endangers it, the most logical use of excavation methods lies in benching or flattening of the slopes. In some instances, of course, flattening of the slope will be of no lasting benefit because the character and geologic structure of the rock cause it to assume very steep slopes with time, regardless of the slope to which it is originally cut.

The following basic principles are involved in arriving at the proper slope for any rock excavation:

1. Primarily, the design must seek to eliminate or minimize future maintenance costs that may arise from weathering or erosion of the exposed bedrock. On highway cuts the debris from the exposed face tends to clog ditches, resulting in pavement failures; to block shoulders at curves, thus narrowing the usable and safe width of the road; and to produce rockfalls on the pavement itself, with consequent danger to drivers and vehicles.

2. The completed slope must be as steep as is feasible in order to maintain excavation quantities, hence costs, at a minimum. This requirement, combined with the first, serves as a bracket for the design problem.

3. The proper design of slopes in rock is directly related to the geologic characteristics of the rock itself. The original nature of the rocks, as well as the degree, character and rate of weathering and other alteration, all play a part in determining the slopes at which they will remain stable. Even more important than these features are the structural ones, such as character, spacing and dip of bedding planes, faults, and joints, as well as the interbedding of rocks with different physical makeup. All of these features vary so much from rock to rock and from place to place that few if any general correlations can be drawn between rock properties and slope design. The design should, if at all possible, be based on the results of a thorough geologic investigation. Lacking this, the best known technique is to base it on observations of artificial and natural slopes on the same geologic materials in the immediate area. Figure 84 shows an ingenious method of maintaining a very steep cut in rock that would otherwise have required a low slope angle or multiple benches.

In addition to the three principal factors just given, proper slope design must also give consideration to such things as the relative costs of moving large or small quantities of material, of possible questions of legal liability, of the effects of climate on future weathering and erosion, of the effect of blasting methods employed, and of the probable increase or decrease in safety to users of the structure.

Three main kinds of slope design are currently used for highway excavations in bedrock. These are (a) a *uniform slope* from ditch-line to the top of the slope, (b) a slope consisting of straight sec-

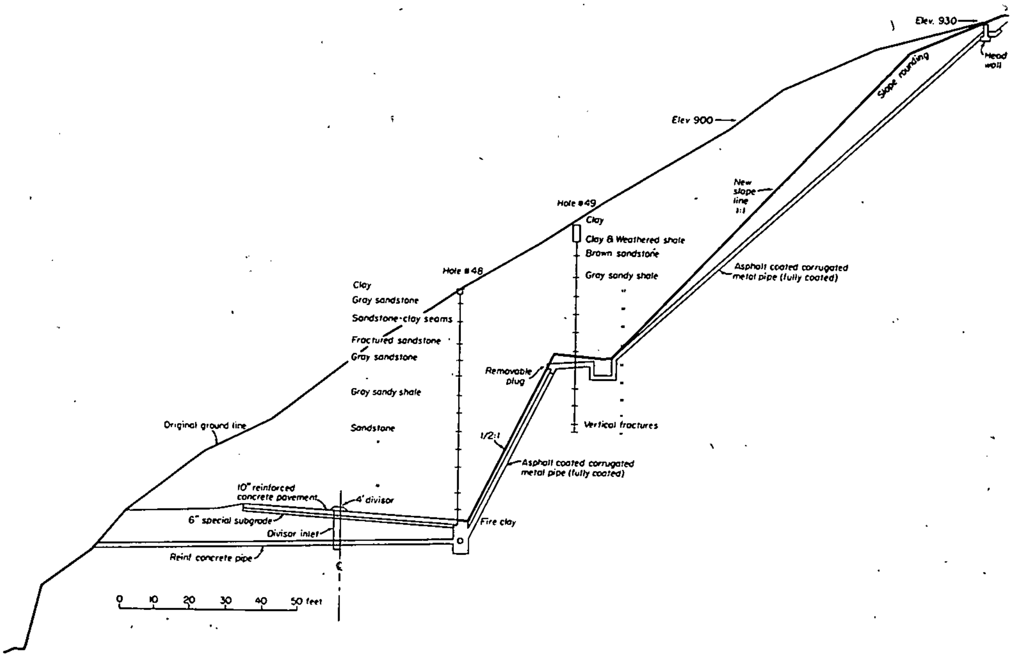


Figure 98. Cuts as high as 190 feet made in alternating horizontal layers of shale and sandstone for the West End Bypass at Pittsburgh, Pa. The design includes use of a single 16-foot-wide bench at varying heights above the roadway, with 1/4:1 to 1/2:1 slopes for the material below the bench and 1:1 slopes above it. This is a typical cross-section for one of the deeper cuts, showing borings and details of slope and slope drainage. (After Roads and Streets, 1950)

tions at *varying angles* (Fig. 98), and (c) straight slopes separated by near-horizontal *benches* (Fig. 65).

The chief problem in designing a uniform slope is to determine its proper angle. This will be related to the height of the cut, as well as to the kind and geologic structure of the material. In any one locality there appears to be a maximum height at which the weaker rock materials tend to maintain stability on a given slope. This factor can only be determined through local knowledge. On the other hand, some of the stronger and more massive kinds of rock may tend to break only along near-vertical faces, thus placing practical difficulties in the way of excavating uniform slopes at lower angles. Again, if different kinds of rock (such as shale, sandstone, and limestone) are interlayered, a uniform slope across the different kinds will commonly result in improper design for one or more of the layers. For most small cuts (those less than 20 ft high) uniform slopes are probably the best and cheapest solution. For all larger cuts, full account must be taken of the geology in determining the angle of a uniform slope.

Variation of the slope angle to correspond with differences in the underlying materials is essential in some situations. It permits use of the proper and most economical slopes for each of the geology. In some cases such a study also reduce erosion on long slopes. Its main drawback lies in the absolute necessity for detailed investigation of the geology. In some cases such a study would cost more than would an over-design, or a uniform slope, based on a minimum of data. In many places, however, differences in durability and per-

meability of the various rock layers absolutely require different slopes for each kind of rock if rockfalls and maintenance expense are to be kept at a minimum. This is true, for example, of the interbedded sandstone, shale, clay and coal of the Allegheny Plateau rocks of Pennsylvanian age (see Fig. 73); it is also true of the less durable limestone, shale and clay of the basal Permian rocks in eastern Kansas. In long cuts, it is not uncommon to pass through the weathered zone and into unweathered rock toward the middle of the cut. If the strata are continuous through the cut this may well result in requirements for two different slope angles for the same bed. Thus, the design slopes on individual beds should take into account the degree of weathering as well as the type of rock.

The choice of benched slope, with either uniform or variable angles of slopes between the benches, assumes that a certain amount of disintegration is inevitable on newly-exposed rock faces. Furthermore, even if observational data are sparse, it is generally possible to establish a reasonable balance between present and probable future costs. Bench designs are based on three variables, as follows:

1. Width of benches.
2. Vertical height between benches.
3. The slope angle between the benches.

If an accurate estimate of the geologic characteristics of the bedrock is not available this method is more satisfactory than the others previously described. For shales and similar rocks, the erosion problem is reduced by use of a bench design because of the reduction of velocity of water that moves down the sloping exposures and onto the benches. Generally, the construction is simpler with benches than with a uniform slope, as steep slopes between benches are feasible. Finally, for most materials the slopes between benches can be steeper than the ultimate, because the weathering products will be intercepted by the benches. The proper location of the benches is directly related to the character and variations in the bedrock encountered, but it also is controlled in part by the safety factor desired for the prevention of rock debris in the ditchline as well as on the shoulder and pavement.

Many engineers consider benches as "clean-off" areas; that is, areas from which debris will be removed periodically, thus making room for additional weathered material. It is true that benches do permit such a procedure, and cleaning the bench may be necessary if the rate of weathering of the bedrock has been underestimated. To produce a maintenance-free condition, however, the debris on the benches should remain as insulation against continued weathering of the bedrock; ultimately the surface should be seeded.

A final factor that must be considered in design of a benched slope is the direction of the transverse slope on the bench itself. Many engineers prefer that the benches slope away from the roadway, whereas others prefer to have them slope toward the road. A roadward slope permits immediate runoff of surface water. This means that there is somewhat less tendency toward sliding of clayey debris that accumulates on the roadward-sloping bench because the material will be well drained. On the other hand, clayey material that piles up on a bench sloped away from the road will hold water, remain plastic, and may eventually slide. Again, the runoff from a roadward-sloping bench may well cause serious erosion of the slopes below it. Where rock is involved, a bench that slopes into the hill tends to resist sliding of rock debris, whereas a roadward slope may encourage movement of debris onto the lower slopes.

In general, it is recommended that the bench should be sloped away from the road in cases where little or no clayey material is expected to accumulate. If this is done, however, longitudinal drains along the inner edge of the

bench are highly advisable if not, indeed, essential. In most other cases the slope should be toward the road.

Figure 65 shows the successful use of multiple benching in bedrock. Figure 99 shows the dimensions and slopes used by the West Virginia Highway Department for designing benched cuts in bedrock. These recommendations are empirically correct for most of the geologic formations in West Virginia; they are also applicable, within limits, to equivalent formations elsewhere. It must be remembered, however, that terms like sandstone and shale are loosely defined by geologists, and are applied to rocks with widely differing physical properties. It must also be remembered that local geologic structure, such as the attitudes of bedding planes or joints, may have a very important effect on the slope stability. It is probable, therefore, that values different from those shown in Figure 99 will have to be applied in many places. In any specific case, the quantity of weathering products to be expected on the benches is the key to the various dimensions. Local experience and observations are essential guides to the design.

Soil Slopes

Excavation methods are applicable to many slides and flows that are made up primarily of soil materials.

Removal of Head. — Removal of the head, or unloading, consists of taking a relatively large quantity of material from the head of the landslide. It is an excellent corrective technique if the quantities involved are not excessive, and if a curved slip-surface exists. The reduction in motivating force achieved by this method is particularly great for slides with curved slip-planes because of the large gravitational forces that act on the upper parts of such slides. The method is best adapted to slides; it is not generally recommended for flows or for movements that are characterized by straight surfaces of rupture. On small landslides there may be a practical limitation, and entire removal may prove more efficient and economical than partial removal. The proper quantity to remove is difficult to estimate, but the theories of soil mechanics are quite helpful in this respect (see Chapter Nine). As a general guide, one to two times the quantity originally removed or to be removed from the toe of the landslide should be excavated from the head. This should be accomplished so as to provide a relatively flat surface (15:1 — horizontal:vertical) at the head of the slide. An example is pictured in Figure 100. As a further check on the design, or where there is no evidence that removal of toe material has or will be accomplished by man or by nature, approximately 15 to 25 percent of the moving mass should be taken from the head. Successful application of this technique may depend on stability requirements above the landslide itself, hence the conditions produced by excavation at the head must be considered in the light of possible movement above the excavation. It is probable that the removal of material at the head will be most successful for slides in which the soil overburden in the stable material above the crown is less than 15 ft deep, and in which any extension of the movement uphill would not produce a serious problem of legal liability.

Lowering of Grade Line. — This method is actually a variation of the technique just described as removal of the head, but it may also be considered as a relocation, discussed in preceding paragraphs. In effect, the method consists of placing the relocated structure on a broad bench cut into the moving or unstable material. If the load removed in cutting the bench is sufficient to produce stability in the entire mass, the solution will be effective.

A significant economic factor in the choice of this method lies in the cost of pavement or track replacement. Geometric considerations, such as grade lines and sight distances, are also involved. In many cases it is necessary to lower the grade by as much as 20 percent of

	Height between Benches, ft		Width of Benches, ft		Backslopes (Hor.: Vert.)	
Type of Rock	H_a	$H_{b'}$, $H_{c'}$, etc.	W_a	$W_{b'}$, $W_{c'}$, etc.	S_a	$S_{b'}$, $S_{c'}$, etc.
1. Major cut in shale with interbedded sandstone	5-20*	20-30	0-30	20-35	1/2:1	1/4:1 to 1/2:1
2. Major sandstone cut	10-30*	30-40	0-20	20-30	1/4:1	1/4:1
3. Major cut in sandstone underlain by shale	10-30*	20-40	0-25	20-35	1/4:1	1/4:1 to 1/2:1
4. Moderate cuts in sandstone and shale	10-40*	20-40	0-20	20-30	1/2:1	1/4:1
5. Major cuts in shale	10-25*	20-30	0-30	20-30	1:1	1/2:1 to 2:1

* Use minimum if $W_a = 0$.

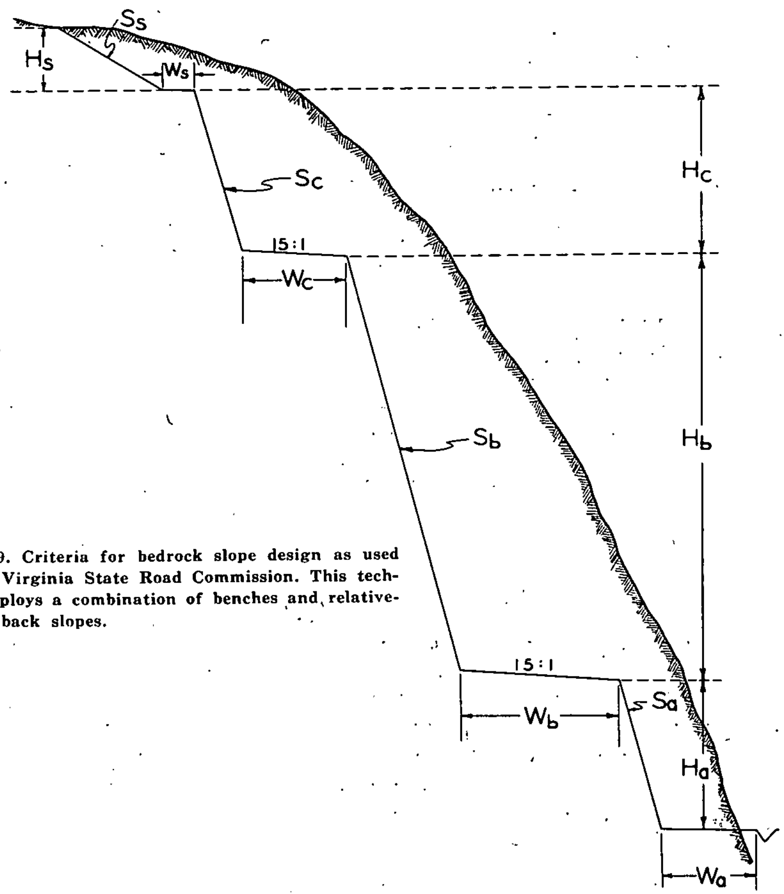


Figure 99. Criteria for bedrock slope design as used by West Virginia State Road Commission. This technique employs a combination of benches and relatively steep back slopes.

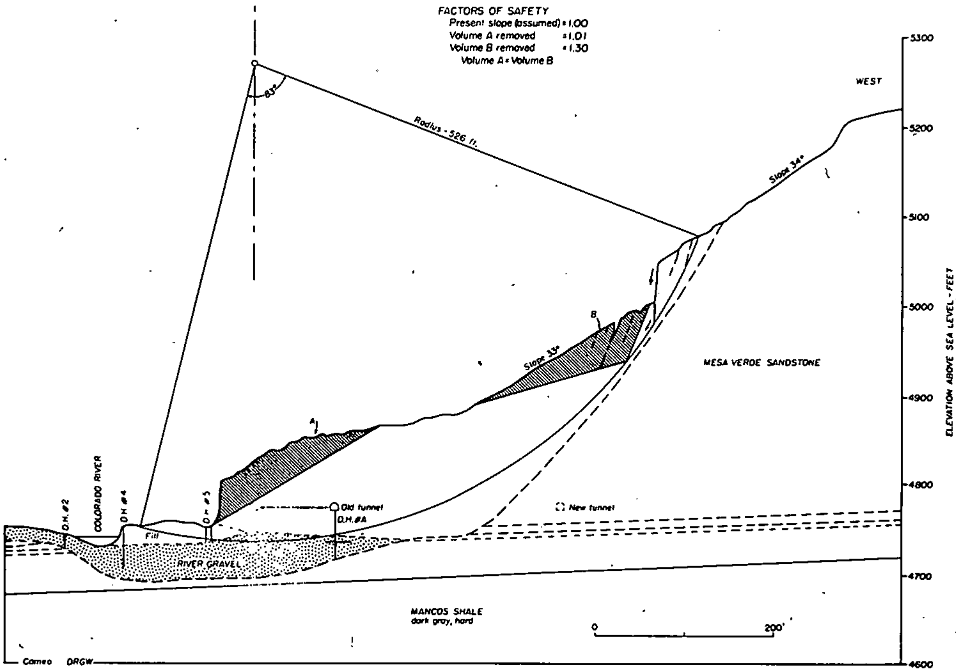


Figure 100. Stabilization of the Cameo slide above a railroad in the Colorado River valley by partial removal of the head. Stability analysis disclosed that removal of the shaded area (B) at the head of the slide would provide a safety factor of 1.3, whereas removal of a similar volume near the toe in the area indicated as (A) would produce a safety factor of only 1.01 based on an assumed safety factor of 1.00 for the existing slope. (After Peck and Ireland, 1953)

the vertical height of the slide; in no case should the grade reduction be less than 10 percent of the height. Figure 101 illustrates a highway problem that was solved by this method.

Reduction of Slope. — Slope flattening is rarely applicable to flows or to slides with straight slip-surfaces. In addition to its use on embankments, the treatment is recommended primarily for cuts where undercutting of slope-forming materials, by nature or by man, has produced a relatively small slide that extends only a short distance above the top of a cut slope. Larger slides are commonly better treated by removal of their heads. In many cases it is necessary to reduce the slope in order to bring about stability at the toe of a slide; the flatter slope, with its reduced motivating force, prevents successive undermining with consequent upslope spread of the failure. In such cases, this method may be used in combination with others.

In some soils, applicability of this method is limited by the direct variation in the shearing resistance of the soils with differences in height of cut. In West Virginia, for example, most talus soils are stable on 2:1 slopes for cuts up to 25 ft in height, but they require 3:1 slopes for cuts of 30 ft in height. For even greater heights, removal of head or other techniques are used in combination.

The dangers of flattening or benching of the cut slope without consideration of all the factors involved are illustrated in Figure 102.

Benching of Slopes. — Benching of slopes, pictured in Figures 102 and 65, is a modification of the slope flattening technique described in the preceding paragraph. On occasion, a straight slope

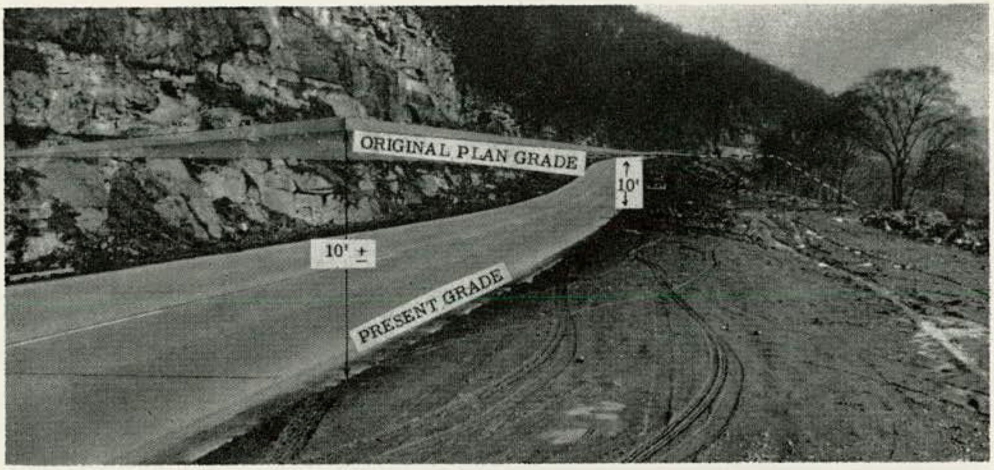


Figure 101. A sidehill fill slide which developed during the construction of State Route 7 near East Liverpool, Ohio, was controlled by lowering the grade about 10 feet at the point where the car is parked. (Courtesy of Ohio Department of Highways)

cannot be cut sufficiently flat through an excavation to provide stability. This situation is most likely to develop on steep hillsides with slopes of 3:1 or steeper. The height at which a given soil will be stable for a given excavated slope can be evaluated through experience and, within limits, by the use of the theory of soil mechanics. For example, in red clay talus deposits in northwestern West Virginia, a 2:1 slope is stable to a maximum vertical height of 25 ft, and not a great deal of variance is noted. In other areas of the country, allowable heights for a given soil can be evaluated through observation and experience.

Benching produces stability by dividing the long slope into segments of shorter slopes connected by benches. The proper width of bench can be estimated analytically for any given soil (see Chapter Nine). In order to make the slope segments act independently, however, the bench should be at least 25 ft wide.

Total Removal. — Removal of all unstable material is a method that is applicable to all types of movement, but it has a practical limitation based on the size of the moving mass. Furthermore, the position of the threatened structure with relation to the landslide mass will influence applicability of this technique. There is no lower limit as to size of slide for which it can be used; the upper limit depends on the money available and the degree of safety desired. For most landslides that involve an excavation of more than 50,000 cu yd, however, it is probable that less expensive techniques, or combinations of several methods, exist.

The total removal method is most applicable if the structure to be protected is at the toe of the slide. Other locations of the structure in relation to the slide would preclude the use of this method in most cases.

Drainage Methods

Drainage is without question the most generally applicable corrective treatment for slides. Surface drainage is of value regardless of the type of slide movement, and can often be used in conjunction with other corrective methods at little additional cost. Subdrainage is infrequently used in correction of falls, but some drainage method is almost a necessity for most flows unless avoidance techniques are followed.

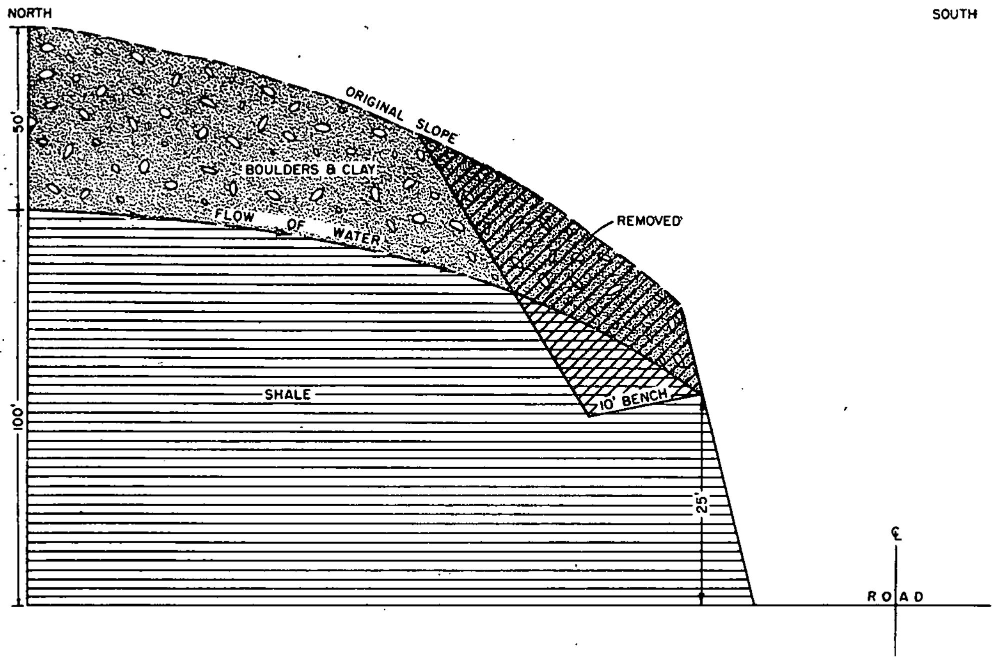


Figure 102. Cross-section of the Keystone slide on Route 145 west of Telluride, Colo. The slide is composed of boulders and clay overlying shale. Irrigation of pasture land about 1/4 mile north of the road furnishes water which lubricates the shale surface and permits continuous movement of overburden. The slide has been active over a period of 60 years. The benched section represents the most recent attempt at partial control of the movement. It is not expected to be a permanent solution, but complete removal, or installation of a deep interceptor drain, are considered less economical. (From drawings and information supplied by the Colorado Department of Highways)

Landslides that involve several million cubic yards cannot be economically controlled except by drainage. The principal use of drainage methods, therefore, lies in the control of very large slides or flows whose control by any other technique would be too expensive. It is important to remember, however, that water will not drain readily through a clay-type, cohesive material unless the internal structure of the material permits (Fig. 71). Thus, unless electro-osmosis treatment is used to change the internal structure, or unless wells are provided to collect seepage water, full benefit of a drainage installation may not be realized for months or years.

The principal advantage in the use of drainage is that it eliminates or minimizes one of the major contributing factors to landslide movement. Drainage may also be the least expensive approach, particularly on large-scale movements. The use of surface drainage is strongly recommended because of its relatively low cost compared with the high potential values that may be derived. For stabilization of flows, particularly, some degree of drainage may be an absolute requirement.

The greatest disadvantage lies in the relatively high cost of subdrainage for the smaller landslides that involve a few hundred to one million cubic yards of material. Another disadvantage lies in the fact that if highly impermeable material is present in the upper layers of a slide it may be impractical to continue the subdrain to the point where it would intercept the source of water. Also, a subdrain structure must be located on solid, unyielding foundations or else the design must permit further movement without completely disrupting the sys-

tem. Unless equipped with efficient filter materials, many subdrains lose their effectiveness because of silting or other reasons; observations should be continued so as to permit immediate action if the drain should become clogged.

The water within a slide mass has two principal detrimental effects — it increases shear stress by its own weight and by increase of seepage forces, and it reduces the shear resistance of the material, particularly along the surface of rupture, by increase of hydrostatic or pore pressures. In addition to these direct effects on shear stress and resistance, either the water itself or the chemicals in it can cause chemical or physical changes within the landslide material. Running water on the surface, of course, leads to increased erosion.

It is generally believed that water acts principally by lubrication of the slip-plane; Terzaghi (1950) and some others, however, hold that there is sufficient water in any earth mass to produce the necessary lubrication. There is also some argument among investigators as to the relative importance of the other factors mentioned in the preceding paragraph. For example, seepage forces and the loss of shear strength due to pore pressure produce identical effects on stability analyses. The increase in weight due to contained water, as well as the reduction of shearing resistance of the soil, have each been considered as insignificant by some investigators. There appears to be general agreement, however, that hydrostatic pressures are commonly a significant contributing factor, as are geochemical and physical changes in some instances.

