216

HIGHWAY RESEARCH BOARD

Bulletin 216

Landslide Occurrence and Analysis

National Academy of Sciences—

National Research Council

publication 669

HIGHWAY RESEARCH BOARD

Bulletin 216

Landslide Occurrence and Analysis

PRESENTED AT THE
Thirty-Seventh Annual Meeting
January 6-10, 1958

1959

Washington, D. C.

Department of Soils, Geology and Foundations

Miles S. Kersten, Chairman
Professor of Highway Engineering
University of Minnesota

COMMITTEE ON LANDSLIDE INVESTIGATIONS

Edwin B. Eckel, Chairman
Chief, Engineering Geology Branch
U. S. Geological Survey, Denver

R. F. Baker, Associate Professor of Civil Engineering, Department of Civil Engineering, Ohio State University, Columbus
Arthur B. Cleaves, Department of Geology, Washington University, St. Louis
Ta Liang, Associate Professor of Civil Engineering, Cornell University, Ithaca
John D. McNeal, Chief Geologist, State Highway Commission of Kansas, Topeka
Harry E. Marshall, Geologist, Bureau of Location and Design, Ohio Department of Highways, Columbus
S. S. Philbrick, Office of the District Engineer, U. S. Corps of Engineers, Pittsburgh
Arthur M. Ritchie, Geologist, Washington Department of Highways, Olympia
A. W. Root, Supervising Materials and Research Engineer, California Division of Highways, Sacramento
Rockwell Smith, Roadway Engineer, Association of American Railroads, Chicago
David J. Varnes, Geologist, Engineering Geology Branch, U. S. Geological Survey, Denver
W. A. Warrick, Resident Manager, John Clarkson, Consulting Engineer, Albany, N. Y.
Eldon J. Yoder, Joint Highway Research Project, Purdue University, Lafayette

COMMITTEE ON STRESS DISTRIBUTION IN EARTH MASSES

D. P. Krynine, Chairman
Berkeley, California

Richard G. Ahlvin, Chief Engineer, Special Projects Section, Waterways Experiment Station, Vicksburg, Mississippi
E. S. Barber, University of Maryland, and Bureau of Public Roads
Louis A. Berger, Louis Berger and Associates, Orange, New Jersey
B. E. Colley, Portland Cement Association, Chicago
Lawrence A. DuBose, Testing Service Corporation, Lombard, Illinois
Austin H. Emery, Associate Soils Engineer, Bureau of Soil Mechanics, New York State Department of Public Works, Albany
Jacob Feld, Consulting Engineer, New York, N. Y.
Robert G. Hennes, Department of Civil Engineering, University of Washington, Seattle
W. S. Housel, University of Michigan, Ann Arbor
F. N. Hveem, Materials and Research Engineer, California Division of Highways, Sacramento
Robert L. McNeill, Staff Engineer, Woodward Clyde and Associates, Oakland, California
Robert L. Schiffman, Department of Civil Engineering, Rensselaer Polytechnic Institute, Troy, N. Y.
Frank. H. Scrivner, Rigid Pavement Research Engineer, AASHO Road Test, Ottawa, Illinois
M. G. Spangler, Iowa State College, Ames

Contents

Regional Concept of Landslide Occurrence

ROBERT F. BAKER, Associate Professor of Civil Engineering; and
ROBERT CHIERUZZI, Research Associate, Ohio State University, Columbus

The report covers the initial phases of a basic study of landslides. The long-range objective is the development or refinement of quantitative methods for analyzing the degree of stability of natural slopes. The underlying principle of the research is that the types of landslides that occur in a given geographic region are relatively limited, and the number of variables present in a given region will be reduced or the range of values limited. Under such an approach a greater possibility exists for the establishment of a comprehensive generalized approach.

The phase of the research reported in the current report covers the basic concepts and the efforts to use physiographic provinces of the United States as the basis for regional considerations. Case histories from the literature, from the files of the authors, and from the questionnaire received by the HRB Committee on Landslide Investigations were the source of data. The landslides were classified in accordance with the new system proposed by the HRB committee, and a summary is included of the types of landslides that occur within the several regions.

Possibilities of immediate use of the results are recognized. If the types of landslides that occur within a region are limited, the highway engineers in a specific area can learn more rapidly and accurately how to analyze and treat the landslides encountered.

THE DEGREE of importance of the landslide problem is a variable quantity. In some parts of the world, major mass movements represent the ultimate in catastrophe and occur with great frequency. On the other hand, some geographic areas rarely encounter the phenomenon, and are only conscious of the tragic implications through newspaper and periodical accounts. For highway engineers, the economic factors are a glaring reality, but vary also with locale.

From the viewpoint of the soils engineer, "landslides" can and do occur in all areas, regardless of the terrain or material because landslides represent a special type of stability problem (1), that is, a failure of the soil in shear. It is true that in the common conception of landslides, natural slopes are envisioned, and such failures as tunnel cave-ins, trench displacements, and foundation failures are not included. Since the latter types of difficulties occur as a result of man's activity, the locale is a function only to the extent that good soil engineering is a function of the geographic area.

By restricting one's consideration of landslides to failures which involve natural slopes, the indications of an "area-problem" become immediately apparent. Many references and implications in technical literature suggest such an approach. However, past studies of landslides have concentrated upon complete classification, historical aspects, individual case histories, or generalized, over-simplified solutions.

No effort is made in the following discussion to supply a rigorous background on landslides. The reader is referred to a recent publication of the Highway Research Board for a comprehensive treatment (2). The classical theories of soil mechanics applicable to the problem are also omitted, but they are available in the several texts on the subject.

REVIEW OF THE LANDSLIDE PROBLEM

Landslides have been the target for study for many years, and quite recently an almost forgotten manuscript by Collin (3) shows that good, quantitative efforts were underway in France in the early 1840's. The bibliography published by the Highway Research Board (4) includes 267 articles or texts on the subject prior to 1950.

The analysis of a landslide can be subdivided into (1) classification, (2) recognition, (3) analysis, and (4) treatment. Since classification has no real significance to the engineer except as it aids in the analysis, one could think of the approach as a three-step operation. However, classification is the common tool for grouping similar landslides and will be considered independently.

The geological sciences have been historically, and by definition, interested in major mass movements. Particularly, the field of geomorphology (science of landforms) has been vitally concerned. With reference to the formation of topographic forms, Ladd (5) has said "erosion, alone, should be given less credit for playing the major role." Geologic studies have been of infinite value in establishing classification and historical implications. While many authorities have produced their own system of classifying, the basic form of the system suggested by Sharpe (6) has prevailed. From a historical viewpoint, hypothesizing as to transportation, sedimentation, and loading will continue as a basis for complete understanding, and perhaps, formulation of new theories. As to analysis and treatment, the field of geology has relied upon experience and judgment to provide the solution. Many case histories attest to the contribution of the geologist and the engineer in such endeavors.

With the advent of the theory of soil mechanics (7) efforts were increased to obtain analyses and treatments based upon quantitative techniques. Unfortunately, the classical mechanics theories were developed for idealized materials, and could rarely be directly applied. The resultant status includes a "missing link," or a break, in the orderly progression from the physical and historical description through a rigorous, rational analysis to the treatment.

As applications of theoretical soil mechanics became more common, a basic weakness in the existing landslide classification schemes became evident, and systems similar to the new HRB classification (2) were developed. The major revision consisted of grouping together those landslides with similar behavior relative to stress-strain conditions.

From a quantitative viewpoint, perhaps the most significant weakness in analysis and treatment is the inability to measure accurately the shearing resistance of the material. Of almost equal concern is the problem of predicting and estimating the stress conditions within an earth mass. A theory which will adequately explain landslide phenomena must relate all types and all conditions, regardless of the variables present. Furthermore, an understanding of the existing geologic and mechanics literature, properly interpreted and incorporated, should provide a tremendous impetus to the development of such a theory. It is within this latter comprehensive framework that the current research at Ohio State University is centered.

PURPOSE AND SCOPE

The current phase of the problem is directed toward a description of the landslide problem in the United States, and the immediate objective is the delineation of the severity of the landslide problem within the several physiographic sections of the country. In addition to focusing attention methodically upon areas established through sound considerations of geography, geology, and climate, landslides will be examined according to the mechanics of failure as represented by the new type of classification system. The advantages of such an approach lie in the ability to (1) obtain maximum utilization of the existing literature, (2) isolate certain variables, and (3) provide immediate educational aid by reducing the problem scope in a given area.

The existing study is preliminary in nature and was intended to serve as an initial or feasibility stage. The data are limited to those available from the questionnaire circulated by the HRB committee (2), case histories from the literature, and files of the authors. Only the continental United States has been considered, and only landslides related to natural slopes. Failures which develop because of man's activities on and around natural slopes are meant to be included.

CLASSIFICATION OF LANDSLIDES

Landslides will be considered as "downward and outward movements of slope-forming materials—natural rocks, soil, artificial fills, or combinations of these materials." (2) Since the basic intent of the current investigation is related to the development of a quantitative theory for the treatment of landslides, a classification system was desired which would reflect stress-strain considerations. The principles of the method described by the Highway Research Board Committee were therefore adopted for the beginning studies. The method consists of dividing mass movements into one of the three following categories (Fig. 1): (1) falls, (2) slides, and (3) flows. A fourth group, complex, consists of landslides which have the characteristics of more than one of the preceding three. The three factors which reflect engineering properties are the bases for subgroupings: (1) type of material before movement (bedrock or soil), (2) amount of moisture, and (3) relative particle displacement within the moving mass.

Falls can be typified by the rock weathering of exposed bedrock slopes, with the talus at the toe representing the accumulation of many "landslides." In actuality, the size of the moving mass can range from small particles to tremendous blocks. Falls are defined as those landslides which develop because of tension failures in bedrock or soil, followed by free-fall, leaping, bounding, or rolling down the slope until equilibrium is established. The tension failures caused by over-stressing are a result of (1) too steep slopes or undermining, (2) weakening of the mass by the formation of cracks or fissures, or (3) pressure within fissures or cracks. The first is common in sedimentary bedrock deposits in which weak shales underlay more resistant sandstones or limestones. The second effect is prevalent in fissured clays (because of processes such as dessication) or in exfoliated bedrock. Landslides under such conditions are also aggravated by steep slopes. Pressure within cracks or fissures can come from either hydrostatic forces or through ice formation. Although such phenomena may appear to be a compressive-type action with the resultant displacement producing instability due to unbalanced moment, inability to withstand tension will always be a factor although at times insignificant. For the purposes of the current discussion only rockfall and soilfall will be considered. The former consists of bedrock (prior to movement) and the latter is concerned with soil or unconsolidated material.

Slides are landslides caused by a shear failure along warped or plane surfaces. Where the shear-surface is reasonably circular in shape (in two dimensions), the movement is called a slump, while a movement with an essentially planar slip-surface is termed a block glide. "Rock" or "earth" is used as a prefix to differentiate between bedrock and unconsolidated material. Following the initial shear, if the moving mass disintegrates or acts as a group of individual particles rather than as a unit block, the slide is further subdivided into rockslide or debris slide. The former is for bedrock, while the latter is for soil. The latter grouping, as to the behavior after shear, is less significant for preventive purposes than for corrective, historic, or geomorphic considerations.

Flows are landslides in unconsolidated material, resulting from plastic deformation of an earth mass or from viscous-type flow and caused by insignificant

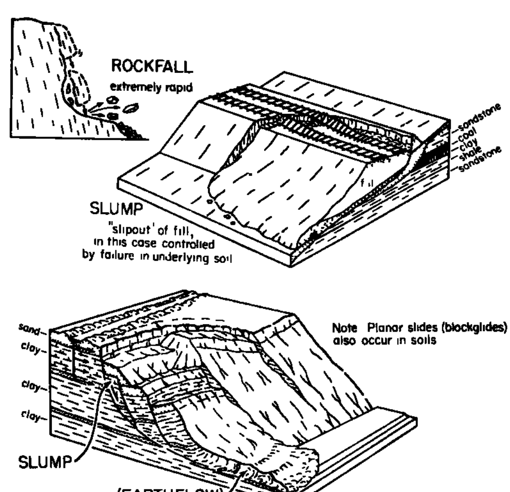


Figure 1. After "Landslides and Engineering Practice." HRB Committee on Landslide Investigations.

TABLE 1

PHYSICAL DIVISIONS OF THE UNITED STATES
(After N. M. Fenneman)

- I. Laurentian Upland
  - 1. Superior Upland
- II. Atlantic Plain
  - 2. Continental Shelf
  - 3. Coastal Plain
    - a. Embayed section
    - b. Sea Island section
    - c. Floridian section
    - d. East Gulf Coastal Plain
    - e. Mississippi Alluvial Plain
    - f. West Gulf Coastal Plain
- III. Appalachian Highlands
  - 4. Piedmont province
    - a. Piedmont Upland
    - b. Piedmont Lowlands
  - 5. Blue Ridge province
    - a. Northern section
    - b. Southern section
  - 6. Valley and Ridge province
    - a. Tennessee section
    - b. Middle section
    - c. Hudson Valley
  - 7. St. Lawrence Valley
    - a. Champlain section
    - b. Northern section
  - 8. Appalachian Plateaus
    - a. Mohawk section
    - b. Catskill section
    - c. Southern New York section
    - d. Allegheny Mountain section
    - e. Kanawha section
    - f. Cumberland Plateau section
    - g. Cumberland Mountain section
  - 9. New England province
    - a. Seaboard Lowland province
    - b. New England Upland section
    - c. White Mountain section
    - d. Green Mountain section
    - e. Taconic section
  - 10. Adirondack province
- IV. Interior Plains
  - 11. Interior Low Plateaus
    - a. Highland Rim section
    - b. Lexington Plain
    - c. Nashville Basin
    - d. Possible western section
  - 12. Central Lowland
    - a. Eastern lake section
    - b. Western lake section
    - c. Wisconsin Driftless section
    - d. Till Plains
    - e. Dissected Till Plains
    - f. Osage Plains
  - 13. Great Plains province
    - a. Missouri Plateau, glaciated
    - b. Missouri Plateau, unglaciated
    - c. Black Hills
    - d. High Plains
    - e. Plains Border
    - f. Colorado Piedmont
    - g. Raton section
    - h. Pecos Valley
    - i. Edwards Plateau
    - k. Central Texas section
- V. Interior Highlands
  - 14. Ozark Plateaus
    - a. Springfield-Salem plateaus
    - b. Boston "Mountains"
  - 15. Ouachita province
    - a. Arkansas Valley
    - b. Ouachita Mountains
- VI. Rocky Mountain System
  - 16. Southern Rocky Mountains
  - 17. Wyoming Basin
  - 18. Middle Rocky Mountains
  - 19. Northern Rocky Mountains
- VII. Intermontane Plateaus
  - 20. Columbia Plateaus
    - a. Walla Walla Plateau
    - b. Blue Mountain section
    - c. Payette section
    - d. Snake River Plain
    - e. Harney section
  - 21. Colorado Plateaus
    - a. High Plateaus of Utah
    - b. Uinta Basin
    - c. Canyon Lands
    - d. Navajo section
    - e. Grand Canyon section
    - f. Datil section
  - 22. Basin and Range province
    - a. Great Basin
    - b. Sonoran Desert
    - c. Salton Trough
    - d. Mexican Highland
    - e. Sacramento section
- VIII. Pacific Mountain System
  - 23. Cascade-Sierra Mountains
    - a. Northern Cascade Mountains
    - b. Middle Cascade Mountains
    - c. Southern Cascade Mountains
    - d. Sierra Nevada
  - 24. Pacific Border province
    - a. Puget Trough
    - b. Olympic Mountains
    - c. Oregon Coast Range
    - d. Klamath Mountains
    - e. California Trough
    - f. California Coast Ranges
    - g. Los Angeles Ranges
  - 25. Lower California province

shearing resistance along a surface of rupture. Generally, flows will develop in very weak materials only, although if the masses are extensive, slow plastic deformations can occur in relatively strong materials that are stressed beyond their elastic range. While flows are most commonly associated with very wet conditions, dry non-cohesive soils can produce rock fragment flows, sand runs, or loess flows, with the nomenclature expressing the particle size. The fine-grained materials produce earthflows or mudflows, where the principal difference is in the amount of water present. A quantitative delineation has not been established for the preceding two types, but the latter term is intended to designate the most fluid movements. Where a mixture of rock fragments and fine-grained soil is involved, the term "debris" describes the material while the terms "debris avalanche" and "debris flow" are used in a parallel sense to earthflow and mudflow. Since little or no cohesion is available in such materials, once failures have developed movement is relatively rapid.

The phenomenon commonly referred to as creep, would fall into the category of a slow earthflow. Under the current status of knowledge, a separate delineation on the basis of speed of movement is not particularly practical for treatment considerations. The differentiation between flows and slides is more critical and, on occasion, very difficult. Earthflows that produce tension cracks at the top of the slide, but distort under plastic deformation without the development of a continuous surface of shear, may be mistaken for a slump or block glide. Also troublesome are the landslides that develop as a slide, but, after a minimum of displacement, take on the appearances of a flow. For preventive measures, the latter type of failure would be analyzed as a slide, but for corrective purposes, as a flow.

From an engineering viewpoint, the size of the landslide has a special significance, and could logically form a basis for classification. However, for the current study, the volume of the moving mass is considered as a landslide variable, but not as a factor in classification.

BASIS FOR PHYSIOGRAPHIC CLASSIFICATION

The division of the United States into physiographic areas is a systematic attempt to divide the topography into homogeneous units with respect to certain fundamental concepts established in the field of geomorphology. Briefly, these principles state that three major factors control the evolution of landforms; namely, the initial structure, the erosive processes continually modifying it, and the stage of its destruction. In general, a change in any of these three factors will produce a uniquely different landform. The converse is also true; that is, differences between landforms can be traced to some differences in any of the three factors (8).

The geologic structure is a dominant influence in the landform modification. Included are characteristics such as the nature of the material; physical hardness of the constituent materials and their susceptibility to chemical weathering; the mode of deposition and subsequent stress history; shearing strength; and structural discontinuities and weaknesses such as joints, bedding planes, faults, folds, and others.

