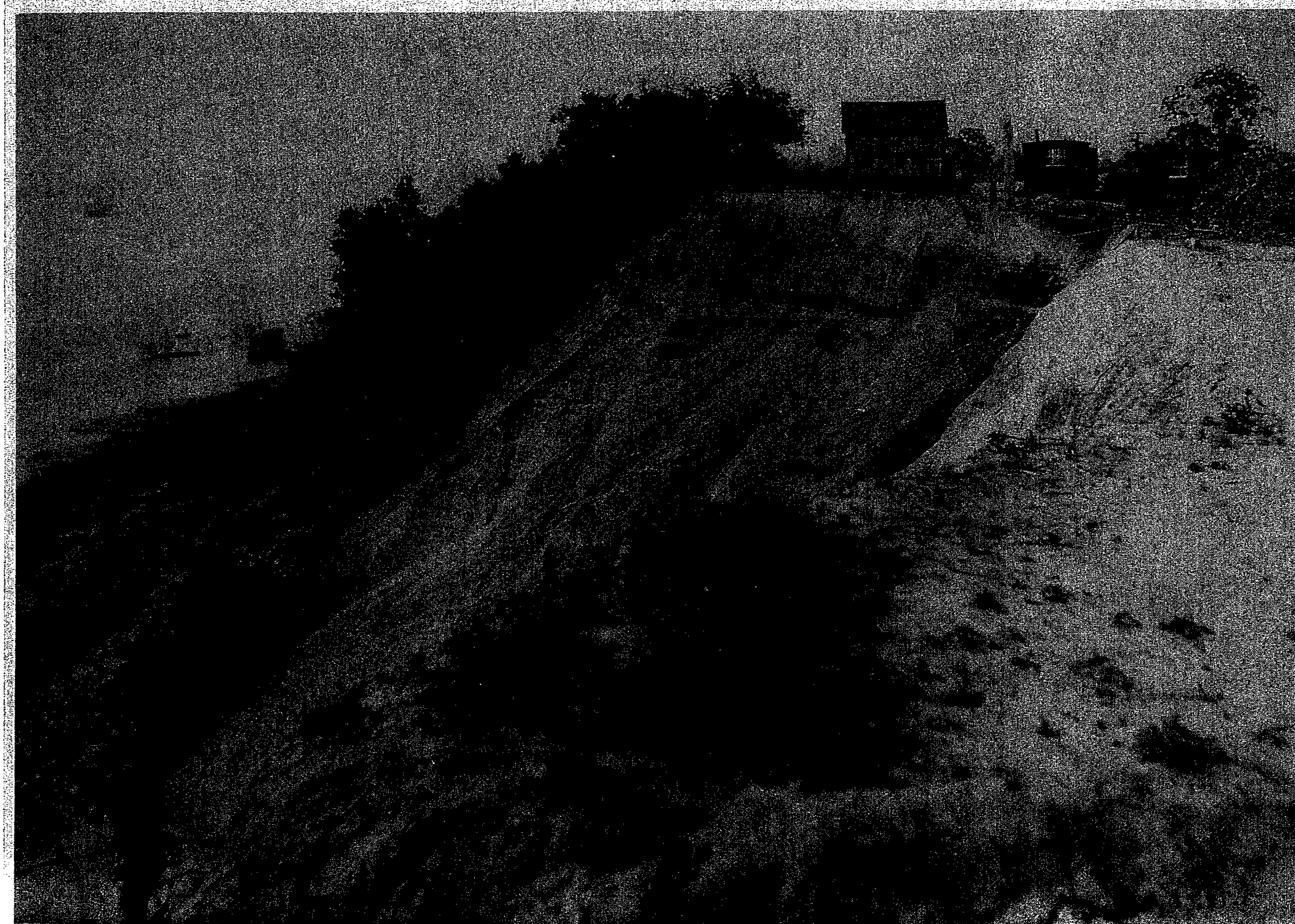
# Ohio River Valley Soils Seminar XVII

# Natural Slope Stability and Instrumentation



October 17, 1986
Clarksville, Indiana

# ORVSS-17
## Natural Slope Stability and Instrumentation
### October 17, 1986

8:00 Registration

<u>Morning Session</u> - Presiding: C. Robert Ullrich, University of Louisville

- 8:45 Welcome
- 9:00 "Instrumentation of Cut and Natural Slopes in Soil and Rock," by John Dunnicliff
- 10:00 "Landslide Recognition and Constructive Prevention in the Appalachian Area," by George A. Hall
- 10:30 Coffee Break
- 11:00 "Red Mountain Landslide Susceptibility Study," by Luther H. Boudra and Gerald T. Vandevelde
- 11:30 "Landslides in the Colluvial Soils of Southwestern Davidson County and Northern Williamson County, Tennessee," by L. C. Weber and L. E. Wilson
- 12:00 "A Case Study of Slope Stability in New Providence Shale," by Mark A. Sites and D. Joseph Hagerty
- 12:30 Luncheon

<u>Afternoon Session</u> - Presiding: D. Joseph Hagerty, University of Louisville

- 1:30 "Methods of Analyzing the Stability of Natural Slopes," by J. M. Duncan
- 2:30 "Repair of Smokey Landslide Using a Tied-Back Wall," by R. Michael Holbrook, Barry K. Thacker, James L. Kennedy, and John Sefton
- 3:00 Coffee Break
- 3:30 "Design of Highway Embankments on Unstable Natural Slopes," by Charles S. Bishop, Donald W. Armour, and Tommy C. Hopkins
- 4:00 "Soil Retention and Slope Stabilization with Geotextile Fabric," by Gilbert M. Camp and James Veith
- 4:30 "Investigation of Forest City Landslide in South Dakota," by V. Bump and S. Bang
- 5:30 Social Hour
- 6:30 <u>Dinner and Evening Session</u>

Presiding: Louis F. Cohn, University of Louisville
Speaker: Edward Cohen, Ammann & Whitney Consulting Engineers
Topic: The Statue of Liberty: Why and How; Then and Now

# PROCEEDINGS OF THE SEVENTEENTH OHIO RIVER VALLEY SOILS SEMINAR

## NATURAL SLOPE STABILITY AND INSTRUMENTATION

October 17, 1986
Sheraton Lakeview Hotel
Clarksville, Indiana

Sponsored by

KENTUCKY GEOTECHNICAL GROUP, ASCE

UNIVERSITY OF LOUISVILLE
DEPARTMENT OF CIVIL ENGINEERING
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# INSTRUMENTATION OF CUT AND NATURAL SLOPES IN SOIL AND ROCK

by

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Abstract. The goal of this paper is to describe the role of geotechnical instrumentation in addressing geotechnical questions that may arise during the design, construction or performance of cut and natural slopes in soil and rock. The paper is divided into four sections.

The first section indicates the general role of geotechnical instrumentation. The second section suggests the principal geotechnical questions that may arise, and indicates the types of instruments that may be used to help provide answers to those questions. The third section provides an overview of what is typically **routine** monitoring and what is monitoring for **special applications.** The fourth section presents a tabular summary of selected case histories that illustrate effective use of geotechnical instrumentation on projects involving cut and natural slopes in soil and rock.

## Introduction

The goal and outline of this paper have been given in the above abstract. The paper is not intended as an exhaustive summary, a state-of-the-art paper, or a "cook-book." It is intended merely to open the minds of readers to the possible role of geotechnical instrumentation, and to guide them towards implementation.

## General Role of Geotechnical Instrumentation

Analysis of slope stability is the principal geotechnical design task for temporary or permanent slopes. Factors influencing stability include stratigraphy, groundwater levels, seepage gradients, strength of the soil or rock mass, geometry and driving moments.

Stability of slopes in soil is controlled by the ratio between the available shearing resistance along a potential failure surface and the shear stress on the surface. Circular or wedge-shaped surfaces are often used in analyses that attempt to model actual conditions. Available strength includes cohesion and frictional components; for long-term considerations, the contribution of cohesion is often reduced significantly.

Stability of slopes in rock is usually controlled by the presence of discontinuities in the rock mass and the presence of water under pressure in these discontinuities. Failures most frequently occur as a result of sliding or separation along discontinuities.

It is imperative that, prior to planning an instrumentation program for a slope in soil or rock, an engineer first develop one or more working hypotheses for a potential behavior mechanism. The hypotheses must be based on a comprehensive knowledge of the locations and properties of discontinuities.

Instrumentation can be used to define the groundwater regime prior to excavating a slope. Results of measurements during excavation can be used as a basis for modification of the designed slope angle. Measurements of ground movement and groundwater pressure can assist in documenting whether or not performance during and after excavation is in accordance with predicted behavior. Measurements can also be used to document whether short- and long-term surface and/or subsurface drainage measures are performing effectively. If evidence of instability appears during or after construction, instrumentation plays a role in defining the characteristics of the instability, hence permitting selection of an appropriate remedy.

Instrumentation programs for natural slopes are essentially the same as for excavated slopes.

### Principal Geotechnical Questions

Every instrument on a project should be selected and placed to assist with answering a specific question: if there is no question, there should be no instrumentation. Before addressing measurement methods themselves, a listing should be made of geotechnical questions that are likely to arise during the design, construction or operation phases.

The following principal geotechnical questions are presented in the normal order of occurrence for a typical excavated slope. The order does not reflect a rating of importance.

#### What are the Initial Site Conditions?

Initial site conditions are determined by use of conventional site investigation procedures, sometimes supplemented by in situ testing. Special attention should be given to defining possible failure mechanisms. If potential failure zones are identified, the need for reinforcement or other methods of slope stabilization can then be addressed. For slopes in rock, comprehensive structural geologic mapping will indicate critical discontinuities. Particular attention should be given to persistent, adversely oriented joint sets, to possible low strength zones, and to continuous features such as faults and shears at the top of the slope that could allow release of potentially unstable blocks and wedges.

Performance monitoring instrumentation sometimes plays a role in defining initial site conditions. For example, groundwater pressures can have a large impact on the stability of slopes. Observation wells and/or piezometers can be installed well before the start of excavation, to define the pre-construction groundwater regime, including any perched or artesian water. If they are installed sufficiently early, seasonal variations can be defined.

If there is evidence of instability prior to excavation of the slope, such as an old landslide, the available data should be analyzed to identify potential failure mechanisms. An instrumentation program can then be planned to test hypotheses and to determine whether adverse conditions are present. Methods of instrumentation for this purpose are described in the following sections.

#### Is the Slope Stable During Excavation?

A program to monitor stability during excavation is not usually required if the design is very conservative, if there is previous experience with design and construction of similar facilities under similar conditions, or if the consequences of poor performance will not be severe. However, under other circumstances a monitoring program will normally be required to demonstrate that the excavation is stable and that nearby structures are not affected adversely.

Deformation and groundwater pressure are the primary parameters that assist in the evaluation of stability during excavation. Deformation measurements are usually the primary interest but, because high groundwater pressures can cause deformation, groundwater pressure measurements are also often needed so that cause and effect relationships can be established.