A landslide that was caused by major excavation at the toe, either naturally or artificially, may not respond to drainage. Even though the material can be drained dry, too much resistance to movement may have already been removed to permit stability. This possibility is greater for slides with curved surfaces of rupture, because in such slides the greater part of the stress derives from the head, whereas the major resistance to stress lies in the toe. If more than one-quarter to one-third of a slide's volume has been removed at the toe, it is doubtful that drainage alone will prove effective in preventing further movement.

Surface waters can be removed from slides, or prevented from entering them, by means of ditches, slope treatment, regrading, or sealing of cracks. The principal methods of removing water from the interior of a slide are horizontal drains, trenches, tunnels and vertical wells. Each of these available methods is described in the following paragraphs.

Surface Drainage

Good surface drainage is highly desirable for treatment of any slide and should be sought regardless of any other techniques that are used. The principal surface drainage methods known, which can be used separately or in combination, consist of open ditches, slope treatment, regrading, and the sealing of cracks.

Open ditches will be useful on virtually all landslides. Particularly desirable are surface drains that are off the moving area and that completely surround the landslide, thus intercepting runoff from higher ground. Their use in locations where debris from above may cause clogging is recommended only on condition that a pipe is placed in the ditch to insure that the water will not be trapped. In many instances, depressed areas on the landslide face have produced ponds; surface ditches can be useful in draining them. A ditch in the slide material itself must be used with caution, however. Unless it is sloped so as to provide fast drainage, or unless its base is sealed with impermeable material, it can easily become a device for feeding water into the slide rather than acting as a remover of water.

Slope treatment can consist of a number of procedures, all designed to promote rapid runoff and to improve slope stability. Some of these methods are

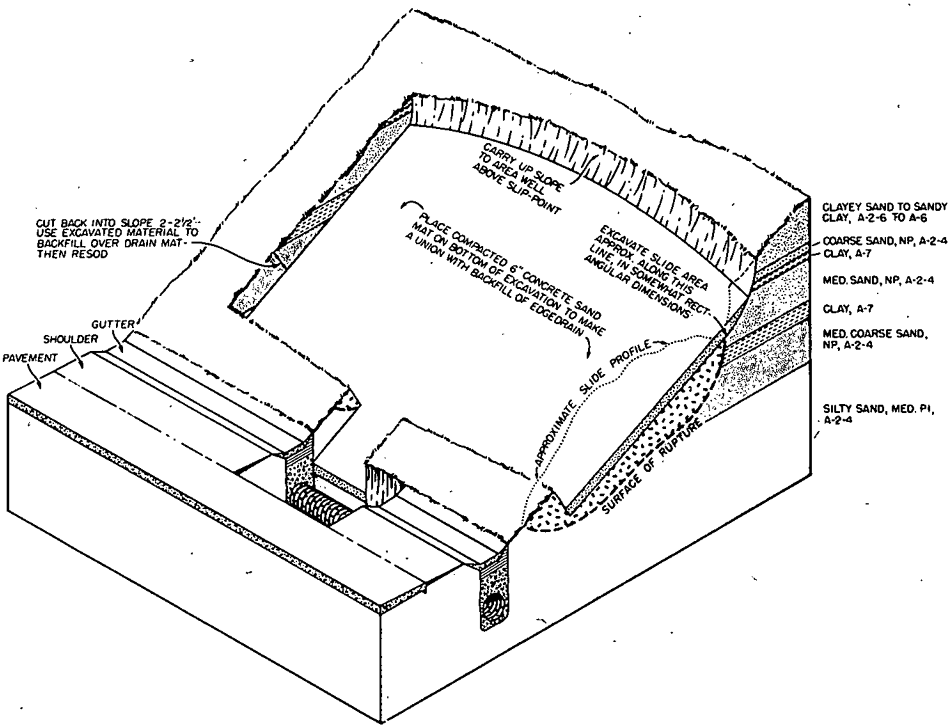


Figure 103. Correction of slide on Delaware Memorial Bridge approach, U. S. Highway 40, Del. The correction consisted of removing the slide mass, placing a sand layer connected to an underdrain system, backfilling, and resodding. In other words, the slope treatment involved improvement of both surface and subsurface drainage to improve the stability. (Sketched from design drawing furnished by Delaware State Highway Department)

seeding or sodding, oiling of surface, gunite, riprap, thin masonry or concrete walls, and rockfills. Gunite and thin masonry walls have been used successfully to protect weak shales or claystones from rapid weathering and subsequent falls. In the Ventura Avenue oil field (Mineral Information Service, 1954) many acres of land were paved with asphalt to promote runoff and reduce infiltration. This technique was merely an adjunct to an elaborate system of horizontal and vertical drains, as well as other methods of control. Here, as elsewhere, surface drainage techniques are valuable in conjunction with other procedures, but rarely provide adequate correction in themselves (see Figs. 82, 89, and 103).

Reshaping of the surface will be beneficial for all landslides that have developed open cracks or depressed areas. The procedure is designed to improve runoff and reduce the entrance of water into the center or bottom of the landslide mass. Reshaping of the surfaces will tend to reduce additional movement, but it is rarely used as a corrective measure in itself.

Sealing of cracks is commonly accomplished by regrading the surface. On occasion, individual cracks may be sealed more economically and rapidly by handfilling with clay, bituminous materials, or cement grout. The sealing of cracks will often materially reduce the amount of movement by preventing the entrance of surface water and the subsequent buildup of hydrostatic pressures or the liquefaction of the landslide mass. Im-

mediate attention to crack sealing is strongly recommended, even though additional correctives will be desirable in most instances.

Subdrainage

Subdrainage is discussed in detail in Chapter Seven. Successful use of the method is dependent on ability to reach the source of water, the presence of permeable material that will permit free access of water to the drain, and the location of the drain on unyielding material so as to insure continuous operation in the future.

Horizontal Drains. — Horizontal drainage gives promise of being a most economical method of correction. During the period 1950-1954 the cost of hori-

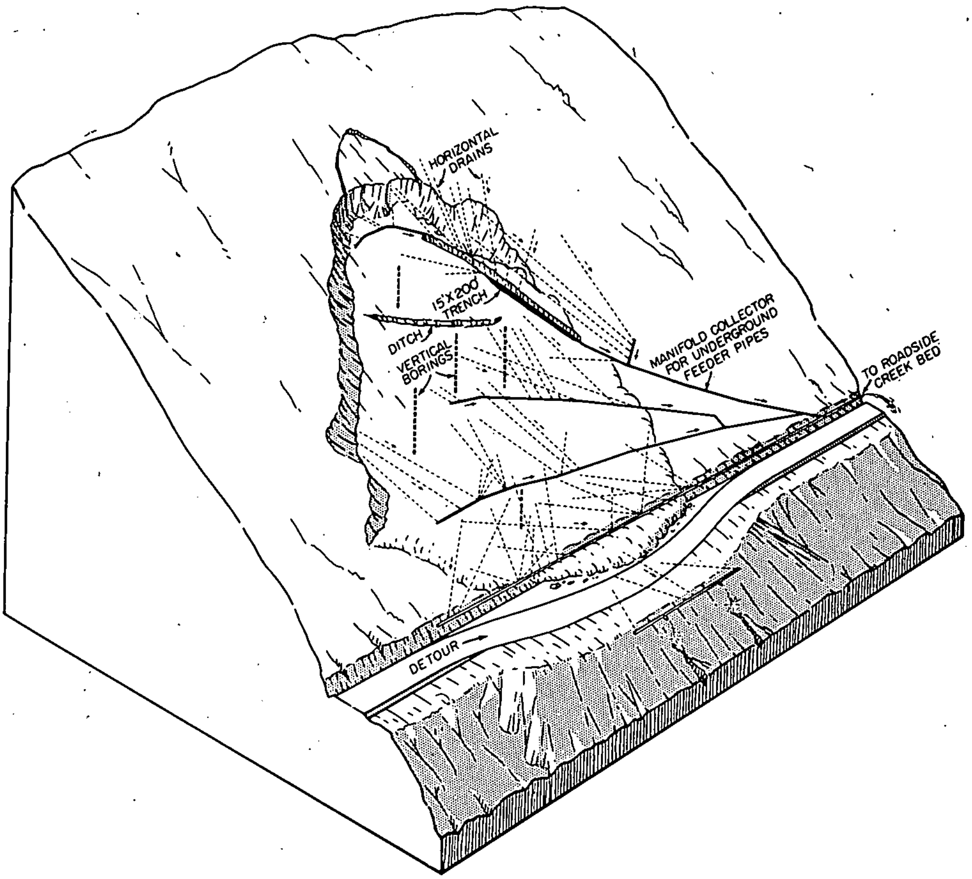


Figure 104. Landslide above State Route 75 near Orinda, Calif. Approximately 250,000 cubic yards of earth, mostly mud, broken rock and shale, were in motion in a slide which covered the road to a depth of 30 feet. Slide area is 300 feet wide and 800 feet high. Corrective treatment included reduction of the slope to approximately 2:1, interception of surface drainage, and subdrainage by means of horizontal borings at various levels. There are 95 horizontal drains with a total length of 10,000 linear feet. A flow of 135,000 gallons per day is reported during the rainy season. Collector system and horizontal drains (single dashed lines) are shown schematically. (Sketched from photographs and plan drawing; Herlinger and Stafford, 1952)

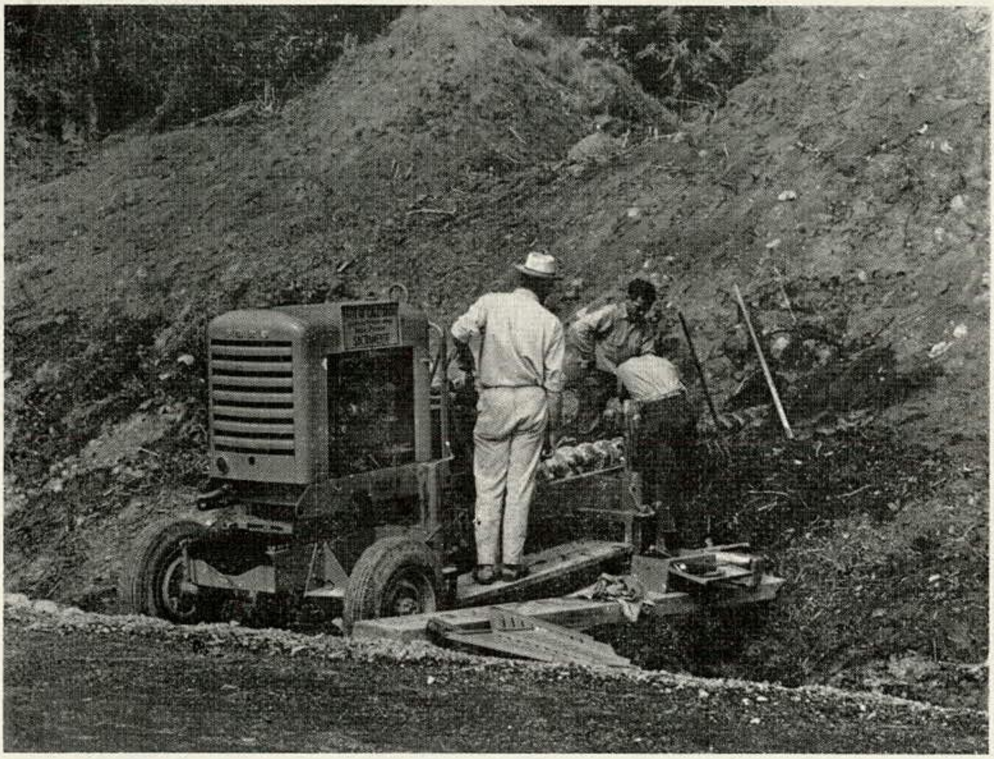


Figure 105. Boring a horizontal hole for subsurface drainage with a rock-boring machine using helical augers. (Courtesy of California Division of Highways)

zontal drain installations was about \$2.00 per foot of pipe in place, or from \$3,000 to \$5,000 for correction of slides that by other corrective methods would have cost from \$10,000 to \$20,000. Extensive application of this technique has been made in the Pacific Coast region. An example of the use of horizontal drainage is given in Figure 104; Figure 105 shows the type of equipment used in California. Only a few applications have been made elsewhere in the country (Figs. 73 and 106).

The object of horizontal drains is to remove water by diversion of the water source or pool, lowering of the water table in the slide mass, or drainage of a pervious stratum. Where the source of the water can be reached and diverted by the drain before it enters the slide material, there should be little question as to the desirability of the installation. For general lowering of the water table, the principles involved with effecting a change in the ground water elevation of a soil mass will prevail.

In the Youghiogheny spillway cut (Fig. 73), horizontal drain holes as much as 300 ft long were drilled during construction by the Corps of Engineers in 1943. Their purpose, successfully accomplished, was to relieve hydrostatic pressures and hence to prevent slump failure.

The indiscriminant use of horizontal drains is to be discouraged. Sufficient drilling should be done to determine whether water is present and whether it can be removed from the ground. The former fact can be determined easily, the latter can be estimated through experience and by observation of the time required for water to flow into the drill-

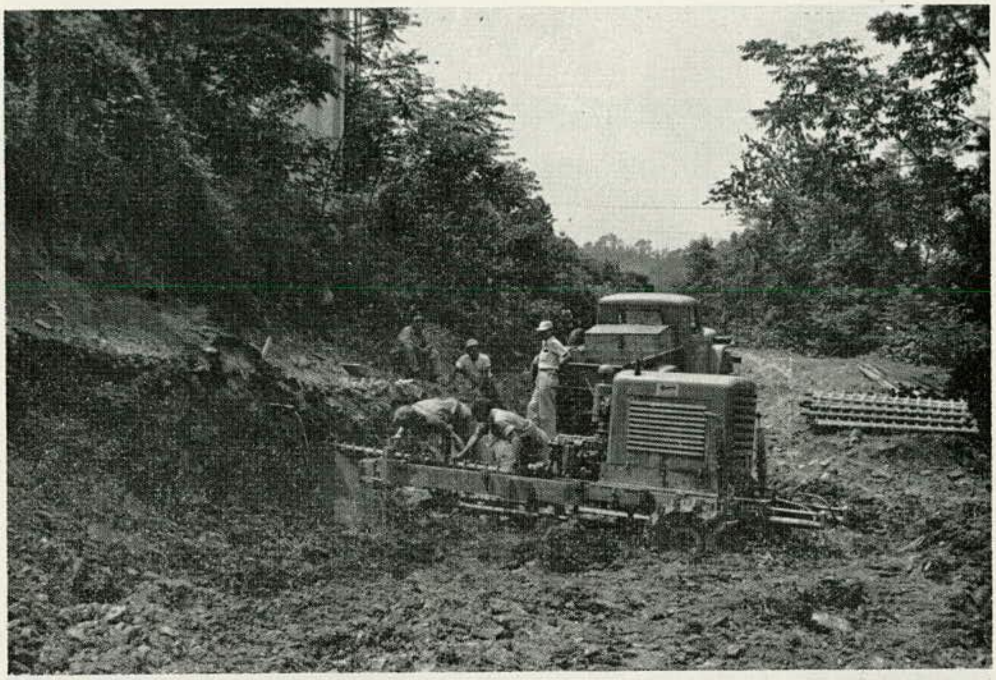


Figure 106. One of the early efforts in eastern United States to apply horizontal drainage techniques to a slide correction problem, near Station 2195, Kanawha County on the West Virginia Turnpike. In addition to the use of benching for the removal of material from the head, a horizontal, continuous helical auger was used to drill 6-inch holes for the placement of 2 1/2-inch O.D. metal pipe. (Photograph courtesy of Armco Drainage and Metal Products, Inc.)

holes. Falling-head permeameters and other devices can be used in the field, and laboratory tests of "undisturbed" samples may help in estimations of permeability.

One of the chief objects of the drilling, which should be based on adequate surface and subsurface geologic investigations, is to determine the relative permeabilities of the various materials within and beneath the slide. That is, in a series of layers of varying permeability, water will be diverted along the top of each relatively impermeable stratum.

On occasion, horizontal drainage can be used in conjunction with other corrective measures, particularly excavation methods. In such instances, the excavation can be held to a minimum and future movement controlled, eliminated, or minimized by horizontal drains (Figs. 104 and 107).

Drainage Trenches. — Drainage trenches or interceptor drains are used for the same purposes as horizontal drains (Figs. 104, 108, and 109). Trench-type drains, however, are generaly limited by practical considerations to those places where water can be intercepted at depths of less than 10 to 15 ft. On some large slides the trench is excavated with power equipment and depth in excess of 15 ft can be reached. This technique is relatively expensive, however, and is used infrequently.

It is most important that the drain pipe be based on unyielding material, which generally means that it must be below the slip-surface. Otherwise, subsequent movement would tend to break or bend the pipe and disrupt drainage

Figure 107. Slump-earthflow 12 miles west of Vallejo, Calif. The slide was 400 feet wide, 860 feet high, and about 60 feet deep, in unconsolidated clayey material with much interstitial water. Movement was apparently hastened by steepening of the slope during road construction. Effective correction consisted of slope-flattening at the head and installation of horizontal subsurface drains. (See also Figure 104) (Photograph by Merritt R. Nickerson, courtesy of California Division of Highways)

before benefit of the drain could be achieved, and the problem would remain unsolved.

Tunnels. — Tunnels to control landslides have been used in this country primarily on the West Coast (Figs. 71 and 72), and only for mass movements of very large proportions. Because of the expense involved, tunneling will not be used frequently. The technique is particularly useful where the endangered structure or structures are extremely valuable.

Vertical Sand Drains. — Vertical sand drains are most commonly used in conjunction with horizontal drains (Palmer,

Thompson and Yeomans, 1950). In many such instances, lenses of permeable material are connected vertically by the sand drain, and a horizontal system is then used to remove the water. This approach is extremely useful in landslides that contain lenses of permeable sand within less permeable material. Very large drains, however, such as are described by Palmer, Thompson and Yeomans, are very expensive, may miss some permeable zones, and may even be destroyed by renewed slide movement. Continued observations and further test drilling are advisable after the installation has functioned for a time in order to make certain that there are no isolated undrained pockets of permeable material.

Vertical drains also have been used to carry the water in a slide mass through an impermeable stratum into a permeable zone (Parrott, 1955; Mineral Information Service, 1954; see also Fig. 110). In the Ventura Avenue oil field, vertical holes were drilled through the slide and into a massive permeable sandstone. Fortunately, the sandstone bed dips at such an angle that it acts as an aquifer and carries water to pumped wells below the toe of the slide area.

One of the most promising applications of subsurface drainage to a landslide has been used in the State of Wash-

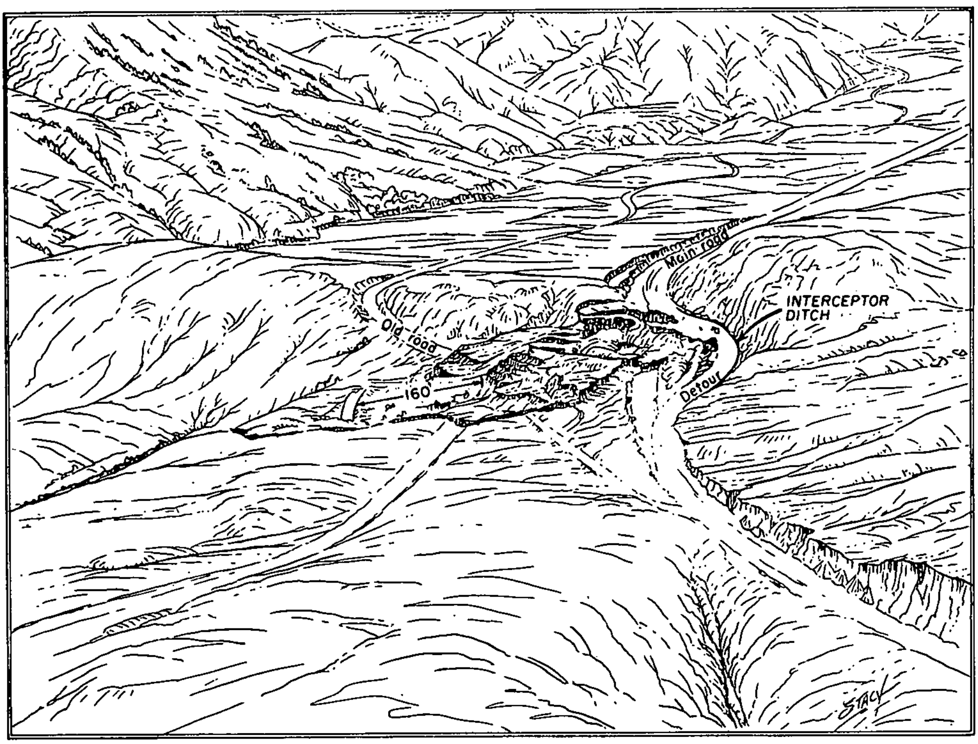


Figure 108. One of twelve landslides on a section of State Route 15 near Horse Shoe Bend in Idaho. Corrective measures included the use of interceptor drains. The sketch pictures the conditions near Station 830, designated as Slide No. 3. Upper portion of slide is a slump, which passes into a flow in the lower parts. The road was partially relocated along the slump scarp. To remove water from seeps at the base of the scarp, an interceptor drain 10 feet in depth was placed in the upper ditchline of the detour road so as to drain from the center toward each end. (Drawn from a photograph furnished by the Idaho Department of Highways)

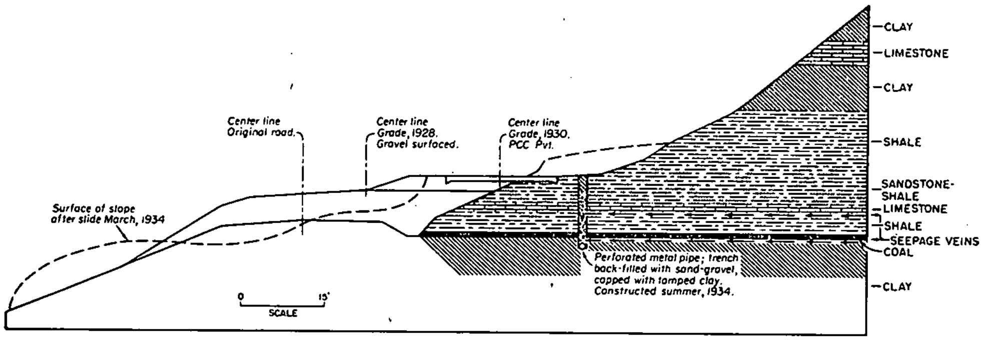


Figure 109. A drainage solution on a Missouri highway. A 15-foot deep interceptor drain was used to stabilize the Mussel Fork slide on State Route 36 two miles east of Bucklin, Mo. The drain intercepted seepage flow through a limestone stratum and a coal seam. This condition typifies the importance of differential permeability in the landslide problem. The relatively impermeable shale and underclay prevent the passage of water, thus producing free water in the more permeable limestone and coal strata. (Courtesy of Missouri State Highway Commission)

ington as reported by Ritchie (1953), who describes a continuous siphon to remove subsurface water (Fig. 75).

RESTRAINING STRUCTURES

Restraining structures, as the name implies, act to control or correct landslides by increasing the resistance to movement. Included here are rock or earth buttresses at the foot of the slide, cribs or retaining walls, piling (fixed or unfixed), dowels, and tie rods. With the possible exception of drainage methods, no other group of corrective treatments appears to have been used more frequently in landslide control than have restraining devices. Results have varied from dismal failures to spectacular successes. Misuse of the techniques, plus lack of understanding of the economic factors involved, have led many engineers to ignore or underrate these important methods.

As indicated in Table 4, Chapter Seven, restraining structures are used primarily to control slides; but they are also occasionally employed to provide underpinning for falls and flows of the other types.

The restraint may be so placed as to protect the main structure from undermining. If so, the landslide may or may not be brought under control. On the other hand, if the restraint is so placed as to protect the structure from encroachment of slide material on or against it, the landslide itself must be controlled.

Buttresses are rarely used except at the toe of a landslide and for the purposes of controlling the slide itself (Figs. 87 and 88). Cribs, retaining walls, and piling are used both at the toe to control movement and as underpinning immediately below a structure to prevent undermining. Use of these devices to control movement will be successful only for small movements, whereas for underpinning purposes they may be used on areas of considerable extent if the depth of the moving mass above the retainer is less than 10 to 20 ft thick.

Tie-rodding of slopes (Cutler, 1932) has been used almost exclusively near the upper limits of slides to protect structures from being undermined. The method is applicable to both large and small slides, but only if the soil overburden is less than about 20 ft thick (Fig. 91). Dowels into rock (Fig. 90) are used exclusively for control of movement of consolidated materials (bedrock) or large boulders.

The usefulness of a restraining structure does not depend on the cause of a

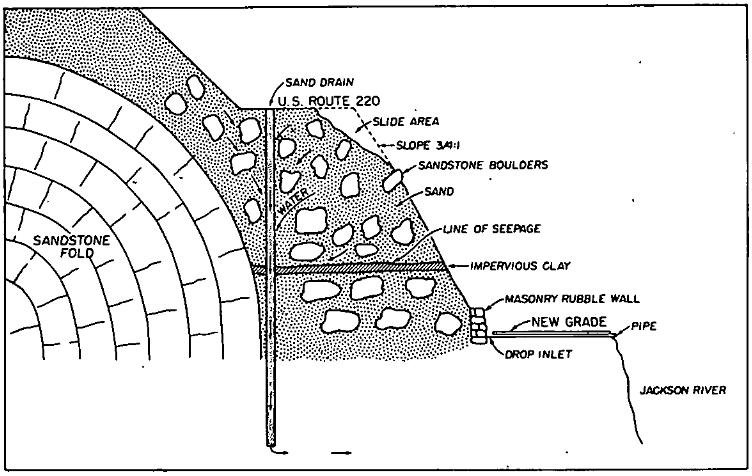


Figure 110. Vertical sand drains used to bypass impervious layer of clay. Water seeping through the thick mantle of sandstone boulders and fine sand was producing slides in the cut above the new highway grade at Ritch Patch Mountain, Va. The water was effectively drained by eight vertical sand drains, which punctured the impervious clay layer shown and discharged the water into the alluvial sand and gravel at the base of the 80-foot holes. (Courtesy of Virginia Deparment of Highways)

landslide. If hydrostatic pressures have played an important part in originating the slide, however, there is difficulty in predicting the ultimate hydrostatic conditions, hence difficulty in designing a restraining structure conservatively. Moreover, flow failures on very flat slopes can produce tremendous pressures; these may be too great for a restraining structure to withstand unless its installation is accompanied by some drainage technique.

The chief advantages of restraining devices lie in their economy under certain conditions. They commonly require less space than other methods, hence right-of-way costs are generally low. Moreover, most restraining structures, such as buttresses or piles, provide relatively high resistance to land movement at low unit costs. Under many conditions, however, particularly on large slides, restraining structures cost more than other methods. There is also the danger that failure of a restraining structure may result in total loss of the investment.