Geomorphic processes and corresponding forces consist of the many physical and chemical actions by which the original structure is modified. Most important of these can be associated with stream, wind, wave, glacial, and weathering actions. The effects of these stresses imposed upon the landforms are reflected in the significantly different erosional, residual, and depositional features produced by each of the geologic agents. For instance, the erosional features produced by streams are gullies, valleys, gorges, and canyons; residual features include peaks and monadnocks; and depositional features consist of alluvial fans, flood plains, and deltas.

Modification, and eventual destruction, of the landforms is considered to occur in stages which are generally designated by the geomorphologists as youth, maturity, and old age. Qualifying adjectives, such as early and late, are often used to designate substages. Chronological age is not inferred, but, rather, the degree of destruction as expressed by topographic characteristics is involved. In youth, topography is relatively undissected with only a few streams. Valleys have V-shaped cross-sections and their depths will depend upon the altitude of the region. Mature topography consists

TABLE 2

LIST OF HRB QUESTIONNAIRES AND RELATED CORRESPONDENCE USED IN THE STUDY

I State Highway Departments (30)

Alabama	Maryland	Ohio
Arkansas	Massachusetts	Oregon
California	Michigan	Pennsylvania
Colorado	Minnesota	Rhode Island
Delaware	Mississippi	South Carolina
Georgia	Missouri	Texas
Idaho	Montana	Utah
Illinois	New Hampshire	Vermont
Kentucky	New Mexico	Washington
Maine	North Carolina	West Virginia

II. State Geological Survey (6)

Florida
Maryland
New Jersey
Oklahoma
Pennsylvania
Vermont

III. Railroad Companies (12)

IV U S Bureau of Reclamation (6)

Region III
Region V
Region VI
Region VII
California
New Mexico

V United States Geological Survey (3)

California
Colorado
Washington

VI. Miscellaneous (9)

mostly of slopes of hillsides and valley-sides. Drainage divides are sharp and maximum possible relief exists. Vertical cutting ceases and lateral destruction becomes important. In reacing old age, valleys become extremely broad with gentle slopes. Considerable development of flood plain and stream meandering prevails. For a more complete treatment of the subject of landform evolution see Sharpe (6) or Lobeck (9).

In accordance with the preceding principles, the United States was divided into eight major divisions representing rather extensive areas of strongly characterized constructional forms such as plains, plateaus, highlands, and mountains. These were subdivided into provinces and sections which represent uniquely different landform areas and destructional history. Delineation of the boundaries of most units corresponds closely to strongly characterized geologic features so that the line of demarcation may be exact to within a few feet. In some areas, however, boundaries are vague and may vary up to several miles (Table 1).

The following quotation from Krynine and Judd (12) are of interest with reference to landslides and physiographic areas:

> "Serious consideration should be given to the regional concept of landslide classification. According to this concept the slides within a geomorphic (or physiographic) province may be defined as an area within which the method of deposition of rocks and soils is approximately the same, landforms are similar, and the climate is approximately identical. This regional concept is accepted by some of the workers interested in landslides, but evidence is still needed."

> "The study of slides leads to the conclusion that the slide characteristics within a given region should depend on the geology, topography, and climate of that region. In fact, often certain slide characteristics are reported either from within a large typical area or from two areas that are similar in some respects. For example, the tremendous Colorado landslides often consist primarily of large boulders. In older glaciated zones both rock and soils are often remarkably stable as in some New England regions, and vice versa, in the regions of the so-called "young geology," e. g., in some parts of the San Francisco Bay area, slide scars on natural slopes are so abundant that they really should be considered as characteristic landforms of the region."

From the foregoing discussion on the destructional stages it appears logical to expect some correlations to exist between the various stages of a specific landform and the severity and type of landslides. According to Sharpe (6) and others, landslides are most likely to occur when the valley walls are the steepest during the transitional period from youth to maturity. Primary road construction in such areas will necessitate considerable cut and fill operations. Any disturbance introduced in the form of a cut or fill invites a situation more favorable for landslide occurrence.

TABLE 3

LIST OF MOST TROUBLESOME LANDSLIDE TYPES
(From Questionnaires of the HRB Committee on Landslide Investigations)

State Highway Department	Rank 1	2	3
Arkansas	Slides	--	--
California	Earth Slump	Earth slump-flow	Debris slide
Colorado	Earth flows	--	--
Delaware	Earth slump	--	--
Idaho	Earth slump-flow	Earth slump	Debris avalanche
Illinois	Earth slump	Earth flow	Debris slide
Kentucky	Debris slide	Earth slump	Rock slump
Maine	Earth slump	Earth flow	--
Maryland	Not major problem		
Massachusetts	Earth slump-flow	--	--
Michigan	Soil fall	Lateral spreading	--
Mississippi	Earth slump	--	--
Missouri	Earth slump	Earth flow	--
Montana	Slides	Rock fall	Solifluction
New Hampshire	Debris slide only		
New Mexico	Rock and earth block glides		
North Carolina	Rock slump	Rock fall	Earth slump-flow
Ohio	Earth slump	Earth flow	Rock fall
Oregon	Earth slump-flow	Earth slump	Debris avalanche
Pennsylvania	Slides	Falls	Flows
Vermont	Flows	--	--
Washington	Earth and rock slumps	Earth slump-flow	Mudflow
West Virginia	Earth slump	Earth flows	Rock fall

PROCESSING DATA

As a preliminary step, the data obtained from the case histories in the questionnaires submitted to the HRB Landslide Investigations Committee (2) were tabulated and analyzed. Information of interest included the total dollars expended by the reporting agency, types of landslides, sizes of the moving masses, and geologic formations associated with slope failures. Additional comments and data on the shape of the slip-surface, type of correction, plan view sketch of the area, causes, etc., were also recorded and used to check landslide classification and areal considerations. The physiographic sections corresponding to specific mass movements were identified from a physiographic map (13). Questionnaires used in the study are listed in Table 2. A total of 24 highway department questionnaires was available, as well as negative statements from six others. The added data from state and federal agencies, railroad companies, and others were also of value.

Table 3 contains a summary of the types of landslides encountered most frequently by the various highway departments. Such information was helpful in determining the landslide types prevalent in the physiographic sections within a state. Furthermore, the preponderance of earth slumps and earth flows throughout the country suggests a degree of similarity as to type of problem encountered.

Tabulations were also made of the type and size of individual landslides reported and the physiographic sections in which they occurred. Tables 4 and 5 are summaries of these data. A total of 527 landslides was studied, with a significantly large group of earth slumps, rock falls, and debris slides. Of the landslides for which sizes were available, more than half were relatively small (less than 50,000 cu yds). The tables also indicate that more case histories were available from the physiographic sections in the eastern part of the country. Whether such a situation developed because of more

TABLE 4

SUMMARY OF NUMBERS OF LANDSLIDES OF VARIOUS MAGNITUDES

Physiographic Section No.	Total Number of Landslides Reported	Physiographic Section No.	Volume less than 5000 cu yds	Physiographic Section No.	Volume 5000-50,000 cu yds	Physiographic Section No.	Volume 50,000-500,000 cu yds	Physiographic Section No.	Volume more than 500,000 cu yds
1. 8d	100	1. 12d	30	1. 12d	27	1. 8e	6	1. 14a	6
2. 8e	83	2. 8e	14	2. 8e	21	2. 8c, 12a,	2	2. 24g	5
3. 24f	67	3. 8d	8	3. 16	8	12f, 13a,		3. 20c	2
4. 12d	64	4. 6b	5	4. 9d, 24g	4	13f, 16,		4. 13f, 19,	1
5. 8c	62	5. 9b, 10,	3	6. 24f	3	20c, 23a		23b, 24b,	
6. 6b	32	7. 4a, 6c	2	7. 4a, 11b,	2	10. 3d, 4a,	1	24d, 24f	
7. 19	25	13f, 14a		12a, 20c,		9b, 12d,			
8. 14a	19	11. 3d, 4b	1	23b, 24a,		12e, 13e,			
9. 12e	14	8c, 12a,		24b		14a, 19			
10. 16	13	12f, 13a,		14. 1, 3c,	1	21c, 24b			
11. 20c	12	13b, 19,		3d, 3e,		24c			
12. 24g	10	22e, 24c		8c, 8d,					
13. 9b	8	24f		9b, 9c,					
14. 4a, 13f	6			11a, 12b,					
16. 9d, 12a	5			12c, 13a,					
18. 12f, 13a,	4			13b, 13f,					
23b,				20a, 21b,					
21. 3d, 6c, 10,	3			24d					
11b, 13b, 24a,									
24b									
28. 9c, 13d,	2								
21b, 23a, 24c,									
24b									
34. 1, 3c, 3e,	1								
9b, 7a, 11a, 12b,									
12c, 13e, 20a, 21c									
Total	587		82		98		33		19

landslides or more thorough coverage is not known.

If one assumes that the amount of money expended is a measure of severity, the results of the questionnaires are of interest. Of the 24 state highway departments reporting, only four indicated that landslide costs exceeded $500,000 per year. Five additional states estimated that between $100,000 and $500,000 were expended annually, while 10 showed an annual cost of less than $100,000. Of the ten railroad companies reporting, only two indicated annual costs in excess of $100,000. Assuming that the 24 states not submitting questionnaires expended less than $25,000 per year, an estimate of the annual expenditure for landslides on highways in the continental United States would approach $10,000,000.

In order to rate the physiographic sections as to the degree of severity, a base was needed for the judgment. Frequency of occurrence as a highway or engineering problem would be different from frequency as a problem in a specific area; that is, landslides may occur in a locale where few highways or other engineering structures are located and could go unreported in an engineering study. Other bases for severity could be size of the moving mass, dollars expended by a company or agency, or number of landslides per unit of area or per mile of highway. For the first attempt, the frequency of occurrence, size of the moving mass, and dollars expended per year were combined in an arbitrary, qualitative manner to arrive at a rating using only the questionnaire data. Another factor in the evaluation was the negative effect of the authors' not having a report from a given state. In effect, it assumes that the problem could not be severe or a questionnaire would have been submitted. Obviously, other reasons could have accounted for the absence of the questionnaire.

Several discrepancies were immediately apparent, and attempts to amplify and to further delineate the areas were made. Such adjustments were partially based upon published records and case histories. The work of Ladd (5), the Highway Research Bibliography (4), and Ta Liang (10) were of significant value in this respect. In many cases, the preliminary classification as to severity was verified. However, the following changes were considered justified, although direct verifications with geologists and engineers were not completed:

1. Sections 16, 19, 20a, and 23a were changed to major severity.
2. Sections 12d, 18, and 20d were changed to intermediate severity.
3. Sections 3a, 13c, 21a, 21d, 21e, 22a, 22e, and 24e were changed to minor severity.

The final results are shown in Table 6 with the preceding list of modifications and are shown graphically in Figure 2.

While the degree of severity as shown in Figure 2 is felt by the authors to be both reasonable and sound, several cautionary statements are in order. First, the delineation between the groups is open to question, although it is probable that even after more study the major severity group will remain as classed. Three groups could be reasonably anticipated for the intermediate and minor groups. It is also probable that the "few to none" class will remain as such.

Secondly, the results tend to indicate the problem severity from an engineering viewpoint of the past and present. As the more remote areas are attacked, some of the areas may become more troublesome. A study and severity analysis from a pure landslide basis is highly desirable.

TABLE 5

SUMMARY OF NUMBERS OF LANDSLIDES OF VARIOUS TYPES REPORTED

Physiographic Section No.	Rock Fall	Physiographic Section No.	Soil Fall	Physiographic Section No.	Rock Slump	Physiographic Section No.	Block Glide	Physiographic Section No.	Soil Slump
8d	56	12a	1	24g	4	24g	5	12d	50
8e	33	8e	1	13d	2	13b	1	8c	40
6b	16			4a	2			8d	30
19	7			9 with	1			8e	25
8c	4							19	12
9 with	1							12e	12
								14a	10
								9b	5
								6b	5
								12f	4
								3 with	3
								10 with	1
Total	125		2		17		6		212

Physiographic Section No.	Rock Slide	Physiographic Section No.	Debris Slide	Physiographic Section No.	Dry Flows	Physiographic Section No.	Earth Flows
6b	8	19	19	16	8	8c	8
8d	2	8e	16	9b	1	8e	5
13f	2	8d	12	9d	1	19	3
19	1	12d	8			23b	2
24b	1	8c	5			24a	
		6b	3			8 with	1
		9b, 9d, 21b	2				
		7 with					
Total	14		76		10		28

Thirdly, the failure of the physiographic sections to relate perfectly to the severity of the landslide problem was noted in a number of cases. For example, section 12e which covers most of the glaciated areas of Ohio, Indiana, and Illinois was interpreted as a major problem in western Illinois, but is certainly not so reported for the same physiographic section in Ohio and Indiana. The eastern portion of West Virginia (section 8e), the western edge of section 3d, northern part of section 12f, and the western edge of section 13f all typify areas that appear to differ from the major portion of the physiographic section.

Finally, the results thus far are quite general, and major differences, particularly in highly localized areas, are to be anticipated. The values of the work thus far completed are highly restrictive, and are of more interest as a guide to further endeavors than for immediate utility.

FUTURE RESEARCH

The immediate goal for the future will be the bolstering of data for the areas where the information available was most sparse. By continuation of the literature study, correspondence with engineers and geologists, and field evaluation, the results shown in Figure 2 can be stated more accurately. Subsequent steps will also include, (1) a closer scrutiny of the magnitude of the mass and the types of landslides which occur in the various areas, (2) development of more data as to the geological formations associated with mass movement, (3) a search for a better basis than pure physiography for regional classification purposes, and (4) evolving a guide for landslide considerations within specific regions.

The conditions in West Virginia can be used to illustrate the degree of detail which will be needed for the various areal groupings, because of the authors' familiarity with

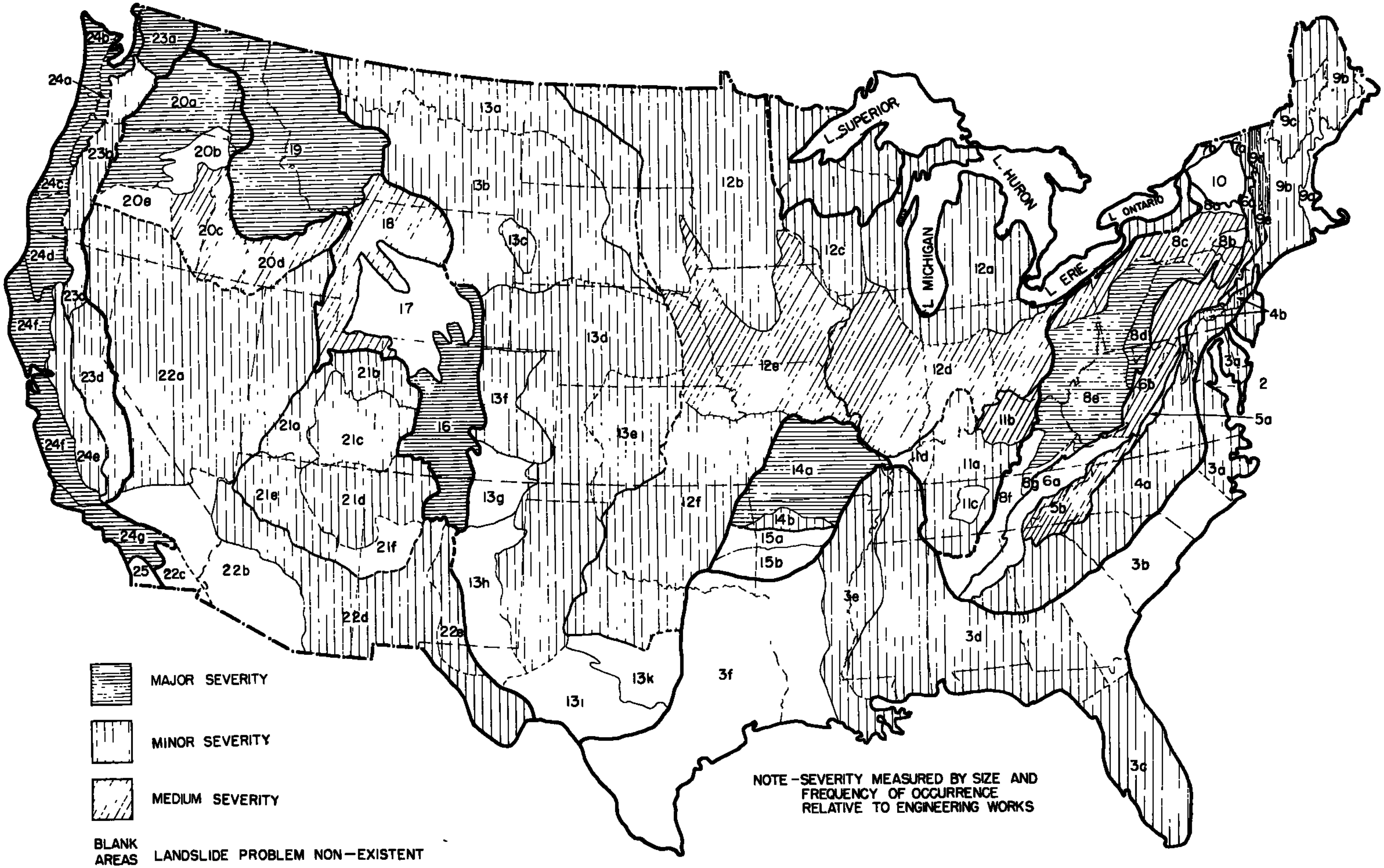


Figure 2. Landslide severity of the United States. Based upon "Physical Divisions of the United States." N. M. Fenneman.