__Surface Monitoring of Deformation.__ Deformation monitoring will often be limited to surface measurements. Vertical and horizontal deformations are normally monitored by surveying methods, with electronic distance measurement playing a significant role. Surface measurements should extend beyond the uppermost limit of any possible movement zone to an area which is known to be stable, so that possible surface strain in advance of cracking can be monitored. Any toe heave should also be monitored. Tension cracks at the crest of the slope may be the first sign of instability. If cracks appear at the crest of the slope or elsewhere, their widths and vertical offsets should be monitored.

Crack measurements give clues to the behavior of the entire slope, and often the direction of movement may be inferred from the pattern of cracking, particularly by the matching of the irregular edges of the cracks. Acoustic emission techniques can sometimes be used by experienced personnel over a wide area in shallow drillholes to determine deformation trends and locations.

For slopes in rock, monitoring the tilt of critical blocks can provide an assessment of stability, and tiltmeters can also be used to monitor stability of slopes in soil if the deformation has a rotational component. Tiltmeters with electrolytic level transducers provide the most precise data, and the high precision allows trends to be determined in a minimum time period. Multi-point liquid level gages have been installed on benches of a large open pit mine where there is concern for a wedge failure: the instruments are used to monitor vertical deformation and are intended to provide a forewarning of any instability.

Subsurface Monitoring of Deformation. Subsurface deformation measurements will be required if sliding occurs, and if the depth of sliding is not readily apparent from surface measurements and visual observations. Measurements of subsurface horizontal deformation are more important than measurements of subsurface vertical deformation.

For slopes in soil, inclinometers are the instruments of choice, although shear plane indicators can be used for crude measurements, and slope extensometers may be preferred if deformation is predicted to occur at well-defined zones. Critical movements of slopes in rock are often smaller than critical movements of slopes in soil, and therefore the required accuracy of deformation measurements is generally greater. Fixed borehole extensometers, installed from the face of the slope following excavation of a rock bench, may therefore be selected for monitoring subsurface horizontal deformation of slopes in rock in preference to inclinometers. Multiple deflectometers and in-place inclinometers can provide real-time monitoring of subsurface deformation, and these instruments can be connected to alarms if required.

Monitoring of Groundwater Pressure. Observation wells are used for monitoring groundwater pressure when there are no perched or artesian water tables, but if either may be present, piezometers should be used.

Open standpipe piezometers are normally selected for slopes in soil, but diaphragm piezometers are appropriate if more rapid response is required. If any sliding is occurring, pore water pressures at or near the sliding surface must be measured to enable an effective stress analysis to be performed.

For slopes in rock, the heterogeneous nature of most rock masses results in a need for comprehensive monitoring of joint water pressure along, above and below each possible failure plane. Patton[1] presents a strong case for use of the movable probe type of multi-point piezometer, because it (a) provides a large number of measuring points, (b) does not create nonconformance, (c) allows the transducer to be calibrated at any time, (d) provides redundancy in field data, and (e) eliminates the problems of creating multiple seals when more than one conventional piezometer is installed in a single borehole. Patton[1] expresses the view that most existing field piezometer installations for rock slope stability investigations are deficient in the number of piezometers (probably by a factor of 5 to 10) unless the geology and hydrology of the slope are very simple. However, simple geology and hydrology cannot be demonstrated without a significant number of drillholes to document the geologic and piezometric data. The Westbay Instruments Ltd. combined piezometer-inclinometer system[2] provides a comprehensive profile of both joint water pressure and horizontal deformation in a single borehole.

If stability has been increased by improving groundwater drainage before the start of excavation, effectiveness of the drainage will usually be monitored by measuring groundwater pressure.

Predicting Stability of Completed Slope. Measurements during excavation can be used to assess the stability of the completed slope. Back-computations are made to examine the validity of parameters used in the original design. On the basis of these back-computations, new design calculations are made to predict the performance of the final slope, and slope angles are modified if necessary. If this approach is taken, clearly the calculations must be made in a timely manner, and a microcomputer will normally be used.

Summary of Possible Instrumentation. In summary, possible instrumentation for examining slope stability during excavation can be selected from the list given in Table 1. Details are given in References 3, 4 and 5.

Table 1. Instruments Suitable for Examining Slope Stability During Excavation

| Measurement | Suitable Instruments |
|---|---|
| Surface deformation. | Surveying methods.<br>Mechanical and electrical crack gages.<br>Tiltmeters.<br>Multi-point liquid level gages. |
| Subsurface deformation. | Inclinometers.<br>Fixed borehole extensometers.<br>Slope extensometers.<br>Shear plane indicators.<br>Multiple deflectometers.<br>In-place inclinometers.<br>Combined piezometer-inclinometer system.<br>Acoustic emission monitoring. |
| Groundwater pressure. | Observation wells.<br>Single piezometers.<br>Multi-point piezometers.<br>Combined piezometer-inclinometer system. |

To the degree possible, key instruments should be installed and initial measurements taken before excavation starts. Additional instruments can be installed as excavation progresses.

## How Much Ground is Moving?

If there is evidence of instability during excavation or after completion of the slope, its characteristics must be defined so that any necessary remedial measures may be taken. The question "how much ground is moving?" can be answered by use of instrumentation. The question "why is the ground moving?" will not be answered by instrumentation alone: the answer of course also requires a complete geotechnical investigation and analysis.

Methods of instrumentation have been described in the previous section. Possible layouts for determining how much ground is moving in excavated slopes are shown in Figures 1 and 2. It must be stressed that these layouts are only examples, and many other configurations are possible. Other configurations are indicated by the case histories described later in this paper.

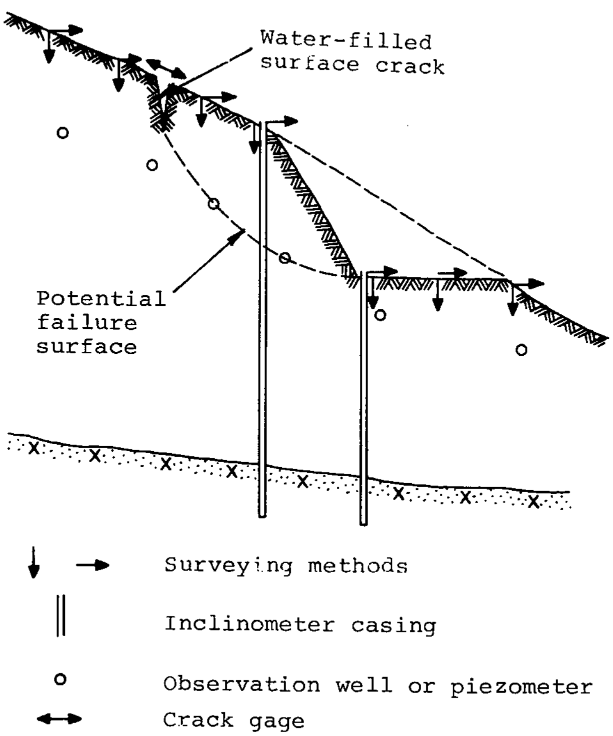


Figure 1. Possible layout of instrumentation for monitoring an excavated slope in soil when there is evidence of instability.[6]

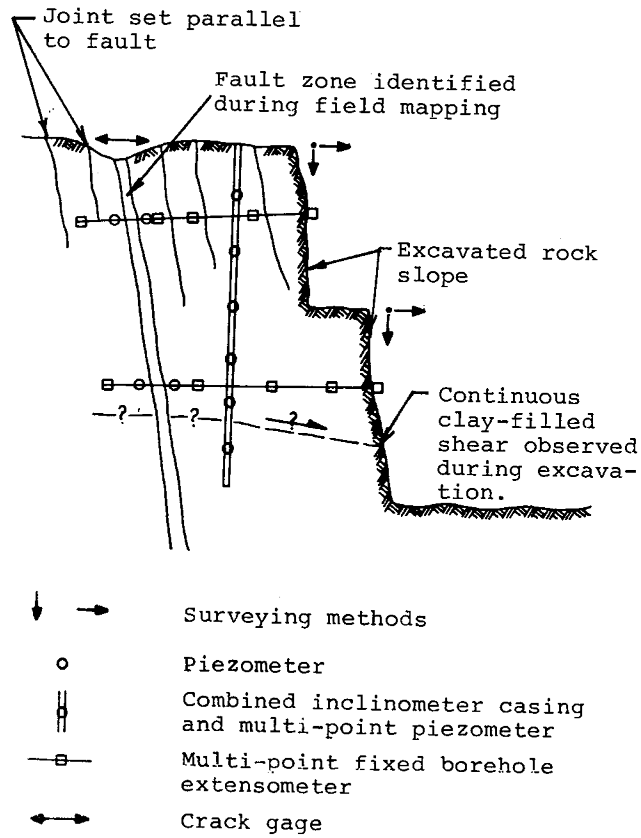


Figure 2. Possible layout of instrumentation for monitoring an excavated slope in rock when there is evidence of instability.

## Is the Slope Stable in the Long-Term?

The question applies primarily to excavated slopes that have been unstable during excavation. However, there are cases where the question applies also to excavated slopes with no history of instability, for example when construction is planned near the toe. The question may also apply to natural slopes.

In general a choice must be made among:

(a) doing nothing, accepting the consequences of slope failure,

(b) monitoring to provide a forewarning of instability, so that remedial measures can be implemented before critical situations arise,

and (c) stabilizing the slope, perhaps including a monitoring program to verify that stability has been achieved.

The choice will be based on many factors, including the consequences of failure and the economics of stabilizing. When planning a monitoring program for either of the second two options, a two-step approach is recommended: first, measurements of deformation on the surface of the entire slope to indicate existence of any instability and second, additional measurements at any locations shown to be unstable. Measurement methods are similar to those discussed in the earlier section describing stability during excavation.

For the first step, regular visual observations play an important role. Electronic distance measurement is the primary method for monitoring any deformation on the surface of the entire slope, and mechanical or electrical crack gages are used to monitor any deformation at existing cracks. If necessary, crack gages can be connected to alarms, set to trigger after a predetermined deformation has occurred. Acoustic emission techniques and tilt monitoring can sometimes be helpful when assessing stability. If stability has been increased by improving groundwater drainage, long-term effectiveness of the drainage will usually be monitored as part of the first step, by measuring groundwater pressure.

If data gathered during the first step indicate an unstable area, the same surface observations and measurements will be concentrated in that area during a second step, to define the instability more closely. The depth of sliding may become readily apparent, and it may therefore be possible to plan and implement remedial measures without the need for subsurface instrumentation. When the depth and thickness of the zone of sliding are not apparent, measurements of subsurface horizontal deformation and groundwater pressure will normally be required.

Two additional factors may cause instability in the long-term, and may merit a monitoring program. First, the wedging action of ice can cause serious instability, and freezing at the surface can cause ice dams to form, which hold unfrozen groundwater inside the ground. The buildup of internal water pressure, unless relieved by drainage, can lead to slope failure. The same effect is observed when sunlight acts on the upper slopes of deep valleys, melting the ice and snow at upper levels, which then percolates downward and exerts pressure against the still frozen ground in the lower, more shaded regions. To combat this effect, drainage of the slope may be required, and drains must be prevented from freezing. Temperature measurements will show the depth and extent of any freezing problems.