Design of a successful restraining structure requires very thorough examination of the foundation conditions. Except in rare instances the method requires that unyielding material be available for anchorage. If such material is not present, the designer must make doubly certain that no failure is likely to develop beneath the foundation of the restraining device.

A mathematical approach to the design of retaining devices is included in Chapter Nine. If no such analyses are planned, the following minimum information will be needed to determine the size of a restraining structure:

1. Areal limits of the slide.
2. Depth of soil overburden or depth to surface of rupture.
3. Relative stability of the moving mass.
4. Foundation conditions for a restraining device.
5. Type of slide movement.
6. Moisture conditions in the moving mass.
7. Value and relative location of structure involved.

Of these factors, one of the most difficult to estimate is the location of the

surface of rupture. Rotation at the toe of a slide or slump normally provides some indication of its location in the lowermost few feet of the slide. At the top of the slide, the location of the slip-surface is also evident. Within the middle of the sliding mass, subsurface exploration will normally give some indication of a change in character or condition of material at or near the slip-surface. If not, it is reasonable to assume that failure took place along the arc of a circle that is tangent to the slip-plane at the top and toe. A quick field method for estimating the position of this arc is given in Chapter Six and Figure 62; see also Chapter Nine.

If the area covered by the slide is known and an estimate can be made of the depth of the moving mass, the size of any particular type of restraining device can be computed (see Table 5).

Retaining walls can be of the massive type (Ladd, 1935) or can merely be used as toe protection (Figs. 81 and 83). Successful installations of crib walls range widely in magnitude; typical ones are shown in Figures 78, 79, 111, and 112.

Piling is perhaps a more controversial corrective treatment than are the other retaining devices. Typical installations, successful and otherwise, are shown in Figures 85, 86, and 113. A piling failure such as is shown in Figure 86 does not necessarily represent a poorly engineered project. In some instances, two or three successive sets of piling installed over a 15- to 20-year period may be more economical than a corrective treatment that controls the movement at one time; such

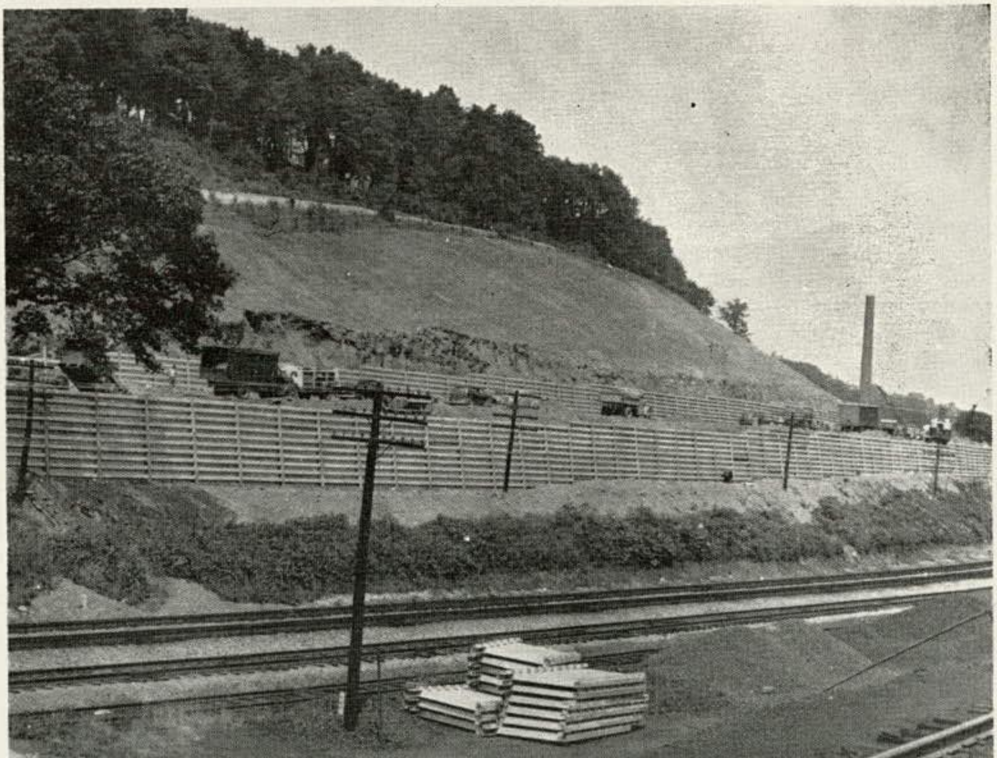


Figure 111. Metal cribbing used as retaining walls above highway and between highway and railroad at Binghamton, N. Y. (Photograph courtesy of Armco Drainage and Metal Products, Inc.)

TABLE 5

EMPIRICAL RELATIONS BETWEEN VARIOUS FACTORS IN THE USE OF RESTRAINING DEVICES TO CONTROL ACTIVE SLIDES*

Type of Treatment	Effect of Quantity of Moving Mass, % by Vol.	Effect of Foundation Conditions	Relative Stability	Type of Movement
1. Buttress at foot (a) Rockfill	Buttress should be 1/4 to 1/3 the volume of total moving mass to be retained	Should extend at least 5 to 10 ft below slip-plane unless stable bedrock is encountered	With the exception of rock buttresses restraining structures are not recommended for controlling very unstable masses at the toe. Near the top of the landslide, piling, cribs, retaining walls, and tie-rodding of slopes can be used successfully	In general, restraining structures are not recommended for falls or flows except as underpinning. If drainage is also provided, a restraining device may be helpful if area is permitted to drain before retainer is built
(b) Earthfill	Recompacted fill should be 1/3 to 1/2 that of total moving mass to be retained			
2. Crib or retaining wall	Volume of crib should be 1/6 to 1/10 that of total moving mass to be retained	Stable bedrock preferred. Otherwise, foundation should extend 4 to 7 ft below slip-plane		
3. Piling (a) Fixed at slip-surface	One pile per 100 cu yd of moving mass; maximum depth of moving mass of 12 to 15 ft	Anchor 1/4 to 1/3 total length of piles in stable bedrock; 1/3 in stable soil		
(b) Not fixed at slip-surface	One pile per 50 cu yd of moving mass; maximum depth of moving mass of 10 to 12 ft	Necessary only where no stable bedrock is beneath slip-plane		
4. Dowels in rock	——	Stable bedrock required		
5. Tie-rodding of slopes	——	Stable material needed for anchorage		

* Subject to evaluation and experience in given locality.

solutions should be neither ignored nor improperly evaluated.

Miscellaneous Methods

Hardening of the slide material by various methods, blasting, and partial removal of toe are all techniques applicable to some correction problems. They are grouped only for convenience rather than because of any similarity in method or effect on landslides. None of these techniques is very widely used, and some are used only provincially. All have been proved successful in some places, however, and all appear to deserve wider consideration and possible adoption.

Hardening of Soil

Some soil materials can be successfully hardened by cementation, chemical treatment, freezing, or electro-osmosis.

All of these methods depend on changing the shearing resistance of the landslide mass, particularly along all or part of the surface of rupture. This is axiomatic, for unless the resistance along this surface is increased there can be no material change in the stability. Thus, it is very important to reach to or through the rupture surface with the hardening technique. This type of correction is not much used anywhere in the United States. The railroads have used cement grouting rather extensively for curing waterpockets, and more recently have been successful in correcting slumping fills by this method (Figs. 92 and 93).

Except for grouting of seams and fractures in bedrock, these methods are not applicable to falls. They can, however, be recommended for either slides or flows, although most flows are less likely to be benefited than slides.

The size of the moving area will be an important factor in the applicability of these techniques. A mass as large as 50,000 to 100,000 cu yd will not be within the economic range of most hardening processes. In a few rare exceptions it is possible to adopt a partial solution by producing a buttress effect at the foot, or a series of columns of solidified earth that can act somewhat like piling.

The principal advantages to these techniques lie in the lack of interference with traffic and the absence of temporary undermining associated with many other procedures. The methods are generally limited to the stabilization of granular materials, but electro-osmosis methods are effective with some clays.

The disadvantages lie in the relatively high cost for hardening masses as large as 100,000 cu yd and in the requirement for a granular material for proper admixture dispersement. Also, few installations have been made, and all deep-soil stabilization is in the experimental and development stage. Finally, the cost is extremely difficult to predict, because predetermination of the quantity of admixture needed will be practically impossible.

In considering these techniques, an investigator should give particular attention to the location of the surface of rupture, the character and gradation of the material, and the presence of aquifers that should be drained to prevent the development of hydrostatic pressures.

Blasting

One of the very controversial methods of correction is blasting. Some noted engineers and geologists have insisted that the technique cannot produce a long-range correction. Other equally competent professional men, however, point to numerous installations which have given satisfactory performance for many years.

The two schools of thought are in apparent agreement on the following points:

1. Blasting can produce better drainage beneath the surface of rupture, thus lowering the water table and reducing hydrostatic pressures in the landslide mass.
2. Blasting can disrupt and relocate vertically upward the critical surface of rupture, thereby either increasing the shearing resistance through changed soil conditions or decreasing the shear-

Figure 112. Installing steel cribbing 1.6 miles south of Mary's River, Randolph County, Ill. Shows excavation for metal bins to retain side slopes of pavement to be constructed where old pavement was undermined by slide. (Photograph courtesy of Illinois Division of Highways)

ing force through removal of load above the slip-surface.

3. Blasting will produce some settlement, most of which can be expected within one year.

4. Relatively competent, firm bedrock must underlie the surface of rupture for any real chance of success.

5. The method does not lend itself to soil material that is greater than 25 to 40 ft deep.

Some of the objections to the control of landslides by blasting are due to the fact that the drainage produced by systematic blasting ultimately will become ineffective due to clogging by fine particles washed into the fractured mass. In some cases the surface of rupture is merely displaced to a slightly higher elevation, and no real change in the landslide is accomplished.

Both of the apparently opposing ideas concerning blasting are possibly correct in part. This could be true if failure of the drainage system and the rupture-surface displacement take 1 to 50 years to develop. That is, if the life of the structure is 20 years and the beneficial effects of blasting continue for 20 years or more, the technique is successful from an economic standpoint. The fact that the benefits of the drainage and the displacement of the surface of rupture would become ineffective after a lapse of time is then immaterial. The major difficulty will lie in reaching a reliable estimate as to the length of time that will be required to produce the damaging changes.

Blasting is most applicable to slide

failures, is not at all desirable for falls except for removal of material, and is not recommended for flows. Very large slides are not susceptible to the blasting method, particularly in areas of deep soils. It is probable that blasting methods should be limited to masses of less than 50,000 cu yd. Although blasting is normally accomplished in bedrock, the technique has been tried in unconsolidated material.

The real advantage of blasting lies in the economy of the method. Where applicable, the technique will cost one-fourth to one-tenth that of other methods. It is so inexpensive that a series of two or three blasting operations over a period of several years may be more economical than a single correction by other possible methods. Blasting is also advantageous in that it involves no serious disruption of traffic and no temporary undermining of the upslope area such as is commonly required for retaining devices and some types of excavation methods.

The disadvantage of blasting lies in the unpredictability of its effects, but the settlement that takes place, possible damage from vibrations caused by the blast, and dangers to humans or property from overshooting, are also to be considered.

The successful use of blasting to control a landslide is closely related to the experience of the operator. A good "powder-man" is essential; he should be instructed to attempt to break up the rock and to lift the fragments vertically so as to penetrate the slip-surface by 3 to 5 ft, and to blast systematically the underlying rock so that cracks and fissures will carry the water out from beneath the slide mass.

Figure 113. The successful piling installation shown was placed by the Northern Pacific Railroad near Noxon, Mont., at Mile Post 76 and 2500. Two rows of 80-foot lengths were involved, one at the shoulder and the other at the river's edge. River erosion was also eliminated (Smith 1949). (Photograph by Rockwell Smith, October 1954)

Figure 114. Top graben of a "piston" slide on U. S. Highway 99 near Napavine, Wash. This is a form of failure by lateral spreading. A layer of plastic clay is overlain by a brittle bed of iron-cemented gravel. Oversteepening of a cut slope permitted the clay to extrude slowly as it was pressed down by the graben block. The recommended correction is to load the graben block (head) and to remove the extruded clay (or unload the toe). (Photograph by A. M. Ritchie, Washington Department of Highways)

Partial Removal of Toe

As pointed out in Chapter Seven, partial removal of the toe is an expedient only, and is seldom to be recommended for control of a landslide of any type. Naturally, the removal of the toe further reduces the stability, since the major part of the resistance in any slide derives from the material at the foot.

The technique is most frequently used in highway engineering as a maintenance operation where there is a requirement for immediate action to open the road to traffic. However, the method is not infrequently used to remove a threat to other engineering structures. Stability should not be expected, and the probability of higher final costs in order to solve an emergency condition must be accepted.

One known exception to the general rule against removal of the toe is in the State of Washington. In constructing a highway near Napavine the Department of Highways encountered a so-called "piston" slide. This is a form of failure by lateral spreading in which a block or graben of rigid material drops vertically and squeezes out a layer of soft plastic clay (see Fig. 114). After detailed investigations, the department's geologist, A. M. Ritchie, recommended that additional material be placed on the downdropping block and that the extruded clay be removed. In other words, the correction method was to load the head and unload the toe of the slide. Such

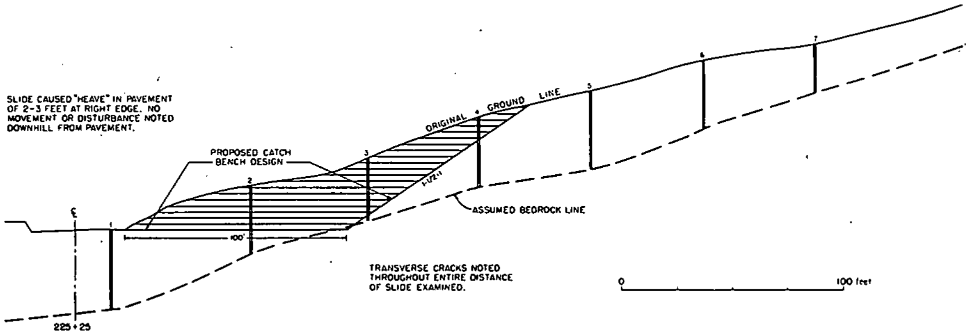


Figure 115. Use of deliberate undercutting. A corrective treatment commonly used in West Virginia consists of deliberately undercutting a landslide with benches provided upon which the slide can come to rest. The cross-section shown represents a slide between New Martinsville and Moundsville in Wetzel County. The work was completed in the Spring of 1955. (Courtesy of West Virginia State Road Commission)

an unorthodox method must, of course, be applied with extreme caution, for even a thin film of clay remaining after the extrusion process could easily lead to further movement.

In rare instances it is possible to produce relative stability by undercutting of the toe; that is, by providing a bench that is broad enough to catch and hold the products of further movement. Figure 115 shows one example of such a deliberate design. In such cases, stability is not anticipated until sufficient movement has taken place to produce a stable slope. The reasoning involved is that stability will ultimately develop through the buildup of toe resistance as the material moves onto the bench area produced by the excavation.

The benching method has been tried successfully on some slides and flows, and is the principle in one approach to correction of rockfalls. Normally, a major landslide is not deliberately treated by this technique, but if a major toe cut is necessary, benching at the toe may be the most economical method. There is no limit to the size of landslide that can be treated by a deliberate undercut for ultimate stability. However, the technique is applicable only when the structure to be protected is at the toe of the landslide and the land above the moving area is worthless. As an aid to achieving stability, other techniques such as horizontal drainage could be used in combination.

The principal advantage to the method is the economy in earth-moving; excavation at the toe of the slope is less expensive than elsewhere. Furthermore, a bench is provided for intercepting debris before the material reaches the ditchline.

A major disadvantage is the difficulty in predicting the size of bench that is necessary for equilibrium to be reached. Also, the surface of the slide must be reshaped in order to reduce movement to a desired minimum. A considerably greater quantity of material may have to be moved if surface water is permitted to enter cracks and aggravate the landslide condition.

Because the method is based on acceptance of the fact that further movement is inevitable, as much material as possible should be left in the new toe. Slopes as steep as ½:1 for vertical heights of 30 to 50 ft may be advisable, since renewed movement will then start the process of producing toe resistance.

In summary, deliberate undercutting at the toe of a slide may be desirable under the following conditions:

1. As an emergency that requires immediate action to clear the road for traffic or to protect a structure from undue pressures.
2. In very mountainous terrain where the land is not valuable and no structures, powerlines, or pipelines will be endangered by subsequent undermining.

3. Where the proximity of the bedrock-overburden contact and the slope line at the toe render it impossible to attain sufficient toe resistance by reshaping the slope.
4. When costs of producing stability by other techniques is relatively high.

It is to be emphasized that indiscriminate use of this technique, or its use with insufficient technical knowledge or investigation, is foolhardy in the extreme.

Warning Devices

Even though they neither prevent nor correct landslides, a few words on warning devices seem appropriate. So far as is known to the committee, the devices used in the United States are confined to railroads and all give warning that slides have occurred or are in progress. None of those used predict impending slides, but it is understood that Japanese engineers have used strain gages successfully for this purpose (Fukuoka, 1953).

For many years the railroads have employed warning devices coordinated with automatic block signals as protection against slides and falling rock areas. Part 205 of the Signal Requisites of the Signal Section of the Association of American Railroads lists three types with typical details.

The falling rock detector as shown in the specifications consists of 35-ft poles with 10-ft crossarms carrying 20 wires. Rock falling on these will break the electrical circuit and throw adjacent track signals into the stop indication.

The rockslide detector is a vertical fence of woven wire fencing supported on poles spaced 15 ft apart. This type actuates the appropriate signals through tension on the fence wires and can also be used for flow slides in earth.

The earth slide detector consists of planking on upright posts equipped with mercury contactors which actuate the signal circuits when pressure of sliding material causes distortion in the fence.

References

Anon., "They Scalped a Mountainside to Build New Pittsburgh Expressway Link." *Roads and Streets*, v. 93, no. 10, p. 54-56, 60, illus., 1950.

Baker, R. F., "Determining Corrective Actions for Highway Landslide Problems." Highway Research Board Bull. 49, 1953.

Berger, L. A., "Three-Dimensional Aspects of Landslides." *Highway Research Board Proc.*, v. 34, 1955.

Cutler, T. H., "Slipping Road-Bed Held Up by Steel Suspenders." *Eng. News-Record*, v. 109, 1932.

Day, Ray, "Giant Spiderweb Protects Washington from Rockslides." *Am. Crosby Clipper* (St. Paul, Minn.), v. 19, no. 4, p. 12, 1953.

Fukuoka, Masami, "Landslides in Japan." Internat. Conf. on Soil Mechanics and Foundation Eng., 3d Proc., v. 2, p. 234-238, Zurich, 1953.

Hennes, R. G., "Analysis of Landslides." Washington Univ. Eng. Experiment Sta. Bull. 91, 57 p., 1936.

Herlinger, E. W., and Stafford, Gifford, "Orinda Slide." *California Highways and Public Works*, v. 31, nos. 1 and 2, p. 45 et seq., 1952.

Krynine, D. P., "Landslides and Pile-Action." *Eng. News-Record*, v. 107, no. 122, 1931.

Ladd, G. E., "Landslides, Subsidences and Rockfalls." *Am. Railroad Engineers Assoc. Proc.*, v. 36, p. 1091-1162, 1935.

Ladd, G. E., "Methods of Controlling Landslides." *Eng. and Contracting*, v. 67, p. 599-608, 1928; also as "Methods of Controlling Highway Landslides." *Roads and Streets*, v. 68, p. 529-538, 1928.

Larew, H. G., "Use of Field, Laboratory and Theoretical Procedures for Analyzing Landslides." *Highway Research Board Proc.*, v. 31, 1952.

Mineral Information Service, "Landslides in Ventura Avenue Oil Field." California Div. Mines, Mineral Information Service, v. 7, no. 5, p. 1, 1954.

Palmer, L. A., Thompson, J. B., and Yeomans, C. M., "The Control of a Landslide by Subsurface Drainage." *Highway Research Board Proc.*, v. 30, p. 503, 1950.

Parrott, W. T., "Control of a Slide by

Vertical Sand Drains." Highway Research Board Bull. 115, p. 51-52, 1955.

Peck, R. B., and Ireland, H. O., "Investigation of Stability Problems." Am. Railroad Engineers Assoc. Bull. 507, p. 1128, Fig. 11, 1953.

Ritchie, A. M., "The Snoqualmie Rock Slide of August 13, 1953." *Pacific Building and Engineer*, v. 59, no. 9, p. 75, Sept. 1953.

Root, A. W., "Correction of Landslides and Slipouts." *Am. Soc. Civ. Eng. Trans.*, v. 120, p. 280-289, 1955.

Simpson, F. H., *et al.*, "Progress Report by the Committee on Roadbed Stabilization." Am. Railroad Engineers Assoc. Bulls. 479 and 486, 1949 and 1950.

Smith, Rockwell, "Roadbed Stabilization." Am. Railroad Engineers Assoc. Bull. 479, p. 674, 1949.

Smith, Rockwell, "Field Studies and Results of Roadbed Stabilization." Am. Railroad Engineers Assoc. Bull. 486, p. 729, 1950.

Tompkin, J. M., and Britt, S. H., "Landslides, A Selected Annotated Bibliography." Highway Research Board Bibliography No. 10, 53 p., 1951.

Terzaghi, Karl, "The Mechanics of Shear Failures on Clay Slopes and the Creep of Retaining Walls." *Public Roads*, v. 10, 1929.

Terzaghi, Karl, "Mechanism of Landslides." In "Application of Geology to Engineering Practice." Geol. Soc. America (Berkey Volume), p. 83-123, 1950.

Terzaghi, Karl, and Peck. R. B., "Soil Mechanics in Engineering Practice." John Wiley and Sons, Inc., New York, N. Y., 1948.

Chapter Nine

Stability Analyses and Design of Control Methods

Robert F. Baker and E. J. Yoder

Discussions throughout this text have emphasized the fact that more than one method can be used to prevent or correct a given landslide problem. In earlier chapters the various corrective and preventive measures have been described and general recommendations for the use of each have been made. Experience has been the basis for the recommendations. It has been shown, however, that the unfavorable experience record of certain treatments has been in part the result of failure to understand the magnitude of the forces involved. The extrapolation of experience with one type of slide in one particular region and type of material to other slide types in other regions and other materials is obviously difficult, if not dangerous. Moreover, where performance records for two different corrective treatments show equal success, some basis is needed for deciding which will be most economical in a new situation. Obviously, some quantitative means of evaluation is needed.

Even though some landslides do not lend themselves entirely to the assumptions commonly used in soil engineering, stability analyses made according to the classic theories of soil mechanics still present the best hope for a quantitative means of evaluating experience and provide a rational basis for extending experience for the purpose of prediction. The analyses cannot be made for every type of landslide and for any type a number of assumptions based on idealized conditions and materials will be required. It is impossible to treat mathematically all of the variables imposed by nature. Further mathematical simplification is required to prevent the analysis from becoming unwieldy. In application, then, the results are always dependent on the validity of the assumptions and simplifications. The results should not be considered as exact solutions of the problem and the possible variance between real and assumed conditions should always be kept in mind. Even with its limitations, applied theoretical analysis has advantages that are useful and it can, on occasion, be of considerable value.

The principal use of a mathematical approach may lie in making it possible to weigh the cost of the treatment against the value received, rather than in the actual quantitative answer. For example, if the use of a given corrective measure is questioned on the basis of experience, the cost of the treatment can be estimated, the before and after safety factors computed, and an evaluation made of how much relative stability is produced for the given amount of money. Even though one may question the accuracy of the mathematics with the attendant assumptions and, therefore, the exact values of safety factors derived, there is less question in considering relative stabilities; that is, in ranking the before and after safety factors.

This chapter is not intended to be a critical review of the methods and theories of soil mechanics as applied to landslides, and space cannot be given to

a detailed discussion of all of the ramifications of stability analyses. The reader is referred to texts on soil mechanics for detailed treatments of the problem and for discussions of the variables and simplifying assumptions that are required in any of the mathematical treatments.

Background material is provided to acquaint management, the field engineer, and the geologist with analytical methods and to permit an understanding of the major part of the discussion, which consists of examples of analyses involving the major methods of landslide control. Principal emphasis is placed on a single method of attack, the Swedish slice method, and on its application to the economics of various treatments. Numerous other methods can be used and are preferred by some workers. Most standard texts on soil mechanics may be consulted for other methods of analysis; the Corps of Engineers Manual (1952) provides a compact discussion with working examples of several of them.

The examples given here are for situations where slides have occurred. With some necessary modifications, however, the same methods are applicable to the analysis of slope stability where construction may create an unstable condition in previously stable slopes. Moreover, the analysis of an existing slide often provides the easiest and perhaps most accurate method of arriving at an estimate of stability for slopes in similar materials in adjacent areas. The discussion and the examples are included in order to demonstrate the method and principles that are involved so that the reader can make similar applications if the principles are applicable. Attention is again called to the need for understanding of the variables and assumptions involved.

Method and Principles

Several methods are available for quantitative study of the stability of slopes. Each varies to a slight degree, and each requires certain assumptions, including one as to the form of the surface of sliding. The real surface of sliding is often a composite surface having a section made up of two or more arcs of circles or approximated by an arc of an ellipse.[6] However, an exact duplication of the potential sliding surface is seldom warranted.

Most methods of analysis, therefore, replace the real surface of sliding with one having a section of either an arc of a circle or of a logarithmic spiral (Rendulic, 1935). The use of the circular arc assumption is based on studies of actual failure surfaces by the Swedish Geotechnical Commission and is fundamental to a method of analysis developed by W. Fellenius (1927, 1936). The general approach of this method has been widely adopted by soils engineers to estimate the factor of safety of slopes against failure.

In addition to an assumption as to the form of the failure surface, conventional stability analyses require certain other facts and assumptions as follows:

1. A shear failure must have occurred or must be a threat. This assumption will be true for slides, but not for falls and some flows. Flow materials will not have significant shearing resistance, so that stability analyses will not generally be made.
2. The average shearing resistance along the slip-surface at the time of failure must be known, as must any major variation from the average. The shearing resistance is at once the most critical value and the most difficult one to obtain unless a failure has occurred.
3. An assumption must be made that the conditions that exist along a narrow slice or cross-section of the slide can be used to design against movement in the remainder of the area. A related assumption (common to all stability analyses) is that no lateral shearing resistance exists along the sides of the slice. It is

[6] Kjellman (1955), indeed, has raised the question of the actual existence of a "surface" of sliding, pointing out that perhaps no true surface exists although all mathematical stability analyses assume such a surface.

believed that this assumption affects the quantitative answer in a minor way. Three-dimensional analyses can be used, but considerably more work is required and an assumption of increased accuracy may not be warranted.