TABLE 6

RATING OF LANDSLIDE SEVERITY
(Based upon HRB Questionnaires and partial literature search)

I. Major Severity
- 8d. Allegheny Mountain section
- 8e. Kanawha section
- 14a. Springfield-Salem Plateaus
- 16. Southern Rocky Mountains
- 19. Northern Rocky Mountains
- 20a. Walla Walla Plateau
- 23a. Northern Cascade Mountains
- 24a. Puget Trough
- 24b. Olympic Mountains
- 24c. Oregon Coast Range
- 24d. Klamath Mountains
- 24f. California Coast Ranges
- 24g. Los Angeles Ranges

II. Medium Severity
- 5b. Southern section of Blue Ridge province
- 6b. Middle section of Valley and Ridge province
- 8c. Southern New York section
- 11b. Lexington Plain
- 12d. Till Plains of the Central Lowland province
- 12e. Dissected Till Plains of the Central Lowland province
- 18. Middle Rocky Mountains
- 20c. Payette section
- 20d. Snake River Plain

III. Minor Severity
- 1. Superior Upland
- 3a. Embayed section
- 3c. Floridian section
- 3d. East Gulf Coastal Plain
- 3e. Mississippi Alluvial Plain
- 4a. Piedmont Upland
- 4b. Piedmont Lowlands
- 6c. Hudson Valley
- 7a. Champlain section
- 9b. New England Upland section
- 9c. White Mountain section
- 9d. Green Mountain section
- 9e. Taconic section
- 11a. Highland Rim section
- 12a. Eastern Lake section
- 12b. Western Lake section
- 12c. Wisconsin Driftless section
- 12f. Osage Plains
- 13a. Glaciated Missouri Plateau
- 13b. Unglaciated Missouri Plateau
- 13c. Black Hills
- 13d. High Plains
- 13e. Plains Border
- 13f. Colorado Piedmont
- 14b. Boston "Mountains"
- 21a. High Plateaus of Utah
- 21b. Uinta Basin
- 21c. Canyon Lands
- 21d. Navajo section
- 21e. Grand Canyon section
- 22a. Great Basin
- 22d. Mexican Highland
- 22e. Sacramento section
- 23b. Middle Cascade Mountains
- 23c. Southern Cascade Mountains
- 23d. Sierra Nevada
- 24e. California Trough

IV. Non-Existent Problem
- 2. Continental Shelf
- 3b. Sea Island section
- 3f. West Gulf Coastal Plain
- 5a. Northern section of Blue Ridge province
- 6a. Tennessee section
- 7b. Northern section of the St. Lawrence Valley province
- 8a. Mohawk section
- 8b. Catskill section
- 8f. Cumberland Plateau
- 8g. Cumberland Mountains
- 9a. Seaboard Lowland section
- 10. Adirondack province
- 11c. Nashville Basin
- 11d. Western section of the Interior Low Plateaus
- 13g. Raton section
- 13h. Pecos Valley
- 13i. Edwards Plateau
- 13k. Central Texas section
- 15a. Arkansas Valley
- 15b. Ouachita Mountains
- 17. Wyoming Basin
- 20b. Blue Mountain
- 20e. Harney section
- 21f. Datil section
- 22b. Sonoran Desert
- 22c. Salton Trough
- 25. Lower California province

TABLE 7

SUMMARY OF TYPE AND SIZE FOR GROUP OF CENTRAL WEST VIRGINIA LANDSLIDES

Landslide Type	Less than 5000	5000-25,000	25,000-50,000	Greater than 50,000	Total
Earthflow	1	4	1	1	7
Slump	4	4	3	4	15
Earth Block Glide	0	0	3	0	3
Total	5	8	7	5	24

TABLE 8

SUMMARY OF 742 VIRGINIA LANDSLIDES[1]

Geologic Formation	Percent of Total Landslides Studied	Approximate Outcrop Area (sq mi)
Alluvium	7	
Dunkard	21	2500
Monongahela	20	1500
Conemaugh	33	1500
Allegheny	2	1500
Pottsville	9	4000
Miscellaneous	8	13000[2]
Total	100	24000

[1] Conducted by John L. Wray, former geologist, West Virginia State Road Commission.
[2] Includes Alluvium.

the region. The state of West Virginia contains approximately one-third of the Kanawha Section (8e), one of the areas of major severity. Much has been written specifically about the landslide problem in the region (14, 15, 16, 17, 18). Other studies not previously reported in the literature have also been conducted. A survey completed in November, 1952, indicated more than 750 known landslides in the state. Unfortunately, complete information on all of these landslides is not available. A partial analysis from the viewpoint of geologic formations was conducted in 1953 by John L. Wray, a geologist formerly with the Department of Soil Mechanics of the West Virginia State Road Commission. The results are shown in Table 7.

All but a small area of eastern West Virginia lies in the Kanawha Section. A map of the location of the many landslides was submitted to the HRB Landslide Committee along with the questionnaire. Practically no landslides were reported northwest of the line that extends southwesterly from the southwest tip of Maryland to Williamson, in southwest West Virginia. The line is considerably west of the eastern boundary of the Allegheny Section. Thus, physiographic sections in themselves are not sufficient to describe landslide susceptibility. In a map which accompanies a Civil Aeronautics Administration report (11), soil boundaries are drawn which come much closer to properly delineating the landslide area of the Kanawha Section.

Also available to the authors were the data from a special study of certain landslides in Braxton and Gilmer counties in central West Virginia. These landslides (Table 8) illustrate the range of sizes and types of mass movement which are most common in West Virginia. It will be noted that the vast majority are relatively small (less than 50,000 cu yds) and more than half involve less than 25,000 cu yds. It is also evident that slumps (soil) and earthflows are most frequently encountered.

Reports by Ladd (5), Baker (19), and others have indicated the presence of numerous rock falls in West Virginia. While very little general data as to size and frequency are available, the problem can be described as general throughout the landslide-susceptible area. With notable exceptions, the masses involved are not great since the falls consist principally of fine-grained weathering products, but they do include individual boulders as large as 25 cu yds. The failures usually develop as the result of differential weathering; that is, a weak, less resistant layer underlies a blocky, exfoliated or jointed strata.

The additional data from West Virginia indicates that the Conemaugh, Dunkard, and Monongahela formations of the Pennsylvanian System are related to 74 percent of the landslides studied. Furthermore, earth slumps, earth block glides, earthflows, and rockfall represent the greatest problem in terms of frequency of occurrence. Most of the landslides involve mass movements of quantities less than 50,000 cu yds. It is also known that the present physiographic boundary is not exact in defining landslide susceptibility. Much more is known and much more is needed concerning West Virginia and the remainder of the Kanawha Section, but the restrictive nature of the problem in the area is evident.

With reference to developing data as to the geologic formations associated with landslides, Table 9 lists the principal offenders as shown by the questionnaire. As in the preceding example for the Kanawha Section, it appears certain that relatively few geologic formations are associated with the landslides in a specific region. The availability of such information should aid in the development of a comprehensive theory.

Landslide types within a state or physiographic section are listed in Table 2, and the range of sizes of the moving masses is given in Table 3. Systematic evaluation of these data will simplify the landslide problem in a specific area and will permit a better understanding by highway engineers. The absence of a clear concept of the limited nature of landslide occurrence in a locale currently requires a complete understanding of all types of mass movements. The result is a hopeless jumble of terminology and

TABLE 9

LANDSLIDE SUSCEPTIBLE FORMATIONS

Formation or Stratigraphic Sequence	Location	Reference[1]
Pierre, Carlile, Graneros Shales, Dakota Sandstone, Denver and Araphoe	Colorado	1
Pierre Shale and Fort Union overlain by till	North Dakota	1
Gaconade and Cherokee	Missouri	1
Morrison and Sundance Clays and Shales	Gros Ventre River Valley-Middle Rocky Mountains Section-Wyoming	2
Chinle Shales	Vermillion and Echo Cliffs-Grand Canyon Section-Arizona	2
Cox Shales	Diablo Plateau-Mexican Highland Section-Texas	2
Kaibab overlying weak shales and gypsum beds	Unikaret Plateau-Arizona	2
Fruitland Shale	LaPlata County-Navajo Section-Colorado	2
Columbia Lava overlying soft lacustrine beds	Northern Cascade Mountains-Washington	2
Navajo Sandstone overlying extensive fractured and jointed formation	Zion Canyon-High Plateau of Utah Section-Utah	2
Eagle Ck. volcanic breccia underlying Columbia lava	Columbia River Gorge-Oregon	1
Eden shales and limestones	Harrison County, Kentucky	3
Quaternary alluvium (37%), Franciscan sandstone (34%), serpentine (7%) and Franciscan greenstone (6%)	San Francisco North Quadrangle, California Coast Ranges Section-California	1
Nespelem silts, Astoria siltstone and Eocene shales	Washington	1
Hawthorne clays	Florida	1
Massive basalt underlain by weaker layers	Columbia Plateaus Province	3
Bearpaw shale and bentonite seams	Fort Peck Dam-Montana	3
Payette	Payette section-Idaho	1
Merchantville and Woodbury clays, Woodbridge	New Jersey	1
Tablot-Wicomica, Wissahickon	Delaware	1
Rincon Shale, Serpentine	California Coast Ranges-California	1
Jackson Clays	Natchez Trace Parkway-Jackon-East Gulf Coastal Plain Mississippi	3
Mancos Shales overlying:		
a. competent Mesaverde sandstone	Montezuma County-Canyon Lands Section-Colorado	3
b. glacial till	Telluride area-Southern Rocky Mountain Section-Colorado	3
Pottsville, Allegheny, Conemaugh, Monongahela, Dunkard	Pennsylvania	1
Eden shales and limestones, Conemaugh and Dunkard shales	Ohio	1
Conemaugh and Dunkard shales	Kanawha section-West Virginia	1

[1] 1. Highway Research Board Questionnaire submitted to Committee on Landslide Investigations.
2. Tompkin, J. M., and Britt, S. B., "Landslides—A Selected Bibliography," Bibliography No. 10, HRB (1951).
3. Liang, Ta, "Landslides—An Aerial Photographic Study." Unpublished Tesis for Degree of Doctor of Philosophy, Cornell University (1952).

empirical relationships for the engineer with insufficient time to develop a comprehensive background.

Perhaps the most significant indication from the preliminary studies reported here is the fact that physiographic section boundaries are not sufficient in themselves for delineating landslide severity. The most striking example is the Dissected Till Plains Section of the Central Lowland (12e) where landslides appear to be heavily concentrated along the Mississippi River, within a relatively small percentage of the total section area. Part of this discrepancy may be because of improper location of boundaries, or of the difficulties inherent in attempting to group the highly variable components of the earth's surface. The fact that unexplained differences exist means that either the physiographic regional bases are not sound for relating landslide susceptibility or the areas are too large and a further subdivision is needed.

An interesting observation relative to apparent physiographic discrepancies is the presence of landslides near major water courses. The Pacific Coast, the Great Lakes, the Mississippi River, and the Ohio River and its tributaries are all prime examples. Ta Liang (10) and others have also noted the relationship between rivers and landslides. The basic tenent of areal erosion and the attendant landslide influence suggests such a relation. However, the difference in severity is not completely explained by the presence or absence of a major water course. Perhaps some combination of physiography, surface drainage system, pedology (11), or other dominant factors will provide the ultimate basis for classification.

CONCLUSIONS

Based upon the Highway Research Board questionnaires submitted by various state and federal agencies, companies, and consultants, and upon a limited literature search the following conclusions are offered:

1. Efforts to relate degree of severity of landslides to standard physiographic sections produced encouraging results, although several deviations were noted. The degree of severity was defined as a function of the effect on engineering works as opposed to general landslide susceptibility. Both magnitude of moving mass and frequency of occurence were considered in assigning the measure of severity.
2. Unquestionably, the most severe landslide problems exist in the following physiographic sections: Allegheny Mountain section (8d), Kanawha section (8e), Springfield-Salem Plateaus (14a), Southern Rocky Mountains (16), Northern Rocky Mountains (19), Walla Walla Plateau (20a), Northern Cascade Mountains (23a), Puget Trough (24a), Olympic Mountains (24b), Oregon Coast Range (24c), Klamath Mountains (24d), California Coast Ranges (24f) and Los Angeles Ranges (24g).
3. Equally evident is the fact that landslide problems in the following sections are practically non-existent: Continental Shelf (2), Sea Island section (3b), West Gulf Coastal Plain (3f), Northern section of the Blue Ridge Province (5a), Tennessee section (6a), Northern section of the St. Lawrence Valley province (7b), Mohawk section (8a), Catskill section (8b), Cumberland Plateau (8f), Cumberland Mountains (8g), Seaboard Lowland section (9a), Adirondack province (10), Nashville basin (11c), Western section of the Interior Low Plateaus (11d), Raton section (13g), Pecos Valley (13h), Edwards Plateau (13i), Central Texas section (13k), Arkansas Valley (15a), Ouachita Mountains (15b), Wyoming Basin (17), Blue Mountains (20b), Harney section (20e), Datil section (21f), Sonoran Desert (22b), Salton Trough (22c), and Lower California province (25).
4. The remaining physiographic sections showed a range between the preceding two, and a precise delineation is somewhat open to question.
5. Continued research in the establishment of landslide-susceptible areas is recommended, with the following major objectives:

   a. Continued literature search in order to incoporate all existing knowledge into the framework, and to lend impetus to the study;

   b. A detailed examination of the several physiographic sections in order to better establish the degree of severity, the types of landslides that occur, the range in magnitude of the landslides, and the geologic formations which are associated with mass movements;

c. Examination of possible modifications and subgroupings which will better define a region's susceptibility to landslide problems;

d. Development of a comprehensive description of the landslide problem on a regional basis in order to provide breadth of knowledge and to simplify landslide treatment within a specific area;

e. With the better understanding that will result from such a systematic delineation it will be possible to study the development of a rational theory for landslide treatment. The existing practice of applying an adjusted theory to the problem is leading to its own form of empiricism because of the absence of a clear concept of the basic components.

ACKNOWLEDGMENTS

The authors wish to express their gratitude to the members of the Landslide Investigations Committee and the administrative officials of the Highway Research Board for making the questionnaires available. The cooperation of the many officials who submitted the questionnaires to the Committee is also sincerely appreciated.

REFERENCES

1. Terzaghi, Karl, and Peck, R. B., "Soil Mechanics in Engineering Practice." John Wiley and Sons (1948).
2. "Landslides and Engineering Practice." HRB Committee on Landslide Investigations.
3. Collin, Alexandre, "Landslides in Clays." Translation by W. R. Schriever, University of Toronto Press (1956).
4. Tompkin, J. M., and Britt, S. B., "Landslides—A Selected Annotated Bibliography." HRB Bibliography No. 10 (1951).
5. Ladd, G. E., "Landslides, Subsidences and Rock Falls." Bulletin, American Railway Engineering Association, Vol. 37, No. 377 (July 1935).
6. Sharpe, C. F. S., "Landslides and Related Phenomena." Columbia University Press (1938).
7. Terzaghi, Karl, "Theoretical Soil Mechanics." John Wiley and Sons (1943).
8. Fenneman, N. M., "Physiographic Divisions of the United States." Annals of the Association of American Geographers, 6:19-98 (1917).
9. Lobeck, A. K., "Geomorphology." McGraw Hill Book Company, Inc., New York (1939).
10. Liang, Ta, "Landslides—An Aerial Photographic Study." Unpublished Thesis for Degree of Doctor of Philosophy, Cornell University (1952).
11. Belcher, D. J., Gregg, L. E., and Woods, K. B., "The Origin, Distribution and Airphoto Identification of United States Soils." Technical Development Report, CAA, No. 52 (1946).
12. Krynine, D. P., and Judd, W. R., "Principles of Engineering Geology and Geotechnics." McGraw Hill Book Company, Inc. (1957).
13. Fenneman, N. M., "Physical Divisions of the United States." Map, United States Geological Survey (1949).
14. Baker, R. F., "Analysis of Landslides." HRB Bull. 49 (1952).
15. Baker, R. F., "An Analysis of Corrective Actions for Highway Landslides." Transactions, ASCE (1953).
16. Ladd, G. E., "Landslides and Their Relation to Highways." Public Roads, Vol. 8, pp. 21-31, 35 (1927); Vol. 9, pp. 153-163 (1928).
17. Woods, K. B., and Litehiser, R. R., "Soil Mechanics Applied to Highway Engineering in Ohio." Engineering Experiment Station, Bull. 99, The Ohio State University (1938).
18. Price, P. H., and Lilly, K. O., "An Investigation of Landslides Affecting Roads in West Virginia." Unpublished report to the State Road Commission of West Virginia (1940).

19. Baker, R. F., "The Design of the Slope of Highway Rock Excavations in West Virginia." Proceedings, Third Annual Symposium on Geology as Applied to Highway Engineering (Feb. 1952).

An Appraisal of Measures for Improvement of Slope Stability[1]

R. G. HENNES, Professor of Civil Engineering, University of Washington;
S. E. HAWKINS[2], Southern Railway System, Asheville, N. C.; and
E. L. McCOY[2], U. S. Corps of Engineers, Portland, Oregon

Generally, attempts to increase the factor of safety of an earth slope involve either drainage or excavation. The object of drainage is a lowering of the water table, with an accompanying reduction in the magnitude of unfavorable forces. In the present paper several graphs are presented which enable the engineer to estimate the amount of drainage necessary to achieve a desired factor of safety. These graphs yield safety factors corresponding to various levels of water table in an earth mass where the failure plane would approximate a Swedish arc located in a clay bank underlain by a permeable stratum.

An alternative procedure improves stability by unloading the slope. It is shown that flattening the slope is much less effective than benching per unit of excavation. Graphs are presented which plot factor of safety against quantity of excavation for both benching and slope reduction. Both $\phi = 0$ and $\phi > 0$ Cases are considered.

● WHEN THE STABILITY of an earth mass is deemed to be unsatisfactory, the engineer entrusted with the task of improving the stability of the slope will elect, generally, either to flatten the slope or to drain the unstable bank. Existing methods of analysis are adequate for the determination of the factor of safety of the bank both before and after the execution of the measures taken for the betterment of stability. The objective of the present paper is to extend the application of existing analytical procedures to the point where an engineer may design his landslide control measures to achieve a preselected factor of safety, rather than by trial and error determine the factor of safety of a tentative design.