Second, loss of tension in rockbolts can cause instability. If rockbolts have been installed for slope reinforcement, the opportunity arises to measure load in the rockbolts, and these measurements may be useful for verifying assumed design loads. However, recognizing that the purpose of rockbolts is to restrain deformation, it is usually more effective to monitor deformation of the rock rather than stress in rockbolts, using fixed borehole extensometers. A typical approach is use of single-point fixed borehole extensometers, with rods and expansion shell anchors, anchored deeper than the rockbolts. If measurements of load in the rockbolts are also required for more comprehensive monitoring, load cells can be used on end-anchored rockbolts but strain gages must be used on fully grouted rockbolts.

## Overview of Routine and Special Applications

For those readers who seek general guidelines on what is typically **routine** monitoring and what is monitoring for **special applications,** Table 2 presents a broad generalization.

In practice the most frequent use of instrumentation for excavated and natural slopes is to investigate the characteristics of a slope that is observed to be moving.

Table 2. Overview of Routine and Special Monitoring.

| Application | Measurement |
|---|---|
| Routine monitoring. | Surface deformation.<br>Groundwater pressure. |
| Special applications. | Subsurface deformation.<br>Load in rock bolts.<br>Temperature. |

## Selected Case Histories

Table 3 summarizes selected case histories of instrumented excavated slopes.

Table 3. Summary of Selected Case Histories of Excavated Slopes.*

| Project | Principal Concerns | Type of Slope | Measurement** | Instrumentation Discussed*** | Special Features | Reference |
|---|---|---|---|---|---|---|
| Vaiont Slide, Italy. Pillar Mountain Slide, Alaska. Downie Slide, Canada. | Cause, physical characteristics, activity status of slides. Planning remedial measures. | Rock. | D<br>D<br>D<br>D<br>D<br>D<br>G<br>G,D | Surveying methods.<br>Crack gages.<br>Inclinometers.<br>In-place inclinometers.<br>Fixed borehole extensometers.<br>Acoustic emission monitoring.<br>Open standpipe piezometers.<br>Combined multi-point piezometers and inclinometers. | | 1 |
| Downie Slide, Revelstoke, B.C., Canada. | Physical characteristics and activity status of slide. | Rock. | D<br>D<br>D | Surveying methods.<br>Inclinometers.<br>Acoustic emission monitoring. | | 7 |
| Mine in Western Canada. Reservoir area of a dam. | Stability of slopes. | Rock. | D<br>D<br>D<br>D<br>D<br>D<br>G | Electrical crack gages.<br>Tiltmeters.<br>Inclinometers.<br>In-place inclinometers.<br>Fixed borehole extensometers.<br>Acoustic emission monitoring.<br>Vibrating wire and pneumatic piezometers. | Data transmitted to a remote location, using telemetry system. | 8 |
| Five open pit mines. | Stability of slopes. | Rock. | D<br>D<br>D<br>D<br>G | Surveying methods.<br>Mechanical crack gages.<br>Inclinometers.<br>Fixed borehole extensometers.<br>Piezometers. | | 9 |
| Cabin Creek Hydro-Electric Project, CO, USA. | Stability of slope. | Rock. | D | Fixed borehole extensometers. | | 10 |
| Fountain Slide, OR, USA. | Stability of slope. | Rock. | D<br>G | Inclinometers.<br>Pneumatic piezometers. | | 11 |
| I-40, TN, USA. I-26, NC, USA. | Stability of slopes. | Rock. | D<br>D | Surveying methods.<br>Inclinometers. | | 12 |
| Minneapolis Freeway, Minneapolis, MN, USA. | Cause and physical characteristics of slide. Planning remedial measures. | Rock. | D<br>G | Inclinometers.<br>Open standpipe piezometers. | Slope failure along bentonite seam. | 13 |

Table 3. Summary of Selected Case Histories of Excavated Slopes.* (continued)

| Project | Principal Concerns | Type of Slope | Measurement** | Instrumentation Discussed*** | Special Features | Reference |
|---|---|---|---|---|---|---|
| Seattle Freeway, Seattle, WA, USA. | Stability of eight slopes. | Over-consolidated clay. | D<br>D<br>G<br>G | Surveying methods.<br>Inclinometers.<br>Observation wells.<br>Piezometers. | | 14 |
| Chuquicamata Mine, Chile. | Prediction of failure. | Rock. | D<br>D | Crack gages.<br>Acoustic emission monitoring. | | 15 |
| Potrero Hill Slide, San Francisco, CA, USA. | Stability of slope. | Rock. | D<br>D<br>D<br>D | Surveying methods.<br>Inclinometers.<br>Fixed borehole extensometers.<br>Acoustic emission monitoring. | Tunnel below toe of slope. | 16 |
| Tripp-Veteran Pitt, Ruth, NV, USA. | Stability of slope. | Rock. | D<br>D | Mechanical crack gages.<br>Acoustic emission monitoring. | | 17 |
| Eight slope failures. | Various. | Soil and rock. | D<br>D<br>G | Surveying methods.<br>Inclinometers.<br>Open standpipe piezometers. | | 18 |
| Steel Plant Expansion, Weirton, WV, USA. | Cause and physical characteristics of slide.<br>Planning remedial measures. | Colluvium. | D<br>D<br>G<br>S | Surveying methods.<br>Inclinometers.<br>Open standpipe piezometers.<br>Strain gages on tension ties. | Slope stabilized with flexible sheet pile wall and tension ties. | 19 |
| Three slides alongside railroads in Japan. | Prediction of failure. | Soil and rock. | D<br>D | Surveying methods.<br>Crack gages. | Recommendations for forecasting failure. | 20 |
| Portugese Bend Landslide, Palos Verdes, CA, USA. | Stability of slope. | Rock. | D<br>D<br>D | Surveying methods.<br>Crack gages.<br>Tiltmeters. | Unsuccessful attempt to control sliding by installing caissons across sliding surface. | 21 |

* Case histories are in chronological order, starting with the most recent.
**D: Deformation. G: Groundwater pressure. S: Strain in structural member.
***Details of instrumentation are given in References 3, 4 and 5.

## Acknowledgement

This paper is based on a chapter of a book by the author, currently in press, that discusses geotechnical instrumentation for monitoring field performance.[3] The chapter has been written with the assistance of Robin B. Dill, Senior Engineer, and Douglas G. Gifford, Associate and Vice President, Haley & Aldrich Inc., Cambridge, MA, whose cooperation and assistance are acknowledged with great gratitude.


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# LANDSLIDE RECOGNITION AND CONSTRUCTIVE PREVENTION IN THE APPALACHIAN AREA

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Abstract. Natural landslides in the Appalachian area tend to develop characteristic topographies which are recurring and recognizable. The great majority of landslide problems resulting from construction develop in these natural landslide areas. Recognition of these areas and suitable preventive construction methods should eliminate most landslides, especially the more costly failures.

The author has inspected and investigated thousands of landslides throughout the Appalachian area (predominantly Kentucky, Ohio, Pennsylvania, Virginia, and West Virginia). This experience has led to the observation that most "new" landslide problems are merely variations of problems encountered at another time and location. In nearly all cases the problem could have been predicted and prevented with a tremendous saving in cost.

The author presents the more important and conspicuous topographic features which serve as precursors for natural landslide areas. In addition, the more common types of failures are discussed, and design and construction measures for prevention of the failures are outlined. Since care must be exercised in applying preventive measures to insure that the "cure" does not create a new problem, a checklist is provided to assist the reader in evaluating preventive measures.

## Introduction

The section of the United States which includes eastern Kentucky, eastern Ohio, western Pennsylvania, western West Virginia, and the western tip of Virginia (and perhaps bits of contiguous states) is infamous for its landslides. Baker and Chieruzzi[1] designated this area as the only area east of the Mississippi with major landslide severity (see Figure 1). Woods, Miles, and Lovell[2] stated,

> "Landslides are extremely prevalent in parts of the Appalachian Plateau, West Virginia probably being the most difficult area from this standpoint on the Continent"

With obvious reference to the same region, Deere and Patton[3] observed,

> "The Allegheny Plateau region, a relatively large area of rugged topography in the east-central United States, is entirely underlain by shales, sandstones, and

related rocks, and many engineering projects in this area are directly affected by the severe slope stability problems that develop."

The geologic history and the role of landslides in the development of the terrain of this region are dominant factors in the slope stability problems which abound. Peck[4] noted,

> "Experts in air-photo interpretation and geomorphologists have been pointing out the implications of old slide areas for years. They have gone so far as to suggest that, if there are no old landslides in an area, it is fairly unlikely that a moderate construction operation will start a new one. On the other hand, if old landslides abound, it is quite likely that even minor construction operations will lead to sliding."

Landslide problems continue to develop throughout the described region, and the majority of the slides develop in topography which is strikingly similar. Is the problem failure to recognize these slide prone areas, or is the problem a lack of knowledge of preventive design? It is hoped that the information provided in the following pages will help to alleviate future landslide problems by providing some information on both recognition and prevention.

## Landslide Recognition

### Scope

Landslides discussed in this paper are limited to those falling into certain categories. First, consideration is limited to failure of soil materials in which shearing along one or more planes or narrow zones is the primary mechanism for mass movement. This omits such failures as rockfalls and mud flows; however, many mud flows are initiated as planar shear failures, and these are tacitly included. The second limiting category is geographic region. Discussion is limited to conditions found in the region cited in the Introduction and delineated in Figure 1. This region is contained in the Appalachian Plateau Province and is characterized by soils derived from rocks of Permian and Pennsylvanian age. The last limitation pertains to the topographic features described for landslide susceptible terrain. Construction in terrain with such features does not always produce landslides nor do all landslides occur in terrain with the indicated features. However, the majority of landslides in the region, and usually the more serious landslides, occur in the types of topography described. If only those landslides in this category could be prevented, many millions of dollars would be saved throughout future years.

### Topographic Features

The single most important topographic feature with respect to landslide potential in the region of interest is the natural hillside drainage channel. Roughly two-thirds of the thousands of landslides inspected by the author occurred in, around, or among hillside drainage channels. This observation holds in both those areas which are shale dominant and those which are sandstone dominant. In fact, in sandstone dominant areas (typical in much of eastern Kentucky, southwestern West Virginia, and western Virginia), colluvium filled hillside drainage channels and colluvium covered footslopes are the primary landslide topographies.

The hillside drainage channel is an obvious path for surface water but is often overlooked as an avenue for subsurface water. These channels often contain colluvium, which is combined slide and slope wash materials. The colluvial materials may even collect to the point that the hillside drainage channel is masked; i.e., the channel becomes filled with colluvium. If disturbed, especially during wet periods, the material often slides right down the channel. This condition is one of the more dangerous and the least easy to recognize of the topographic features. There are nearly always at least two indications of the condition: a subtle depression along the buried channel; and a channel, or at least a strong indication of a surface water concentration area, on the hillside above. Springs may or may not be found below.