4. An assumption must be made as to the location of the piezometric or the ground water surface at the instant of failure. This will apply to those movements where hydrostatic pressures could have played a significant part. One of two assumptions will be necessary, for rarely will it be possible or practical to obtain the necessary hydrostatic or ground water data. The first is to assume a reasonable location for the piezometric surface based on subsurface water conditions. If the shearing resistance is known, the location can be checked against the fact that a failure developed (or has not yet developed); that is, if a failure has developed, then certain hydrostatic pressure conditions could have produced the failure (higher pressures would have brought failure sooner, and lower pressures would have produced no failure). The other approach is to use a value of shearing resistance which incorporates the effect of hydrostatic pressure. This approach is more useful if a correction other than drainage is to be analyzed, and if the value of the shearing resistance is based on the developed slide.

5. The value of the safety factor to apply must be established. This facet can be a very difficult one to handle, for a relatively minor change in safety factor may more than double the cost of the treatment. Also of some importance is the selection of the type of safety factor to be used. Safety factors can be expressed in terms of the ratio of slide resisting forces to slide-inducing forces, or they may be expressed in terms of the relationship between soil strength factors (for example, in terms of the developed unit cohesion as compared to the unit cohesion adopted for design). The definition selected will vary with the method of analysis and the conditions of the individual situation. For an excellent discussion of safety factors as related to slope stability analyses, see Corps of Engineers (1952).

In the discussion and examples of this chapter the most frequently used expression of the safety factor will be as a ratio between total shearing resistance and total shearing force. In analyses of failed slopes the concept will be used that failure occurred when total shearing force just exceeded total shearing resistance. Thus, for the analysis of the failed slope a factor of safety of one is assumed. This assumption is fundamentally sound and, it is felt, allows the best estimate of the values of cohesion, c, and angle of internal friction, ϕ, as they existed in the ground prior to movement.

Swedish Method of Slices

The Swedish Method of Slices was developed to a relatively high degree by W. Fellenius (1927, 1936). This method applies to most cohesive soils above the water table which have a shearing resistance, s, approximately equal to

$$s = c + \sigma \tan \phi \qquad (1)$$

in which

c = cohesion;
σ = stress normal to the slip-surface; and
ϕ = angle of internal friction.

Difficulty is generally encountered in establishing accurate values of cohesion and angle of internal friction, due to inadequate sampling and testing techniques. However, the method can be applied to materials that are non-uniform in character and is most useful in estimating factors of safety against failure.

In the analysis, the assumption is made that the surface of failure of a slope can be defined as having a section represented by the arc of a circle, and that the soil within the circle rotates about point 0, the center of the circle (Fig. 116). The arc along which the soil may be assumed to move will be deter-

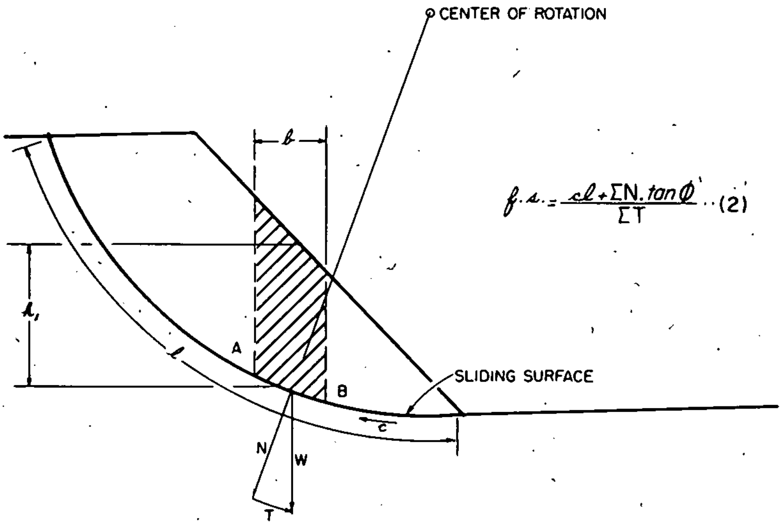


Figure 116. Forces acting on a slide wedge.

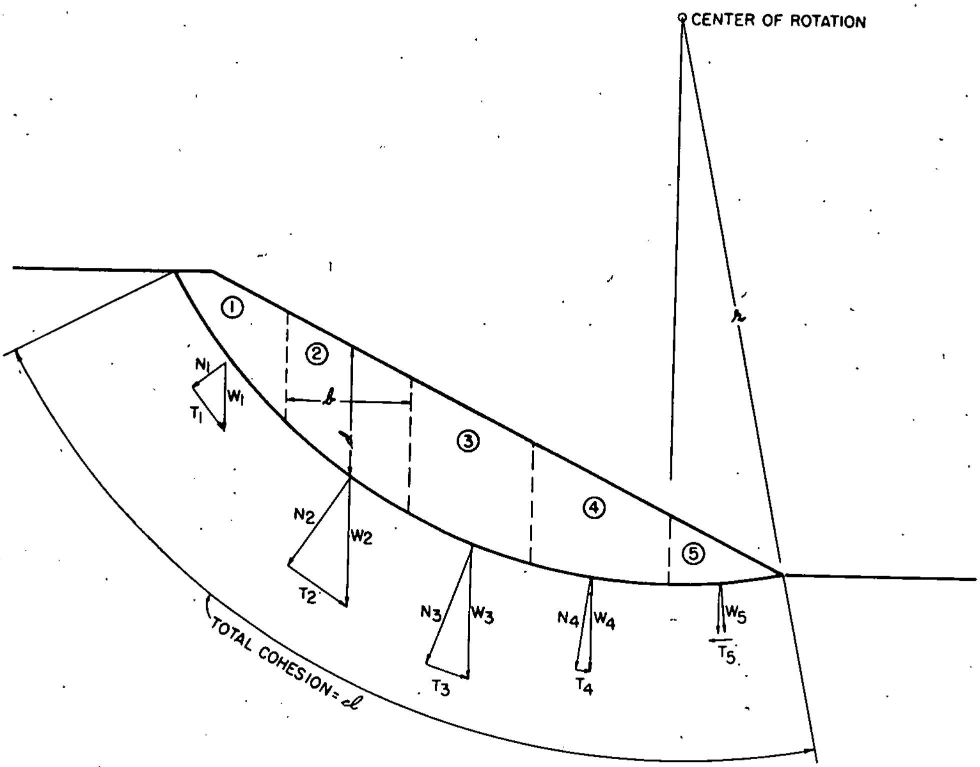


Figure 117. Graphical solution of forces for the method of slices.

mined by stratification within the sliding mass, depth to a firm material, and several other factors. In many cases the sliding surface will not approximate that of an arc of a single circle, but will be made up of composite arcs.

The procedure requires a cross-section, plotted to scale, of the slope being analyzed. The circular arc that represents the failure surface is then drawn on the cross-section, forming a circular segment representing the sliding mass. The segment is then divided into several slices of equal width, as shown in Figure 117. The shaded area of Figure 116 represents a single slice.

The forces acting on this slice are indicated at the sliding surface. Neglecting the forces acting on the sides, the forces acting on the slice are the normal and tangential components, N and T, of the weight W; the unit cohesion per unit of slice width, c, acting along the arc, BA, and the frictional force induced by N. The tangential vector, T, represents the slide-inducing force, whereas the resisting forces are the cohesion plus the normal force, N times the tangent of the angle of internal friction. Thus, the factor of safety (f.s.) against sliding along an arc of length l can be written as

$$\text{f.s.} = \frac{\text{shearing resistance}}{\text{shearing force}}$$

$$= \frac{c\,l + \Sigma N \tan\phi}{\Sigma T} \qquad (2)$$

In which ΣT and ΣN represent the sum of values of T and N for all the slices. ΣT is the total slide-inducing force; $cl + \Sigma N \tan\phi$ represents the total resisting force, l being the length of the sliding surface.

This method of analysis lends itself readily to the design of corrective measures. If the critical slide surface of a slope can be established, and the shearing forces evaluated, the increase in factor of safety realized by placing an additional resisting force at the toe can be calculated readily. The expression then becomes

$$\text{f.s.} = \frac{c\,l + \Sigma N \tan\phi + P}{\Sigma T} \qquad (3)$$

in which P is the additional resisting force per unit of width.

NEUTRAL PRESSURES

In the foregoing equations the weight of the soil mass is equal to the volume of soil times the soil's unit weight. Where the ground water table is below the failure surface (thus no seepage forces are encountered) the unit weight used in the calculations is the weight of a unit volume of the soil and its included water. However, should the ground water table be at some point above the failure surface, the resisting force is reduced due to the neutral pressure, μ, of the soil water. In this case the factor of safety against sliding is given by

$$\text{f.s.} = \frac{c\,l + \Sigma(N - \mu) \tan\phi}{\Sigma T} \qquad (4)$$

in which μ represents the total force of the soil water exerted on the bottom of the soil slice (Fig. 116). For example, if the water table is at the ground surface in Figure 116 and no flow of water exists, the neutral pressure acting on a slice is given by

$$\mu \doteq h_1\,\gamma_\omega\,\text{BA} \qquad (5)$$

in which γ_ω is the unit weight of water.

Expressed in another way, the slide-inducing forces are determined by using the weight of the soil plus water; the resisting forces are determined using the submerged unit weight of the soil

$$\gamma'_m = \gamma_m - \gamma_\omega \qquad (6)$$

in which γ'_m is the effective or submerged unit weight of the soil, γ_m is the mass unit weight of soil plus water, and γ_ω is the unit weight of the water.

Method of Estimating Stability

In the ideal case of relatively homogeneous soil, the factor of safety of a slope against sliding can be determined conveniently by graphical procedures, as illustrated in Figure 117.

The sliding elements of equal width are obtained. The weights (W_1, W_2, W_3 . . . W_n) or areas of each slice are laid out, respectively, as a vertical vector to any convenient scale at the center of each slice at the sliding arc. If the slices are of equal width, this may be done by making the vector distance numerically equal to the average depth of the slice. Lines are then drawn through the center of the circle and through the origin of each W vector at the sliding surface; this locates the line of action of the normal forces. The tangential forces are next drawn at right angles to these lines and to the lower end of the W vectors. The T and N forces may then be determined by use of an engineer's scale. As an aid in the solution it is best to set up the problem in the form of a table (see Table A, Fig. 118).

Table A, Figure 118, applies to a slump or rotational type of failure. Where the slip-surface is nearly a straight line in a planar failure, the same approach may be used (see Fig. 119). In this case the resisting force is again made up of the unit cohesion times the length of sliding plane plus the product of the normal force times the tangent of the angle of internal friction, or $c\,l + \Sigma N \tan\phi$, and the sliding force is equal to T.

Location of Sliding Surface

The success of this method of analysis, and of any mathematical treatment of slides, is contingent upon adequate boring and strength data. A sound field exploration program is essential before any type of theoretical analysis is made. Moreover, many landslides are not adapted to mathematical analysis; among these are rockfalls of all types. For the purpose of analysis, failures for artificial embankments are generally broken down into (a) slope failures, (b) toe failures, and (c) base failures. The first two of these are perhaps self-explanatory. The last, base failure, denotes a deep circle that intersects the ground line well below the toe of the slope. This type of failure, if influenced entirely by soil, is generally a midpoint failure; that is, one where the center of the circle exists at some point on a vertical line drawn midway between the toe and the top of the original slope. The location of the center on this line must be found, however, by trial and error.

The location of the circle must be compatible with the known conditions. If a layer of weak, soft material exists at some depth, the circle will be so situated that its major portion lies within this layer. If materials of different shearing resistance are present, such as soil overburden on rock, or on a firm base such as gravel, the circle will generally be tangent to the firm base. Seepage planes may likewise influence the location of the circle.

Methods are available for mathematically estimating the potential sliding surface of unfailed artificial slopes in homogeneous soils (see Taylor, 1948). After an estimate is made of the potential failure surface, taking into account the natural soil conditions, calculations are made as illustrated in previous paragraphs. The factor of safety is then computed and a new trial is made by shifting the center of rotation to both the left and the right. By repeating the process after the center of rotation is moved vertically, one can determine the critical center which is the one giving the least factor of safety.

If, after a slide occurs, the positions of at least two points on the slide can be fixed in relation to the positions which they had on the original ground, the sliding surface may be determined by simple geometry. This is done as illustrated in Figure 120. Straight lines are drawn from the original to the final location of the known points. Perpendicular bisectors of these lines will intersect at the center of rotation of the mass. In

practice it is best to utilize at least three points, more if possible. In this way any error which arises from inaccurate measurements in the field or which arises from the fact that the sliding plane is not the arc of a circle will be averaged. The lines will be found to intersect at several points and the true center can then be taken as the average of these. A slightly different empirical method of determining the location of the slip plane is described in the section on "Estimating Depth of Slump Slides: Slip Circle Method" (Chapter Six) and illustrated in Figure 62.

Determination of Strength Factors

It is extremely important that proper estimates be made of the values of cohesion and internal friction. In Chapter Three under "Factors that Contribute to Low Shear Strength," several items that contribute to shear failures of earth and rock masses are listed. Among these are neutral pressures and pressures caused by percolating water, sensitivity of clays, inherently weak materials, and others. If the rational approach to slope design is to be adequate, each of these must be evaluated.

In the special case of saturated natural clay deposits the shearing resistance can be approximated by

$$s = c \tag{7}$$

This is true because the permeability of clay is very low; therefore, if a shearing force is applied rapidly before drainage can take place, the load is taken in large part by interstitial water and the apparent angle of internal friction is equal to zero. Thus, when the analysis is made for saturated clays, the shearing resistance is made up of cohesion alone. For this case, the value of cohesion is determined by performing the unconfined compression test. Cohesion is then equal to one-half the ultimate strength in compression. Rewriting Eq. 2 with $\phi = 0$ gives

$$\text{f.s.} = \frac{c\,l}{\Sigma T} \tag{8}$$

In another special case, that of clean uncemented sand, essentially no cohesion exists and Eq. 1 becomes

$$s = \sigma \tan\phi \tag{9}$$

For these cohesionless sands the angle of repose, the natural slope assumed by sand when poured loosely on a flat surface, although not in exact agreement with the angle of internal friction, will give results which are sufficiently accurate.

For the more general case where both cohesion and internal friction must be considered, the laboratory determination of these factors is more complicated. The shear resistance may be determined by direct shear tests or, preferably, by triaxial shear tests. However, many analyses of failed slopes which have been based on laboratory values for c and ϕ have given safety factors greater than 1.0. In other examples, natural slopes of known stability have given computed safety factors as low as 0.75. This failure of laboratory-derived values to give the expected results in computations is no doubt caused by irregularities in the soil, difficulties in obtaining undisturbed samples, problems in laboratory technique, and the effect of seepage forces.

In the redesign of failed slopes an estimate of the average shearing resistance which is safe and which lessens the effects of the troublesome variables previously listed can be arrived at by basing the computations on the conditions in the failed slope.

The average shearing resistance along the failed surface can be computed by balancing forces around the center of rotation (see Fig. 121). For the redesign of slopes it is not advantageous to divide the resisting forces into their components (cohesion and friction) unless a drainage solution is involved; instead, a composite figure acting along the sliding surface should give the desired accuracy. If the latter method is used (see Fig.

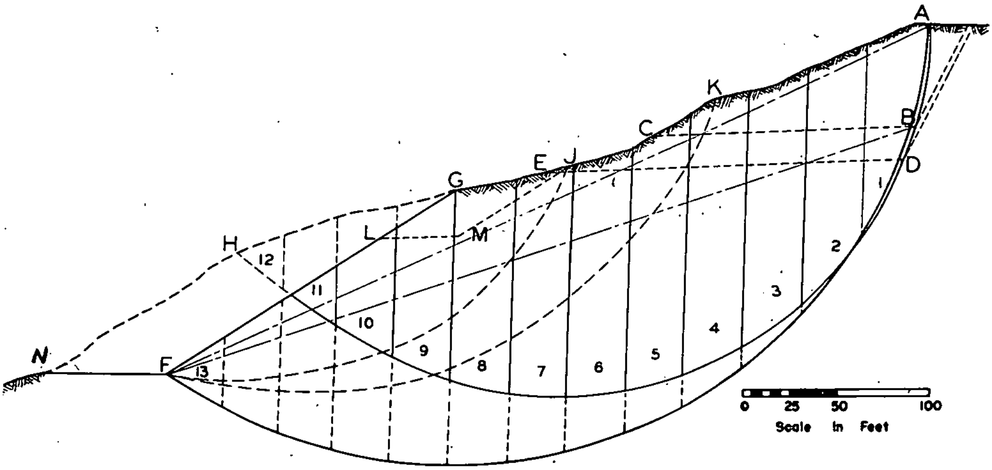


Figure 118. Determination of shearing resistance and design for excavation methods.

Table A. For use in determining original values of ϕ and c.

Segments Within Arc AH

	1	2	3	4	5	6	7	8	9	10	11	12	Total
Area (A), sq ft	2580	3730	4120	4450	4160	4140	3600	3430	2860	2230	1260	320	36 880
Normal (N_A)	1000	2500	3350	4000	4000	4100	3600	3400	2750	2050	1050	250	32 050
Tangential (T_A)	2400	2800	2400	1900	1150	600	0	−400	−800	−1000	−700	−200	8 150

Table B. For use in determining stability if toe of slide (FNG) is removed.

Segments Within Arc AF and Below Excavation Line GF

	1	2	3	4	5	6	7	8	9	10	11	12	13	Total
Area (A), sq ft	2350	3780	4460	4980	4780	4750	4730	4740	4110	3340	2290	1570	740	46,620
Normal (N_A)	650	1800	3200	3950	4650	4500	4600	4700	4100	3300	2200	1500	650	39,800
Tangential (T_A)	2250	2870	3050	2850	1350	1500	1000	400	−200	−500	−600	−500	−350	18,120

Table C. For use in determining stability if head (ABC) and toe (FNG) of slide are removed.

Segments Within Arc AF and Below Excavation Lines ABC and GF

	1	2	3	4	5	6	7	8	9	10	11	12	13	Total
Area (A), sq ft	610	2750	3400	4240	4620	4750	4730	4740	4110	3340	2290	1570	740	41,890
Normal (N_A)	250	1450	2400	3400	4100	4500	4600	4700	4100	3300	2200	1500	650	37,150
Tangential (T_A)	550	2350	2400	2500	2100	1500	1000	400	−200	−500	−600	−500	−350	10,650

Table D. For use in determining stability if larger head (ADE) and toe (FNG) of slide are removed.

Segments Within Arc AF and Below Excavation Lines ADE and GF

	1	2	3	4	5	6	7	8	9	10	11	12	13	Total
Area (A), sq ft	350	2030	2960	3780	4220	4510	4630	4740	4110	3340	2290	1570	740	39,270
Normal (N_A)	150	1100	2100	3100	3750	4250	4400	4700	4100	3300	2200	1500	650	35,400
Tangential (T_A)	300	1700	2100	2150	1900	1450	1000	400	−200	−500	−600	−500	−350	8,850

Table E. For use in determining stability if a straight 2:1 slope is excavated.

Segments Within Arc AF and Below 2:1 Excavation Line AF

	1	2	3	4	5	6	7	8	9	10	11	12	13	Total
Area (A), sq ft	2130	3360	3870	4450	4480	4680	4210	3930	3410	2640	1750	1350	500	40,760
Normal (N_A)	650	1850	2850	3600	4000	4400	4100	3900	3400	2600	1700	1300	400	34,750
Tangential (T_A)	1700	2800	2600	2600	2000	1550	900	400	–200	–350	–400	–550	–300	12,750

Table F. For use in determining stability if a straight 3:1 slope is excavated.

Segments Within Arc AF and Below 3:1 Excavation Line BF

	1	2	3	4	5	6	7	8	9	10	11	12	13	Total
Area (A), sq ft	670	2050	2820	3270	3480	3480	3380	3170	2770	2520	1650	1120	470	30,850
Normal (N_A)	400	1100	2000	2650	3100	3300	3300	3150	2750	2450	1600	1050	450	27,300
Tangential (T_A)	550	1700	1950	1900	1550	1100	750	300	–150	–350	–400	–400	–250	8,450

Table G. For use in determining stability if toe of slide (FNG) is removed (assuming arc JF is potential sliding surface).

Segments Within Arc JF and Below Excavation Line GF

	7	8	9	10	11	12	13	Total
Area (A), sq ft	765	1880	2160	1840	1550	910	465	9,570
Normal (N_A)	400	1400	1850	1700	1500	900	450	8,200
Tangential (T_A)	650	1250	1100	700	350	100	–100	4,050

Table H. For use in determining stability if a bench (JGLM) is cut.

Segments Within Arc JF and Beneath Bench JGLM

	7	8	9	10	11	12	13	Total
Area (A), sq ft	470	1100	1525	1800	1550	910	465	7,820
Normal (N_A)	100	850	1200	1700	1500	900	450	6,700
Tangential (T_A)	450	700	900	700	350	100	–100	3,050

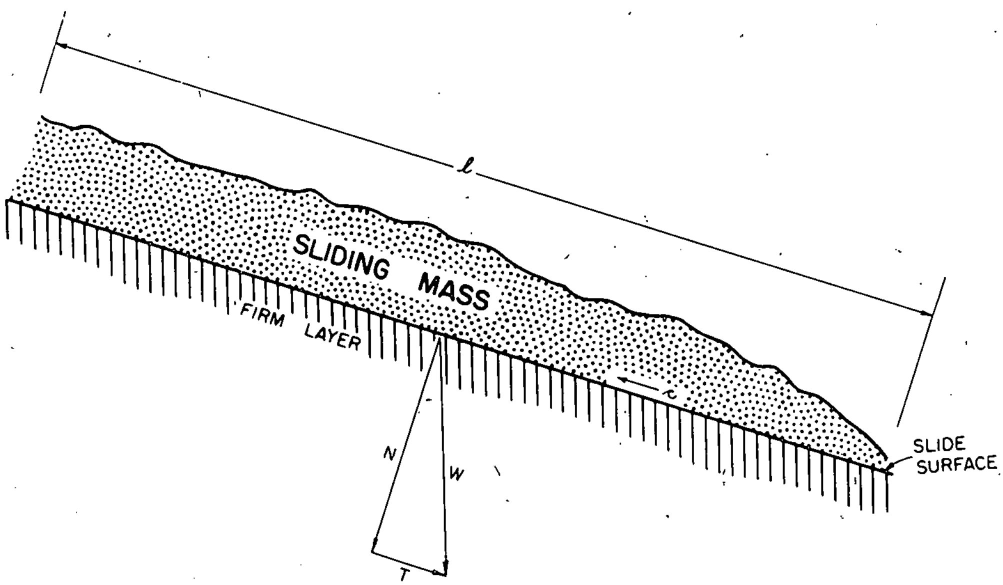


Figure 119. Forces acting on sliding mass, planar slide surface.

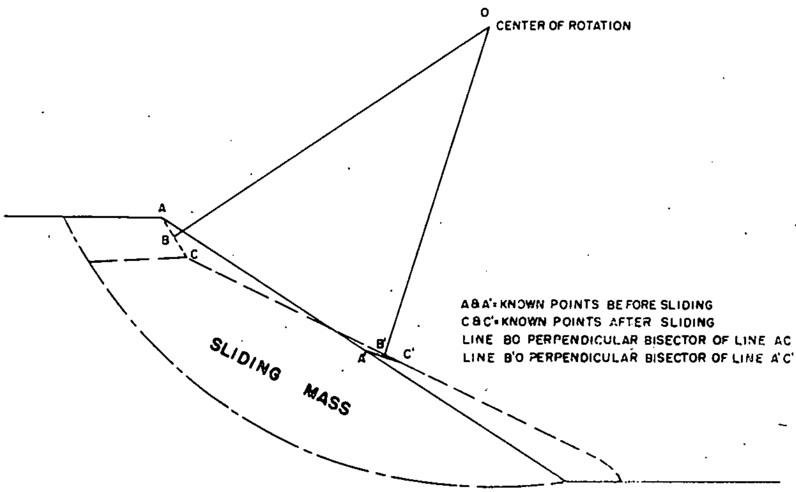


Figure 120. Method of locating center of rotation of slide mass.

121), moments can be balanced about the center of rotation and the average shearing resistance at failure is calculated from

$$s = \frac{W_1\, d_1 - W_2\, d_2}{l\, r} \qquad (10)$$

in which s is the average shearing resistance, composed of either, or both, cohesion, c, and $N \tan\phi$.

Use of Slide Data for Determining Shearing Resistance

As has been pointed out, the values for ϕ and c to be used in Eq. 2 can be obtained by laboratory shear tests on undisturbed samples taken from the zone of the shear surface (slip-plane). For computations where the pore pressures are to be ignored, however, technique of the type described in the preceding paragraph is recommended.

The method of slices (see Fig. 117 and discussion) can also be used in estimating the shearing resistance at the time of failure. A condition of safety factor = 1.0 is assumed. In the example that follows, such pairs of values for c and ϕ are used as to permit computation of either c or ϕ if the value of the other quantity is assumed. The average unit weight of the soil mass must be determined by sampling and measuring. An estimate of the location of the slip-surface is required, and the cross-section must be divided into increments as shown in Figure 118. In this and the following figures in this chapter the areas and the normal and tangential forces of each segment of the diagram are shown in tables. The components for normal (N_A) and tangential (T_A) forces are expressed in terms of area so as to simplify computations. Assuming that the average unit weight of the soil has been determined to be 125 lb per cu ft, that a 1-ft slice is used, and that prior to the excavation and the subsequent slide the arc AH represents a first estimate of the slip-surface, then the total normal force, in pounds, is

$$\sum_1^{12} N = \sum_1^{12} N_A\ (\gamma_m) \qquad (11a)$$

and the total tangential force, in pounds, is

$$\sum_1^{12} T = \sum_1^{12} T_A\ (\gamma_m) \qquad (12a)$$

in which

$\sum_1^{12} N_A$ = summation of the normal forces for increments 1-12, inclusive, in sq ft of area;

$\sum_1^{12} T_A$ = summation of tangential forces for increments 1-12, inclusive, in sq ft of area; and

γ_m = unit weight of the landslide mass, in lb per cu ft.

Thus, the following values are determined:

$$\sum_1^{12} N = 32{,}050 \times 125 = 4{,}010{,}000 \text{ lb}$$

$$\sum_1^{12} T = 8{,}150 \times 125 = 1{,}020{,}000 \text{ lb}$$

$$l = 505 \text{ ft (scaled)}$$

$$\text{f.s.} = 1.0 \text{ (assumed)}$$

With these four factors known, Eq. 2 now contains only two unknowns, ϕ and c. By assuming a value for one of these factors, the other may be computed. For example, assuming that $\phi = 5°$, and expressing Eq. 2 as

$$c = \frac{\text{f.s. } \Sigma T - \Sigma N \tan\phi}{l} \tag{13}$$

c has a value of

$$\frac{1{,}020{,}000 - (4{,}010{,}000 \times 0.0875)}{505 \times 1}$$

or 1,320 lb per sq ft.