REMEDIAL EXCAVATION

The increase in stability can be achieved by flattening a clay bank is illustrated in Figure 1. It is assumed that seepage forces are negligible in a mass of medium clay 40 ft high. The pertinent soil properties are:

unit weigth, γ = 120 lb per cu ft
cohesion, c = 1000 lb per sq ft, and
friction, ϕ = 6 deg

It is evident that flattening this slope (increasing the value of b) serves to increase the factor of safety. If a rise of 1 ft is

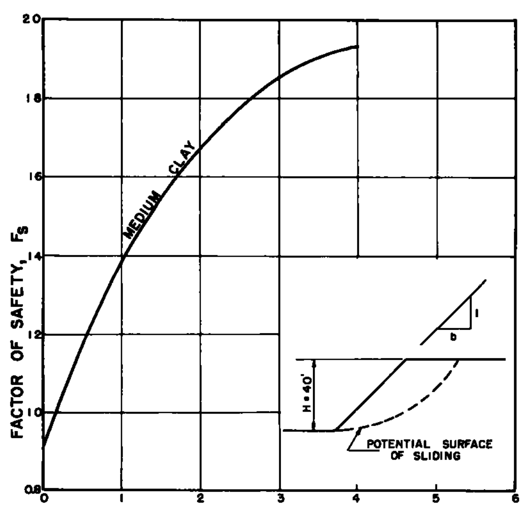


Figure 1.

[1] This paper is based on two MSCE theses: "The Effect of Drainage in Landslide Control," by Eugene L. McCoy (1955); and "Slope Deformation and Safety as Related to Railway Landslide Problems," by Sidney E. Hawkins (1955). The research was supervised by R. G. Hennes.
[2] Formerly Research Assistants, Engineering Experiment Station, University of Washington.

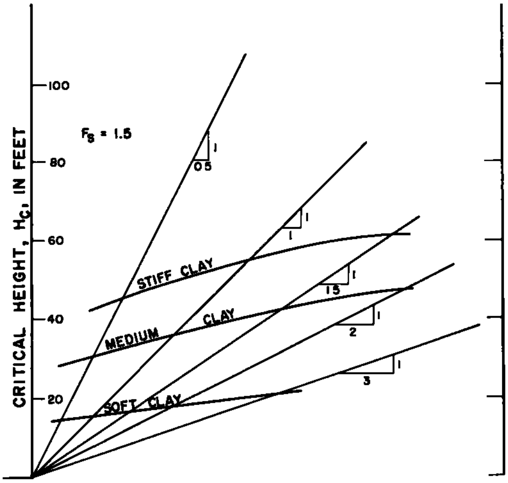


Figure 2.

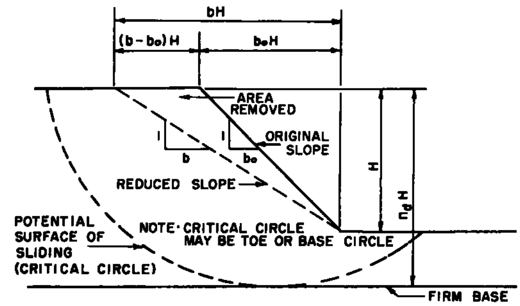


Figure 3.

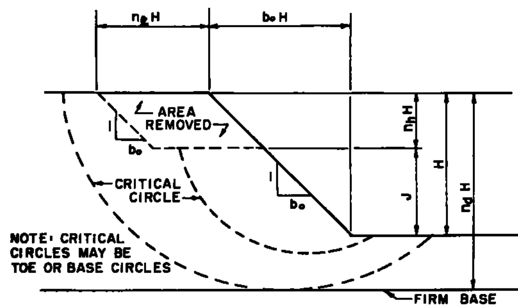


Figure 4.

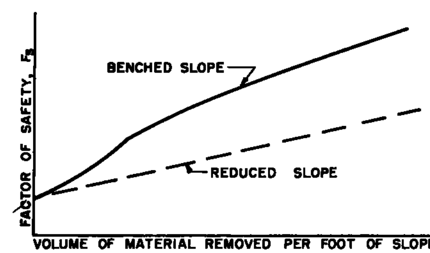


Figure 5.

achieved in a horizontal run of 2 ft instead of 1 ft, the factor of safety is increased from about 1.38 to about 1.67.

More information on the relationship between slope angle and stability is presented in Figure 2, which shows the critical heights corresponding to various slope angles in specific soils. Conventionally, critical height is the maximum height at which a bank will stand (factor of safety of unity) at a given slope angle. However, Figure 2 is based upon a factor of safety equal to 1.5 rather than unity, as a more useful value. The assumed properties of the selected clays are:

	Stiff	Medium	Soft
γ =	130	120	110 pcf
c =	1500	1000	400 psf
ϕ =	8 deg	6 deg	4 deg

It is noteworthy that slope angle is not a dominant factor in the stability of the more plastic clay banks.

Both Figures 1 and 2 were derived from curves showing the stability factor, N_s, for different slope angles and different soil properties by N. Janbu (1). N_s is a pure number showing how stable a slope is for a particular homogeneous earth material. The factor of safety, F_s, as used on these plates is found by

$$F_s = N_s \frac{C}{\gamma H}$$

in which

c = cohesion of soil in lb/ft^2
γ = unit wt of soil in lb/ft^3
H = height of slope

Janbu's curves give information similar to the stability curves presented by D. W. Taylor (2). However, factors of safety derived from Taylor's curves will generally be somewhat higher.

<u>Reduction of Slope Angle</u>. A procedure for removing material is to reduce the slope

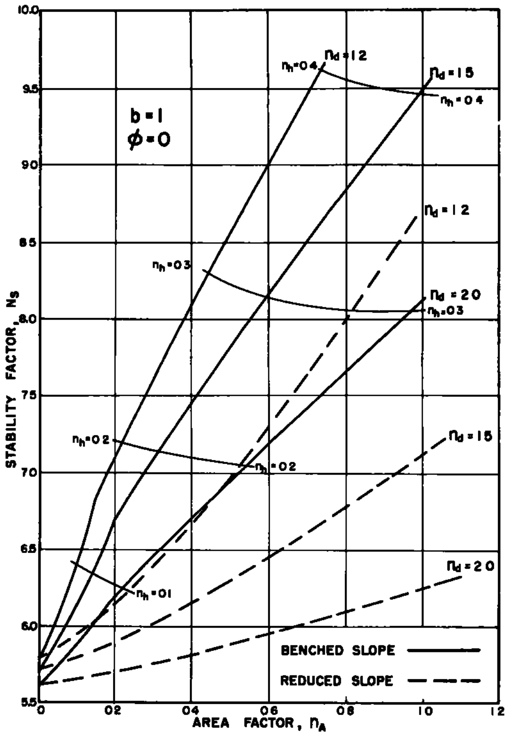


Figure 6.

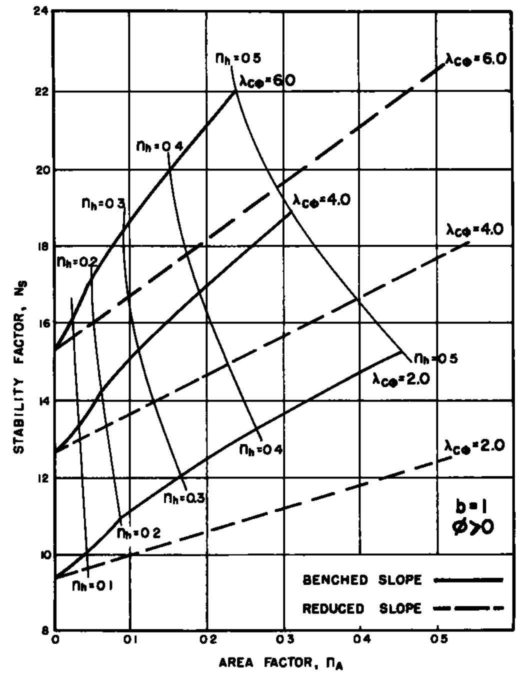


Figure 7.

angle as in Figure 3. Referring back to Figures 1 and 2, it is evident that this procedure would increase the slope safety.

Already one objection to decreasing the slope angle has been mentioned. In addition, for large reduction of the slope angle, new right-of-way would be required to accommodate the flatter slope. Later discussion will also reveal that this method is not the most economical with respect to the amount of material moved.

Benching. A second method for unloading a slope is benching. Here the material near the crest of the slope is removed, leaving a bench part way down the slope. This type of removal takes material from the part of the slope where it will do the most good. Figure 4 is a cross-section of a benched slope.

The benching method is more effective because material is removed from the upper part of the slope. All of the material in this location provides a large driving force to produce slope failure.

Figure 5 shows the increase in factor of safety due to benching and slope reduction. Both curves start from the same point which is the factor of safety for the original slope prior to unloading by either

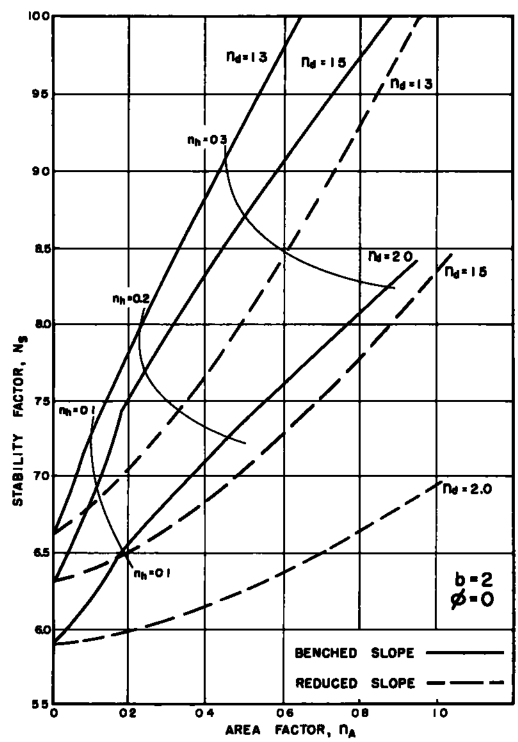


Figure 8.

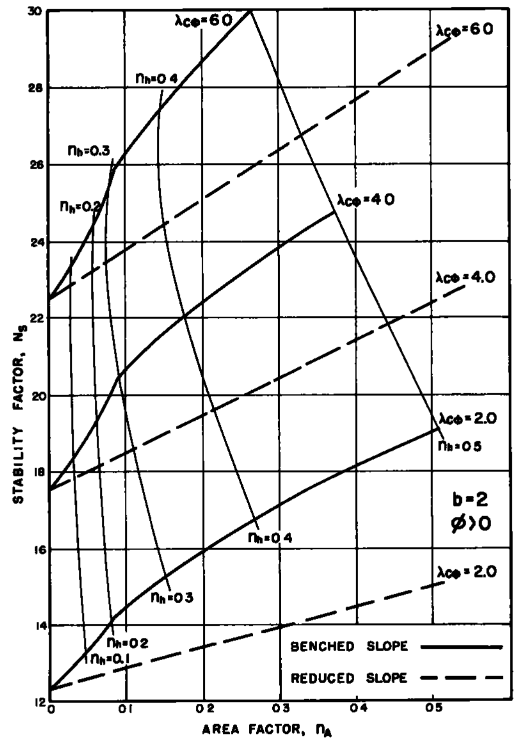
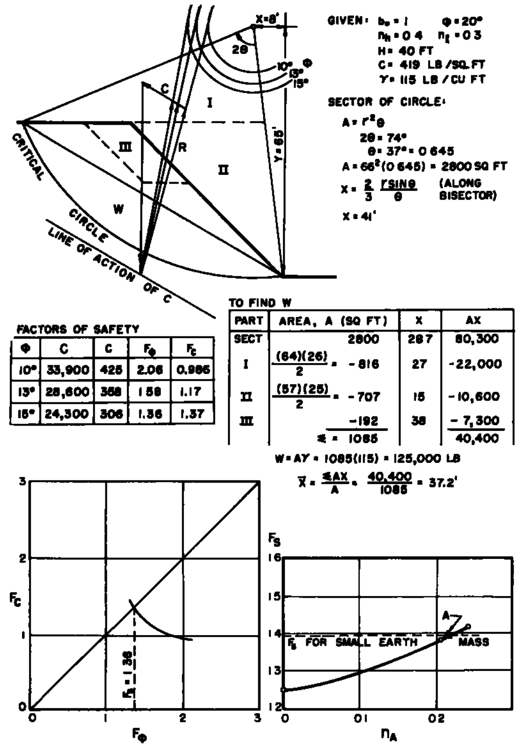


Figure 9.



FACTORS OF SAFETY

ϕ	C	c	F_ϕ	F_c
10°	33,900	425	2.06	0.985
13°	28,600	358	1.58	1.17
15°	24,300	306	1.36	1.37

TO FIND W

PART	AREA, A (SQ FT)	X	AX
SECT	2800	28.7	80,300
I	$\frac{(64)(26)}{2} = -816$	27	-22,000
II	$\frac{(57)(25)}{2} = -707$	15	-10,600
III	-192	38	-7,300
	Σ = 1085		40,400

Figure 10.

method. The increase in F_s due to benching is achieved by lowering and widening the bench. F_s will increase until the bench is halfway down the slope. Any further lowering will not increase F_s. The proper relationship between depth and width of the bench can be obtained from curves to follow.

As shown in Figure 5, the safety of a slope can be increased any set amount for less excavation by benching than by slope reduction. The results of this study also show that the greater the soil strength the less is the excavation required to produce a given increase in F_s by benching or slope reduction.

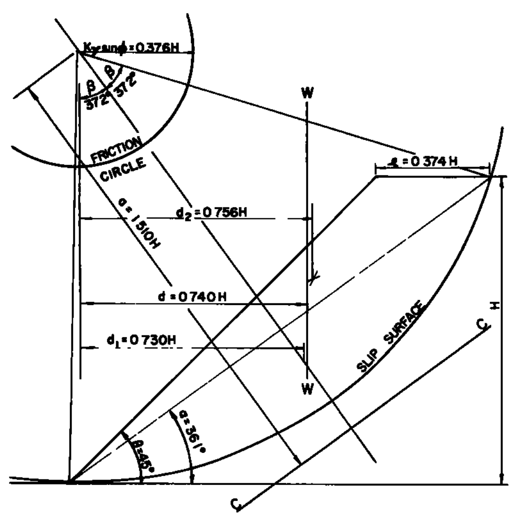
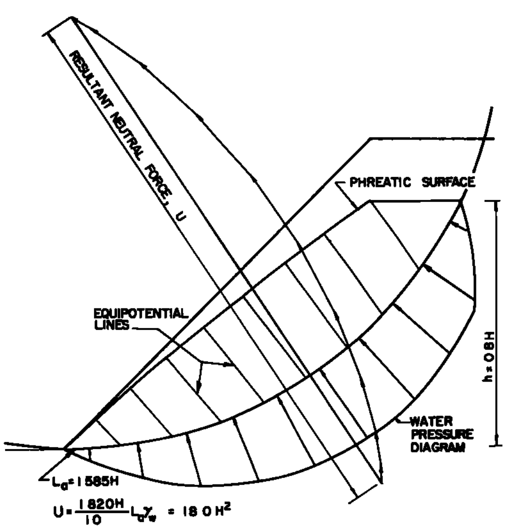


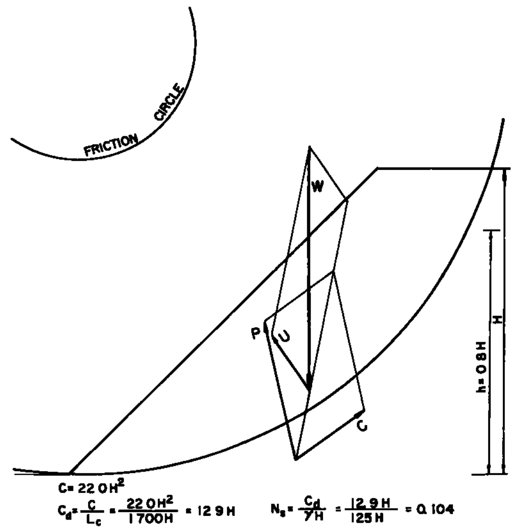
Figure 11.

To aid in the determination of the proper bench dimensions and to compare the benching and slope reduction methods, the curves in Figure 6 through 9 have been prepared. They pertain only to slopes of homogeneous earth material with negligible seepage forces. These curves are in general the same as found in Figure 5 except F_s has been replaced by N_s, the dimensionless stability factor already mentioned. This group of curves includes original slopes of 1 to 1 and 2 to 1. Figures 6 and 8 are for purely cohesive soils such as clays in which the angle of internal friction is zero.



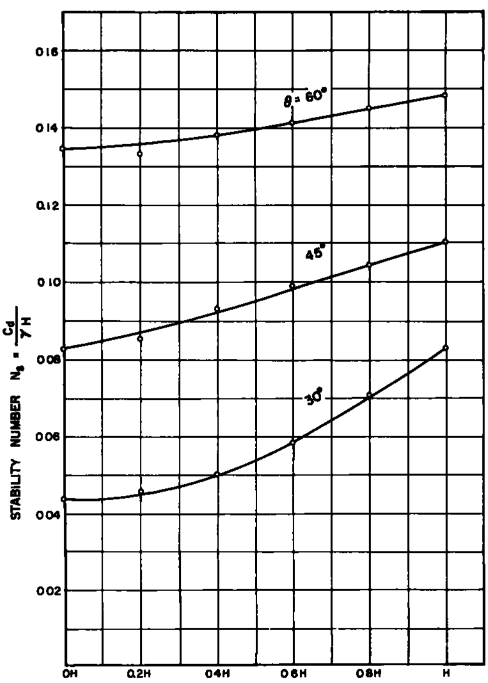
ϕ = 15°
h = 0.8H

Figure 12.



ϕ = 15°
h = 0.8H

Figure 13.