Even deeply incised hillside channels, with the wet weather water flow paths in the bottom being on bedrock, are often slide susceptible. The sides of the channels are often composed of deeply weathered residual soils, colluvium from past infilling, or both. The sides are prone to fail by sliding into the channel.

Drainage channels along hilltops and ridges are not to be ignored. This is especially true at saddles in ridges. The author never ceases to be amazed at the quantity of groundwater that can be accumulated a very short distance below a small hilltop or narrow ridge. Colluvium infilling (perhaps left from a time when the hill or ridge was higher than present), residual soils, and wet weather groundwater seepage combine to make poor foundation conditions for fill material placed in the head of these channels.

Since colluvium has been moving down slopes for many millennia, it is not surprising that considerable accumulations may be found in footslopes of hillsides. For this reason, any construction in the footslope area of a hillside should be undertaken with caution. There are certain topographic conditions, however, which indicate special cause for alarm. These include concave shaped hillsides, amphitheatered hollows and bulges in footslopes at the terminus of one or more hillside channels.

Concave hillsides tend to funnel surface water, colluvium, and subsurface water into a more limited area at the toe of the slope. The concave shape is undoubtedly the result of erosion by a meandering stream during formation of the valley, and this erosion process continues to promote accumulation of colluvium at the toe of the slope. In concave hillsides composed of interbedded sandstones and shales in which the shales are dominant, multiple layers of colluvium are sometimes found which contain water under considerable artesian pressure. The author has seen borings in footslopes of such concave hillsides which have flowed water at the surface for several hours. Needless to say, these hillsides are replete with old slip planes. Accumulations of colluvium of 60 to 80 feet are not uncommon on major streams (rivers and creeks), while accumulations of 20 to 40 feet are common on smaller streams (brooks or runs). Concave hillsides with considerable accumulations of colluvium are also found in sandstone dominant hillsides; the author has not encountered artesian water pressure in these hillsides, but very high wet weather water tables are common. Deere and Patton[3] have given an excellent discussion of the development of colluvium accumulations on hillsides.

Amphitheater shaped heads of hollows are somewhat similar to concave hillsides in that the concave shape tends to promote a downslope funnel type movement of both water and colluvium. The amphitheater is full of hillside drainage channels with their associated slides. Yet, the amphitheater is also different from concave hillsides. The greater concavity and the heavy concentration of intermittent water flow causes more active erosion. Hence, the very deep accumulations of colluvium are not common. While the concave hillside, especially the footslope area, is prone to yield a massive, difficult landslide, the amphitheater is more apt to produce a multitude of smaller landslides. Nevertheless, amphitheaters, especially in the footslopes, occasionally produce deep, massive failures.

A bulge or protrusion of the toe of a slope into a valley or hollow deserves attention. If examination of the hillside above the projection discloses a hillside drainage channel (or sometimes more than one channel if the projecting mass is large), the bulge is probably a mass of colluvium which can be unstable during wet periods. Often these bulges are "in the way" of construction projects in the valley, or they are viewed as good starting points for roads up the hillside. Disturbance is likely to lead to slope failure.

The last topographic feature to be discussed is the natural bench or gently sloping areas on hillsides. There are many instances where natural benches on hillsides are manifestations of a weak, weatherable stratum - normally clay shale. The underlying stratum is usually a strong, weather resistant sandstone. Differential weathering not only creates the bench but brings hordes of slope debris from above onto the bench. Three factors become involved during development of the bench:

1. the bench becomes loaded with colluvium;

2. the low permeability of the weak stratum catches water percolating through the hill and along the hillside and promotes groundwater seepage through the residual soil and colluvium on the bench;

3. these two factors combined with the weakness of the residual soil lead to repeated failures which keep flattening the bench.

The width of such benches appears to depend on several factors including the thickness of the weak stratum or strata and the competence of the overlying strata. The benches may occasionally become several hundred feet wide, which makes them attractive for streets and subdivisions. The benches may also become discontinuous because of dissection by hillside drainage channels, changes in lithology, or other changes. Sometimes the condition is so limited that it merely appears as an anomalous flat area on the hillside, probably due to a localized weak zone, a concentration of groundwater, or both.

A description of the foregoing topographic features is now presented in a series of figures as a summary of the landslide recognition portion of this paper. Figure 2 is a photograph of a typical landslide in a hillside drainage channel. The roadway cut which caused the slide was only 10 or 12 feet high on a 2 horizontal to 1 vertical slope. Notice the saddle in the ridgeline above.

It is difficult to show larger slide areas in photographs; further, modern preliminary engineering is usually blessed with topographic mapping. Consequently, Figures 3 through 6 show the limits or perimeters of actual landslides on topographic maps in order to demonstrate some typical landslide topographies.

Figure 3 shows a landslide which was precipitated by a moderate excavation to straighten the stream channel. Observe the hillside drainage channel evidenced by concave contours almost to the top of the hill. Also notice how the slope flattens toward the footslope area and how multiple, distinct drainage channels appear but seem to become discontinuous. These are typical indications of a dangerous collection of colluvium in the footslope. The soil depth would normally be relatively small at the top of the slide but would increase appreciably with accumulated colluvium depth toward the toe.

Figure 4 illustrates an area much like that shown in Figure 3 except for scale. Any attempt to place natural phenomena into categories always encounters gray areas, and this is a good example. The area shown in Figure 4 might be considered either a concave hillside or a hillside drainage channel. Nevertheless, the more gently sloping area in the lower half of the slope is typically slide susceptible. The concave contours in the upper slope overlie the more dangerous accumulation of colluvium. Also the contours at the bottom of the slope protrude into the valley. If the reader will recall that water and colluvium will tend to move perpendicular to the contours and, ignoring the slide limits shown, will attempt to envision the long term downslope movement of materials, the most likely slide area can be imagined. The protruding contours further help delineate the greatest slide potential. Look again at Figure 3 and attempt to envision the slide prone area. Don't be concerned if you envision areas somewhat larger than the actual slide areas. The actual landslide limits were influenced by the construction taking place, and the construction was localized excavation in both instances. The hillside shown in Figure 4, in particular, could produce a much more extensive slide mass.

Figure 5 is an excellent example of a concave hillside complete with meandering stream cutting at the toe of the slope. The landslide developed during initial stages of embankment construction for a highway. The roadway grade was rising to the right, so the embankment height increased in that direction. The embankment section transitions to a cut section to the right of the slide area. Again, the construction significantly affected the size and shape of the slide, but the reader should recognize the area as slide susceptible.

Figure 6 shows an anomalous flat area. The formation of such areas is apparently akin to the formation of slide susceptible benches, as described before. Note the relationship of the slide limits to the original contours.

## Landslide Prevention

The previous discussion has pointed out how the limits of landslides are influenced by the construction which disturbed the slide susceptible areas. Other facts also indicate the important effects of construction. For example, there are hundreds of houses within the region under consideration which were built on old landslides and which have shown no movement. Other houses built in similar old slide areas have been damaged or destroyed. It is apparent that, whether by design or happenstance, the nature of construction in a slide prone area is very important to future stability.

There have been many more papers written describing landslides and slope stability problems and analyses than have been written describing the prevention of landslides. Those papers written concerning prevention are usually either very general or very site specific. This is quite understandable since each landslide preventive measure must be tailored to the conditions imposed by individual projects and sites. Much thought and considerable experience are needed to arrive at the "best" solution. Nevertheless, there are some common methods and "dos and don'ts" which must be kept in mind, and these will be summarized below.

The most commonly propounded preventive measure is avoidance. While there are certain circumstances where economy dictates avoidance, construction on the same piece of property may well be quite feasible for another project. For example, a slide prone lot may be a poor site for a moderately priced home but may be a good site for a sizable apartment building. The size of the slide prone area relative to the size of the project must be considered. The author prefers to consider avoidance an option of site selection rather than a landslide preventive measure.

Before looking at preventive measures, it is informative to look at common causes of failures. Knowing actions which cause landslides may assist in developing converse actions or at least may tell us what not to do. Certainly the foremost causes of landslides are those provided by nature - downslope groundwater seepage and continuing landslide activity. Hence, areas which produce "man-made" landslides are often only barely stable to begin with.

Examples of slide susceptible topography in the previous section also give some examples of causes of landslides in such topography. Cut slopes or excavations at the toe of slide prone areas represent a very common cause. Any embankment construction which reduces the factor of safety of the slope normally causes failure. Even embankment construction at the toe of slide prone areas, which might be considered as a buttress, can cause failure as discussed later.

Introduction of water into a slide prone area is a common causal action, and there are many sources of water. Poor surface drainage, poor choice of exit locations or conditions for foundation drainage, roof drainage, and other storm drainage, and similar drainage shortcomings cause many slides. Even when drainage is properly installed, a characteristic of slide prone areas is frequently overlooked - continued downslope creep or adjustment. Creep occurs in all hillside soils and is often more pronounced in slide prone areas. Also, preventive measures oftentimes require time to become fully effective. For example, subsurface drainage in low permeability soils may require a few years to fully lower the water table. Creep and adjustment movements can rupture sewer lines, water lines, and drain lines, thereby leading to failure where failure had been "prevented."

With these more common causal actions in mind - excavation or filling which reduces slope stability and introducing water into the ground - we look at some common counteractions or preventive measures. Again, the most suitable measures are not only site specific but also project specific; there are usually many options open in treating a particular problem that cannot be discussed here because they are unique to the problem or project.

Drainage, both surface and subsurface but especially the latter, is no doubt the foremost preventive measure. While good surface drainage can be planned effectively, subsurface drainage may offer problems. First, groundwater seepage in natural soils in general and colluvium in particular is far from uniform, and the drainage pattern is usually either unknown or not completely known. Hence, small, localized subdrains should be avoided except to convey water from specific, known groundwater sources. Rather, general drainage designed to intercept all random seepage zones should be used. These may be trenches in a dendritic pattern, blanket drains, or similar systems.

A second problem with subsurface drainage is the time required for the drainage to be effective. In low permeability soils, months or even years may be required to accomplish a significant increase in the factor of safety by drainage alone. Sometimes the landslide will move before the drainage has done its job, and the movement may interrupt the drainage and negate its future functioning. Consequently, when drainage of low permeability soils is the primary preventive measure, a secondary measure which will provide at least a small increase in factor of safety may be in order.

A classic preventive measure is to remove material from the top of the potential slide area and to place the material at the bottom as a buttress. This is an excellent choice where applicable. There are many variations of this approach. For example, if a building with a basement is to be constructed, the building may be located so that excavation for the basement removes weight from the top of the potential slide area while the excavated material is placed as weight at the toe. Other alternatives involve using only excavation in the upper area or fill in the lower area (but not both). These involve transportation of material away from or onto the site, and the added expense may sometimes be minimized by selling fill material to or buying waste material from a nearby site.

Retaining and restraining structures other than buttressing fills may also be used. Retaining walls, piling, and various methods of earth reinforcing are often invaluable, especially where property lines or other physical limitations restrict cut or fill slopes.