Assuming that $\phi = 10°$, then c has a value of

$$\frac{1{,}020{,}000 - (4{,}010{,}000 \times 0.1763)}{505}$$

or 619 lb per sq ft.

Therefore, for future computations, the pairs of values to be used together are: $\phi = 5°$ and $c = 1{,}320$ lb per sq ft; $\phi = 10°$ and $c = 619$ lb per sq ft.

The shearing resistance thus computed indicates the strength needed to maintain equilibrium prior to any recent movement. Any other stability analyses made on the basis of these shear values will represent a stability with reference to that before recent movement. For example, a safety factor of 1.0 after treatment will mean that a condition will exist that is approximately as stable as the original hillside. For very stable slopes that may be encountered, this approach may represent an overdesign. For very unstable slopes, a greater relative safety factor may be desired for the corrective treatment.

The preceding method for estimating shearing resistance is of primary use for analysis of landslides that have occurred. The technique has been used on relatively stable slopes, but more danger of overdesign exists. The cross-section of the ground surface after movement (or the original ground surface for a potential slide area) is used, which represents an assumption that little or no change in shearing resistance has taken place. This will be true except where extensive movement has taken place, such as when sensitive clays are encountered. For these materials, the movement will resemble a flow rather than a slide and a stability analysis probably will not be attempted. Where sensitive clays are suspected (potential slide case), laboratory tests will disclose the truth very quickly.

The fact that the shearing resistance does not change radically in a slump-type failure (except where sensitive clays are involved) may not be readily acceptable. However, if shear resistance computations are made for the slip-surface and ground line before movement and the results are compared with those for the ground line after movement, only a slight difference will be developed. In fact, minor changes in driving and resisting forces make such a result self-evident.

For relatively stable hillsides, the technique is slightly more involved. The probable slip-surface location must be considered in light of subsurface data on bedrock location, weak strata, etc. In general, the circle that progresses farthest uphill will produce the highest shearing resistance, which is the value desired; that is, if the resistance to shear

was lower, a failure would have occurred.

Another means of estimating the shearing resistance is to compute the value along a failed surface after excavation (arc AF in Fig. 118 after the slope FG has been cut). Possibilities exist for errors, however, if pore pressures can be expected to increase at a later date. The advantage to considering the slope as it existed before failure is that, within the lifetime of the slope, any pore pressures that have existed will be reflected in the stability of the natural slope.

Examples of the Method Applied to Specific Control Measures

Excavation

If the approximate location of the surface of rupture and the average shear strength characteristics are known, and if the influences of hydrostatic pressure are neglected, Eq. 2 can be used to estimate the effect of excavation anywhere on the slope.

Removal of Material at Head of Slide

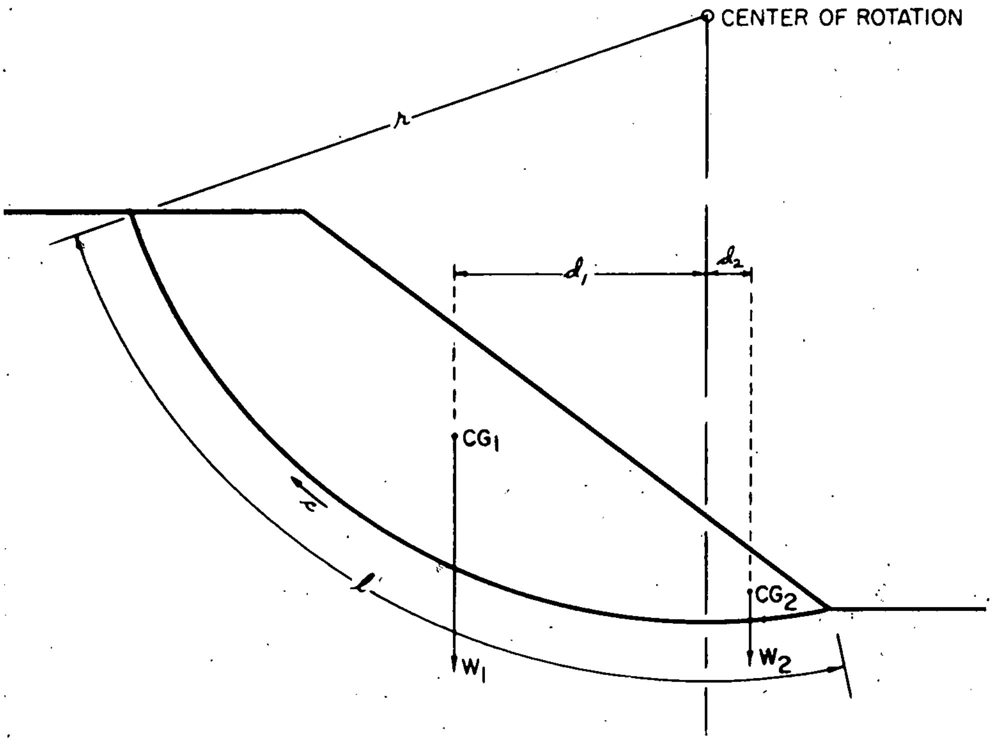
Considering first the removal of material from the head, one can use a technique consisting of a trial-and-error method to develop the desired safety factor. From Figure 118, an area (ABC) approximately 10 to 25 percent of the moving mass is selected. Eq. 2 can then be used to determine the safety factor after area ABC is removed and the lower slope is excavated to line FG. The stability will be improved due to the decrease of ΣT, but it will be lessened by decrease in the length of the slip-plane and the loss of forces normal to the slip-plane. However, as the major portion of the shearing force comes from the head, the net result of such excavations is an improvement of stability conditions. As shown in Figure 117, the ratio of T to N is relatively larger in the head region of a slide than it is in the middle and toe regions.

If the area selected (ABC, Fig. 118) does not produce a sufficient increase in the safety factor, a larger area is then tried (ADE). Conversely, if the increase is too great, a smaller area is considered for economic reasons.

The following is an example of the computations required, neglecting the effect of pore pressure. Referring to Figure 118 for a slump failure, and to Eq. 2, assume that undisturbed samples have indicated an average unit weight of 125 lb per cu ft. Also assume that laboratory tests or slide analyses indicate that $\phi = 5°$ and $c = 1{,}320$ lb per sq ft. For the first computations, the arc AF will be used as the slip-surface, and the effect on the stability by an excavation along FG will be determined. By graphical methods described previously, the values in Table B, Figure. 118, are computed. Using the method of computation given in the previous section, $\Sigma N = 4{,}970{,}000$ lb; $\Sigma T = 1{,}640{,}000$ lb; $l = 550$ ft; and f.s. $= \dfrac{(4{,}970{,}000 \times 0.0875) + (1{,}320 \times 550 \times 1)}{1{,}640{,}000} = 0.700$. To estimate the influence of removing the upper portion of the slide (area ABC), the following factors are determined for the slip-surface AF, with the upper and lower areas (ABC and FNG) excavated (Table C, Fig. 118): $\Sigma N = 4{,}650{,}000$ lb; $\Sigma T = 1{,}330{,}000$ lb; $l = 495$ ft; and f.s. $= \dfrac{(4{,}650{,}000 \times 0.0875) + (1{,}320 \times 495 \times 1)}{1{,}330{,}000} = 0.795$. The larger area at the head (ADE), together with the same toe removal (FNG), are then assumed to be removed (Table D, Fig. 118), the values becoming: $\Sigma N = 4{,}430{,}000$ lb; $\Sigma T = 1{,}100{,}000$ lb; $l = 475$ feet; and f.s. $= \dfrac{(4{,}430{,}000 \times 0.0875) + (1{,}320 \times 475 \times 1)}{1{,}100{,}000} = 0.923$.

Flattening the Slope

For a comparison of the foregoing removals of head and toe with the stability for straight slopes, assume that the 2:1 (horizontal:vertical) slope, AF, is cut (Table E, Fig. 118). The values then be-



CG_1, CG_2 = Centers of gravity, respectively, of driving and resisting masses;
W_1 = Weight of driving mass;
W_2 = Weight of resisting mass;
d_1, d_2 = Lever arms, respectively, of W_1, W_2;
c = Cohesion per unit of length and width of slice; and
l = Length of slip surface.

Figure 121. Determination of average shearing resistance by balancing of forces.

come: $\Sigma N = 4{,}350{,}000$ lb; $\Sigma T = 1{,}591{,}000$ lb; $l = 550$ ft; and f.s. =

$$\frac{(4{,}350{,}000 \times 0.0875) + (1{,}320 \times 550 \times 1)}{1{,}591{,}000} = 0.697.$$

The stability of the flatter 3:1 slope, BF (Table F, Fig. 118), would be: $\Sigma N = 3{,}410{,}000$ lb; $\Sigma T = 1{,}055{,}000$ lb; $l = 495$ ft; and f.s. =

$$\frac{(3{,}410{,}000 \times 0.0875) + (1{,}320 \times 495 \times 1)}{1{,}055{,}000} = 0.902.$$

Thus, removal of material near the top of the slide produces a greater influence on the stability than do the other corrective measures for which calculations were made. An economic comparison of excavation at the head of the slide over slope-flattening can also be made from the examples. The removal of area ADE requires only 785 cu yd of excavation per yard of slide length measured normal to direction of movement. The excavation of a 3:1 slope gives nearly the same safety factor, but requires removal of nearly 2½ times as much material, or 1,720 cu yd, for the same length of slide.

The effect of excess hydrostatic pressures (seepage forces) and the reduction in shearing resistance due to removal of the load were not considered in the foregoing. In the following section on drainage methods, a theoretical approach is suggested for considering the hydro-

static forces when sufficient data are available.

For a slide having a planar sliding surface (Fig. 119) it is obvious that removing the head has no more effect on the measure of stability as obtained from Eq. 2 than the same removal from any other place in the moving mass. This is true because the relation of N to T is the same at any point on the slide. One exception would be at the toe of the slide. If the cut slope were not sufficiently flat, a failure could develop on that slope and progress uphill, successively undermining the upper areas. There is another factor to be considered for toe excavation. If the slip-plane is curved at the toe but elsewhere straight, toe removal would be more severe in terms of undermining. The increased detrimental effect is caused by (a) a decrease in shearing resistance resulting from the removal at the toe of a mass which contributes to the frictional part of the shearing resistance; and (b) an increase in the tangential component (shearing force), as the values of T would be negative at the toe (see Fig. 117).

Benching of Slopes

Computations relative to the benching of slopes are essentially the same as described previously for other excavation methods. Because slopes containing cohesive materials are limited to a "critical height" (above which failure occurs) for a given angle, many too-steep slopes that are within the limits of their individual critical heights can be separated by a bench and thus be made stable.

The following is an example of designing benches in cohesive soil slopes. Referring to Figure 118, Table G, and using Eq. 2, assume that the slope FG has been excavated, that $\phi = 5°$ and $c = 1{,}320$ lb per sq ft[7], and that arc JF represents a potential slip-surface (Table G, Fig. 118). The values then are:

[7] It must, of course, be assumed also that the danger of sliding along arc AF and similar planes has been removed by excavation at the head of the slide or by other means.

$\Sigma N = 1{,}025{,}000$ lb; $\Sigma T = 506{,}000$ lb; $l = 260$ ft; and f.s. $= \dfrac{(1{,}025{,}000 \times 0.0875) + (1{,}320 \times 260)}{506{,}000} = 0.855$.

In order to determine the effect on the stability, assume that the bench, JGLM, is excavated (Table H, Fig. 118), the resulting values being: $\Sigma N = 837{,}000$ lb; $\Sigma T = 371{,}000$ lb; $l = 260$ ft; and f.s. $= \dfrac{(790{,}000 \times 0.0875) + (1{,}320 \times 260)}{371{,}000} = 1.12$.

The same slip-surface, JF, was used in both computations. More dangerous slip-surfaces (KF, for example), farther up the slope, also should be checked.

Drainage

The summary of "Drainage Methods" in Chapter Eight indicates five possible detrimental influences of water in a slide area. The factor of reduction in weight, the change in shearing resistance of the material at the slip-surface, and the effect on the shearing resistance due to geochemical and physical changes are difficult to evaluate quantitatively. Necessarily, a stability analysis based on these three factors will lie in the realm of conjecture until better techniques have been developed. In particular, the decrease in weight by the installation of drains is likely to show little influence on the stability as pictured by the safety factor.

However, the lowering of the ground water table or the elimination of excess hydrostatic pressures (or seepage forces) can materially influence the value of the safety factor. In this respect, subdrainage measures can be evaluated analytically.

Two factors that are difficult to determine are: (a) drain spacing, and (b) the prediction of the water table or piezometric surface after the drains have become effective. At the moment, trial-and-error methods must be used and field observations are needed. For example, in clay soils the drain spacing might need to be as close as 10 to 25 ft (perpendicular to direction of landslide movement).

$\mu 1_A$ = Neutral pressures before drainage (area);
$\mu 2_A$ = Neutral pressures after drainage (area);
h_1 = Height of piezometric surface above slip-plane before drainage; and
h_2 = Height of piezometric surface above slip-plane after drainage (bottom of pipe).

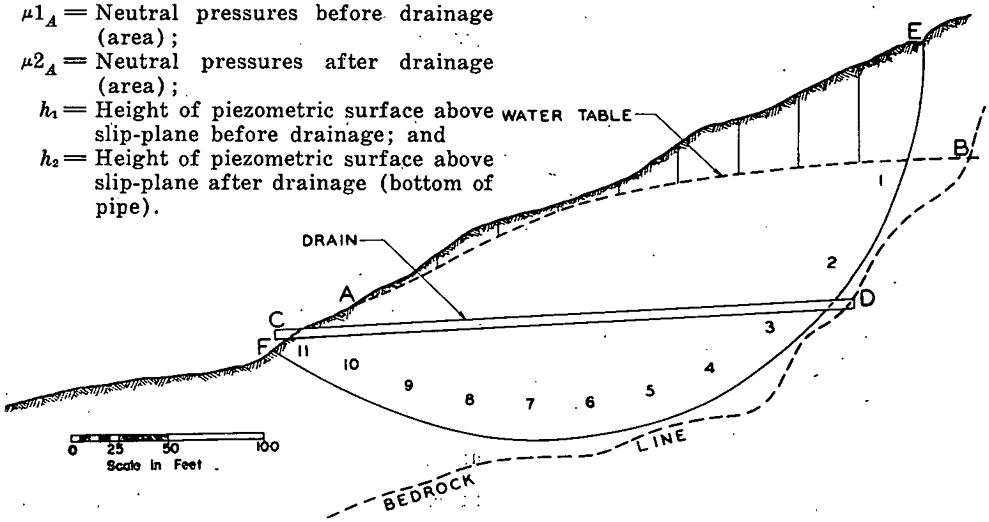


Figure 122. Analysis of horizontal drainage installation.

Table I. For use in determining stability if a drain CD is installed.

	Segments Within Arc EF											
	1	2	3	4	5	6	7	8	9	10	11	Total
Area (A), sq ft	2460	3730	4120	4450	4150	4100	3650	3280	2300	1340	390	33,970
Normal (N_A)	900	2500	3300	4000	3900	3950	3600	3200	2150	1150	300	28,950
Tangential (T_A)	2300	2800	2450	1850	1100	500	–200	–700	–800	–700	–250	8,350
Average h_1	30	80	98	119	125	123	115	96	70	45	18	
Average l	65	50	40	38	32	32	32	32	35	35	30	
$\mu 1_A$, sq ft	1950	4000	3920	4520	4000	3930	3680	3070	2450	1575	540	33,635
Average h_2	0	2	23	44	55	61	60	56	45	30	13	
$\mu 2_A$, sq ft	0	100	920	1670	1760	1950	1920	1790	1575	1050	390	13,125

For the drainage of permeable layers, a 25- to 50-ft spacing might be adequate. Complete interception of all water, of course, calls for a system of drains that is fitted to the local geologic conditions, rather than one that follows a set geometric pattern.

In the following example, four simplifying assumptions are involved (Fig. 122), as follows:

1. The new water table will lie at the elevation of the drain.
2. No change in ϕ and c occurs with the lowering of the water table.
3. There is no reduction in the average unit weight of the soil.
4. A static condition (no seepage) exists for the conventional flow line.

The first three do not represent a conservative approach, and estimated adjustments can be made if desired. The fourth eliminates the development of a flow net, and a conservative answer is obtained unless deep flow exists. As the water table is drawn in Figure 122 it is obvious that some seepage will actually take place at A, where the water table intersects the surface. The assumtion that no seepage exists is made for problem simplification, but is justifiable. The quantity of flow is not important and static hydrostatic pressure conditions will nor-

mally be more severe than those for seepage.

Figure 122 and its table are used for the following examples and the following conditions are known or assumed. The average unit weight of the soil, γ_s, is 125 lb per cu ft, the drain CD has been installed, the line AB represents the highest original water table or piezometric surface to be anticipated, and $\phi = 10°$.

$$\sum_1^{11} N = \sum_1^{11} N_A \gamma_m \qquad (11b)$$

$$= 28{,}950 \text{ x } 125 = 3{,}620{,}000 \text{ lb}$$

$$\sum_1^{11} T = \sum_1^{11} T_A \gamma_m \qquad (12b)$$

$$= 8{,}350 \text{ x } 125 = 1{,}040{,}000 \text{ lb}$$

$$\mu_1 = \mu_{1A} \gamma_\omega \qquad (14a)$$

$$= 33{,}635 \text{ x } 62.4$$
$$= 2{,}100{,}000 \text{ lb (before drainage)}$$

$$\mu_2 = \mu_{2A} \gamma_\omega \qquad (14b)$$

$$= 13{,}125 \text{ x } 62.4$$
$$= 819{,}000 \text{ lb (after drainage)}$$

Also, $l = 465$ ft, $\Sigma(N - \mu_1) = 1{,}520{,}000$ lb, and $\Sigma(N - \mu_2) = 2{,}801{,}000$ lb.

In order to estimate shear characteristics, a safety factor of 1.0 is assumed for the original hillside and Eq. 13 gives

$$c = \frac{1{,}040{,}000 - (1{,}520{,}000 \text{ x } 0.1763)}{465}$$

= 1,660 lb per sq ft. For the influence on stability produced by the drain, Eq. 4 can be used to find the factor of safety, or f.s. =

$$\frac{(2{,}801{,}000 \text{ x } 0.1763) + (1{,}660 \text{ x } 465)}{1{,}040{,}000}$$

= 1.22.

Thus, lowering the water table under the stated conditions produces a safety factor of 1.22 as compared to 1.0 without drainage.

The preceding example illustrates the importance of ϕ with regard to drainage solutions. Unless ϕ is at least 10° to 20° the influence of drainage may not be great. The safety factor for a cohesive material is given by Eq. 8. For such materials, the benefits from drainage must result from loss of weight and increase in shearing resistance. Once more using Figure 122, but this time assuming that $\phi = 0°$ and that the unit weight can be reduced from 125 to 115 lb per cu ft, the value for cohesion can be obtained by expressing Eq. 8 in terms of c for a safety factor of 1.0, giving $c = \dfrac{1{,}040{,}000}{465}$ = 2,240 lb per sq ft. For the assumed loss of weight accomplished by drainage, and assuming no increase in shearing resistance, f.s. $= \dfrac{1{,}040{,}000}{960{,}000} = 1.09$. Furthermore, in order to improve the safety factor to a value of 1.25, the increase in shearing resistance required would be $c = \dfrac{960{,}000 \text{ x } 1.25}{465} = 2{,}580$ lb per sq. ft. Laboratory testing could be used to estimate whether the assumed decrease in unit weight and increase in shearing resistance are reasonable for the type of soil and for the drainage characteristics of the landslide.

Restraining Structures

For a quantitative approach to determination of the size of a restraining structure, an estimate is needed of the force against the restraining device, the shearing resistance along the slip-surface, and the resistance afforded by the retainer. For flow conditions, there will be very little resistance along any surface of separation that develops and the forces against a structure will be relatively large. For slides, if an estimate can be obtained of the sizable resistance along the slip-surface at the time of movement, the factors which must be known are the thrust against the retaining device and the point and direction of its application.

The degree of stability (or relative safety factor) should be at least 1.5 for restraining structures. However, values as low as 1.25 may be required due to economic considerations.

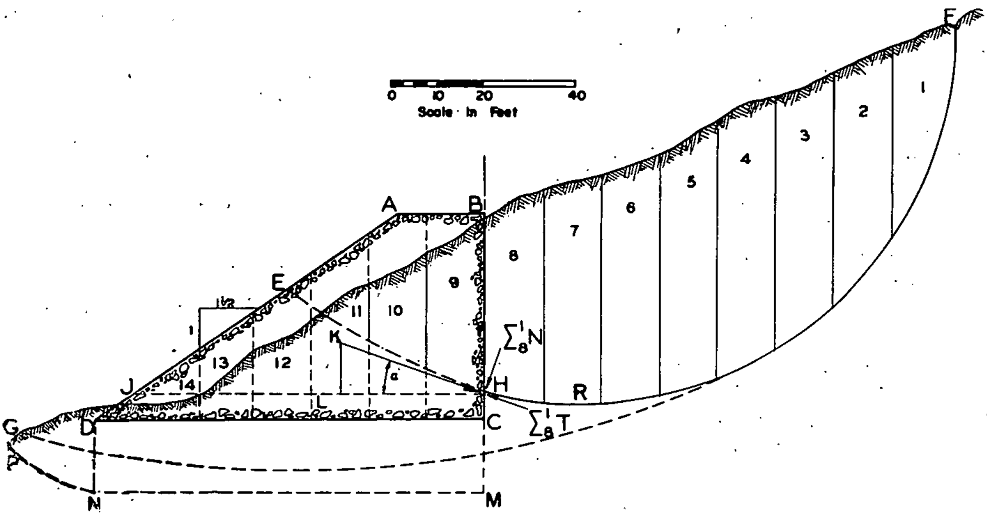


Figure 123. Design of a rock buttress.

Table K. For use in determining the shearing resistance, assuming a slip plane EF in the original hillside.

Segments Within Arc EF

	1	2	3	4	5	6	7	8	9	10	11	Total
Area (A), sq ft	385	465	640	680	650	620	550	545	335	195	60	5,125
Normal (N_A)	160	400	540	620	630	615	550	540	330	180	55	4,620
Tangential (T_A)	350	420	350	280	155	75	0	–35	–70	–70	–20	1,435

Table L. For use in determining stability of a rock buttress if shear failure develops along arc EH through the buttress.

Segments Within Arc EH

	9	10	11	12	Total
Area (A), sq ft	435	420	200	60	1,115
Normal (N_A)	425	390	185	30	1,030
Tangential (T_A)	–115	–170	–80	–45	–410

Construction of this type will frequently involve special procedures in order to permit the near-vertical, BH. This will be particularly true if the slope is relatively unstable.

Buttresses

For buttresses, failure may develop in one of three ways (see Fig. 123), as follows:

1. Shear through the buttress (along line HE or HJ).
2. Foundation failure beneath the buttress (along arc FG).
3. Friction or shear failure between the buttress and the foundation (along line CD).

Rock buttresses can be constructed on either a solid rock foundation or on soil. Where it is possible or practicable, a solid rock foundation should be obtained, as foundation failures through rock are unlikely. Thus, the possibilities of failure at the contact surface between buttress and foundation are minimized, and the costs of adequately providing against a foundation failure beneath the buttress are eliminated.

For a rock buttress with foundations on soil, it will be necessary to determine the value of the shearing resistance of the underlying soil. This can be done by laboratory testing or by the evaluation of performance in a manner similar to the determination of the shearing resistance elsewhere in the slide area. The shearing resistance of the soil in the foundation material is needed both for a check on stability against a foundation failure and for the estimate of the stability along the contact surface between the buttress and the foundation.

The primary difference between a rock buttress and an earth buttress lies in the distinction between a granular, non-cohesive material (rock buttress) and a fine-grained, cohesive soil (earth buttress). The granular material develops shearing resistance, due to friction, which is proportioned to the weight of the material above the shear plane. A cohesive material develops a shearing resistance from cohesion (which is not materially affected by weight above the shear plane) and from friction. The shearing resistance of a clay soil develops primarily from cohesion, with little or no friction benefit. This fact has led to the $\phi = 0°$ approach, a simplification that may be warranted in many instances. However, the frictional component may be tangible, with values ranging from 5° to 15° for clays and silty clays.

Rock Buttress. — Consider first a rock buttress. The slip-surface can be extended through an assumed buttress (line HE in Fig. 123) and a stability analysis used to determine the degree of stability. Such an analysis is difficult because trial-and-error methods are involved and because curved slip-surfaces must be faced. The curved slip-surface complication can be avoided without serious error by assuming a straight line extension of the slip-surface through the buttress. Normally, it will be easier to determine the size of the buttress on a preliminary basis by checking the stability against a friction or shear failure at the base of the buttress (line JH). Stability with reference to a shear failure through the buttress and the relative stability against a foundation failure beneath the buttress can then be checked.

The location of the buttress with reference to the toe of the movement is related to the position and shape of the slip-surface. An effective location for the back of the buttress is near that part of the slip-surface that is tangent to the horizontal (point R). It is recommended that for the first computations one edge of the buttress (line BC) be placed so that the tangent to the slip-plane at point H makes an angle of less than 10° with the horizontal.

Using the principles explained previously for excavation methods, one can estimate the summation and direction of application of the tangential and normal stresses at any point along the slip-surface. In order to obtain a preliminary estimate of the size of the buttress, the summations are made for the upper portion of the mass between the top of the slide and the upper edge of the buttress (increments 1-8, inclusive, in Fig. 123). To determine the amount of resistance required from the buttress, a safety factor for design is established, and the following form of Eq. 3 is used:

$$P_R = \text{f.s.} \left(\sum_a^b T + \sum_b^c T \right) - \sum_a^b N \tan\phi - c_s l_{a-b} \quad (15a)$$

or

$$P_R = \text{f.s.} \sum_a^b T - \sum_a^b N \tan\phi - c_s l_{a-b} \quad (15b)$$

in which

c_s = cohesion in the natural soil;
P_R = resistance required from the buttress, in pounds.

Eqs. 15*a* and 15*b* represent the general equation where *a*, *b*, and *c* are any three increments between which summations are desired. For example, in Figure 123

$a = 1$, $b = 8$, and $c = 14$. For preliminary estimates, the value of $\sum_b^c T$ can be assumed to equal zero (Eq. 15a), which will give a conservative result (algebraic value is normally negative).