ϕ = 15°

Figure 14.

Curves are presented for three values of n_d the depth factor. The reader is referred to Figure 3 for the definition of n_d. Figures 7 and 9 show soils exhibiting internal friction. Curves for three values of $\lambda_{c\phi}$ are included on each.

$$\lambda_{c\phi} = \frac{\gamma H \tan\phi}{c}$$

in which ϕ = angle of internal friction,

On all curves the volume of excavation is found by:

$$V = n_A H^2$$

The dimensionless quantity n_A, the area factor, comes from the curves.

The depth of the bench for a particular value of N_s is found by interpolating between the lines marked n_h, the bench depth factor, also a dimensionless number.

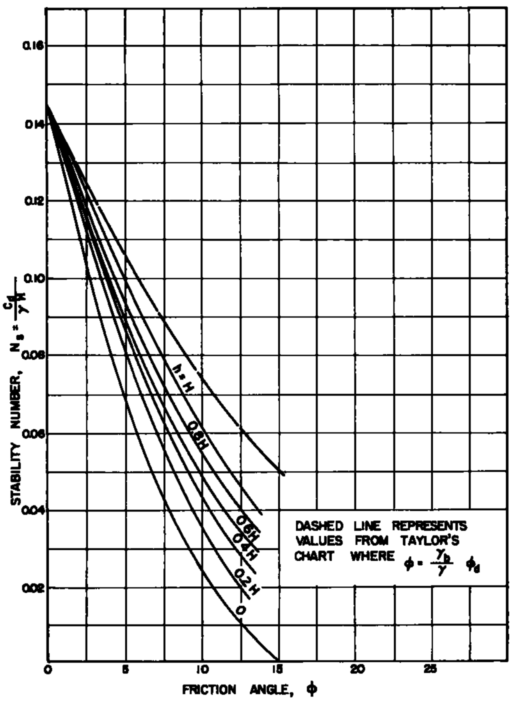
$$\text{Depth of bench} = n_h H$$

The bench width can now be found by

$$\text{Width of bench} = \frac{n_A}{n_h} H$$

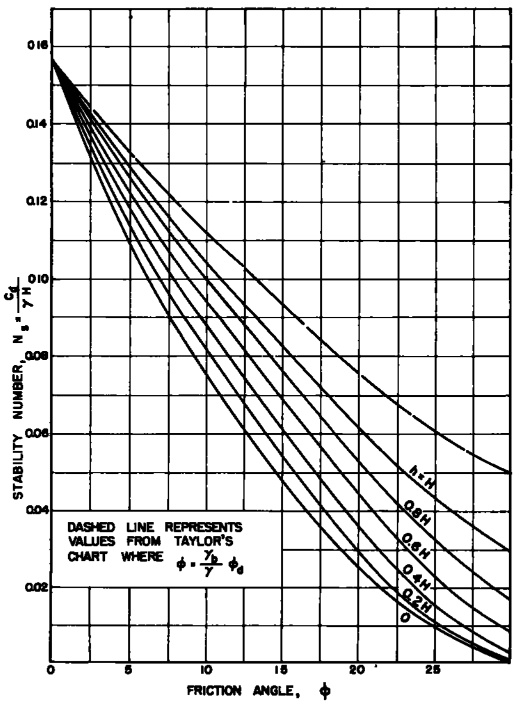
For the slope reduction curves, the reduced slope may be found by the use of n_A or

$$b = 2n_A + b_o$$



15° SLOPE

Figure 15.



30° SLOPE

Figure 16.

in which

b = reduced slope

n_A = area factor

b_o = original slope

Example Problem 1. A problem is presented in the determination of proper bench dimensions to increase the safety factor of a slope to 1. 5.

Given: Initial slope, $b_o = 1$

$c = 600 \text{ lb/ft}^2$

$\phi = 22$ deg

$\gamma = 120 \text{ lb/ft}^3$

$H = 60$ ft

For these given conditions the evaluation of $\lambda_{c\phi}$ is

$$\lambda_{c\phi} = \frac{\gamma H \tan\phi}{c} = \frac{(120)(60) \tan 22 \text{ deg}}{600} = 4.85$$

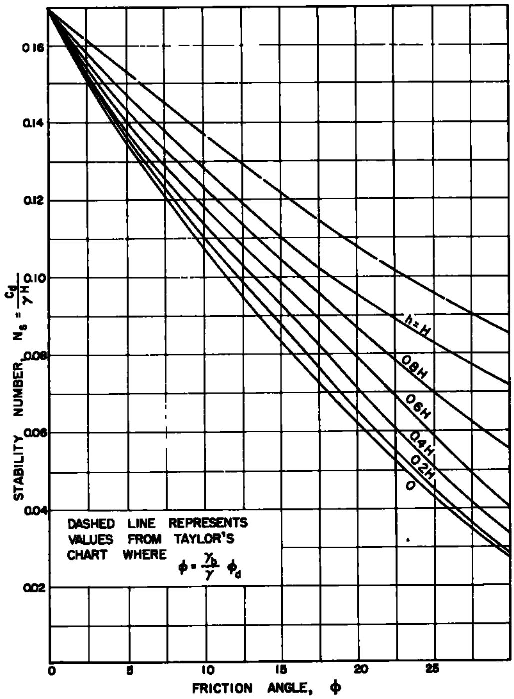
The stability number desired is

$$N_s = F_s \frac{\gamma H}{c} = 1.5 \frac{(120)(60)}{600} = 18$$

By use of Figure 7, n_h and n_A can be determined by interpolation between $\lambda_{c\phi} = 4$ and $\lambda_{c\phi} = 6$.

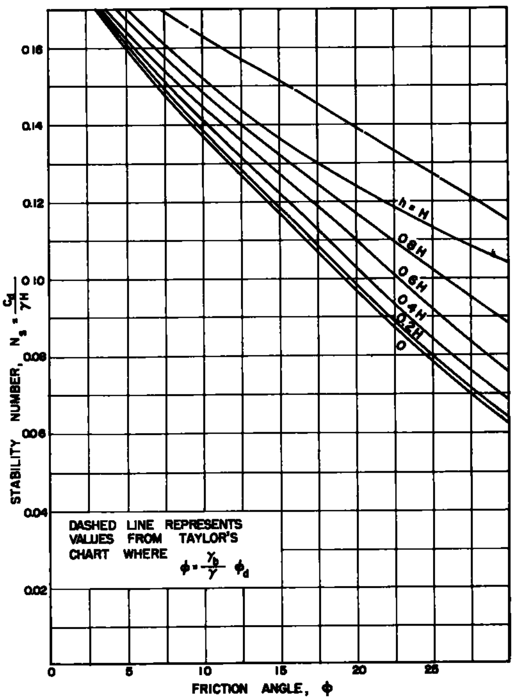
$n_h = 0.38$

$n_A = 0.16$



45° SLOPE

Figure 17.



60° SLOPE

Figure 18.

Now the volume of excavation, depth, and width of the bench can be computed.

$$V = n_h H^2 = 0.16(60)^2 = 575 \text{ ft}^3\text{/lineal foot of slope}$$

$$\text{Depth of bench} = n_h H = 0.38(60) = 23 \text{ ft}$$

$$\text{Width of bench} = \frac{n_A}{n_h} H = \frac{0.16}{0.38}(60) = 25 \text{ ft}$$

Example Problem 2. In this problem the same given quantites found in problem 1 will be used but the factor of safety will be increased to 1.5 by reduction in slope angle.

Given: Initial slope, $b_o = 1$

$c = 600$ lb/ft^2
$\phi = 22$ deg
$\gamma = 120$ lb/ft^3
$H = 60$ ft

$\lambda_{c\phi}$ and N_s will also be the same as in problem 1.

$\lambda_{c\phi} = 4.85$
$N_s = 18$

Interpolating between the dashed lines for $\lambda_{c\phi}$ of 4 and 6 in Figure 7, n_A will be

$$n_A = 0.34$$

The volume of excavation and reduced slope are

$$v = n_A H^2 = 0.34(60)^2 = 1220 \text{ ft}^3\text{/lineal foot of slope}$$

$$b = 2n_A + b_o = 2(0.34) + 1 = 1.68$$

As seen in Figure 3, the slope crest must be moved back a distance $(b - b_o)H$. For this problem

$$(b - b_o)H = (1.68 - 1)\,60 = 41 \text{ ft}$$

From the results of these two problems it can be readily seen how much more effective benching is than slope reduction for increasing the safety of a slope against failure. Slope reduction requires more earth removal and a wider area of excavation at the slope crest. If slope unloading is being considered for controlling a landslide situation, the results of the curves presented here indicate that benching should be considered as a method for accomplishing the desired result.

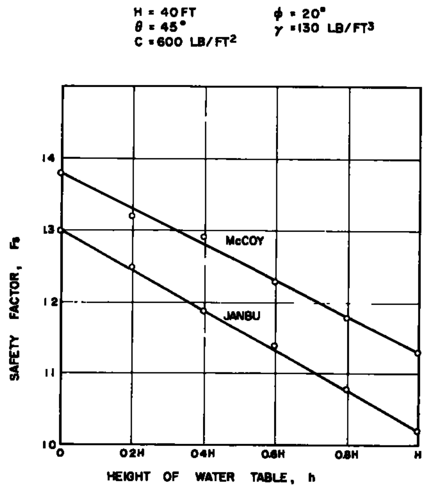


Figure 19.

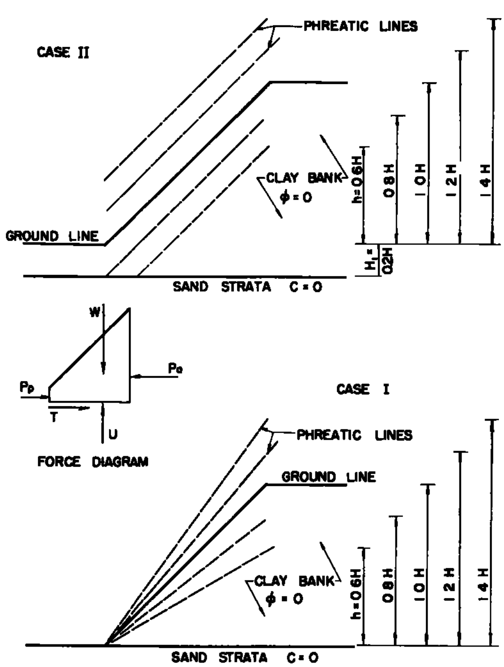


Figure 20.

Remedial Drainage

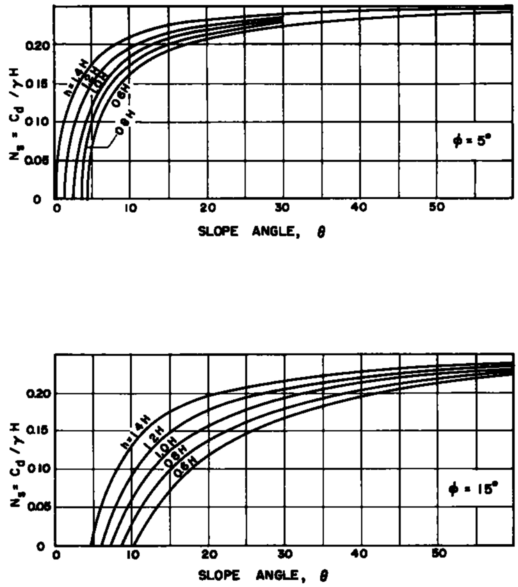
It is generally recognized by engineers that the installation of drainage works in an earth embankment or cut has as its purpose the relief of hydrostatic pressures rather than any substantial reduction in the water content of the soil. In this connection the term "water table" refers to that surface which is the locus of atmospheric pressure in the porewater, and not to any significant boundary between saturated and unsaturated soil. For twenty years engineers have had available procedures for determining the location of the water table subsequent to the installation of drainage, at least in idealized situations; and they have been able to compute the factor of safety corresponding to predetermined seepage patterns. Such analyses, however, have involved a laborious trial-and-error process for each drainage situation. The engineer has had little guidance in his preliminary appraisal of the over-all effects of any specific depression of the water table. The graphs presented herewith are intended to repair this deficiency.

Reduction of Porewater Pressure in Clay Banks

In order to present graphically with some quantitative values for a changing water table level, computations were made illustrating the effect of different water table levels upon several slopes with different soil strengths. The procedure used was taken from Taylor (2). Henceforth material from this reference will be labeled "Taylor's values."

The method of slope analysis employs the Swedish circle, solving for the forces graphically by the ϕ-circle method. As an illustration of the method used to obtain the curves, a slope of 45 deg with a friction angle of 15 deg was chosen and is shown in the figures. The solution is independent of height of the slope. Figure 11 shows the critical slip circle and lines of action of the weight of the soil mass and of the cohesion derived from Taylor's values. Figure 12 illustrates the method used in determining the resultant neutral force for the different water table levels.

The shape of the upper part of the flow net is one that will never be found in nature. However, it is felt that in order to standardize the flow nets for comparison and to avoid introducing other variables into the solution, the assumption shown is necessary. The top flow line is assumed to be tangent at the toe of the slope. This may also vary in natural conditions; however, the variation would be difficult to determine and unnecessarily complicate the solution. In any case, the error involved between computing the pressures used in this paper and actual pressures is very small and would not affect the values obtained for the stability numbers. Figure 13 shows the force diagram used in the computations of the stability numbers.



CASE I, $H_1 = 0$

Figure 21.

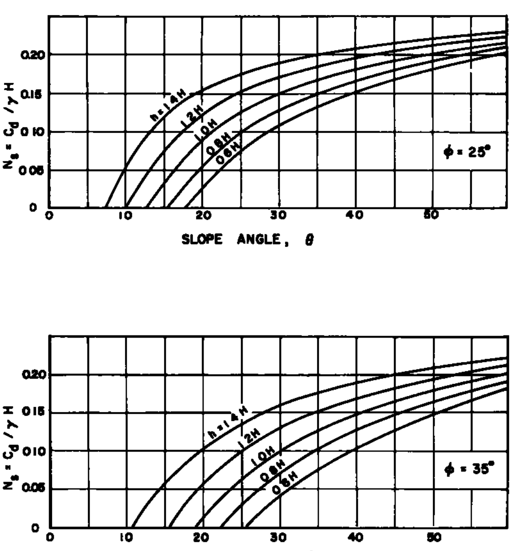
The stability numbers computed for the several slopes and friction angles were plotted against the water table heights and a smooth curve drawn through the points as shown in Figure 14. Values of the stability number for a water table height of 0.2H were rather scattered in all curves because the graphical solution of the force diagram required the use of very small angles.

The results of the computations are shown in Figures 15 through 18. The curve for h= 0 is the curve shown on Taylor's charts for the zero boundary neutral force. The dashed curve represents values from Taylor's curves for sudden drawdown, while the intermediate curves give values for steady seepage with the water table level at the elevations shown.

The curves should prove useful in estimating the values of lowering the water table upon the stability of the slope by the use of a drainage plan. They may also be used to determine the maximum height of slope of a cut to be made where soil strengths and the water table levels are known or may be estimated.

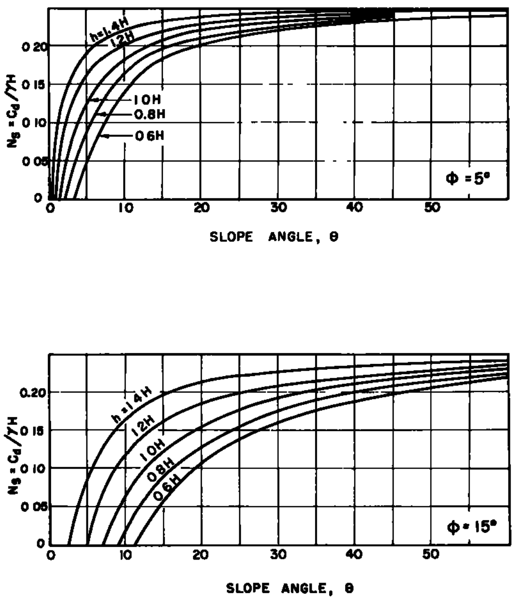
The curves presented assume that in every case the failure will occur as a toe circle, that is, the failure arc cuts the toe of the slope.

In a paper by Nilmar Janbu a straight line interpolation of the depth of the water table was used in working out an example (1). The same example was used here with the results shown in Figure 19.

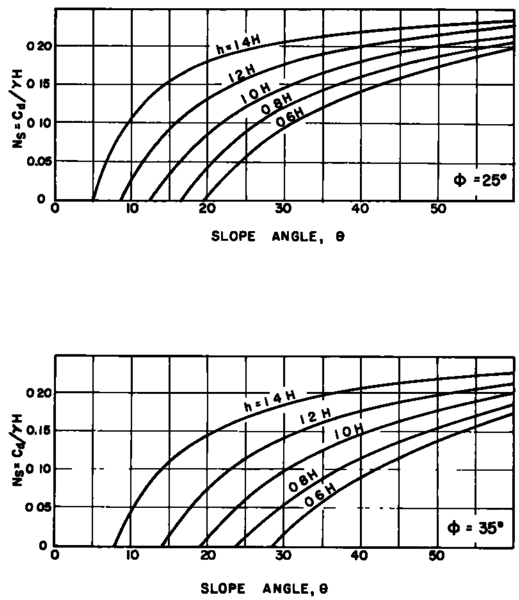


CASE I, $H_1 = 0$

Figure 22.



CLAY BANK OVER SAND STRATUM
CASE II, $H_1 = 0.2H$
Figure 23.



CLAY BANK OVER SAND STRATUM
CASE II, $H_1 = 0.2H$
Figure 24.