## Preventive Design

The design of preventive measures for a project to be constructed on a site which is recognized as slide prone does not deviate greatly from normal practice. Recognition of the potential problem is an asset in planning the subsurface exploration program; a good soils and groundwater profile must be established to assess slope stability. Location of borings is more important than quantity, and carefully installed multilevel piezometers in selected borings yields much better groundwater information than a multitude of measurements in open boreholes. Borings cost money - plan carefully, obtain quality information, and never use a shotgun approach. If the reader can begin to visualize potential slide areas by inspection of the topography, then locations for needed borings are much easier to rationalize.

Stability analyses within our region of interest will yield reliable results if used as follows:

1. use effective stress analyses;

2. remember that groundwater conditions can change greatly with climatic conditions in a short period of time, so treat groundwater conservatively;

3. remember the existence of old slip planes - use residual strength if recent slide movement is evident, otherwise simply ignore or severely limit cohesion intercept values.[5]

Landslide preventive measures and landslide corrective measures are quite alike, and there is much to learn from failure of efforts to hold slopes by various means. Some common failures and their causes are now discussed, and a checklist is then provided to assist in evaluating the adequacy of the design of common protective measures.

Problems with subsurface drainage have already been discussed. Problems with surface drainage usually result from inadequate slope on unpaved channels and failure to extend the drainage beyond the limits of the slide prone area. Sometimes neglect in maintaining surface drainage during construction causes trouble, and poor maintenance of ditches after construction may contribute to failure.

Failures associated with retaining walls depend on the use of the wall. If the retaining wall is merely to contain earth buttress material, as is often the case where property lines or other restrictions prevent use of the full buttress, the wall needs only to be designed to normal standards. If, however, the wall is intended to restrain potential slide movement, three modes of failure may occur: failure under the wall, failure through the wall, and failure over top of the wall. These modes can occur regardless of the type of wall (cantilever, gravity, etc.).

Piling is normally used in cantilever beam fashion to retain or restrain a soil slope mass. The bottom portion of the piling must be fixed into reasonably competent material (usually bedrock) below the deepest practicable level of slope movement. The piles are rarely driven; instead, they are placed in predrilled holes and grouted. Failure of the piling itself may occur by three modes: by being pushed over (insufficient fixity); by bending (insufficient bending strength); by shearing (extremely rare). This method of correction can also fail by movement of slope material downhill of the piling, failure over top of the piling, and movement of soil between the piles. See Baker and Yoder.[6]

Excavation at the top and buttressing at the bottom of a slide prone area usually fails in the following manners: excavation at the top extends into a slide susceptible area above (e.g., a bench); excess pore water pressure under the buttress leads to failure which can be retrogressive; inadequate drainage under the buttress leads to saturation and subsequent failure; excavation and buttress are inadequate.

Table 1. Checklist to Assist in Evaluating Preventive Measures

| | |
|---|---|
| Site Selection Option | Is project big enough to warrant use of a potential slide area? |
| Retaining Wall | Can failure overtop the wall?<br>Can failure pass under the wall?<br>Is the wall designed for added load to resist potential slide movement? |
| Piling | Fixed below deepest failure depth?<br>Strong enough to resist bending?<br>Strong enough to resist shear?<br>Is earth movement below (downhill of) the piling possible (or harmful)?<br>Can failure pass over the piling?<br>Is soil movement between the piling possible? |
| Excavation At Top (Unloading) | Can excavation cause failure from above?<br>Is excavation adequate? |
| Earth or Rock Buttress | Is subdrainage necessary or adequate?<br>Were excess pore pressures under weight of buttress considered in analyses?<br>Is buttress size adequate? |
| Subsurface Drainage | Is drainage deep enough to sufficiently lower the water table?<br>Is drainage extensive enough to intercept random seepage paths?<br>Has time for effective drainage been assessed?<br>Is movement possible which could damage drainage?<br>Have drainage outlet locations been judiciously selected? |

## Summary and Conclusions

Information has been given on recognition of landslide susceptible terrain in the Appalachian area and on prevention of landslides. The information is far from exhaustive but should be helpful as an abridgment of these subjects. If readers are assisted in preventing even a few landslides, the effort will be worthwhile.

Except for references to shale dominant and sandstone dominant areas, lithology was not discussed. This does not mean that rock character is not important. It suffices to say that the number and severity of landslides tend to increase with the abundance of clay shale.

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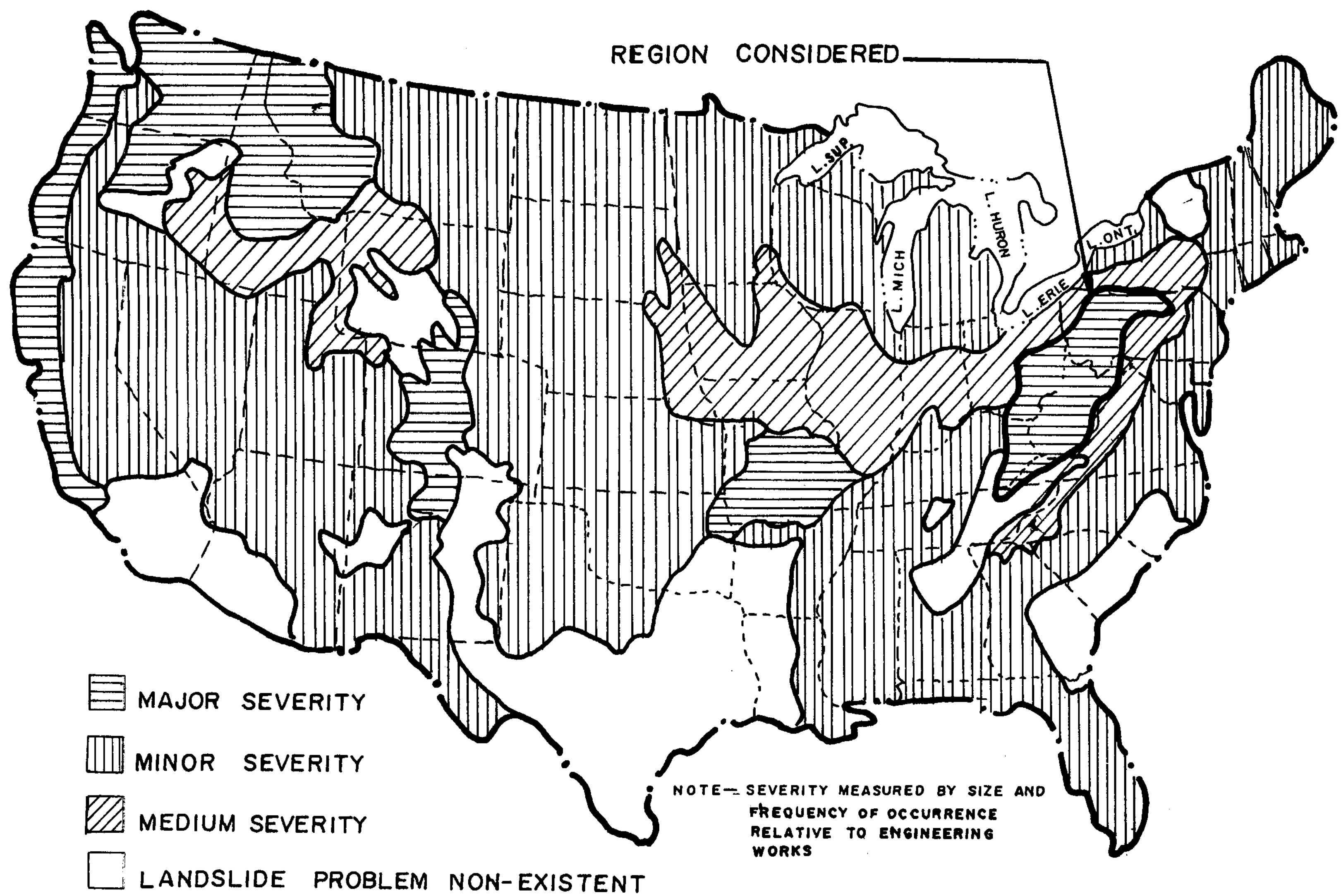


Figure 1. Landslide Severity of the United States (After Baker and Chieruzzi[1])