Given the summation of the shearing resistance required from the buttress along the extension of the slip-surface (EH in Fig. 123) and the direction of its application (tangent to the slip-plane at point H), the value for P_R represents the summation of the resistance within the buttress (increments 9 to 14, inclusive). The source of the shearing resistance will be the frictional component of the weight of the mass for (a) shear surfaces within the buttress, and (b) at the contact between the rock buttress and bedrock. Most of the available resistance will be friction. The resistance offered by the tangential component for a nonhorizontal shear plane is included in $\sum_b^c T$. By graphics, the horizontal thrust against the buttress (required frictional resistance) can be determined (LH) so as to resist a shear failure through the buttress (along line HJ), or can be obtained by multiplying P_R by $\cos\alpha$. Therefore, the following equations can be used to express the resistance required from the buttress per unit of width.

For horizontal shear through the buttress:

$$P_R \cos\alpha = \gamma_B A_B \tan\phi_B \qquad (16a)$$

or

$$A_B = \frac{P_R \cos\alpha}{\gamma_B \text{ x } \tan\phi_B} \qquad (16b)$$

in which

α = angle formed by the tangent to the slip-surface and the horizontal at back of buttress;

γ_B = unit weight of the buttress, in lb per cu ft;

A_B = area of the buttress, in sq ft; and

ϕ_B = angle of internal friction for the rock in the buttress.

For shear failure at contact — soil foundations:

$$P_R \cos\alpha = \gamma_B A_B \tan\phi_s + c_s\left(\frac{A_B}{h} + \frac{1.5h}{2}\right) \qquad (17a)$$

$$A_B = \frac{P_R \cos\alpha - \frac{1.5h}{2}c_s}{\gamma_B \tan\phi_s + \frac{c_s}{h}} \qquad (17b)$$

in which

ϕ_s = angle of internal friction for the foundation soil;

c_s = unit cohesion of the natural soil; and

h = height of buttress, in ft.

Assuming that the buttress is to be constructed with one vertical face (BC) and the other (AD) on a 1.5:1 (horizontal to vertical) slope, the length of the bases can be expressed as

$$\text{Length of bases} = \frac{A_B}{h} \pm \frac{1.5h}{2} \qquad (18)$$

After obtaining a preliminary estimate of the dimensions of the buttress, the stability with reference to the other conditions of failure should be determined. Assuming that Eqs. 15a and 16b have been used for the previously determined values, one needs to check for the degree of stability with reference to a shear failure through the buttress (line HE). The values for $\sum_a^b T$ and $\sum_a^b N$ between the top of the slide and the edge of the buttress are obtained and the $\sum_b^c T$ and $\sum_b^c N$ within the buttress and above the slip-plane are then determined. From these values, the safety factor can be determined by

$$\text{f.s.} = \frac{\sum_a^b N \tan\phi_s + c_s l_{a-b} + \sum_b^c N \tan\phi_B}{\sum_a^b T + \Sigma \sum_b^c T} \qquad (19)$$

If the safety factor is too low, additional material will be needed above the shear plane. The needed amount can be estimated from Eq. 3 by trial and error. An alternate solution would be a new location for the buttress.

To determine the safety factor relative to a foundation failure (soil foundations), a slip-plane extending below the buttress should be studied (FG in Fig. 123). Unless drilling has indicated the presence of a very weak stratum, the same values for shearing resistance can be used as for the upper portions of the slide, and a circular slip-plane assumed.

The following is an example of the computations (Table K, Fig. 123) for a rock buttress with the unit weight of the soil and buttress material assumed equal to 125 and 100 lb per cu ft, respectively. If one assumes no hydrostatic pressures present, a value of $\phi = 10°$, and a safety factor of 1.0 for the original hillside, the cohesion can be determined as follows: $\sum_1^{11} N = 577{,}000$ lb; $\sum_1^{11} T = 179{,}000$ lb; $l_{1-11} = 188$ ft; and (Eq. 2) $c = \dfrac{179{,}000 - (577{,}000 \times 0.1763)}{188}$ $= 410$ lb per sq ft.

To obtain an estimate of resistance required from the buttress for a safety factor of 1.5, $\sum_1^{8} N = 507{,}000$ lb; $\sum_1^{8} T = 199{,}000$ lb; $l_{1-8} = 149$ ft; and (Eq. 15*b*) $P_R = 1.5\ (199{,}000) - (507{,}000 \times 0.1763) - (410 \times 149) = 148{,}000$ lb.

For preliminary estimates of the size of the buttress, and assuming that the buttress will be founded on bedrock, Eqs. 16*b* and 18 can be applied with the following data: $\alpha = 20°$; $\phi_B = 35°$; $h = 40$ ft; $P_R \cos\alpha = 148{,}000 \times 0.9397 = 139{,}000$; and (Eq. 16 *b*) $A_B = \dfrac{139{,}000}{100 \times 0.700}$ $= 1{,}985$ sq ft.

To determine the length of the bases for a buttress with a rear vertical face and a front slope of 1.5:1 (horizontal: vertical) (Eq. 18), bases $= \dfrac{1{,}985}{40} \pm \dfrac{60}{2}$; therefore, upper base = 20 ft, and lower base = 80 ft.

The stability with reference to a shear failure through the buttress (EH), can be checked by Eq. 19 (Table L, Fig 123): $\sum_9^{12} N = 103{,}000$ lb; $\sum_9^{12} T = -41{,}000$ lb; and (Eq. 19) f.s. = 1.41.

To increase this value, another location of the buttress can be selected or the height can be increased, and the stability estimates recomputed.

For conditions where bedrock is not encountered, Eq. 17*b* can be used for obtaining the dimensions required for the buttress: $A_B = \dfrac{139{,}000 - \dfrac{1.5 \times 40 \times 410}{2}}{100 \times 0.1763 + \dfrac{410}{40}}$ $= 4{,}545$ sq ft; length of lower base $= \dfrac{4{,}545}{40} + \dfrac{1.5 \times 40}{2} = 144$ ft; and length of upper base $= \dfrac{4{,}545}{40} - \dfrac{1.5 \times 40}{2} = 84$ ft.

An alternate to the use of the larger buttress that is required with soil foundations would be one with deeper foundations (MN). Although drainage would be required, additional resistance would be afforded along NP, and the slip-plane FG (failure benenath buttress) would be lowered.

Earth Buttress. — The procedure for designing an earth buttress is quite similar to that described for one composed of rock. Eq. 15*a* can be used to determine the needed resistance. The general procedure described for rock buttresses can be followed, or a method based on assumed dimensions is quite useful. A buttress of approximately one-third to one-half the volume of the mass to be retained is selected. Laboratory tests on remolded samples of the earth buttress material will produce the necessary values for ϕ and c. The resistance produced by the buttress is then checked against the required resistance by extending the slip-plane through the buttress and by use of the following equation (see Fig. 123):

$$P_R = \text{f.s.} \sum_b^c N \tan\phi_b + c_b l_{b-c}) \qquad (20)$$

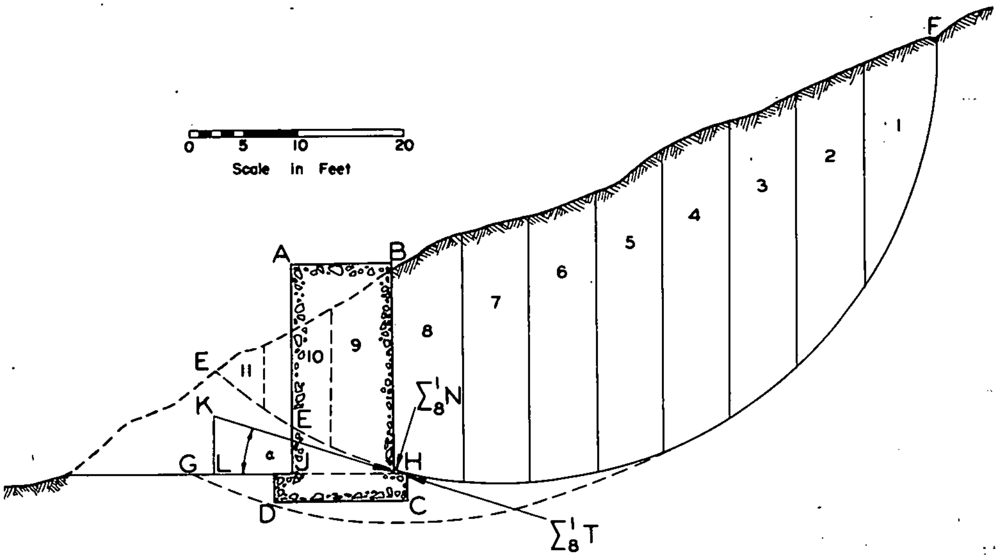


Figure 124. Design of a wall.

Table M. For use in determining stability of a crib wall ABCD. Construction of this type will frequently involve special procedures in order to permit the near-vertical cut, BH. This will be particularly true if the slope is relatively unstable.

	Segments Within Arc EF											
	1	2	3	4	5	6	7	8	9	10	11	Total
Area (A), sq ft	100	120	160	170	165	160	140	140	85	50	20	1,310
Normal (N_A)	40	100	135	155	160	155	140	135	80	45	15	1,160
Tangential (T_A)	90	105	90	70	40	20	0	−10	−20	−20	−5	360

in which

c_b = cohesion for soil in buttress, in lb per sq ft;

ϕ_b = angle of internal friction for soil in the buttress; and

l_{b-c} = length of slip-plane within the buttress, in ft.

If the buttress selected does not produce sufficient shearing resistance, a larger buttress should be considered; if too much resistance is developed, estimates for a smaller one can be made.

In order to determine the adequacy of the buttress with regard to a failure at the contact with the foundation soil, Eq. 17a can be used. The shear characteristics of the foundation material can be assumed to be equal to those for the natural hillside soil, unless obvious differences exist. In the latter event, previously described methods for determining shearing resistance will be required. The failure beneath the earth buttress is checked in the same manner as for a rock buttress (FG of Fig. 123).

Cribs and Retaining Walls

In determining the size or adequacy of a crib or retaining wall, the same basic equations are employed as for the buttresses. The required resistance is obtained by using Eq. 15a. This type of structure must be checked for the following types of failures (Fig. 124):

1. Shear failure through the wall (arc EH or line JH).
2. Foundation failure beneath the wall (arc FG).

3. Friction or shear failure at contact between the wall footer and the foundation (line CD).
4. Overturning.

The possibilities of a foundation failure are checked by the same technique as explained in the foregoing for a rock buttress, and the procedure will not be discussed further. It should be remembered that if the wall is founded in or on bedrock, the possibilities of a foundation failure are remote.

For shear failure of the wall and overturning, the reader is referred to structural texts for the design of a retaining wall. For determining the force required due to inherent instability of the slide area, Eq. 15*a* is used. Knowing the direction and magnitude of the force, KH, that is acting at the slip-plane, normal design methods can be used. The resisting force can be assumed to be applied at the one-third point between the slip-surface and the top of the wall.

For crib walls, shearing resistance is primarily developed in the material used to backfill the crib. The shear values, either for rock or soil, can be determined by shear tests in the laboratory. Soil is rarely, if ever, desirable for the backfill. Although some resistance can be attributed to the interlocking members of the crib, such resistance will be relatively small and can, for safe and conservative design, be disregarded.

Overturning of a crib wall is not a problem if standard recommendations for batter are followed. A combination shear failure and overturning may develop, but the lack of tensile strength precludes normal overturning. The resistance to a friction or shear failure at the contact between the footer and the foundation can be determined as for a buttress, using a form of Eq. 15*a*.

For bedrock foundations

$$P_R \cos\alpha = W \tan\phi_F \tag{21}$$

and for soil foundations

$$P_R \cos\alpha = W \tan\phi_s + c_s l_w \tag{22}$$

in which

W = weight per foot of wall length, in lb;

$\tan \phi_F$ = coefficient of friction between wall and foundation material; and

l_w = length of slip-surface beneath the wall, in ft.

Another estimate of the stability of a retainer can be made if the device is to replace an excavation in a reasonably stable slope. By comparing the natural resistance of the soil removed to that afforded by the retaining device, relative stability can be determined. If a slight movement has developed but halted, or if a stable slope is excavated, the technique may be useful. The approach consists of determining the toe portion of the shearing resistance along a surface of rupture by the previously described methods. The shearing resistance of the excavated soil and, consequently, that required from the retainer is

$$P_R = \text{f.s.}\left(\sum_b^c N \tan\phi_s + c_s l_{b-c} - \sum_b^c T\right) \tag{23}$$

Thus, the stability with reference to the original soils is equal to the ratio of the retainer resistance to the soil resistance before excavation; that is, if a safety factor of 1.5 is used, the retainer will produce a state of stability (as reflected in the safety factor) 1.5 times that developed by the soil before excavation. The over-all safety factor of the hillside may be less than 1.5 in such cases, however. Obviously, this method of computing stability is useful only if the natural hillside appears relatively stable.

The following example of the design of a crib wall refers to Figure 124. Although in practice the wall will be constructed on a batter (approximately 1:6), computations are somewhat simplified and the results are conservative if one assumes a vertical wall. The unit weight of the soil is assumed to be 125 lb per cu ft and the unit weight of the rock backfill for the crib wall to be 100 lb per cu

ft. The length of the slip-surface EF shown in Figure 124 is 94 ft. To determine the shearing resistance under an assumed safety factor of 1.0, and a value of $\phi = 10°$ (Table M); $\sum_1^{11} N = 145{,}000$ lb; $\sum_1^{11} T = 45{,}000$ lb; and $c_s = 45{,}000$ $\dfrac{-(145{,}000 \times 0.1763)}{94} = 207$ lb per sq ft.
To determine P_R (the resistance required from the wall) for a safety factor of 1.5 (relative to the original stability), $\sum_1^8 N = 127{,}000$ lb; $\sum_1^8 T = 51{,}000$ lb; $l_{1-8} = 75$ ft; and (Eq. 15b) $P_R = 1.5(51{,}000) - (127{,}000 \times 0.1763) - (207 \times 75) = 38{,}600$ lb.
The weight required from the wall if the angle of internal friction of the backfill is 35° and α (Fig. 124) is 22° is:

$$W = \frac{-P_R \cos\alpha}{\tan\phi_w} \tag{24}$$

$$= \frac{38{,}600 \times 0.927}{0.700} = 51{,}100 \text{ lb}$$

in which

ϕ_w = angle of internal friction on the backfill material of the crib wall.

The weight available per foot of height can be estimated for a single-cell crib, closed face, with standard 6-in. by 8-in. by 6-ft concrete stretchers. Each of the stretchers weighs 300 lb. If one assumes eight per foot of height, the total weight available is 2,400 lb, or 400 lb per foot of length. The weight of the rock backfill is approximately 600 lb per foot of length for each foot of height. Therefore, the crib wall will weigh 1,000 lb per foot of height per foot of length.

If a double cell is used for 12 ft and the remaining 8 ft of wall height is a single cell, the weight of the double-cell portion is 12 x 2 x 1,000 = 24,000 lb, that of the single-cell portion is 8 x 1,000 = 8,000 lb, and the total is 32,000 lb. Thus, an additional 19,000 lb of weight is required to produce a safety factor of 1.5. To determine the safety factor of the preceding design (Eq. 19), f.s. = 1.18. To use a crib wall in this instance, a backfill between the wall and the slope would be desirable. Furthermore, a factor of safety as high as 1.5 may be impractical in this case.

PILING

For analyzing the benefits derived from piling, consideration is given to the two basic types of piling installations that are currently employed: one type anchors the piling to an unyielding foundation, whereas the other drives to refusal, and may or may not be properly fixed at the surface of rupture. The use of the latter type will not be generally acceptable except as an expedient. The resistance developed at the foot of such a pile cannot be great, but in cases involving small quantities of material, nonfixed piles have proven adequate for an extended period of time.

For a piling installation considered as fixed, the piles should penetrate one-third their total length into a stable foundation material. Where the foundation is bedrock and the piling is grouted at the toe, the depth of anchorage can be reduced to one-fourth the total pile length. Fixed piling fails in one of the following ways:

1. Shear through the pile.
2. Flexure through bending by cantilever action.
3. Soil shear around and past the piles.
4. Foundation failure beneath the piles.

Foundation failures of the type that would follow the arc FG beneath the pile (Fig. 125), have been discussed by Krynine (1931) and Hennes (1936). The increase in safety factor for conditions of a foundation failure produced by piling can be checked in the same manner as for failures beneath a buttress or a wall by the use of Eq. 2 or Eq. 4.

The preliminary spacing of the piling can be determined on the basis of the bending moment developed. The thrust

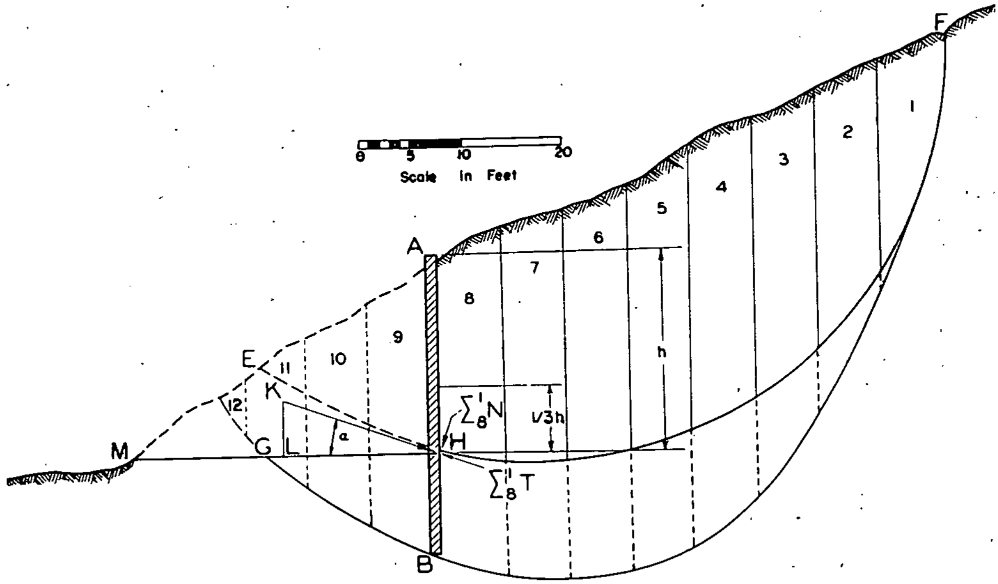


Figure 125. Design of piling.

Table N. For use in determining stability of a piling installation if foundation failure should occur along arc FG. The original slip-plane is arc EF.

	Segments Within Arc FG												
	1	2	3	4	5	6	7	8	9	10	11	12	Totals
Area (A), sq ft	115	160	230	260	260	230	210	195	135	90	65	5	1,965
Normal (N_A)	45	80	150	230	240	225	200	180	120	80	50	–	1,500
Tangential (T_A)	110	140	170	140	80	10	−30	−65	−55	−45	−45	5	405

(P_R) against the piling is determined by Eq. 15*a* for the slip-surface FE, and the horizontal force HL is obtained graphically or from the expression P_R cosα. The slip-surface FE is determined to be the most critical slip-surface — the one for which the piling is being installed. Furthermore, P_R is the shearing force exerted on the piling at the slip-surface. If the piling is assumed fixed in the area BH, and if it is further assumed that the loading diagram on the pile is triangular, a cantilever exists with the total load equal to the shearing force. This load is then assumed to be acting one-third the distance between points H and A. The moment can be expressed as

$$M = P_R \cos\alpha\, D \frac{h}{3} 12 \qquad (25)$$

in which

M = maximum bending moment per foot of width, in in.-lb;
h = length of pile above surface of rupture, in ft; and
D = center-to-center spacing of piling divided by the number of lines of piles, in ft per pile.

Having obtained the bending moment, the size of the pile can be determined by the method appropriate to the type of material being used. Eq. 25 assumes no resistance from the surrounding soil, so that the size of the pile obtained will be

conservative in some instances. However, for the piling to be effective, full bending moment is very likely to develop.

For preliminary design purposes, an estimate of the pile spacing should be made in order to determine the total thrust against a pile. If too large a pile is required, the spacing should be reduced by trial and error.

To determine the resistance to shear developed by the piles at the surface of rupture for a unit slice, the following form of an equation applied by Hennes (1936) to this problem may be used:

$$v_p = \frac{A_p f_v}{D} \tag{26}$$

in which

v_p = shearing resistance of the pile installation per foot of slide width, in lb per ft;
A_p = cross-section area of the pile, in sq in.; and
f_v = allowable shearing stress for the piles, in lb per sq in.

The ratio of v_p to $P_R \cos\alpha$ is the stability of the pile with reference to shear.

Hennes (1936) has also suggested use of an equation for determining the stability with reference to a shear failure of the soil around and past the pile. A form of that equation is

$$S_s = \frac{2\ c\ h\ d}{D} \tag{27}$$

in which

S_s = shearing resistance of the soil per foot of slide width, in lb per ft;
c = cohesion of the soil, in lb per sq ft;
h = height from surface of rupture to ground surface, in ft; and
d = diameter of pile, in ft.

The ratio of S_s to $P_R \cos\alpha$ (total force per foot against pile, from Eq. 15*a*) is the relative stability with reference to soil shear around the pile.

The following is a typical example for steel piling (Fig. 125) assuming, (a) that P_R, c, and ϕ have been determined in the same manner as for the example for a crib wall, and (b) that the shearing resistance of the material downslope from the pile is neglected:
$P_R = 38{,}600$ lb; $\phi = 10°$; $\alpha = 22°$; $c = 207$ lb per sq ft; $f_s = 40{,}000$ lb per sq in.; $f_v = 25{,}000$ lb per sq in.; $h = 20$ ft; and $l_{1-8} = 75$ ft. Assume that four lines of piling are driven with an 18-in. center-to-center spacing in each line, determine the size of beam necessary to resist a bending moment (Eq. 25, in which $D = \frac{1.5}{4} = 0.375$ ft) of 38,600 x 0.927 x 0.375 x 80 = 1,074,000 in.-lb.
For steel I-beams and from the AISC Handbook,
$f_s = \frac{M\ c}{I} = \frac{M}{S}$, and in this case $S = \frac{1{,}074{,}000}{40{,}000} = 26.8$.
For a 10-in. beam weighing 25 lb per ft, $A_p = 7.35$, $S = 26.4$, and $f_s = \frac{1{,}074{,}000}{26.4} = 40{,}680$ lb per sq in.
To determine the shearing resistance of the pile (Eq. 26), $v_p = \frac{7.35 \times 25{,}000}{0.375} = 490{,}000$ lb per ft, and the factor of safety is

$$\text{f.s.} = \frac{v_p}{P_R \cos\alpha} = \frac{490{,}000}{35{,}700} = 13.7 \tag{28}$$

To determine the stability with reference to shearing of the soil around the pile, laboratory tests on undisturbed samples from the piling area should be conducted. Assuming such tests produced a value of 750 lb per sq ft for c, by Eq. 27
$S_s = \frac{2 \times 750 \times 20 \times 0.833}{0.375} = 66{,}700$ lb.
The safety factor is

$$\text{f.s.} = \frac{S_s}{P_R \cos\alpha} = \frac{66{,}700}{35{,}400} = 1.88 \tag{29}$$

To determine the stability with refer-

ence to a foundation failure (FG): $\sum_1^{11} N = 187{,}000$ lb; $\sum_1^{11} T = 50{,}600$ lb; $l = 114$ ft; and (Eq. 2) f.s. $= \dfrac{(187{,}000 \times 0.1763) + (207 \times 114)}{50{,}600} = 1.12.$ This safety factor is too low and a longer pile would be required, thus forcing the slip-plane to a lower elevation. Other reasonable positions of the slip-surface should also be checked. If differences appear to exist between the shearing resistance of the strata, adjustments based on laboratory, field, or analytical studies will be necessary.

If neglecting the resistance that is offered by the mass on the downslope side of the piles is considered unduly conservative, the estimated force against the piles can be adjusted.

Assuming a safety factor of 1.0, the resistance needed from the piling is equal to the increase in safety factor multiplied by the original shearing resistance (or shearing force). With reference to Figure 125, P_R will be equal to $\dfrac{\sum_9^{11} T}{2}$ for a desired safety factor of 1.5 and for slip-surface EH. From the example for a crib wall (Fig. 124) this value for P_R can be obtained, and is equal to 22,500 lb. Assuming three lines of piling at 18-in. center-to-center spacing, $M = 22{,}500 \times 0.927 \times 0.5 \times 80 = 832{,}000$ in.-lb. The required section modulus is $S = \dfrac{832{,}000}{40{,}000} = 20.8.$ For a 10-in. beam weighing 21 lb per ft, $A = 6.19$ sq in; $S = 21.5$; and $f_s = \dfrac{832{,}000}{21.5} = 38{,}700$ lb per sq in. For the shearing resistance of the pile (Eq. 26), $v_p = \dfrac{6.19 \times 25{,}000}{0.5} = 309{,}500$ lb per ft, and the safety factor (Eq. 28) is f.s. $= \dfrac{309{,}500}{20{,}800} = 14.8.$ For stability with reference to the shearing of the soil past the piling (Eq. 27), $S_s = \dfrac{2 \times 750 \times 20 \times 0.833}{0.5} = 50{,}000$ lb, and the safety factor (Eq. 29) is f.s. $= \dfrac{50{,}000}{20{,}800} = 2.40.$

Miscellaneous Methods

From the viewpoint of stability analyses, the miscellaneous methods referred to in Chapter Eight do not lend themselves to a theoretical investigation. The changes produced by the hardening of the soil mass can be estimated by laboratory methods. In turn, the slope can be analyzed in the same fashion and with the same equations as used for excavation methods. However, incorporation of admixtures is so difficult to predict or to measure that little good can be accomplished by stability analyses.

Blasting

The value obtained by blasting is more amenable to prediction by a stability analysis. However, since an element of drainage is involved, a prediction of a change in ground water or piezometric surface is necessary. The other benefit of blasting is the relocation of the slip-surface. Here again, there will be considerable conjecture as to the effect of the blasting, but if the minimum slip-surface displacement is assumed, the stability as indicated by the safety factor will be conservative. Unless the drainage factor is included, no great change in the safety factor is to be anticipated by the minor displacement of the slip-surface.

For a stability analysis of a blasting operation, the shearing resistance should be obtained by methods described previously for existing slope failures. Then, assuming the displaced location of the slip-surface, Eq. 2 (for no drainage value) or Eq. 4 (including drainage values) can be used to estimate the new safety factor.