Reduction of Water Pressure in an Underlying Aquifer

During rainy seasons an excess hydrostatic head may be built up in the underlying strata, threatening the stability of the slope. This force is resisted by the weight of the material above. As the hydrostatic pressure increases the shearing resistance of the pervious layer is reduced and when the active earth pressure at the crest becomes greater than the passive pressure at the toe of the slope plus the shearing resistance, the slope will fail. This failure takes place as the phenomenon of "sudden spreading." The clay slope fails and spreads very rapidly. The graphs presented here will enable one to relate the piezometric head in the pervious strata, the internal friction of the pervious strata, and the cohesion of the clay bank to a critical bank height. Thus, if the piezometric head in the sand strata is recorded periodically, a warning of impending danger can be given and action taken to forestall failure of the slope, or the effect of a drainage plan upon a slope that is sliding or in danger of sliding may be estimated.

Figure 20 shows the boundary conditions assumed for the solution of the graphs. The method used follows the suggestions given by Terzaghi and Peck for the analyses of sudden spreading of clay slopes (3). The phreatic lines were assumed to be straight and emerge at the toe of the slope where the sand strata was located at the toe of the slope. Where the pervious layer was deeper the phreatic line was assumed to be parallel to the ground surface. The sand strata was assumed to have no cohesion while the clay bank was assumed to have no internal friction. The curves showing values of the stability number vs slope angle for the different friction angles and water table heights are presented in Figures 21 to 24.

CONCLUSIONS

The preceding analyses confirm a general impression that engineering efforts to improve slope stability, whether by excavation or by drainage, are more apt to be successful with problems of instability in stiff soils (and steep slopes) than in soft soils (and flat slopes). The graphs presented in this paper should aid the engineer to decide in any ordinary instance whether excavation or drainage offers the more economical solution of this problem. If excavation, then it has been made clear in the preceding pages that benching requires less excavation than slope reduction in achieving a required factor of safety.

The value of lowering the water table increases when the slope has greater strength or internal friction. Therefore, in weaker slopes where trouble is likely to occur, lowering the water table will increase the safety of the slope by only a small amount. However, this small increase may assure the stability of the slope. It is difficult to make any general statement regarding the increased stability for all slopes upon lowering the water table as each slide will have different variables and conditions to be faced. However, in all cases the stability is increased, and the value of that increase may be estimated with the aid of the curves presented.

REFERENCES

1. Janbu, Nilmar, "Stability Analysis of Slopes with Dimensionless Parameters." Harvard Soil Mechanics Series No. 46, Cambridge, Mass. (1954).
2. Taylor, D. W., "Fundamentals of Soil Mechanics." Chapter 16, John Wiley and Sons (1948).
3. Terzaghi and Peck, "Soil Mechanics in Engineering Practice." John Wiley and Sons (1948).

Appendix A

<u>Derivation of Figure 6 and 8, $\phi = 0$</u>

In general the curves for benched slopes are obtained by the solution of two problems. For a particular bench, as in Figure 4, the stability of the mass below the bench must be considered as well as the stability of the entire slope. This is shown by the two dashed critical circles. The curves in Figures 6 and 8 were found, then, by obtaining the factor of safety, F_s, for the small mass below the bench for a particular bench depth, $n_h H$. Then F_s for the entire slope was determined for this value of bench depth and several bench widths, $n_e H$. When F_s for the entire slope was found equal to F_s for the small earth mass, this determined the widest bench that could economically be cut for that particular bench depth and constitutes a point on the curve of N_s versus n_A.

<u>Example Problem A-1</u>. This problem illustrates the above discussion by showing the procedure used in obtaining a point on the bench curve for $n_d = 2$, Figure 6.

Given: $b_o = 1$ $\quad\phi = 1$

$n_d = 2$ $\quad c = 800 \text{ lb/ft}^2$

$\quad\gamma = 120 \text{ lb/ft}^3$

$n_h = 0.1$

$H = 30 \text{ ft}$

The F_s for the small earth mass is obtained by determining N_s from Janbu's curve in Figure 2-1 (<u>1</u>). This figure is similar to the curves presented by Terzaghi and Peck.

$$F_s = N_s \frac{c}{\gamma J} = 5.64 \frac{800}{(120)(27)} = 1.39$$

The x and y coordinates for the critical circle of the small mass below the bench is obtained from other curves of the same plot. The intersection of the critical circle and the bench line determines the width of the first trial bench in this example for the first trial

$$n_e = 2.05$$

For this bench width F_s for the entire slope was obtained by following the procedure outlined by Janbu (<u>1</u>). Benching is taken care of by considering the earth removed as constituting an upward force on the slope.

$$F_s = 6.2 \frac{800}{(120)(30)} = 1.38, \text{ for } n_e = 2.05$$

Since F_s for the entire slope is slightly smaller than F_s for the small mass, a wider bench may be cut necessitating another trial n_e. If F_s for the entire slope had been larger for this first trial, the F_s for the smaller mass would be used for a point on the final curves. This condition occurs for small values of n_h and determines the first part of the final curves which is concave upward.

For a second trial let $n_e = 2.4$. Using the same procedure as above

$$F_s = 6.4 \frac{800}{(120)(30)} = 1.42, \text{ for } n_e = 2.4$$

Now F_s for the entire slope is larger than F_s for the small mass below the bench. To find the bench width where both are the same, all trial values are plotted as in Figure 10. The volume of the material removed per lineal foot of slope is

$$V = n_e n_h H^2$$

$$\text{Let } n_A = n_e n_h = (2.05)(0.1) = 0.205$$

$$V = n_A H^2 = 0.205H^2$$

Point A then represents the widest bench which should be excavated for a bench depth of $n_h = 0.1$.

The entire bench curve for $n_d = 2.0$ is determined by the method outlined above with n_h being taken from 0.1 to 0.5 at 0.1 intervals. For the final curve F_s was converted to N_s.

Example Problem A-2. The dashed lines on Figures 6 and 8 represent an increase in N_s for slope reduction and were obtained by the following method. Referring to Figure 8, a slope less than b_o was considered, such as $b = 1.5$. The N_s value (5.75) for this slope was obtained from Figure 2-1. The volume of excavation per lineal foot of slope was found by

$$V = \frac{(b - b_o)}{2} H^2$$

Here a quantity is multiplied by H^2, as in the case for a benched slope. Therefore, to plot this curve along with the bench curve

$$n_A = \frac{b - b_o}{2} = \frac{1.5 - 1}{2} = 0.25$$

The volume excavated would then be

$$V = n_A H^2 = 0.25H^2$$

This allows comparison of volumes of excavation for benching and slope reduction for the same value of N_s. The entire slope reduction curve was obtained by increasing b by intervals of 0.25.

Appendix B

Derivation of Figures 7 and 9, $\phi > 0$

These curves were derived similarly to those on Figures 6 and 8 in that F_s for the entire slope was compared to F_s for the small earth mass below the bench for a certain value of bench depth.

Example Problem B-1. Consider the determination of a point on the bench curve of Figure 7 for $\lambda_{c\phi} = 4.0$.

Given: $b_o = 1$ $\quad\phi = 20$ deg
$n_h = 0.4$ $\quad c = 419$ lb/ft²
$H = 40$ ft $\quad\gamma = 115$ lb/ft³

F_s for the small earth mass is obtained from Janbu's curve in Figure 6 (1).

The x and y coordinates for the critical circle of the small earth mass are also obtained from that figure. The intersection of this circle and the bench line determined the initial bench width of $n_e = 0.3$.

F_s for the entire slope for a particular value of n_e must be accomplished by first finding the approximate location of the critical circle for failure of the entire slope. Then F_s is determined by the us of the ϕ-circle method. Locating the critical circle will be discussed first.

It was felt that the critical circle for a benched slope of a given slope angle would fall near the critical circle of a slope which had been reduced somewhat below the given slope angle, or a reduced slope would exist that had essentially the same driving moments due to the soil mass as the benched slope. At least if this condition was met the critical circle should give a reasonable value for F_s since minor changes of the critical circle do not change F_s appreciably.

Consequently F_s was determined for the critical circles of several reduced slopes until a minimum F_s for the benched slope was found. Usually the slope was reduced 2 deg between each trial.

In the current example the three trial critical circles used for $n_e = 0.3$ were for reduced slopes of 38 deg, 36 deg, and 34 deg. It was found that F_s for a reduced slope of 36 deg was the smallest and is bracketed by two higher values. Therefore this value of F_s was selected for $n_e = 0.3$.

Figure 10 shows the graphical procedure used in finding F_s by the ϕ-circle method. The resultant weight vector, W, was located by taking moments about the toe of the slope. From the intersection of W and the line of action of C, the total cohesive force acting along the critical circle, lines were drawn tangent to three different ϕ-circles. C is then evaluated for the three cases.

The factors of safety with respect to c and ϕ for the three values of ϕ, were determined by

$$F_c = \frac{c \text{ (effective)}}{c \text{ (required)}}$$

$$F_\phi = \frac{\tan\phi \text{ (effective)}}{\tan\phi \text{ (required)}}$$

F_s was then found from the small graph, Figure 11, such that

$$F_s = F_c = F_\phi$$

$$F_s = 1.36, \text{ for } n_e = 0.3$$

Since this value is smaller than F_s for the small earth mass, the bench may be widened. The second width trial value is $n_e = 0.4$. This time the same critical centers were used and the minimum F_s is

$$F_s = 1.45, \text{ for } n_e = 0.4$$

Still smaller, a third trial bench width of n = 0.5 was computed in a similar manner.

$$F_s = 1.53, \text{ for } n_e = 0.5$$

Again, as in Appendix A, all trial values are plotted (see Fig. 10) and a point on the final curve is obtained. Points were figured for values of n_h from 0.1 to 0.5 at 0.1 intervals.

The curves for reduced slope on Figures 7 and 9 were found by the same method outlined in Appendix A except that N_s values came from Janbu's Figure 6.

Computer Solution of Swedish Slip Circle Analysis for Embankment Foundation Stability

ROGER V. LeCLERC and ROBERT J. HANSEN, Supervising Highway Engineers,
Washington State Highway Commission

Personnel of the Materials Laboratory and of the Computer Section of the Washington Department of Highways have developed a program for the IBM 650 magnetic drum data processing machine which will analyze a given foundation problem in a matter of minutes. At present the program will handle analyses of embankment stability where the foundation is composed of as many as 3 layers or strata of different material. Although the present program is restricted to homogeneous embankments, suitable modifications should enable it to handle any number of embankment or foundation materials if they are placed or occur in a known geometric pattern in the cross-section.

The following data are necessary for the machine analysis: cohesion, angle of internal friction, and unit weight of the soils involved; initial slope to be analyzed; thickness of the foundation soil strata; height of embankment; and design safety factor.

The program may be used in two ways: (1) To investigate a given range of slopes, automatically advancing to the next flatter slope if the safety factor against failure is found to be less than the predetermined value; and (2) To investigate a range of slopes, in individual analyses for each slope.

● THE SWEDISH SLIP CIRCLE method of analyzing slope and embankment foundation stability is based upon studies which have indicated that failures in slopes usually occur along a cylindrical surface within the soil rather than along a plane surface, and that rotation takes place about the center of the cylinder. Early work on this was done by Petterson (1) and Fellenius (2) and later, D. R. May (3) who proposed the use of a planimeter in connection with a graphical solution of the problem. Tables and charts by Fellenius, Taylor (4), and others were also developed to aid in solution of the problem, using the ϕ circle modification.

In the past, the most common method of analysis made use of the graphical solution of the problem. This approach, though simplified to a great extent, is a time-consuming trial and error procedure which often lacks thorough analysis. The complex mathematical capabilities of the electronic computers appeared to offer a method for further simplifying the analysis.

At the suggestion of the Materials Laboratory, the Computer Section of the Washington Department of Highways initiated the development of a computer program for the complete analysis of embankment foundation stability. It was intended that this program would analyze all potential failure circles, or arcs, so that confidence in the thoroughness of investigation could be realized. The potential saving in engineering time was obvious from the start, and this point alone would justify the development of the program.

Although time has not permitted as rigorous a check as might be desired, the completed program is thought to be thorough in its coverage. Computation of a single safety factor requires only 3 to 5 seconds. Complete analyses, involving many trials, may take anywhere from 1 to 5 minutes. However, as might be expected, any computer program which attempts to be all-inclusive, such as this one, has certain limitations, most of which appear to be potentially insignificant at this time. The limitations are covered in detail in a subsequent section.

METHOD OF ANALYSIS

It is not the intent of this paper to describe the theory of the Swedish Slip Circle analysis, but to discuss its implementation through use of a medium size electronic computer—in this particular case, the IBM 650.

The purpose of the program is to solve for the minimum safety factor for a designated slope, the safety factor being the ratio of the withholding force divided by the tendency to "slip." For equilibrium, or a safety factor of 1.0, the sum of the moments about point "0", the center of the revolving cylinder, must be equal to zero.

The determination of the minimum safety factor necessitates the investigation of a series of circles until the circle is found with the conditions that yield the minimum safety factor. This phase is ideally adapted to an electronic computer because of the decisions that can be built in to a computer program. Programming techniques have also made it possible to generate selected dimensions, by increments, which locate and describe the circles investigated.

There are two general ways in which the program may be used.

1. By using certain code numbers, in this case a code 8, the computer will determine the minimum safety factor beginning with the steepest slope designated for analysis. Computation of a safety factor less than 1.0 will cause the program to branch and investigate flatter slopes in succession until a minimum safety factor greater than 1.0 is found and punched out. Slopes of 1, 8:1, 2:1, 3:1, 4:1, 6:1, and 10:1 can be investigated in the present program form. Others may be included, or substituted, if desired.
2. If the code number 9 is used, the minimum safety factor, regardless of value, is punched out for each designated slope. Any slope flatter than 1:1 may be investigated in this case.

A typical cross-section of an embankment resting on questionable foundation soils is shown in Figure 1. The terminology of various dimensions used in the development of equations for the computer solutions is shown thereon. The coordinates of the center of the circle are represented by the distances A and B which are, respectively, the lateral distance from the crest of the slope and the vertical distance from the top of the embankment. All equations are in terms of A, B, R (the radius), the slope, and the values which describe the thickness and properties of embankment and foundation soil layers. The complete mathematical equations involved are shown in Appendix A, together with explanatory notes and sketches showing their derivation.

The program has been set up to handle 4 soil layers, one of which is the embankment. Input data and their significant figures consist of the following types of information:

W = Density, XXX lb/ft^3
C = Cohesion, XXX lb/ft^2
tan ϕ = Coef. of friction, .XXX
H = Height of fill or soil layers, XX.X ft

The following data are required for the problem:

1. Problem number;
2. Identification number to indicate method of analysis desired (automatic slope change to a safety factor greater than 1, or safety factor for beginning slope), Code 8 or 9;
3. Beginning slope;
4. Fill height (H_1);
5. Unit weight, cohesion and tangent of angle of internal friction of fill material (W_1, C_1 tan ϕ_1);

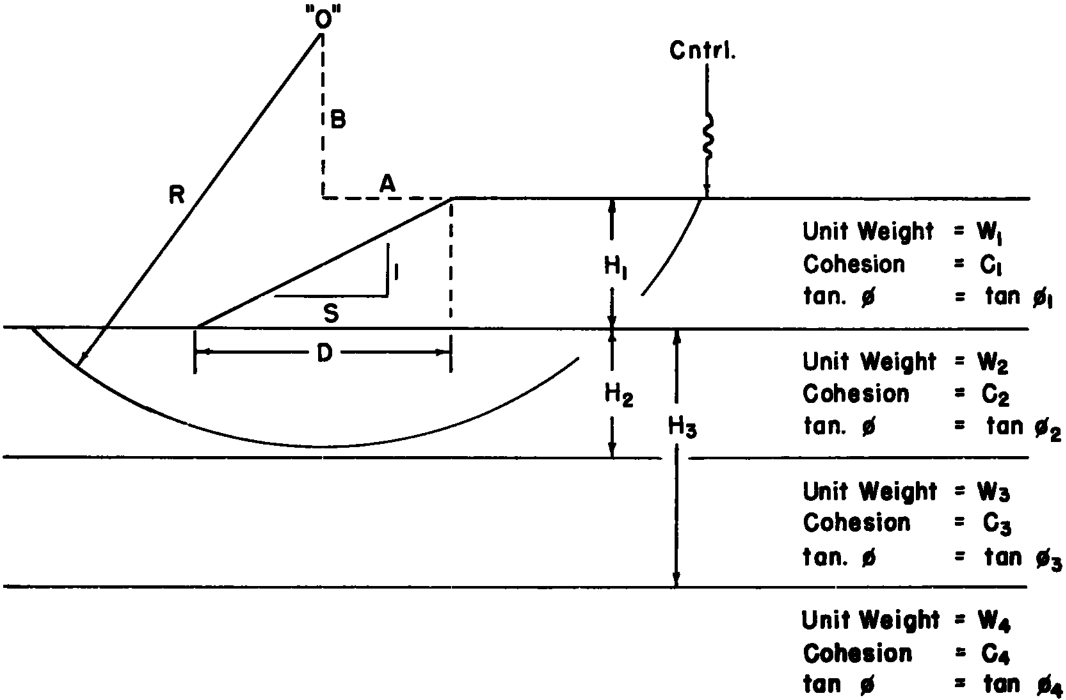


Figure 1. Terminology used in computer solution of Swedish Slip Circle.

6. Depth, unit weight, cohesion and tangent of angle of internal friction of first foundation soil layer (H_2, W_2, C_2 $\tan\phi_2$);
7. Depth from ground surface to bottom of second foundation soil layer (H_3); unit weight, cohesion and tangent of angle of internal friction of second foundation soil layer (W_3, C_3, $\tan\phi_3$);
8. Unit weight, cohesion, and tangent of angle of internal friction of third foundation soil layer (W_4, C_4, $\tan\phi_4$); and
9. Width of embankment to which analysis should be confined (CNTRL)

A sample input data form is shown in Appendix B, in conjunction with the sample problem. It will be noted that as many as 3 foundation soil layers may be considered. If the particular problem should not encompass that many foundation strata, only the data for the layers involved need be entered, subject to the condition that the last layer described automatically will be considered infinite in depth. For example, if the foundation profile for any problem should consist of 30 ft of organic clay underlain by relatively firm sand, which persists in depth beyond a point of concern, data for both the clay and the sand should be entered, even if those for the latter have to be assumed.