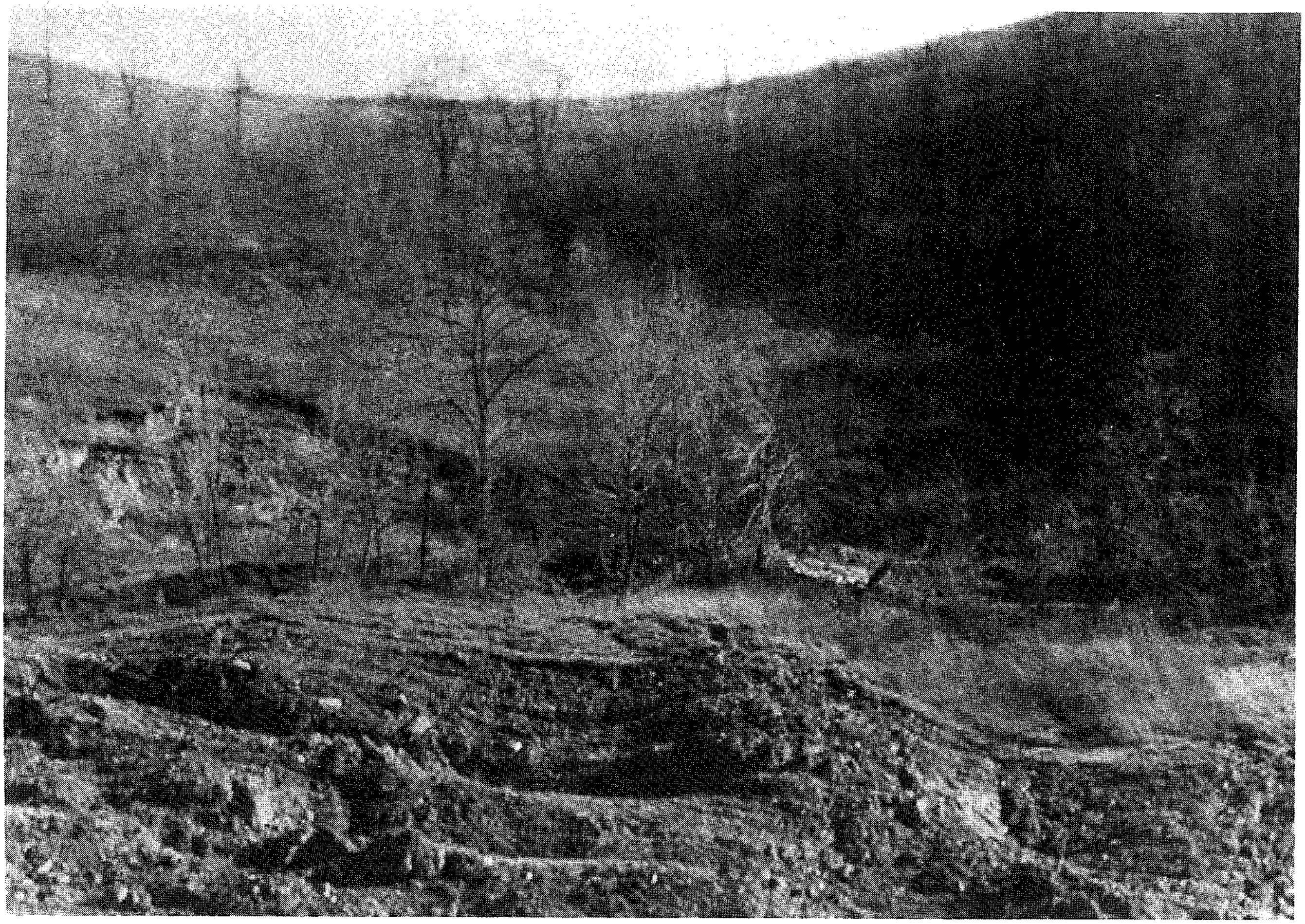

Figure 2. Characteristic Slide in Hillside Drainage Channel

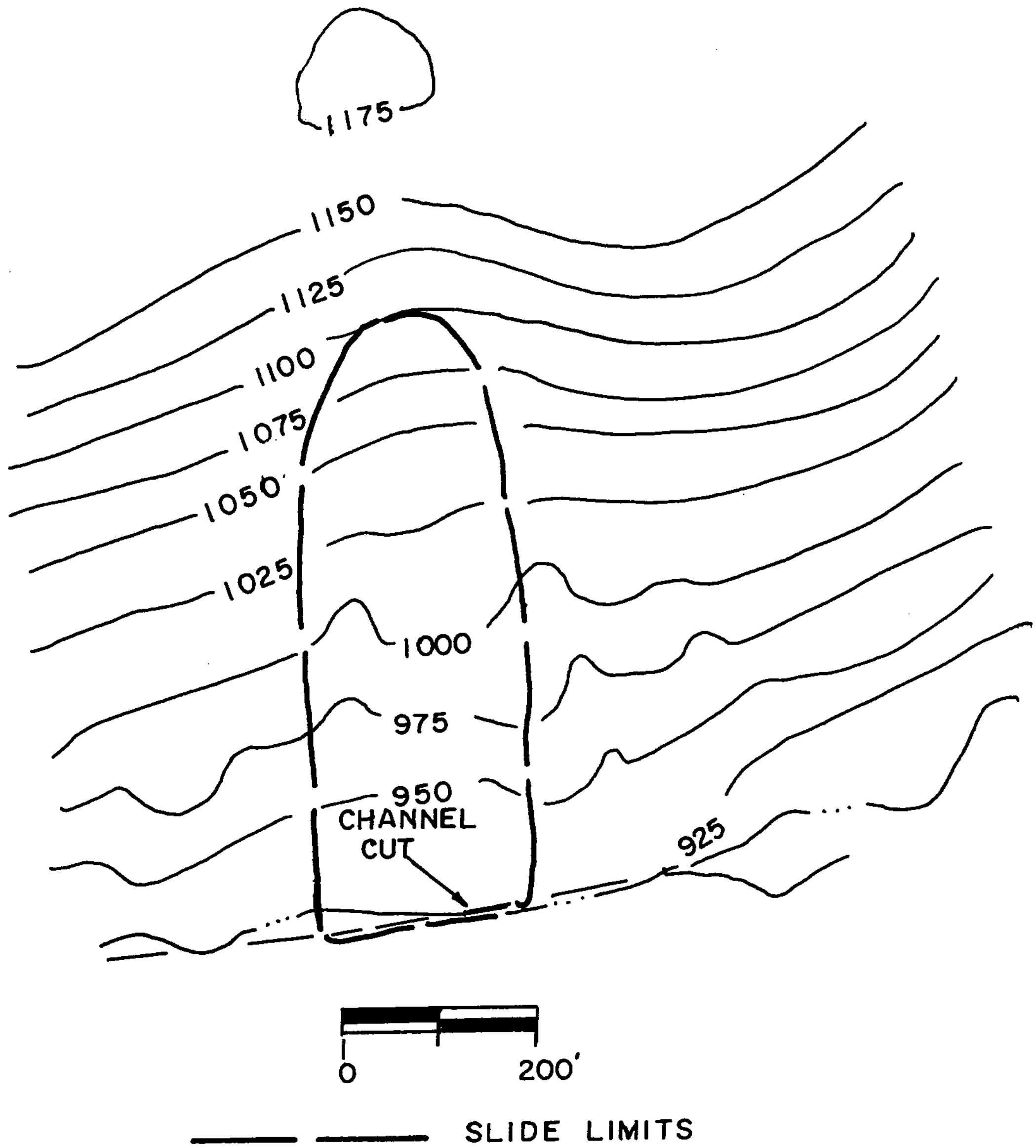


Figure 3. Characteristic Slide in Footslope and Drainage Channel

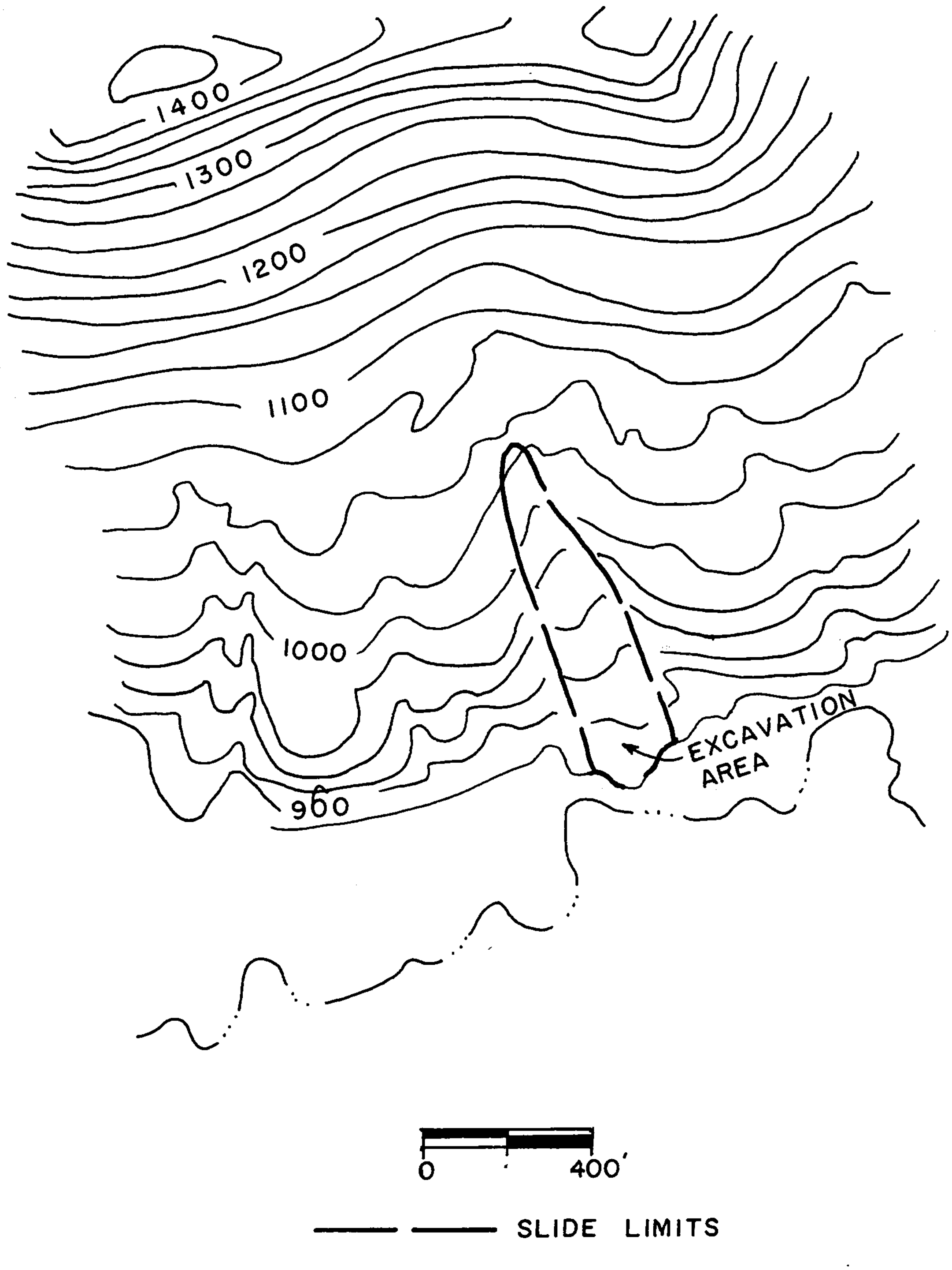


Figure 4. Characteristic Slide in Footslope of Hillside Drainage Area. Note Bulging Contours.