For the following example, the data of Figure 126 and Table O apply. The unit weight of the soil is 125 lb per cu ft and

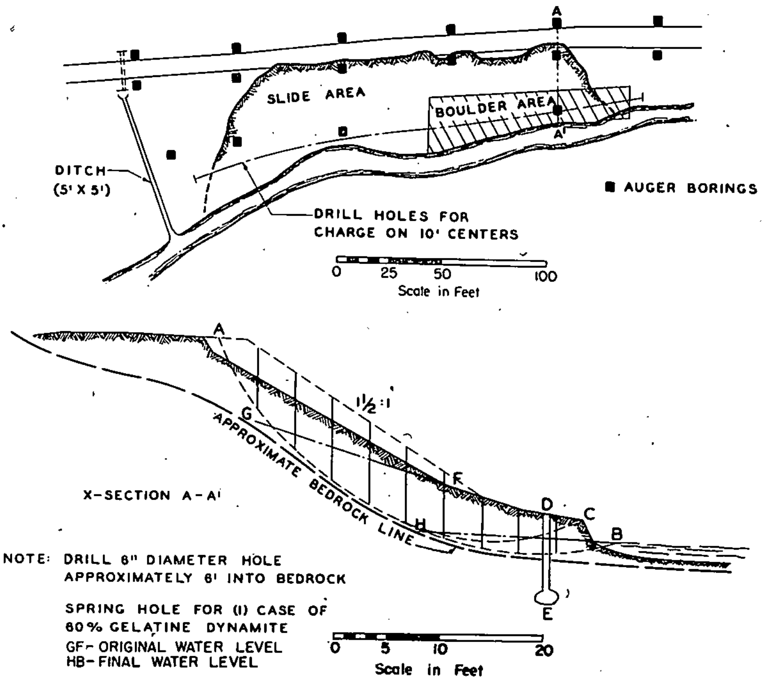


Figure 126. Design of blasting installation.

Table O. For use in determining stability along a slip-surface AB.

Segments Within Arc AB

	1	2	3	4	5	6	7	8	9	10	Total
Area (A), sq ft	12	23	28	29	29	29	20	14	13	9	206
Normal (N_A)	7.00	16	22	25	24	28	19	13	12	8	174
Tangential (T_A)	10.0	16	16	14	15	9	4	1	–1	–1	83
Average h	0	0.5	2.5	4.4	5.0	5.5	5.0	4.5	1.5	1.0	
	–	3.0	5.0	5.0	4.5	4.5	4.5	4.5	4.5	4.0	
(sq ft)		1.5	12.0	20.0	22.5	24.5	22.5	20.0	6.5	4.0	133.5

Table P. For use in determining stability after blasting has shifted the slip-surface to AC and has lowered water table from FG to BH.

Segments Within Arc AC

	1	2	3	4	5	6	7	8	9	10	Total
Area (A), sq ft	12	23	28	29	29	28	17	10	7	2	185
Normal (N_A)	7	16	22	25	24	26	16	10	6	1	153
Tangential (T_A)	10	16	16	14	15	9	1	0	–2	–1	79
Average h	0	0	0	0	0	0	0.5	1.0	0	0	
	–	–	–	–	–	–	4.0	4.0	–	–	
(sq ft)	0	0	0	0	0	0	2.0	4.0	0	0	6

a ϕ value of 10° is assumed. Considering first the slip-surface, AB, a value for c, corresponding to $\phi = 10°$, is determined. Also, $\Sigma N = 21{,}800$ lb; $\Sigma T = 10{,}400$ lb; $\Sigma\mu = 8{,}300$ lb; $l = 48$ ft; and (Eq. 4)

$$c = \frac{10{,}400 - (21{,}800 - 8{,}300)\ 0.1763}{48}$$

$= 167$ lb per sq ft.

If blasting is to be accomplished as indicated on Figure 126, one must assume in the design stage that the slip-surface will take a position such as AB, and that the water table will drop from FG to approximately BH. After blasting has been accomplished, the assumptions can be checked and the stability recomputed. The effect on the stability can be estimated as follows, using Table P, Figure 126: $\Sigma N = 19{,}200$ lb; $\Sigma T = 9{,}870$ lb; $\Sigma \mu = 375$ lb; $l = 47$ ft; and (Eq. 4)

$$\text{f.s.} = \frac{(19{,}200 - 375)\ 0.1763 + (167 \times 47)}{9{,}870} = 1.13.$$

Thus, the blasting creates a maximum increase to 1.13 for the safety factor, as compared to a condition of f.s. = 1.0 at the time of failure. Additional blasting, or the replacement of the upper part of the fill with lightweight material (such as cinders) would tend to further increase the relative stability.

The fact that blasting did not make a significant change in the safety factor is of interest. It could mean either that the stability analyses do not measure adequately the degree of stability, or that this type of blasting is not very effective. The quantitative approach used in this preceding solution required several major assumptions with regard to the lowering of the water table and the displacement of the slip-surface. Even with most favorable assumptions the safety factor was not materially affected. One is reminded, however, that with such a quantitative approach, comparisons of the stability produced by various techniques are possible. Empirical methods do not provide adequate bases for such comparisons.

References

Allen, Harold, *et al.*, "Report of Committee on Classification of Materials for Subgrade and Granular Type Roads." *Highway Research Board Proceedings*, v. 25, 1945.

American Association of State Highway Officials, "Standard Methods of Determining the Liquid Limit of Soils." Am. Assoc. of State Highway Officials Designation T89-42; and "Standard Methods of Determining the Plastic Limit of Soils." Am. Assoc. of State Highway Officials Designation T90-42.

American Society for Testing Materials, "Procedures for Testing Soils. Nomenclature, Standard Methods and Suggested Methods." 1950.

American Society for Testing Materials, "Triaxial Testing of Soils and Bituminous Mixtures." John Wiley & Sons, New York, N. Y., 1950.

Corps of Engineers, U.S. Army, "The Unified Soil Classification System." Tech. Memo. No. 3-357, Waterways Experiment Station, Vicksburg, Miss., 1953.

Corps of Engineers, U.S. Army, "Soil Mechanics Design; Stability of Slopes and Foundations." U.S. Corps of Engineers, Eng. Man., Part CXIX, Chapter 2, 1952.

Fellenius, W., "Calculations of the Stability of Earth Dams." Transactions, 2nd Congress on Large Dams, v. 4, Washington, D.C., 1936.

Fellenius, W., "Erdstatische Berechnungen, etc.," W. Ernst u. Sohn, Berlin, 1927 (revised edition, 1939).

Hennes, R. G., "Analysis and Control of Landslides." Bull. No. 91, Univ. of Washington Eng. Exp. Sta., Seattle, Wash., 1936.

Kjellman, W., "Do Slip Surfaces Exist?" *Geotechnique*, v. 5, No. 1, p. 18-22, 1955.

Krynine, D. P., "Landslides and Pile Action." *Engineering News-Record*, v. 107, No. 122, Nov. 26, 1931.

Lambe, T. William, "Soil Testing for Engineers." John Wiley & Sons, New York, N.Y., 1951.

Rendulic, L., "Ein Beitrag zur Bestimmung der Gleitsicherheit." *Der Bauingenieur*, v. 16, p. 230-233, 1935.

Taylor, Donald W., "Fundamentals of Soil Mechanics." John Wiley & Sons, New York, N.Y., 1948.

Terzaghi, Karl, "Theoretical Soil Mechanics." John Wiley & Sons, New York, N.Y., 1943.

Terzaghi, Karl, and Peck, Ralph B., "Soil Mechanics in Engineering Practice." John Wiley & Sons, New York, N.Y., 1948.

Chapter Ten

Trends

John D. McNeal

Prophecy is seldom wise for the prophet who can expect to live to see its fulfillment or failure. On the other hand, no one can afford to ignore the indications of the future as these are revealed in the conditions and trends of the present. Neither can one be content with the tools of today for the task of tomorrow.

There are factors and trends of today which indicate that landslides will be of even greater significance in engineering practice in the future than they are now. These same signs appear to point to a shift in emphasis in landslide treatment from correction to prevention. What are these signs?

The first is the tremendous expansion in traffic. A second is the change in thinking in regard to financial responsibility of governing bodies; a gradual change away from the precept of the sovereign right of a state to refuse to be sued. A third and related indication of the future is seen in the decline of the concept of a landslide as an "act of God" and the growing realization that many landslides are initiated by "acts of man."

In spite of the scientific methods and great effort devoted to traffic prediction today, the volume increase continues to exceed expectation. At the same time the demand for greater safety, shorter routes, wider pavement, and improved grades combines with the volume increase to require ever higher design standards. As a result, the engineer is faced with more and deeper cuts, higher fills, and alignments which must overcome rather than avoid obstacles. In each case the chances for landslides are increased. Urban expansion; the increase in irrigation use; and the growth, both in volume and complexity, of other civil works, are also increasing the number of locations where landslides must be anticipated.

The remarkable growth in urbanization, so apparent across the entire country, is causing man to make even greater demands on nature. As cities become larger and more congested, and as land values increase, land that was once considered too waterlogged or too steep or otherwise unfit for use is developed for industrial or residential construction. With such construction comes an increase in the number of potential and actual landslides that is out of all proportion to the relative area of the land that is so used.

The engineer is faced not only with the increased possibility of landslides, but also with greater economic losses and greater chances for injuries and deaths when landslides occur. For example, the probability of an accident caused by a rockfall increases in direct proportion to traffic density. In addition, in heavy traffic other vehicles may collide with the car actually hit by a rockfall, thus multiplying the damage and injuries. Property damages also will increase out of proportion to those which can be expected today. Higher design standards require higher priced roads and structures. Failures, thus, involve more cost. Greater losses chargeable to traffic delays and detours result from increased traffic density. Higher land values and urban expansion

lead to increased right-of-way costs. Under the conditions described, economics and public safety will dictate the expenditure of money for the detailed investigations needed to effect prevention of landslides.

Although every engineer and highway department is keenly aware of public safety and property rights, the increasing extension of liability to Federal, State, and municipal governments can scarcely fail to increase the emphasis on these basic engineering considerations. Neither is it to be expected that the establishment of courts of claim and the passage of tort claim acts will reduce the number of demands for damages.

The number of damage claims (and the number of awards made) will probably also increase with the growth of public and legal awareness of the true nature of landslides. It was shown in Chapter Two that the courts have at least on occasion been influenced by the "act of God" concept of landslides. It is logical to assume that at least a part of the public has held and still holds a view similar to that of the courts. The growing realization that some slides are caused or at least triggered by the activities of men, combined with the extended rights of legal action, will cause many more claims, just and unjust, to be filed. More awards will be made and, inevitably, blame will be assessed in some cases where no blame exists. The result must be that engineers will turn more and more to prevention. For self-protection, they may even be tempted toward uneconomic decisions.

Faced with the necessity for preventing landslides, the engineer and all those who work with him on the problem will find it necessary to re-evaluate and improve their present knowledge and procedures and to develop new or better methods of study and control. The remainder of this chapter is devoted to a consideration of some of the changes and improvements that may be expected or that at least are needed. The organization of the discussion in general follows the order of chapters in the book.

Economic and Legal Aspects

The probable future in regard to economic and legal aspects has been discussed in the preceding paragraphs. However, one additional point deserves consideration. The legislation passed by the City of Los Angeles concerning grading, excavation, and foundations (Chapter Two) probably will be followed by similar regulations for other cities, and perhaps even for counties and states. For example, while this book was in preparation, a fatal accident involving a foundation excavation in New York City resulted in a demand for increased regulation of such operations. Legislation of the general nature of that passed in Los Angeles is undoubtedly needed in the interest of public safety. In view of the present state of public knowledge of slope stability, however, engineers, geologists, and soils engineers should take an active part in the drafting of such legislation. They must also assume the responsibility for keeping the laws in step with changing conditions or knowledge. Otherwise, regulations of this type may become either inadequate or unduly restrictive, and thus uneconomic.

Landslide Types and Processes

The multiplicity of existing classification systems of landslides (Chapter Three), each based on somewhat different factors, would seem to rule out either the need for or the probabilities for significant improvement. Many more classification schemes will no doubt appear and each one will have some usefulness or appeal to certain groups or individuals. Some additional landslide subtypes will probably be discovered and will need to be assigned a proper classification.

The anticipated increase in emphasis on prevention of slides, the interest of the public in their cause, and particularly the expected demand of the legal profession for better, more exact, and more inelastic definitions, will lead to a need for more general agreement on a single system of classification which would be fur-

ther subdivided than existing schemes. It might well be based on several interacting factors, including cause and materials involved. It must be remembered, however, that lawsuits are decided on the basis of evidence and that classification of a slide is likely to be less important than is quantitative evidence based on field and laboratory analyses.

Increased emphasis in classification on the description of materials involved and the correlation of materials to slide types and motions offers some possibilities for a system of use in the prediction of slides. Data (Table 6) from the replies to the landslide questionnaire show some interesting, apparent correlations between occurrence and lithologic type. For example, the Pierre, Bearpaw, and Graneros shales are all superficially similar to each other and, in lesser degree, to the Maquoketa shale of a very different age. Each of these formations is associated with landslides in a number of different climatic environments and reliefs.

Much more research into the basic properties of different materials as related to different environments is needed before data like those in the table can be used safely. Certainly, on the basis of information now available, one cannot make a blanket condemnation of any one of the formations or rock types listed, nor can one make any useful statement correlating rock or material type with slide type.

Field Recognition and Airphoto Interpretation

The characteristics by which landslides and dangerous ground may be recognized in the field are rather well known and it is doubtful that many new features of importance will be described in the future. Emphasis on recognition of potential landslides is a necessary prelude, however, to an increase in the use of preventive measures.

The greatly increased use of aerial photogrammetry in the location and surveying of highways is already in evidence. The preliminary survey of some highway projects, as well as the final cross-sectioning, is now being done entirely by photogrammetry with a field survey only for control. With the expanded highway program recently authorized, use of slower field methods of survey will tend to lose ground to the newer technique. This trend, of course, will restrict the opportunity for field investigation and recognition. Airphoto interpretation is a logical substitute, but in its best application it should supplement and guide rather than substitute for field study.

Rapid advances are being made in aerial photographic techniques and equipment. Aerial photographs in color and at much larger scales than currently used, while not now in general engineering use, offer many advantages in landslide studies and should increase in application. In many areas the relatively new techniques of airphoto interpretation can provide a fairly complete and accurate picture of geologic features of importance in landslide studies with a saving in time and, often, money.

Field and Laboratory Investigations and Stability Analyses

The most fertile fields of the landslide problem for future research and development are those of field and laboratory investigations and quantitative analyses of stability. For both field and laboratory studies the immediate need is for better and more complete application of present techniques and knowledge. There is a wealth of geologic and soils data which could and should be more generally applied to landslide problems. Unfortunately, engineers and others working with landslides in the field are too often unfamiliar with such material.

Information on availability of basic geologic maps and reports, as well as some help in interpreting them in engineering terms, can commonly be obtained from the U.S. Geological Survey, from the appropriate State Geological Survey, or from the geology department of the

TABLE 6

STRATIGRAPHIC UNITS SUSCEPTIBLE TO LANDSLIDING

(Data compiled from replies to H.R.B. landslide questionnaires)

Region and State	Geologic Series or Formation	Description
Northeast	Glauconite beds in Cretaceous sediments	
Vermont		Pervious material beneath clay or soil
Maine		Soft clays
Middle East		
New Jersey	Upper Cretaceous clays such as Merchantville and Woodbury	Sand or gravel overlying clay strata
Delaware	Talbot/Wicomico; Wissahickon	
West Virginia	Conemaugh; Monongahela; Dunkard	
Ohio	Ordovician shales and limestones Conemaugh; Monongahela; Dunkard	Fire clays
Illinois		Shales
Pennsylvania	Conemaugh; Monongahela; Catskill; Wissahickon	
Southeast		
No. Carolina	Blue Ridge Province	Jointed surfaces filled with manganese
Florida	Miocene-Hawthorne	Fullers earth type clay
North Central		
Iowa	Des Moines series (Pennsylvanian) Maquoketa (Ordovician)	
Kansas	Pierre shale, Graneros shale, Dakota formation	
South Central		
Texas	Tertiary lava, tuffs and agglomerates Animas sediments	
Mountain	Pierre shale; Bearpaw shale; Graneros shale; top Dakota sandstone	
Montana	Precambrian Belt sediments	Clay shales, bentonite, serpentine
Idaho	Payette; Triassic sediments	
Colorado	Fort Union; Denver, Arapahoe	Glacial till; ground moraine
New Mexico	Dakota on Morrison	
Pacific		
California	Franciscan; Pico; Rincon	Serpentine; clay shale, Quaternary alluvium
Oregon	Eagle Creek volcanic breccia	Basalt talus resting on breccia
Washington	Nespelem silts; Astoria siltstone; Eocene shales	

nearby colleges or universities. The Bureau of Soil Survey, U.S. Department of Agriculture, the State Agricultural Surveys, and the agriculture departments of universities or colleges are equally good sources of information on soils and soil maps. Perhaps the easiest way for the engineer to obtain help in seeking available source material, however, is to enlist the friendly services of a professional in the special field of interest. Such an expert also can be expected to help in translating the basic data into terms of strength or other engineering characteristics.

However, the need exists for new and improved testing and investigative procedures. In this respect, attention is called to the mapping techniques being used in terrain intelligence and other military geology reports (Hunt, 1950). Refinement and extension of this type of mapping could be very beneficial to landslide investigations. Some of the recently published U.S. Geological Survey maps, such as those of Crandell (1955), Hansen and Bonilla (1956), and Lindvall (1956) represent an important step toward correlation of geologic and engineering data.

Opportunities exist for extension of basic research, both in fields now being studied in connection with landslides and into fields not now receiving proper attention. One new field for research has been mentioned in connection with the discussion of Table 6. Few rock types have received as little study as have shales, yet they are of vast significance both to landslide problems and to many other phases of engineering and the general economy. There is widely recognized association of many engineering problems and specific shales, yet little is known of the cause of troublesome properties of these rocks. For the solution of landslide problems perhaps it would be sufficient to develop methods of determining cohesion and internal friction for shales, such that the laboratory values would be indicative of field performance. Again, the engineering significance of the changes which occur during weathering in physical and other properties of shales and other rocks needs much study.

Current knowledge of both the role and the occurence of subsurface water in relation to landslides is highly inadequate. The influence of hydrostatic pressures on slope stability is very little known, as are the quantitative effects of water on the mechanics of failure. A part of the study needed should be directed to the physicochemical changes which result from the interaction of subsurface water and various minerals. Much is known of the occurrence and movement of subsurface water below the ground water table and studies of soil water have received much attention in recent years. Between the water table and the soil water zone, however, all too little is known about either the occurrence or movement of water.

The use of stability analyses often furnishes the only quantitative evaluation of a landslide, and in many cases offers a reliable means of comparing alternate designs. With constantly improving techniques and equipment for sampling and testing, the application of stability analyses and other tools of soil mechanics should have even greater application in the future. The numerous assumptions and simplifications referred to in Chapter Nine as necessary for any stability analysis in themselves condemn our lack of knowledge of basic soil information. The engineering profession will not be satisfied to allow these assumptions to remain unchallenged, for, although the assumptions are well understood and to some degree can be evaluated by the soil engineer, he must continue to seek replacement of assumption by fact.

Laboratory soil tests of all types give results which have meaning principally in relation to their correlation with field behavior. Among other things, case histories are needed that compare actual field behavior with that predicted from laboratory tests and stability analyses. No doubt many of the difficulties encountered in applying test results in the field result from inadequate sampling techniques and procedures. Engineers, including soil engineers, and geologists

have too long ignored the fundamental properties of soils, including soil structure, mineralogy and genesis. Fortunately, the field has not been entirely ignored. The soil scientist and the clay mineralogist, among others, have available much of the basic information which is needed, but it remains for the engineer to interpret the information and to put it to work.

Prevention and Correction

The major prediction of a shift in the future from emphasis on correction to prevention has been discussed previously in this chapter. This emphasis on prevention is not meant to imply that correction will be no problem in the future. Landslides will continue to occur, and it will never be economically feasible to prevent slides completely. This applies particularly, perhaps, to relatively small landslides, many of which occur in mantle soil or weathered material. These are far more numerous, and more troublesome to many agencies, than are larger and more spectacular slides, yet it is both technically and economically infeasible to predict and to prevent many of them.

Several other less inclusive predictions can be made. Many are dependent for fulfillment on the developments which are predicted or indicated as needed in the discussions of previous sections of this chapter. Most are related to the expected increased interest in prevention.

The prediction of sliding as to time remains one of the most difficult problems. It will not always be possible or economically feasible to prevent a landslide, even when its occurrence can be predicted. For some areas of anticipated slides it will be sufficient and advisable to be able to predict the time of their occurrence. A start has been made on solution of this problem, but far more research is needed. Some railroads and other agencies have maintained repeated checks of the positions of reference points on an area subject to sliding and are able to predict the time of any catastrophic movement. The Japanese have made real progress in time prediction of landslides by the use of strain gage data correlated with the results of laboratory tests (Fukuoka, 1953).

Related to the time prediction of sliding is the use of automatic devices to warn the traveling public that a slide has occurred. Railroads have led in the use of warning devices. These usually consist of signal wires which when broken or moved actuate block signals to halt traffic. Highway departments have been slow in adopting automatic warning signals and have relied on permanent signs warning that an area is potentially dangerous. It is reasonable to assume that both railroads and highways will increase and improve the use of automatic warning systems.

Fundamental to accomplishment of prevention is prediction. Several of the chapters of this book have discussed methods of predictions. Field and air-photo recognition of the characteristic surface indicators of dangerous ground has been discussed. The recognition of dangerous situations which are not evident on the surface are less well understood. There is a trend, however, which will help in the identification of subsurface danger spots. There has been a rapid growth since 1940 in the use of engineering geologists in highway departments. Their use has been restricted in many states to functions which are not connected with landslides. However, the geologists in a few states now prepare a complete, three-dimensional view of subsurface conditions by showing detailed soil and geologic conditions in both plan-profile and cross-section. With this information, dangerous situations that will arise entirely from construction and that are not visible on the original ground can be readily seen. Provision for correcting such locations in the design (prevention) stage can often be handled in a balanced design with very little additional cost.

It is not to be expected that many really new preventive or corrective measures will be developed. The increasing knowledge of the physico-chemical prop-

erties of clays, soils and shales, however, may make the use of electro-osmosis, electro-chemical hardening of clays, and chemical injections more economical and effective than they are today.

The principal development in prevention and correction measures will probably come from better understanding and use of present-day methods. A better understanding of the forces involved in a landslide should eliminate some of the failures which have occurred in the past through misuse of piling and restraining structures, or indeed of every other known method of treatment.

With all of the refinements and advances that are expected in investigation, testing, and mathematical analyses of slides, experience will no doubt remain a most important guide for field use. The experience which has been gained and is recorded in the files of many highway departments and other agencies needs to be organized and analyzed and the results published for all to use. Eckel (1956) has summarized this need, as follows:

> We need many, many more studies of actual slides — not simply descriptions of their size and shape and the damage they did. To get much further with our understanding of slides these descriptions must include detailed records of the physical properties of the soils or rocks that were involved, detailed histories of the movements that took place, and enlightened inquiries into the causes of movement. With this kind of facts we can go a long way in comparing slides and in extrapolating the knowledge thus gained to the solution of new slide problems.
>
> Finally, we need to record many more of our failures as engineers or geologists. The literature is full of descriptions of preventive or corrective methods that were successful. On the other hand, records of installations that failed to do the job expected of them are notably lacking. Nobody enjoys publicizing his mistakes, of course — but just the same there is little hope of improving our knowledge of how to handle landslides until we are able to study and compare the case histories of methods that have failed with those that have been successful.

References

Crandell, D. R., "Geology of the Oahe Quadrangle, South Dakota." U. S. Geol. Survey Geol. Quadrangle Map GQ 53, 1955.

Eckel, E. B., Proc. 55th Ann. Conf., *Proc. Am. Railway Eng. Assoc.*, v. 57, p. 1081-1085, 1956.

Fukuoka, Masami, "Landslides in Japan." Proc. 3d Internat. Conf. on Soil Mechanics and Foundation Engrg., v. 2, p. 234-238, Zurich, 1953.

Hansen, W. R., and Bonilla, M. G., "Geology of the Manila Quadrangle, Utah-Wyoming." U. S. Geol. Survey Misc. Inv. Map I-156, 1956.

Hunt, C. B., "Military Geology." In "Application of Geology to Engineering Practice." Berkey Volume, Sidney Paige, Chairman; Geol. Soc. America, p. 295-327, 1950. (See also "Interpreting Geologic Maps for Engineering Purposes." Hollidaysburg Quadrangle, Penna., 1953, U. S. Geol. Survey.)

Lindvall, R. M., "Geology of the Big Sandy Quadrangle, Montana." U. S. Geol. Survey Misc. Inv. Map 1-130, 1956.

Appendix

Questionnaire on Landslides and Engineering Practice

The questionnaire whose results formed the basis of this book, and which is described in Chapter One, is here reproduced in full as to contents, but condensed as to format. In its original form the questions were preceded by a two-page statement of the objectives of the questionnaire and a solicitation for help from the many engineers and geologists to whom it was sent. It was also accompanied by an early version of the classification chart that appears as Plate 1 in this volume.

The questionnaire is included here mainly because it gives in succinct form a guide to the thinking and observations that the Committee feels must go into the investigation, solution and description of any landslide problem. It may also have some value in giving the reader an idea of the kind of data that are contained in the completed questionnaires as filed with the Highway Research Board Library. These questionnaires contain an abundance of basic facts that will be useful, it is hoped, to future students of landslide problems. They also contain many excellent photographs and drawings of actual slides and of methods used for combating them.

General Questions

Please indicate restrictions as to use of the information you supply.

(a) No restrictions.

(b) Permission granted to publish with due credit.

(c) Permission granted to publish if identity of organization and location are withheld on all items marked "X."[8]

(d) Permission granted to publish all information furnished except those marked "C" (*i.e.*, C VI) which are supplied as confidential information to the committee.[8]

If exact figures are not readily available, the committee would appreciate approximate figures or informed guesses throughout this questionnaire rather than no answer. Wherever possible, the following questions have been designed for objective answers for your convenience. More detailed remarks will, of course, be appreciated.

I. What is the approximate yearly cost to your organization of all landslides (include construction, relocation, and maintenance costs)?

(a) Less than $25,000

(b) $25,000 to $100,000

8 Editor's note: Questionnaires that contained confidential material have been returned to the authors and are not on file with the Highway Research Board.

(c) $100,000 to $250,000
(d) $250,000 to $500,000
(e) $500,000 to $1,000,000
(f) In excess of $1,000,000
(g) ______________________________

II. Please cite figures for an unusually troublesome year under the following headings.
(a) Loss of life (number).
(b) Number of persons injured.
(c) Cost of relocations to avoid landslides.
(d) Cost of all preventive and corrective (maintenance) measures other than relocation.
(e) Direct damages not included under a, b, or c (*i.e.*, damage to rolling stock, damages awarded in lawsuits as a result of encroachment of slide on other property, estimated damages resulting from traffic delays including construction of temporary detours, etc.)
(f) ______________________________
(g) Year for which above figures are given.