The flow chart for the IBM 650 program is shown in Figures 2 through 5. The first step of the calculation involves the computation of the initial values of A, B, and R which define the first trial circle. The following empirical equations are used to obtain these values:

$$A = \tfrac{1}{2}(H_1 + D) \qquad (1)$$

$$B = \frac{H_1 s^2 + 2D - 3H_1}{8} \qquad (2)$$

$$R = \sqrt{(H_1 + B)^2 + (D - A)^2} + 1 \qquad (3)$$

in which

s = slope
H_1 = ht of embankment

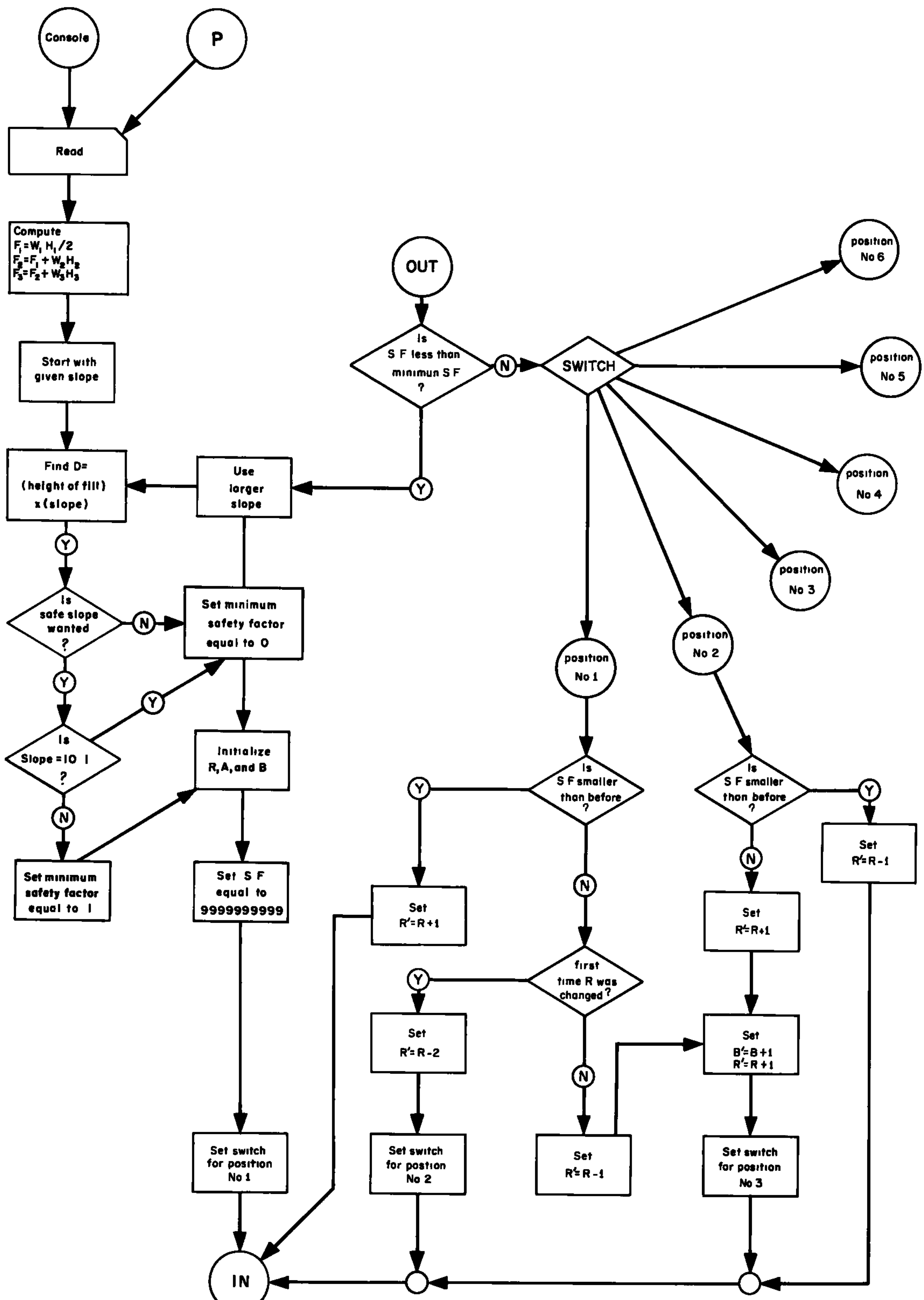


Figure 2. Flow chart (1), Swedish Slip Circle analysis on IBM 650.

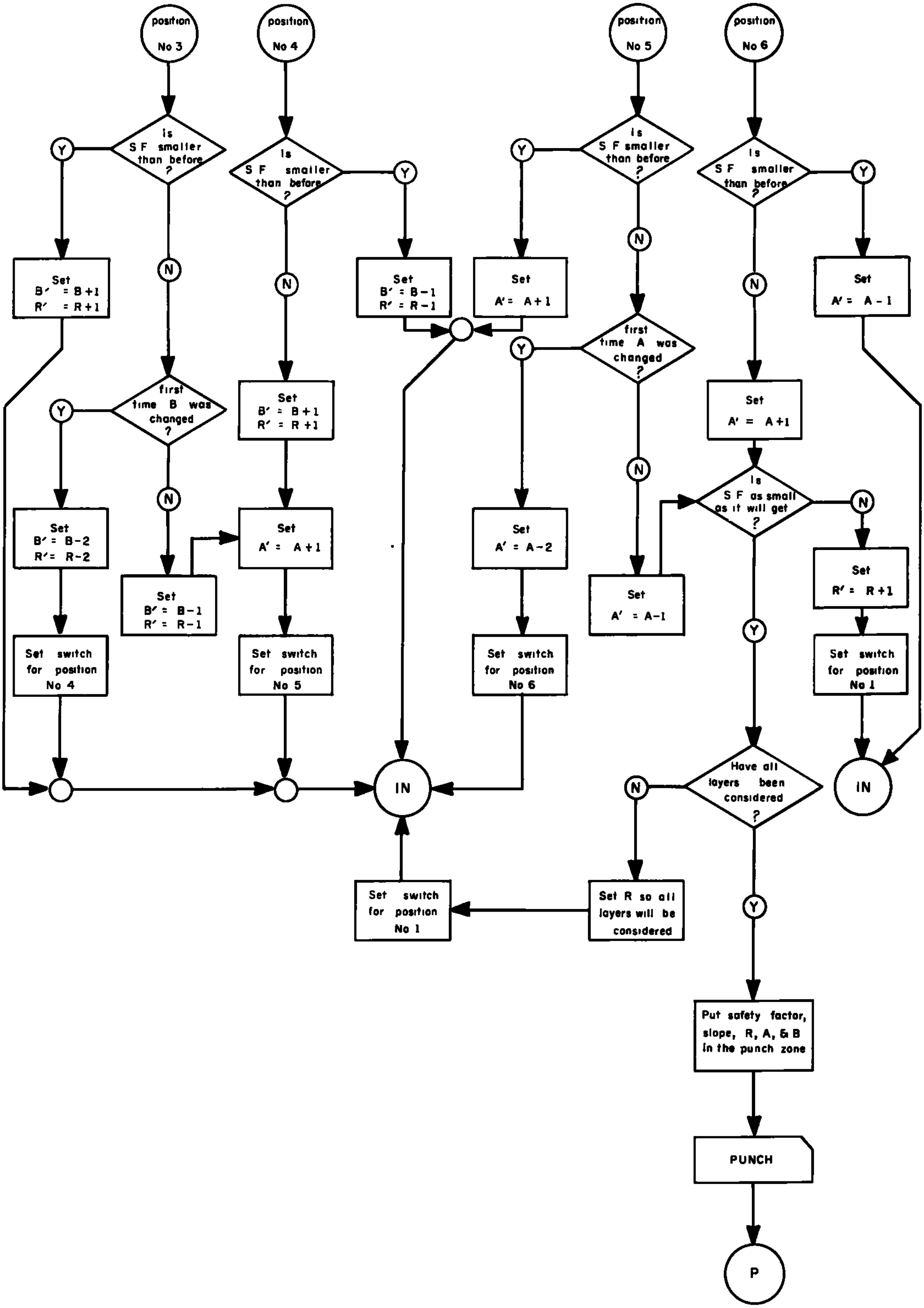


Figure 3. Flow chart (2), Swedish Slip Circle analysis on IBM 650.

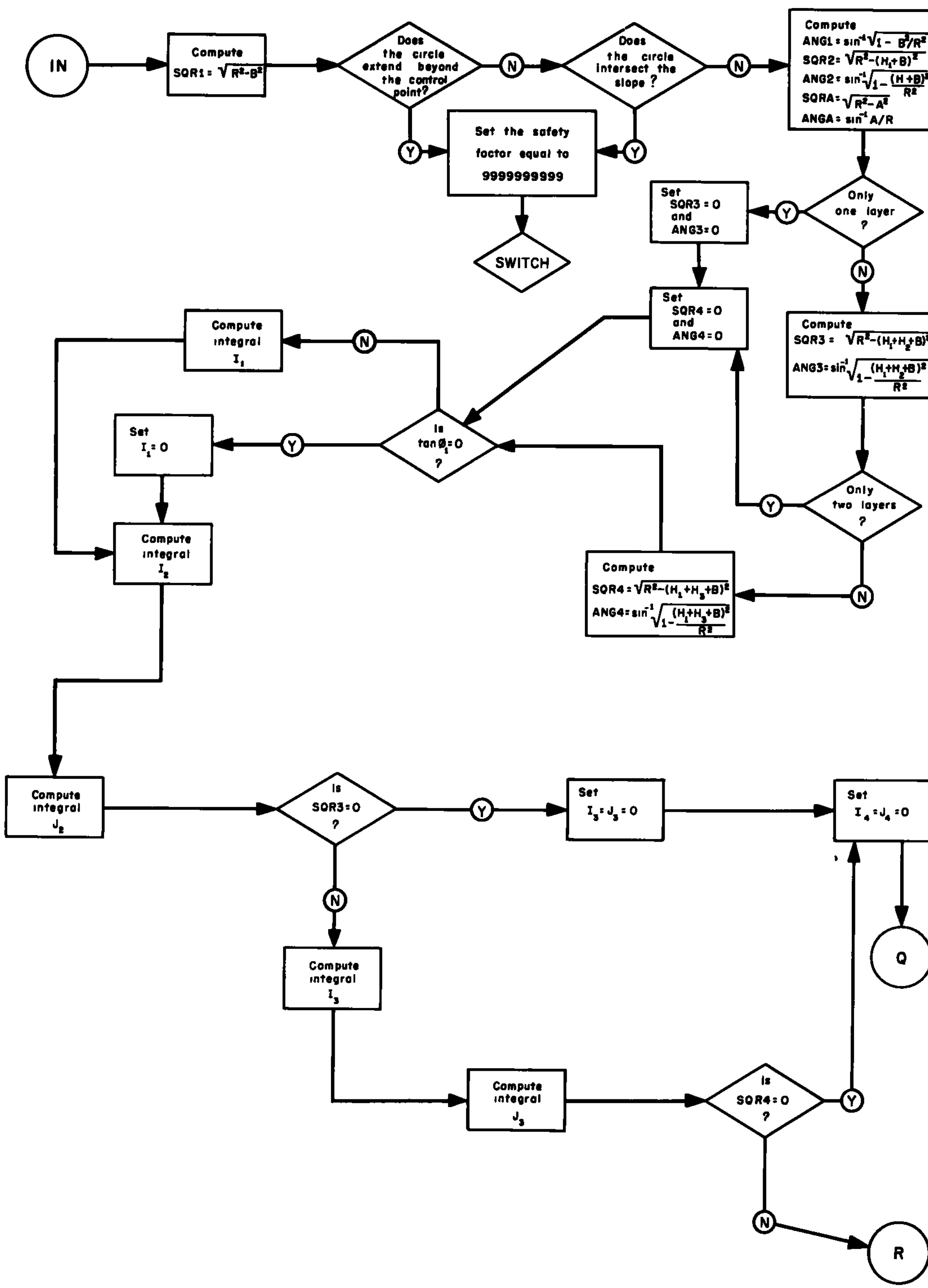


Figure 4. Flow chart (3), Swedish Slip Circle analysis on IBM 650.

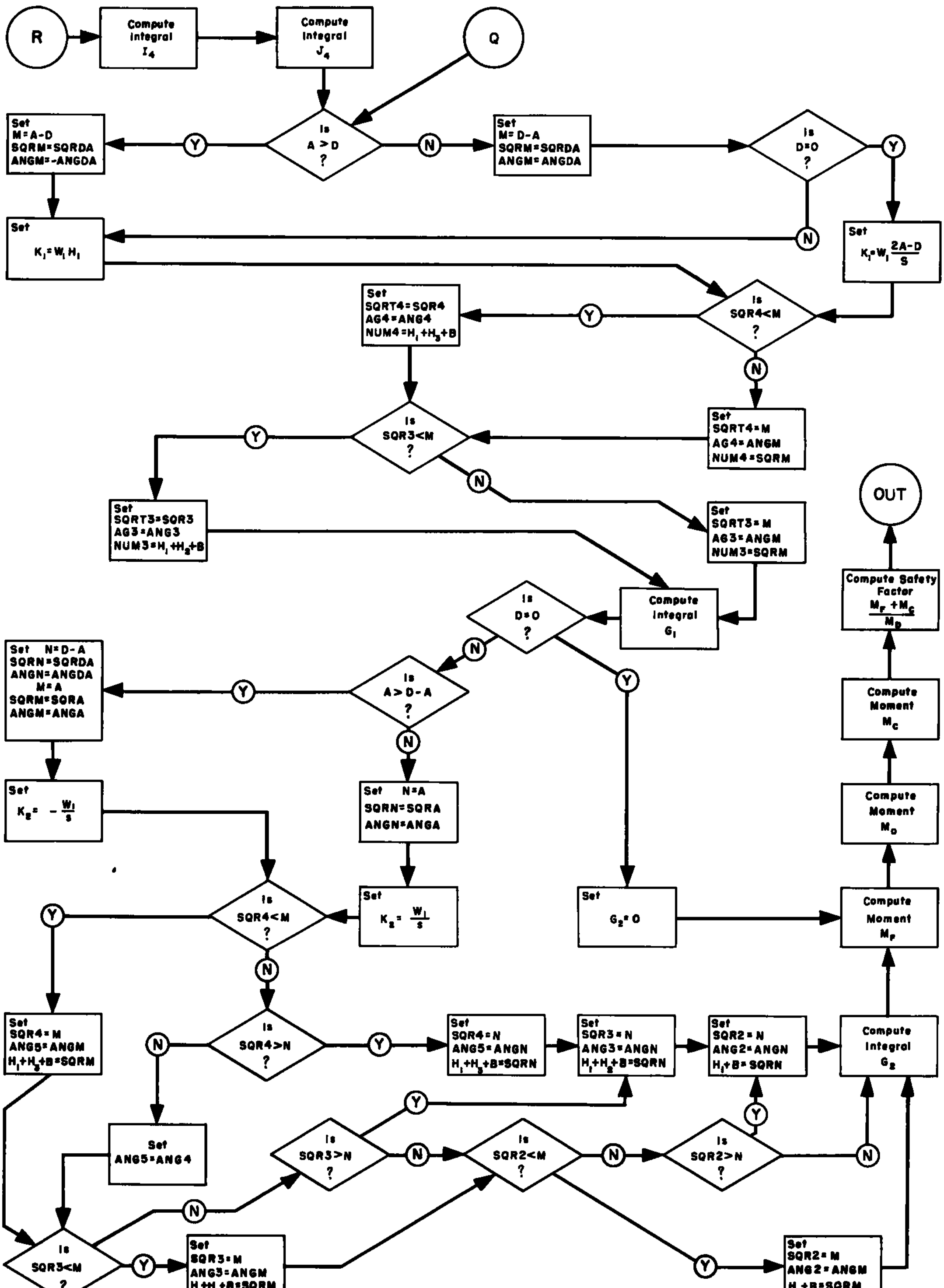


Figure 5. Flow chart (4), Swedish Slip Circle analysis on IBM 650.

These empirical equations give results such that the circle defined thereby will not intersect the slope, will not pass beyond the control point (cntrl), and will penetrate a minimum depth into the first foundation soil layer.

Using the initial values of R, A, and B together with the pertinent input data, a safety factor for the first circle is calculated. The value of R is then increased and another safety factor computed. The second safety factor is compared to the first, and if less, R is increased again until a minimum safety factor is reached. If the second safety factor is greater than the first, the value of R is decreased until a minimum safety factor is reached. After R is varied until a minimum safety factor is reached, A is varied, then B and R are varied together in the same manner. With the resultant values of R, A, and B, the entire procedure is repeated until the safety factor found equals the previous safety factor, thus establishing a minimum. This minimum safety factor, together with the related values of R, A, and B, is stored in a special comparison area.

The program then checks to see if all foundation layers have been investigated. Since the starting conditions are such that the upper foundation layer is investigated initially, this check operation consists mainly of resetting the initial values of R, A, B so that the circle passes into the second and/or third layer. The entire procedure is then repeated with these initial conditions, and the minimum safety factor resulting from all the above calculations is punched out of the special comparison storage area along with the radius and center coordinates of the circle providing this safety factor.

The above sequence of operations is followed in the calculation of the minimum safety factor, regardless of value, for any one designated slope (code 9). In the automatic operation (code 8) the same procedure is followed except that calculation of a safety factor less than 1.0 will cause the program to branch and investigate the next flatter slope. This will continue until a minimum safety factor greater than 1.0 is found or until a slope of 10:1 is investigated, whichever occurs first.

Output data consist of the problem number, the code number specifying the type of analysis, the initial slope investigated, the slope for which the minimum safety factor is calculated, the value of this safety factor, and the values of R, A, and B which locate and describe the circle that provides this safety factor. The form used in tabulating these data is shown in connection with the sample problem, Appendix B.