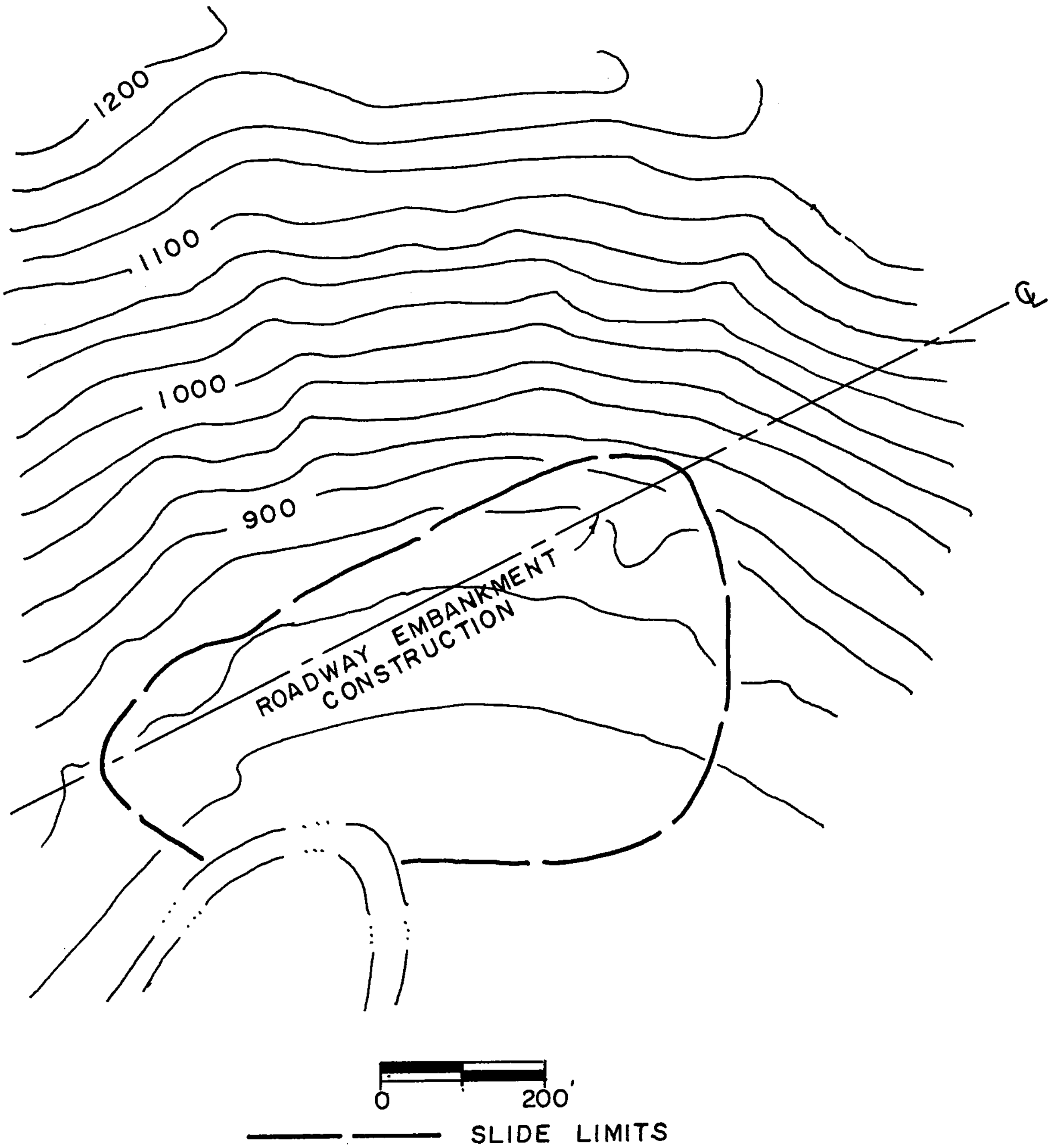


Figure 5. Characteristic Slide in a Concave Hillside

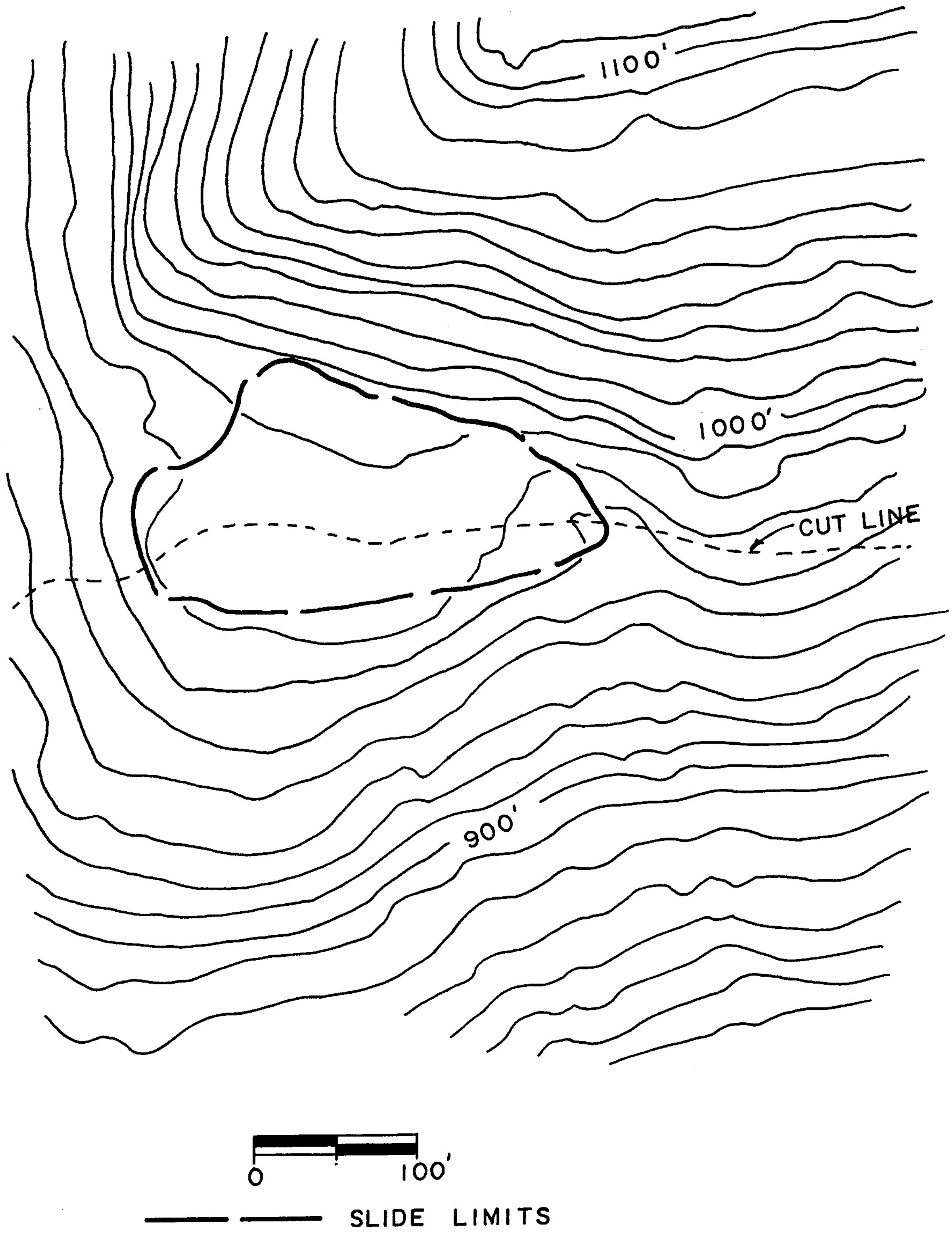


Figure 6. Characteristic Slide in an Anomalous Flat Area

# RED MOUNTAIN LANDSLIDE SUSCEPTIBILITY STUDY

by

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Abstract: The north face of Red Mountain within the City limits of Birmingham, Alabama is relatively steep in its natural condition and is covered with colluvial and residual soil. With increased utilization of the steeper portions of the mountain, problems have risen with the stability of the slopes. Consequently, a study was undertaken of a portion of the north face of Red Mountain to determine its susceptibility to the formation of landslides. Almost all the observed landslides were found to be triggered by modification of the slope by man. The critical characteristics of the slide prone areas were preslide slope, the presence of shallow, shaley bedrock, and the presence of colluvial soil. The results of the study were compiled into a series of maps which classified all areas within the study boundaries as having a "low," "moderate," or "high" susceptibility to landsliding.

## Introduction

Red Mountain is a generally linear physiographic feature which forms the southeast boundary of Jones Valley within the city limits of Birmingham, Alabama. A number of landslides have occurred along the north face of Red Mountain causing damage to buildings and other improvements. Therefore, the City of Birmingham authorized a study to: (1) characterize the type of landslides that were occurring along a portion of the north face of Red Mountain; (2) identify factors contributing to the frequency of landslides in the study area; and (3) identify those portions of the study area which are more susceptible to landsliding.

The study was conducted by first accumulating and reviewing existing published and unpublished information about the study area and its physical characteristics. Information was obtained from many federal, state and local agencies in the form of maps, reports, aerial photographs and verbal communication. A site reconnaissance was then made of areas known or suspected to contain landslides. Approximately 50 landslides were documented, and the specific characteristics of the landslides were noted. The accumulated landslide data were then reviewed to assess which factors were common to a large number of landslides and could be used as an indicator of landslide susceptibility.

## Descripton of Study Area

### Study Area Boundaries

The study area included approximately 4.5 square miles along the north face of Red Mountain within the city limits of Birmingham, Alabama. The crest of Red Mountain was typically the southern boundary of the study area. The northern boundary was less definite, but by description it followed the relatively flat area north of a lower, secondary ridge. The study area was roughly 8.2 miles long and 0.6 miles wide (Figure 1).

### Ground Surface Description

The north face of Red Mountain has steep to moderate topographic relief (slopes of 25 percent and steeper), and the direction of the slope is generally downward toward the northwest. Areas not currently developed commonly have a heavy brush and pine tree cover except where previous clearing has occurred or rock is exposed. In cleared areas, vines, such as Kudzu, densely cover the surface during the spring and summer.

Development of the north face has been ongoing for a number of years, and roadways and structures are well established, particularly on the lower regions of the study area. Large rock outcroppings are common in the upper steep portions where only minimal development has occurred. Mining activity has greatly altered some areas near the crest including widespread strip mining of the dipping iron ore beds and displacement of mining spoils onto the north face.

Several characteristics of the area increase the potential for landslide frequency. These include: (1) the high annual precipitation (53 inches); (2) the increased desirability of the area for development resulting in concentrated man-made modifications to natural slopes, drainage patterns and vegetation; and (3) the extensive mining of the Red Mountain iron ore between 1870 and 1971 resulting in tailings being pushed onto the already unstable or marginally stable colluvial/residual soil veneer.

Areal Geology

Red Mountain forms the southeast boundary of Jones Valley which is a portion of the larger Birmingham Valley. The Birmingham Valley is included in the Appalachian Valley and Ridge Physiographic Province which extends into Central Alabama and is characterized by northeast-southwest trending valleys and ridges. The geology of Red Mountain and the Valley and Ridge Province, in general, is a result of intermittent deposition and erosion periods combined with and followed by complex folding and faulting of older underlying rock beds. The north face of Red Mountain is actually the steeply eroded south dipping limb of a large anticline. A layered pattern of geologic formations projects to the face, terminating with the resistant sandstone formations at the crest[3]. The youngest geologic formation extensively exposed in the study area is the Red Mountain Formation, which is found at the crest of the mountain. The Red Mountain Formation includes iron-rich sandstone, siltstone, and limestone, of the Silurian System. The oldest formation exposed is the Knox Group of the Ordovician System, locally known as the Copper Ridge Dolomite, which is exposed at lower elevations on the north side of the study area (Figure 2).

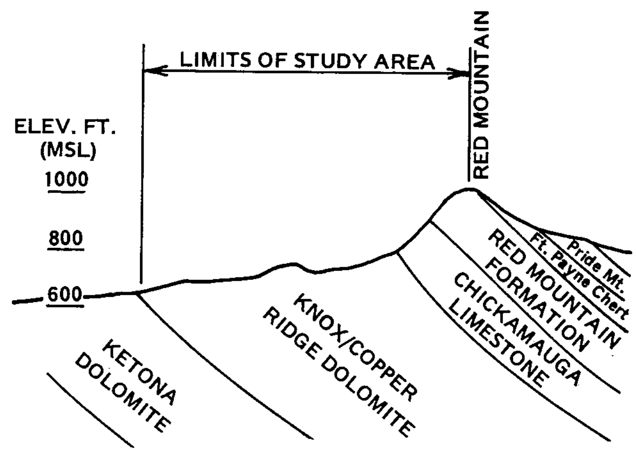


FIGURE 2. GENERALIZED GEOLOGIC CROSS SECTION OF RED MOUNTAIN

Soil Overburden Conditions

Overburden materials occurring on the face of Red Mountain typically consist of one or more of the following soil types: (1) residual soils, (2) colluvial soils, and (3) fill soils.