III. Was the year cited under II above one of:
(Indicate yes or no)
(a) Particularly heavy precipitation?
(b) Unusually severe winter?
(c) Unusual drouth?
(d) Had the previous season been one of particularly heavy construction on new alignments?
(e) Other unusual aspects? (Please cite) ______________________________

IV. In the year cited, which type of landslide (see enclosed classification chart) caused most of your problems?
(a) Falls
(b) Slides
(c) Flows

V. Which of these types is most frequent in a normal year?
(a) Falls
(b) Slides
(c) Flows

VI. Do most of your landslide problems occur in:
(a) Bedrock
(b) Unconsolidated material (including soils)

VII. Please indicate on a standard base map, highway planning map, or railway system map, the location of all slides with which your organization has been troubled or of which you have knowledge. Please assign an index number to each slide.* This number may be used in referring to the slide on the detailed description sheets which have been enclosed. We are particularly desirous of obtaining detailed information on at least one slide of each type (see enclosed classifica-

* If the number of slides to be plotted makes impractical the assignment of an index number to each slide, use a black dot for each slide location and assign index numbers only to those slides for which you complete the enclosed data sheets or supply additional information.

tion charts) which your organization has encountered. In addition, for other representative slides of each type which you have shown by number on the map, we would appreciate at least the following information:

1. Slide type as indicated by a figure number selected from the enclosed charts.
2. Location (*e.g.*, route, direction, and distance from city or town).
3. Volume of slide.

We can plot this information on large geologic, physiographic, and climatic maps of the area and thus obtain considerable additional information.

VIII. What existing texts or books have you found most satisfactory?
For example:

Ladd, G. E., "Landslides, Subsidences and Rockfalls as Problems for the Railroad Engineer." Proceedings, American Railway Engineering Association, v. 36, p. 1091-1162, 1935.

Sharpe, C. F. S., "Landslides and Related Phenomena." 125 p. New York, Columbia University Press, 1938.

Heim, A., "Ueber Bergsturge." Naturf. Gesell. Zurich, Neujahrsblatt 84, 1882.

Terzaghi, K., "Erdbaumechanik." Franz Deuticke, Wien, 399 p., 1925.

Terzaghi, K., "Mechanism of Landslides:" (chapter in) "Application of Geology to Engineering Practice." Berkey Volume, Geol. Soc. of America, p. 181-194, 1950.

IX. Why do you prefer the book or article cited, or what features of the book have you found most satisfactory?

X. If you have developed a classification of landslides within your organization, or favor one of the standard classifications such as those included in the references cited under Question IX above, please give details on the back of this page.

XI. Have you had experience with any landslide which you feel does not fit into one of the classifications (figures) shown on the enclosed chart? If so, please sketch below and/or on the following blank sheet. Please assign a type name to each sketch.

XII. Of the types of landslides shown on the enclosed chart or in your sketches, which has most frequently been a problem to you?
List by figure number (enclosed chart) or type name in order of frequency.

XIII. One state has reported that a majority of the landslides on the highway system has resulted from undercutting of the toe of a slope by stream action; another, faced chiefly with the problem of rockfalls, attributes the problems to frost wedging in strongly jointed rock. *What single factor do you consider most important in causing each of following major landslide types in your state or on your system?* If applicable,

qualify your answer for a particular section (physiographic or geologic provinces).

1. Falls; 2. Slides; 3. Flows.

XIV. What percentage of your landslides occur under the listed conditions?

1. In undisturbed natural slopes above the construction.
2. Above the roadway but on slopes cut by the construction.
3. Below construction but apparently "triggered" by natural causes, such as undercutting of the downhill slope by streams.
4. Below construction and apparently "triggered" by the construction.
5. Entirely within the cut section of construction.
6. Entirely within constructed fills.
7. In ancient slides reactivated by construction.
8. Other ________________________________

XV. In general, what relationship have you found between ground water and landslide occurence?

XVI. Many of the landslides of southwestern Colorado occur in the Mancos shale where overlain by the Mesaverde formation, and many landslides occur in the Conemaugh formation in Pennsylvania and adjacent states. What formations or stratigraphic sequences have you found to be particularly susceptible to landsliding?

XVII. What is the flattest original slope on which you have observed each of the following?

(a) Rockfall (Fig. 1)
(b) Soil fall (Fig. 2)
(c) Bedrock slump (Figs. 3, 4)
(d) Block glide (Fig. 5)
(e) Soil slump (Figs. 6, 7, 8)
(f) Rockslide (Fig. 9)
(g) Debris slide (Fig. 10)
(h) Failure by lateral spreading (Fig. 11)
(i) Debris flow (Fig. 19)
(j) Other flows

XVIII. Rate the methods listed below (plus any additions which you may make) in the order of their effectiveness in prediction of "troublesome ground."

(a) Study of aerial photographs.
(b) Interviews with local citizens along proposed line.
(c) Predictions based on geologic formations to be encountered.
(d) Predictions based on soil types to be encountered.
(e) Soil mechanics studies.
(f) Ground reconnaissance by experienced personnel.
(g) ________________________________
(h) ________________________________

XIX. In the recognition of old slide areas, every engineer or geologist has his "pet" signs (hummocky ground, bent trees, etc.) What are your favorite signs for recognizing such ancient or potential trouble areas?

XX. Some organizations use laboratory identification of the various clay minerals to determine the probability of slides. If you have used such methods, can you summarize your experience with examples?

XXI. We can all profit by knowing how the other fellow goes about his work. Supply, if you can, a copy of a representative field study of a slide indicating the field methods used and the recommendations drawn from observations.

XXII. Supply a copy of a typical soils laboratory report on a landslide problem that indicates the methods of testing and analysis used in your organization.

XXIII. Describe briefly any special methods of field or laboratory investigations that you have tried other than those exemplified in the examples asked for above.

XXIV. In your organization are landslide problems considered principally the concern of: (underline one)
(a) The location engineer
(b) The soils engineer
(c) The staff engineering geologist
(d) The design engineer
(e) The maintenance engineer
(f) ______________________________

XXV. The Swedish Geotechnical Commission has devised an automatic, electrically-operated, warning system which operates a block signal at the ends of a railroad cut when a slide occurs in the cut; other organizations take regular readings of observation stakes driven in suspected slide areas in order to detect a movement in its earliest stage. Have you used either of these methods? __________ What other methods for predicting or warning of the occurrence of landslides in known dangerous areas have you used or encountered?

XXVI. If you have made any studies using observation wells or pore-pressure gauges to determine the subsurface water conditions in suspected slide areas, please give brief description.

XXVII. The State of Washington has recently installed strong, wire mesh blankets over the face of deep rock cuts to prevent damage from rockfalls. Other states often use a series of berms on high backslopes to form catchment shelves for rockfalls. Have you used either method for controlling rockfalls? __________ With what success? ______________ ______________________________ What other methods have you used in preventing or controlling rockfalls?

XXVIII. What method or methods of correction have you found most successful in correcting or controlling slides (Types II, A and B of classification chart)?
What methods have you found to be unsuccessful or of doubtful worth? Use back of this page for further details.

XXIX. What method or methods have you found successful in controlling or correcting flows (Type III of classification chart)?
What methods have you found to be of doubtful or no worth?

XXX. The courts have held (*Boskovich* vs. *King County, Wash.*, 1936) that where warning of the danger of landslides would not promote the safety of those using the highway, there is no duty to give it. At the location in question slides had occurred in the past, but the court felt that since no slide actually blocked the road, no warning would have been suitable or required. What do the laws of your state have to say about the necessity of warning signs in areas susceptible to landslides or rockfalls?
Specifically, does the establishment of such things as "Danger, Falling Rock" relieve the state of liability if a motorist hits a fallen rock in the road?

XXXI. In a number of cases landslides onto a highway or railroad right-of-way have been attributed to irrigation canals or detention dams constructed at some point above the right-of-way. If you know of such a case, where have the courts placed responsibility and to what extent were damages awarded?

XXXII. Describe briefly other types of legal problems which may have arisen in connection with landslides.
Citations of specific suits or details of specific litigation will be greatly appreciated. Please indicate restrictions upon publication of such material.

Description of Individual Slides

It is desired that you complete one of these sheets for at least one representative of each type of slide with which you are troubled. Data for any additional slides listed on the location map would be greatly appreciated. Space is provided on the back of these sheets for additional remarks, sketches, photographs, etc., any of which would be extremely valuable. If you prefer, substitute a narrative description of each slide for this information sheet. The questions listed below cover the principal points in which we are interested, but a description in your own words will undoubtedly furnish us more information on the important points of slide characteristics and corrective measures.

Slide Reference Number (from location map) ______________________________
Location: __________ miles ______________ (direction) from __________ (town, city, etc.) on Route No. ______________.
Landslide type (figure number from enclosed classification chart).
Landslide type (your classification or nomenclature).
Date of Occurrence ________________________
Estimated quantity of material involved in slide:

__________Less than 5,000 cu yd
__________5,000 - 50,000 cu yd
__________50,000 - 500,000 cu yd
__________500,000 - 1,000,000 cu yd
__________Greater than 1,000,000 cu yd

Approximate dimensions of slide block or flow
Width (along scarp): ______________
Length (normal to scarp): ______________
Depth (maximum): ______________
Greatest distance through which material moved: ______________
Approximate original slope on which slide occurred: ______________
Estimated rate of movement (select one or indicate measured movement under (e):
______________(a) Slow (less than 5 ft per month)
______________(b) Moderate (5 ft per month to 5 ft per day)
______________(c) Rapid (5 ft per day to 1 ft per minute)
______________(d) Very rapid (in excess of 1 ft per minute)
______________(e) Measured rate was __________ feet per __________
Precipitation conditions preceding slide were:
______________(a) Dry
______________(b) Moderate rainfall
______________(c) Heavy and/or prolonged rains
______________(d) Rapidly melting snow cover
______________(e) Fast spring warmup
______________(f) ______________
Was subsurface water involved? __________ Remarks: ______________
Estimated monetary damage: ______________
Number of persons injured ______________ Killed ______________.
For what period of time was traffic blocked or delayed until slide could be cleared or detour route established ______________

TYPE OR TYPES OF MATERIAL INVOLVED IN SLIDE (Underline): If preferred, by sketch of slide show types of material involved.

A. Slide entirely in unconsolidated material: Yes_______No_______
I. Material was: (a) Residual (b) Non-residual
II. Material was (a) Relatively homogeneous (b) Non-homogeneous.
III. Material was:
(a) Waterlaid:
1. Floodplain alluvium 2. Terrace deposits
3. Alluvial fan deposits 4. Other ______________
(b) Glacial deposits:
1. Till 2. Stratified drift 3. Other ______________
(c) Wind deposit:
1. Dune sand 2. Loess 3. Adobe
(d) Other:
1. Talus material 2. Old landslide material
3. Manmade fill 4. Mine tailings
5. Other ______________
(e) Agricultural soil type was (*e.g.*, Putnam clay) ______________
(f) Please give available B.P.R. soil classification and other available pertinent soil mechanics data under remarks where applicable.
Remarks ______________

B. Slide involved chiefly consolidated material. Yes_______No_______
1. Sedimentary:
(a) Shale (b) Sandstone (c) Limestone

(d) Interbedded limestone or sandstone and shale.
(e) Other ______________________________
(f) Stratigraphic unit(s) involved (*e.g.*, Pottsville fm.) ________

2. Metamorphic:
   (a) Schists (b) Phyllite (c) Marble
   (d) Serpentine (e) Gneiss
   (f) Other ______________________________
3. Igneous
   (a) Volcanic:
      1. Volcanic ash.
      2. Tuffs and breccias.
      3. Interbedded lava flows of composite cones.
   (b) Lava flows:
      1. Movement entirely in the lava.
      2. Lava flow underlain by weaker rock.
   (c) Acid igneous rocks
   (d) Basic igneous rocks
   (e) ______________________________

CAUSE OR CAUSES OF THE SLIDE: If several causes or factors are involved check as many as necessary, indicating, if possible, their relative importance.

(a) Gravity primarily.
(b) Rounded sand and pebbles.
(c) Clay materials.
(d) Micaceous minerals.
(e) Serpentine.
(f) Water, free, capillary.
(g) Raising water content through increase of supply.
(h) Raising water content through blocking natural flow outlets.
(i) Prevention of evaporation by blanketing with earth, cinders.
(j) Rapid drawdown of water table.
(k) Rise of water table in distant aquifer.
(l) Seepage from artificial source (reservoir or canal).
(m) Removal of toe, natural causes.
(n) Removal of toe, construction.
(o) Earthquakes.
(p) Blasting.
(q) Temperature changes.
(r) Frost action.
(s) Dipping joint planes.
(t) Shrinkage cracking.
(u) Increase of load, construction.
(v) Increased load, ice and snow.
(w) Increased load, talus and slide accumulation.
(x) Dipping bedding planes.
(y) Dipping schistosity and cleavage.
(z) Fault planes.
(aa) Chemical decomposition of rocks (particularly volcanic).
(ab) ______________________________

TYPE OF TREATMENT: (after Baker, R. F., "Determining Corrective Action for Highway Landslide Problems." Highway Research Board Bull. 49, p. 6, 1952.)

I. Elimination methods
   A. Relocation of structure, complete
   B. Removal of the landslide
      1. Entire
      2. Partial at toe
   C. Bridging
   D. Cementation of loose material, entire

II. Control methods
   A. Retaining devices
      1. Buttresses
         a. Rock
         b. Cementation of loose material at toe
         c. Chemical treatment, flocculation, at toe
         d. Excavate, drain and backfill at toe
         e. Relocation, raise grade at toe
         f. Drainage of the toe
      2. Cribbing — concrete, steel, or timber
      3. Retaining wall — masonry or concrete
      4. Piling — steel, concrete, or timber
         a. Floating
         b. Fixed, no provision for preventing extrusion
         c. Fixed, provision for preventing extrusion
      5. Tie-rodding slopes
   B. Direct rebalance of the ratio between resistance and force
      1. Drainage
         a. Surface
            (1) Reshaping landslide surface
            (2) Slope treatment
         b. Subsurface (French drain type)
         c. Jacked-in-place or drilled-in-place pipe
         d. Tunneling
         e. Blasting
         f. Sealing joint planes and open fissures.
      2. Removal of material, partially at top
      3. Lightweight fill
      4. Relocation, lower grade at top
      5. Excavate, drain, and backfill, entire
      6. Chemical treatment, flocculation, entire

III. Other ____

____

____

Has the above listed treatment been successful? ____________________

Were any previous treatments unsuccessful? __________ What was the type of treatment?

Sketches, photographs and written reports or descriptions from your files will be appreciated.

TYPE OF MOVEMENT

I. FALLS
Mass in motion travels most of the distance through the air. Includes free fall, movement by leaps and bounds, and rolling of rock and debris fragments without much interaction of one fragment with another.

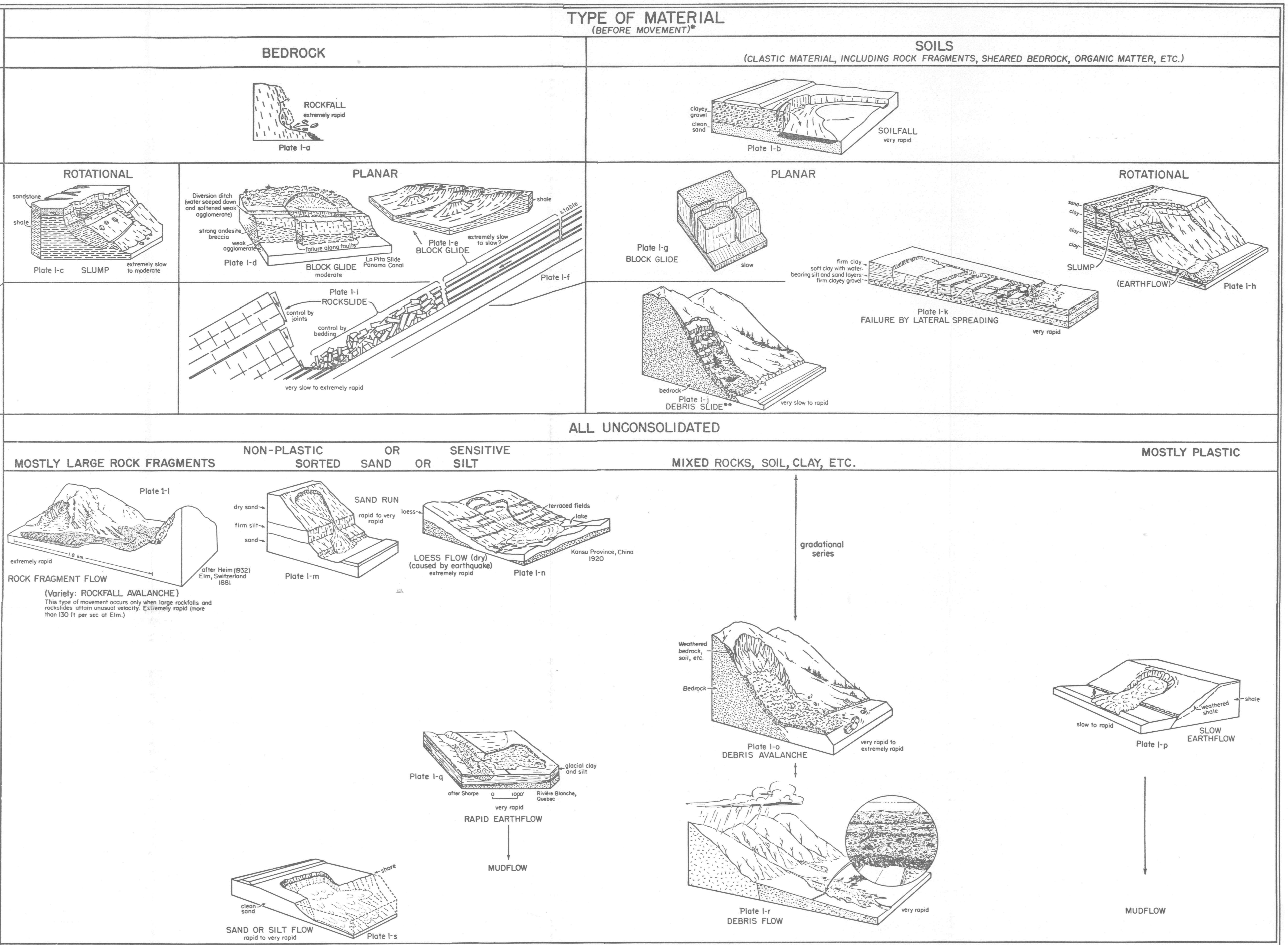
II. SLIDES
Movement caused by finite shear failure along one or several surfaces which are visible or whose presence may reasonably be inferred.

A. Material in motion not greatly deformed. Moving mass consists of one or a few units. Maximum dimension of units is greater than displacement between units. Movement may be controlled by surfaces of weakness such as faults, bedding planes or joints.
1) SLUMP: Movement only along internal slip surfaces, which are usually concave upward. Backward tilting of units is common. 2) BLOCK GLIDE: Movement of a single unit out and down along a more or less planar surface of weakness, generally a bedding plane. Block may glide far out on original ground surface.

B. Material in motion is greatly deformed or consists of many semi-independent units. Movement frequently is structurally controlled by surfaces of weakness such as faults, joints, bedding planes, variations in shear strength between layers of bedded deposits, or by the contact between firm bedrock and overlying detritus. Maximum dimension of units is comparable to or less than displacement between units, and generally much smaller than displacement of center of gravity of the whole mass. Movement may progress beyond original slip surface so that parts of mass slide over the ground surface.

III. FLOWS
Movement within displaced mass such that the form taken by moving material or the apparent distribution of velocities and displacements resemble those of viscous fluids. Slip surfaces within moving material are usually not visible or are short-lived. Boundary between moving mass and material in place may be sharp or a zone of distributed shear.

GRADATIONAL WATER CONTENT
DRY
WET



IV. COMPLEX LANDSLIDES
Movement is by a combination of one or more of the three principal types of movement described above. Many landslides are complex, although, as illustrated in Pls. I-k and I-l, one type of movement generally dominates over the others at certain areas within a slide or at a particular time in the evolution of a slide.

Nomenclature of the parts of a landslide
(see drawing at right)

MAIN SCARP – A steep surface on the undisturbed ground around the periphery of the slide, caused by movement of slide material away from the undisturbed ground. The projection of the scarp surface under the disturbed material becomes the surface of rupture.
MINOR SCARP – A steep surface on the disturbed material produced by differential movements within the sliding mass.
HEAD – The upper parts of the slide material along the contact between the disturbed material and the main scarp.
TOP – The highest point of contact between the disturbed material and the main scarp.
FOOT – The line of intersection (sometimes buried) between the lower part of the surface of rupture and the original ground surface.
TOE – The margin of disturbed material most distant from the main scarp.

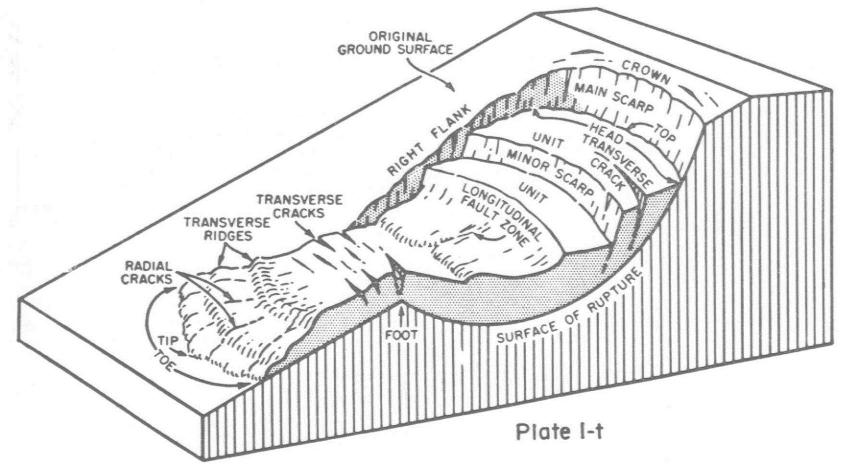


Plate I-t

TIP – The point on the toe most distant from the top of the slide.
FLANK – The side of the landslide.
CROWN – The material that is still in place, practically undisturbed, 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.

The following definition of a landslide has been adopted for use in this book:
Landslide – The term "landslide" denotes downward and outward movement of slope-forming materials composed of natural rock, soils, artificial fills, or combinations thereof.
Landslides move along surfaces of separation by falling, sliding, and by flowing. Parts of a landslide may move upward while other parts move downward. The lower limit of the rate of movement of landslide material is restricted in this book by the economic aspect to that actual or potential rate of movement which provokes correction or maintenance.

*The type of material involved is classified according to its state prior to initial movement or, if the type of movement changes, according to its state at the time of the change in movement. Thus, the Elm slide (Pl. I-l) began as a rock slide and rock fall in bedrock, but at the time a flowing type of movement started the material was an "unconsolidated" mass of extremely rapidly moving rock fragments.

**By debris is meant natural soil and rock detritus.

Approximate ranges of rates of movement are according to the scale below
Plate I-u

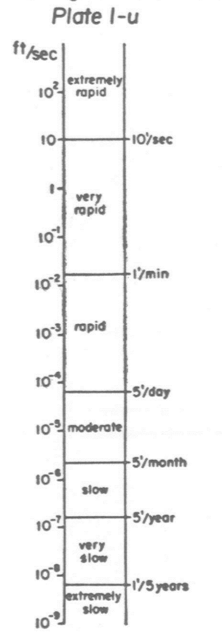


PLATE I
CLASSIFICATION OF LANDSLIDES
Highway Research Board Landslide Committee

HIGHWAY RESEARCH BOARD

NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL
DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH

2101 Constitution Avenue Washington 25, D. C.

EXecutive 3-8100

BOOK ANNOUNCEMENT

LANDSLIDES AND ENGINEERING PRACTICE

HRB Special Report 29

Prepared by the Committee on Landslide Investigations
Edited by Edwin B. Eckel, Chairman

Publication date: January 1958 Price: $6.00
126 illus., 6 tables, 1 plate 232 pp.

The Highway Research Board, a unit of the National Academy of Sciences—National Research Council, announces publication of its Special Report 29, entitled "Landslides and Engineering Practice." The result of several years of effort by the Board's Committee on Landslide Investigations, this book may well become a standard reference work for those ~~concerned with this important field of~~ engineering.

It is the purpose of this volume to bring together in coherent form and from a wide range of experience such information as may be useful to any engineer who must recognize, avoid, control, design for, or correct the more important types of landslide movement.

Because the book is designed for practical use, theoretical discussions are minimized, whereas those phases held to be of greater interest to the practicing engineer are emphasized over others.

The book is divided into two parts. Part I, called "Definition of the Problem," is intended to provide the engineer with the tools and methods he needs to solve an actual or potential landslide problem.

Part II, called "Solution of the Problem," summarizes the methods known to have been applied to the prevention and control of landslides; it also discusses the methods of making stability analyses and of using them in the solution of design problems. In this part every effort has been made to distinguish between those methods that have proved successful under given circumstances and those that

have not. The brief closing chapter points out the kinds of information on landslides and their control that are still lacking and suggests methods by which such information may be obtained.

In its attempt to cover the entire field of landslides, from causes to cures, the volume is, to the authors' knowledge, unique in the English language.

There is no expectation that the reader of this book will become an expert on all phases of the investigation and treatment of landslides. Rather it has been the aim of the compilers to provide an introduction to all of the main factors that go into the solution of a given landslide problem. The average engineer, to whom a landslide is only one of many different problems that he encounters in his work, should be able to use the tools presented here himself or else should be able to determine from the facts given here when it is time to call in a specialist on one or another phase of his investigation. On the other hand, the specialist in some phase of landslide studies should gain an appreciation of the many facets of a landslide problem and of how his specialized knowledge of one facet can best be applied toward solution of the total problem.

Normal surficial creep is arbitrarily excluded from consideration, as are subsidence without downslope movement and most types of movement due to freezing and thawing of water. Similarly, landslide phenomena in tropic and arctic climates, and their treatment, are almost entirely neglected here. A few examples are drawn from other countries, but as the writers and their informants are largely experienced in the United States, most of the descriptions of landslides and of engineering techniques are drawn from this country.

It was perhaps inevitable, considering the makeup of the committee that compiled the book and the sources of information easily available to it, that the volume should seem to stress the landslide problem related to highways and railroads almost to the exclusion of many other landslide problems, such as those of shorelines and waterways, of city, suburban, and resort developments, and of farmlands. This apparent neglect has not been intentional—nor should it necessarily detract from the applicability of the facts contained herein to the solution of landslide problems other than those encountered by highway and railroad engineers. The factors of geology, topography, and climate that interact to cause landslides are the same regardless of the use to which man puts a given piece of land.