LIMITATIONS

It has been necessary to include certain limitations to the scope of the analysis. First, the stratification of foundation soil layers must be essentially horizontal, or capable of being represented so, for correct operation of the program.

A second limitation is that the circle should not intersect the slope. This does not appear to be a serious limitation inasmuch as the weakest circle, or the circle with the lowest safety factor rarely passes through the slope on a simple embankment cross-section.

Another control point is the width of embankment to which the analysis should be confined. Many limit the investigation to half the top width of the embankment. Since this distance is a variable, provision is made for this point to be included in the input data.

At present the program is compatible with homogeneous fill material only. It is not, as now constituted, capable of analyzing a heterogeneous embankment such as an earth fill dam, nor will it handle the analysis of an embankment incorporating berms in the cross-section. A modification to accomodate this latter feature is currently underway.

Some difficulty is being experienced in efforts to apply the program to analyses of embankments placed over a relatively thin layer of soft foundation soil, particularly where this layer is underlain by a fairly firm material. Although the extent of this limitation has not been pinpointed, it does appear that the program will be satisfactory for all cases where the ratio of the effective embankment width to the thickness of the soft foundation layer is less than 4.0. When this ratio exceeds 4.0, it is probable that the failure surface will be other than cylindrical. Such conditions should be analyzed by other methods. Answers given by this program analysis are subject to possible error in these cases, and should be checked in light of the above statements.

Another limitation to the present program is that the water table cannot be above ground surface. Modifications to handle this condition are also underway.

APPLICABILITY

In its present form the program is quite versatile. With very minor modifications, it can be used to calculate a safety factor from a given set of coordinates defining any desired circle—and can do so in less than 5 seconds. With the machine on automatic operation, slight changes in the program can be made so that each safety factor calculated, together with data on the slope and coordinates defining the circle, will be punched out. This gives a check on the thoroughness of scanning.

This program should be a valuable aid to those who desire to construct tables or charts for use in design or control of embankments. It will eliminate the long and tedious calculations usually associated with this procedure.

The program provides a means of evaluating situations involving 3 different foundation soil layers, which are often encountered in the work. Water table elevation is considered as a foundation soil boundary since the effective weight of material below this point is equal to the submerged weight. This, then, represents a change in soil properties necessitating the consideration of this portion of the foundation soil as a different material in the analysis.

CONCLUSION

It is felt that the program for use of the IBM 650 computer in analyzing embankment foundation stability by the Swedish Slip Circle method will be of definite value to those concerned with the design of embankment foundations. This program is available for use and is offered to those who might desire it. It is hoped that it will serve as a stepping stone to even more versatile programs for this analysis—programs brought about by the modifications others will discover and incorporate to meet their particular needs.

ACKNOWLEDGMENT

The major part of the program development was accomplished by Jon Petersen, presently an undergraduate at Stanford University, while employed with the Washington Department of Highways during the summer of 1957. The credit for the program is rightfully his. Recognition should also be given to personnel of the Computer Section, namely Paul Yeager, Charlene Travis, L. C. Reynolds and Eugene Lee for a great amount of assistance in the incorporation of minor modifications and to M. P. O'Neill and Harold Dunn of the Planning Division Drafting Section for the preparation of flow charts and drawings. Acknowledgment of the beneficial suggestions of Carl E. Minor and Lloyd Morgan of the Materials Laboratory relative to the development of the program and the prepartion of the paper is gratefully made. And last, but far from least, sincere appreciation of the able assistance and advice in checking the program and suggesting modifications is given to Henry Sandahl of the Materials Laboratory.

REFERENCES

1. Petterson, K. E., "Kajraseti Goteborg des 5te Mars 1916." Tekniske Tidskrift, Vol 46, p 289 (July 29, 1916).
2. Fellenius, W., "Erdstatische Berechnungen mit Reibung und Kohasion (Adhasion)." Wilhelm Ernst und Sohn, Berlin (1927); 4th edition (1948).
3. May, D. R., "Application of the Planimeter to the Swedish Method of Analyzing the Stability of Earth Slopes." Report on the Second Congress on Large Dams, Vol IV (1936).
4. Taylor, D. W., "Stability of Earth Slopes." Journal of the Boston Society of Civil Engineers (July 1937).

Appendix A

Equations

$$I_1 = 6 \tan\phi_1 W_1 \int_{\sqrt{R^2 - (H_1 + B)^2}}^{\sqrt{R^2 - B^2}} \left(R^2 - X^2 - B\sqrt{R^2 - X^2}\right) dx$$

$$I_2 = 12 \tan\phi_2 W_2 \int_{\sqrt{R^2 - (H_1 + H_2 + B)^2}}^{\sqrt{R^2 - (H_1 + B)^2}} \left(R^2 - X^2 - [H_1 + B]\sqrt{R^2 - X^2}\right) dx$$

$$I_3 = 12 \tan\phi_3 W_3 \int_{\sqrt{R^2 - (H_1 + H_3 + B)^2}}^{\sqrt{R^2 - (H_1 + H_2 + B)^2}} \left(R^2 - X^2 - [H_1 + H_2 + B]\sqrt{R^2 - X^2}\right) dx$$

$$I_4 = 12 \tan\phi_4 W_4 \int_{0}^{\sqrt{R^2 - (H_1 + H_3 + B)^2}} \left(R^2 - X^2 - [H_1 + H_3 + B]\sqrt{R^2 - X^2}\right) dx$$

$$J_2 = 12 \tan\phi_2 F_1 \int_{\sqrt{R^2 - (H_1 + H_2 + B)^2}}^{\sqrt{R^2 - (H_1 + B)^2}} \sqrt{R^2 - X^2}\, dx$$

$$J_3 = 12 \tan\phi_3 F_2 \int_{\sqrt{R^2 - (H_1 + H_3 + B)^2}}^{\sqrt{R^2 - (H_1 + H_2 + B)^2}} \sqrt{R^2 - X^2}\, dx$$

$$J_4 = 12 \tan\phi_4 F_3 \int_{0}^{\sqrt{R^2 - (H_1 + H_3 + B)^2}} \sqrt{R^2 - X^2}\, dx$$

$$G_1 = 6 K_1 \Bigg[\tan\phi_4 \int_{0}^{\sqrt{R^2 - (H_1 + H_3 + B)^2}} \sqrt{R^2 - X\, dx} + \tan\phi_3 \int_{\sqrt{R^2 - (H_1 + H_3 + B)^2}}^{\sqrt{R^2 - (H_1 + H_2 + B)^2}} \sqrt{R^2 - X^2}\, dx + \tan\phi_2 \int_{\sqrt{R^2 - (H_1 + H_2 + B)^2}}^{M} \sqrt{R^2 - X^2}\, dx \Bigg]^{a}$$

$$G_2 = 6 K_2 \Bigg[\tan\phi_4 \int_{M}^{\sqrt{R^2 - (H_1 + H_3 + B)^2}} (A - X)\sqrt{R^2 - X^2}\, dx + \tan\phi_3 \int_{\sqrt{R^2 - (H_1 + H_3 + B)^2}}^{\sqrt{R^2 - (H_1 + H_2 + B)^2}} (A - X)\sqrt{R^2 - X^2}\, dx$$

S = slope

$F_1 = \frac{W_1 H_1}{2}$

$F_2 = W_2 H_2 + F_1$

$F_3 = W_3(H_3 - H_2) + F_2$

$K_1 = W_1 H_1 \quad (A > D)$

$K_1 = W_1 \frac{2A - D}{S} \quad (A \leq D)$

$K_2 = \frac{W_1}{S} \quad (A \geq D - A)$

$K_2 = - \frac{W_1}{S} \quad (D - A > A)$

If $D - A > A$, $N = D - A$, $M = A$

If $A > D - A$, $N = A$, $M = D - A$

[a] The limits of integration are as given if these values are $\leq$M. If a limit is >M, then it is set equal to M.

$$+ \tan\phi_2 \int_{\sqrt{R^2 - (H_1 + H_2 + B)^2}}^{\sqrt{R^2 - (H_1 + B)^2}} (A - X)\sqrt{R^2 - X^2}\, dx \qquad + \tan\phi_1 \int_{\sqrt{R^2 - (H_1 + B)^2}}^{N} (A - X)\sqrt{R^2 - X^2}\, dx \Bigg]^{b}$$

$$M_D = W_1 H_1 \left[3R^2 + 3A(D - A) - 3B(H_1 + B) - D^2 - H_1^2 \right]$$

$$M_c = 6R^2 \Bigg[C_1 \operatorname{Sin}^{-1} \frac{\sqrt{R^2 - B^2}}{R} + (2C_2 - C_1) \operatorname{Sin}^{-1} \frac{\sqrt{R^2 - (H_1 + B)^2}}{R}$$

$$+ 2(C_3 - C_2) \operatorname{Sin}^{-1} \frac{\sqrt{R^2 - (H_1 + H_2 + B)^2}}{R} + 2(C_4 - C_3) \operatorname{Sin}^{-1} \frac{\sqrt{R^2 - (H_1 + H_2 + B)^2}}{R} \Bigg]$$

$$M_F = I_1 + I_2 + I_3 + I_4 + J_2 + J_3 + J_4 - G_1 - G_2$$

DISCUSSION OF EQUATIONS

In the equations modified for computer use, all the forces are multiplied by 6R to eliminate fractions and a number of divisions by R. This multiplication gives results which are six times the moments due to these forces. The intergral giving M_d, which is six times the driving moment, can be easily evaluated to give the result shown above.

The frictional moment, however, cannot be explicitly presented as easily, so all equations for the I, J and G integrals are left in integral form. The integrals are all of the form: $6R \cos\theta \, dW$, or $6\sqrt{R^2 - X^2}\, dW$. The program initially considers the problem to have the geometrics shown in Figure 6(a). The regions with which each of the

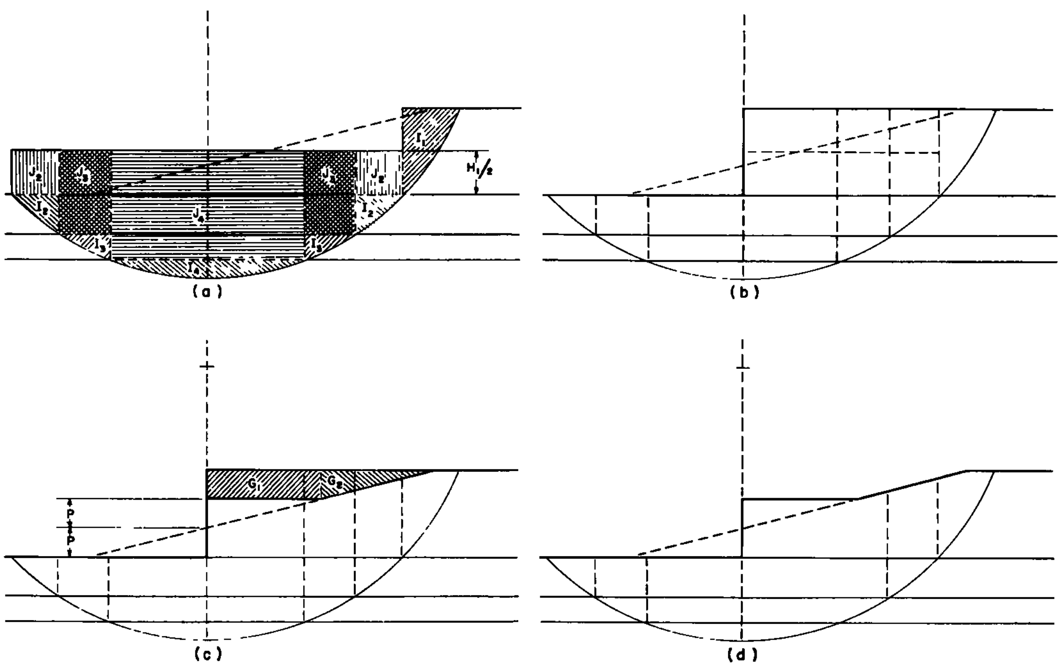


Figure 6.

---

[b] The limits of integration are as given if these values are ⋗M and ⋖N. If a limit is <M, then it is set equal to M. If a limit is >N, then it is set equal to N.

I and J integrals are associated, are labeled on this drawing. The shape of the embankment cross-section in Figure 6(a) is equivalent (as far as friction forces are concerned) to the shape in Figure 6(b). From this shape the G integrals (shown in Figure 6(c) are subtracted, leaving the configuration shown in Figure 6(d), which is frictionally equivalent to that shown in Figure 1—the usual embankment cross-section. Thus M_F, which is 6 times the frictional moment, equals the sum of the I and J integrals, minus the sum of the G integrals.

The cohesive force is calculated by multiplying the length of the arc by the cohesion of the material through which the arc is passing. The sum of these products for each materials considered is multiplied by 6R to obtain M_c. The formula for this is given explicitly in the section, "Equations."

The safety factor then equals:

$$(M_c + M_f)/M_d$$

Appendix B

Sample Problem

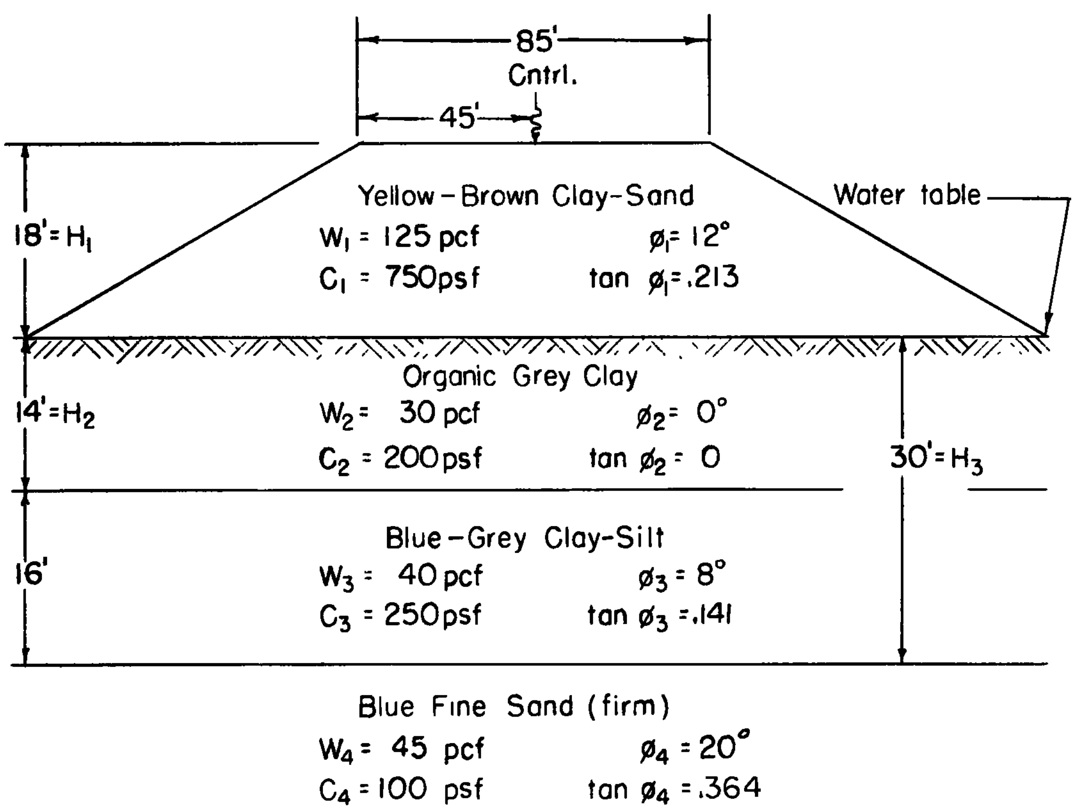


Figure 7. Sample problem.

INPUT DATA
SLIP CIRCLE PROGRAM

8/9: 8 - Punch values for safe slope only
9 - Punch values for starting slope only

IDENT.					8/9	STARTING SLOPE (To one)				FILL SOIL PROPERTIES									H_1			FIRST LAYER SOIL PROP.								
										W_1			C_1			Tan ϕ_1						W_2			C_2			Tan ϕ_2		
						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
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
0	0	1	1	1	9	0	1	8	0	1	2	5	7	5	0	2	1	3	1	8	0	0	3	0	2	0	0	0	0	0
0	0	1	1	1	8	0	1	8	0	1	2	5	7	5	0	2	1	3	1	8	0	0	3	0	2	0	0	0	0	0

Leave Blank if there is Only One Layer												Leave Blank if There are Only Two or Less Layers														
H_2			Second Layer Soil Prop.									H_3			Third Layer Soil Prop.									Cntrl.		
			W_3			C_3			Tan ϕ_3						W_4			C_4			Tan ϕ_4					
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.
32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58
1	4	0	0	4	0	2	5	0	1	4	1	3	0	0	0	4	5	1	0	0	3	6	4	0	4	5
1	4	0	0	4	0	2	5	0	1	4	1	3	0	0	0	4	5	1	0	0	3	6	4	0	4	5

OUTPUT DATA
SLIP CIRCLE PROGRAM

IDENT.					8/9	STARTING SLOPE					FINAL SLOPE					SAFETY FACTOR					R				A				B		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
0	0	1	1	1	9	0	1.	8	0		0	1.	8	0		0.	8	9	7		0	4	9.		0	1	7.		0	1	7.
0	0	1	1	1	8	0	1.	8	0		0	4.	0	0		1.	1	1	6		0	8	0.		0	3	6.		0	4	8.

Figure 8.

THE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

Receiving funds from both public and private sources, by contribution, grant, or contract, the ACADEMY and its RESEARCH COUNCIL thus work to stimulate research and its applications, to survey the broad possibilities of science, to promote effective utilization of the scientific and technical resources of the country, to serve the government, and to further the general interests of science.

The HIGHWAY RESEARCH BOARD was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the NATIONAL RESEARCH COUNCIL. The BOARD is a cooperative organization of the highway technologists of America operating under the auspices of the ACADEMY–COUNCIL and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the BOARD are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.