The thickness of residual soil varies widely. In the area of study, residual soil thicknesses commonly vary from less than 5 feet to more than 30 feet. Residual soils occur over most rock formations in the study area except where rock outcrops exist or where the residual soil has been replaced by colluvium following ancient landsliding. Soil characteristics vary, based on the mineral makeup of the parent rock.

Residual soils derived from the Red Mountain Formation grade from silty or sandy clays to silty or clayey sands, and typically exhibit a low to moderate plasticity. A distinctive red coloring and staining effect distinguish the residual soils of the Red Mountain formation from those of other formations with similar soil types but lacking the high concentration of hematite (iron ore).

Residual soils of the Chickamauga Limestone Formation are typically silty clays or clayey silts of moderate to high plasticity containing angular to subrounded chert fragments. Residual soils of the Copper Ridge Dolomite Formation are commonly moderate to high plasticity silty clays containing abundant fragments of light gray to grayish-pink chert.

The colluvial soils, by the nature of their mode of deposition by sliding and sloughing, are usually loosely bonded and have not been subjected to sufficient overburden pressures to achieve the level of compaction more common in the residual soils. The low densities and irregular particle orientations result in low strengths and higher compressibilities. The thickness of the colluvial deposits vary greatly; however, within the study area the thickness appears to be greatest where slopes become flatter. These conditions are common overlying the Chickamauga Formation where the colluvium can be more than 10 feet thick.

Fill soils are also common in the study area. Fill materials were often placed as a result of road building or leveling required for development. Some of the fill soils were placed in an uncontrolled manner and exhibited densities, moisture contents and slopes which are not consistent with generally accepted engineering practice for materials intended to support roadways or structures.

Uncontrolled fill from the old mining operations was also found extensively. These spoils were typically pushed or dumped onto the face of an existing slope. They received little compaction beyond the effects of mining equipment operating on their surface and they commonly contain boulders within the fill. They are usually prone to both subsidence and sliding.

### Groundwater

Available groundwater data was limited. However, springs located near the northern boundary of the study area and available groundwater data from previous geotechnical explorations and from the Alabama Geological Society publications suggested that the groundwater roughly follows the upper rock-soil interface and fluctuates seasonally.

## Description of Study

### Literature Review and Research

A review of both published technical information pertinent to the study and unpublished sources of landslide information were sought. A partial listing of the published information sources used is contained in the list of References. Engineering files from past geotechnical projects within and near the study area also provided valuable data.

### Aerial Photography Study

Black and white, stereo pair aerial photographs were available from both the City and from Jefferson County with full coverage of the study area. These sets of photographs were both made during the winter, but 6 years apart, and were prepared at scales of 1:6,000 and 1:12,000. The photos were most useful in providing a preview of the study area land characteristics and identifying areas where conditions were more conducive to landslides. Actual identification of individual slides by air photo interpretation was possible only in a few cases due to the extensive disturbance to some areas by old mining operations and the degree of development of the topographically lower areas.

### Interviews and Information from Public Agencies

Federal, state and local agencies were contacted for information on existing landslides. Few documented landslide reports were available; however, much assistance was provided through verbal histories by experienced inspectors and engineering personnel. A partial listing of agencies contacted is contained in Table I.

### Field Observations

Landslides or suspected areas of landslide activities revealed through the literature review and aerial photography interpretation or through information provided by individuals were first checked in the field before they were classified as landslides. During the site visits, we attempted to describe the relevant characteristics of the landslide including areal extent; type of movement (rotation or translation); geologic formation; presence of water seepage; soil fill; residuum (or colluvium); presence of human disturbance; preslide slope; direction of movement; and remedial measures taken.

Field observations were made to enable identification of as many slides as possible. These observations were accomplished by dividing the study area into a number of sections and performing on-foot reconnaissances aided by interpretations of more critical areas from aerial photographs. Initial field observations were concluded in June when dense vegetation prevented further meaningful observations. Additional field observations were made in conjunction with aerial photo interpretation and literature research as required throughout the study. Finally, at the conclusion of the map preparation, a follow-up reconnaissance of the mountain was performed to check that the areas generally exhibited the characteristics of the susceptibility level at which they were mapped.

Compilation of Data

The identified landslides were plotted on topographic maps (scale: 1"=100') provided by the City. Each landslide was given a reference number. Data sheets were completed for each landslide including information obtained in the field, together with information taken from public sources, such as Alabama Geological Society maps, Soil Conservation Service maps and the topographic maps mentioned above. Typical data compiled on each slide is shown in Table II.

As an additional evaluation aid, cross sections were plotted for selected segments of the north face and the nearby landslides were marked at their appropriate position on the slope. Exploratory data from previous engineering reports were used to estimate the soil thickness at the corresponding elevation on the profiles.

The collected data were then reviewed to assess which characteristics were common to most landslides. The factors having the highest frequency were labeled as critical factors and were used to define areas of higher susceptibility to landsliding.

## Findings

Description of Landslides Observed

Landslides are defined as the downward and outward movement of slope-forming materials such as natural rock, soils, fill materials, or combinations of these materials. Such movements can be further divided into five groups; falls, topples, spreads, flows and slides[8]. In this study, most movements noted were of the "slide" type. Slide movements can be further characterized as rotational or translational landslides. The former includes the more common, spoon shaped slumps resulting in an arch shaped scarp at the head and bulge at the toe of the landslides. Rotational slides move along a surface of rupture that is curved concavely upward, like the bowl of a spoon.

Landslides are a normal result of the continuing erosion, weathering and deposition processes which form and reform the surface of our planet. Landslides occur in nature when soil stresses exceeed soil strength due to weakening of the rock and soil through weathering, natural removal of previously counterbalancing masses or the influences of transitory forces such as fluctuating groundwater or seismic activity. However, since these natural processes are measured in a geologic time frame, the frequency of landslides that are caused by or accelerated by human disturbances is orders of magnitude greater than due purely to natural phenomenon.

In addition to the sudden concentrated movements of a slope failure or landslide, movements may also occur more slowly due to natural forces in the form of creep. Creep is a slow or intermittent form of ground strain that is reflected in the tipping of fence posts or gently curving of trees, with the convex side pointing downhill in the direction of movement.

The mechanism of creep is not fully understood. On slopes in which the safety factor is low, the movement is probably true creep at stresses close to those producing shear failure. On flatter slopes, creep may be a result of alternate shrinking and swelling of the soil with seasonal changes in moisture coupled with the continuous downhill pull of gravity. Generally, creep is confined to the upper 15 to 20 feet of the soil or broken rock mass and is most rapid close to the ground surface.[10] Due to the slow rate of movement, the specific areas experiencing creep as well as the rate of movement may not be identifiable except through studies involving observations and instrumentations extending over several years.

The presence of creep is an indication of potential trouble because it suggests a quasi-equilibrium state than can be easily upset and turned into a landslide by man-made alterations such as a deep cut or a heavy fill or structure.

Frequency of Occurrence

Key factors associated with the majority of the landslides on Red Mountain included; steep slopes, shallow rock, colluvial soils, groundwater, shaley bedrock and human disturbance.

Since the study area was limited to a 4.5 square mile area along the north face of Red Mountain which has a sequential order of geologic formations and slopes which generally steepen with increasing elevation, the variability of areal characteristics was limited. Therefore, those characteristics recorded at each slide location maintained a degree of uniformity. Generally, the landslides occurred on preslide slopes steeper than 35 percent and some landslides, which encompassed relatively large areas, included ranges of slopes extending as low as 25 percent.

Rock outcrops were commonly exposed in landslide scarps or nearby features. Rock depths of less than 25 feet were common in slide-prone areas.

Two geologic formations, the Red Mountain and Chickamauga Formations, underlie most of the slides identified. These formations both contain shale interbedded with their principal constituents (sandstone and limestone, respectively). Generally, the landslides did not extend into the parent rock but rather into the material which weathers from the bedrock - the residual soil or colluvial soil overburden. Landslides were more common in colluvium derived from the upper, shaley formations. Although there were fewer landslides in the colluvium from the lower shale-free beds, a comparison of the effect of the shale in the colluivum was difficult because the average slope was less over the shale free formations.

Groundwater typically plays a major role in the stability of soil or rock slopes. An increase in the moisture content of the soil and/or the seepage of groundwater along joints or cracks in the soil or rock decreases soil and rock shear strength and increases weight. Either or both of these conditions may initiate or aggravate sliding. In addition, regrading or filling on a natural slope can block natural drainage paths in the soil or provide new entry points for water - both of which aggravate any tendency to slide. These factors together with wet weather springs observed along roadways, seepage observed coming from scarps of existing landslides and the historically higher incidence of landslides during periods of heavy rainfall, demonstrate that groundwater plays an important role in triggering slides in the study area.

Landslide frequencies were greatest just downslope of the steeper slopes in zones where colluvial buildup from old sliding, sloughing, and creep had occurred. Available subsurface data and site observations confirm the presence of a colluvial veneer near many slide locations. Most of the slides occurred above elevation 800 feet, MSL. Further analyses indicates that most movement projects along a bearing roughly perpendicular to the crest of the mountain and in the direction of steepest slope.

Causes of Landslides

Man-made disturbance was the most important contributing cause of landslides aside from steepness of slope and water. Virtually every landslide recorded could be readily traced to the effects of a cut, fill, drainage change or a combination of these. Toe cuts were the most common form of human disturbance which triggered landslides.

## Map Preparation and Uses

### Purpose of Maps

The purpose of the maps was to provide a guide to general areas within the study area with "high", "moderate" or "low" susceptibility to landsliding.

### Susceptibilty Categories

Areas designated as being highly susceptible to landsliding included those areas "exhibiting many of the characteristics of landslide prone areas and areas containing identified landslides". The characteristics signaling high susceptibility included bedrock of the Red Mountain and Chickamauga Geologic formations and topography with slopes greater than 30 percent.

Areas identified as having moderate susceptibility to landsliding have "some of the characteristics of landslide prone areas but lack evidence of landslides". Generally, areas in this category presently have slopes between 30 and 20 percent and are underlain by any of the formations present in the mountain.

Areas characterized as having low susceptibility to landslides, "exhibit no evidence of landsliding and lack most of the characteristics of areas where landslides have occurred." Such areas would typically have slopes less than 20 percent and are underlain by formations other than the Red Mountain or Chickamauga. Figure 3 shows a typical map.

Although susceptibility to landsliding in the later category is low, slopes could, of course, become unstable if proper excavation and construction practices are not followed. For example, low landslide susceptibility areas may develop landslides in steep cuts or poorly designed or constructed fills.

## Development of Categories

Evaluation of the characteristics of areas with high landslide frequencies enabled the selection of the few factors which were common to all or nearly all landslides identified. The absence or presence of these characteristics at locations throughout the study area was used to define areas with similar susceptibility to landsliding. The criteria used included; percent slope, geologic formation, elevation or position on the face of the mountain and evidence of creep in the general region.

Evaluation of specific portions of the study area for factors critical to landslide development was accomplished by first dividing the study area into 250 x 250 foot blocks (or grid cells) on topographic maps (scale 1"=100'). Each block as a whole was then rated for each factor in accordance with the average or dominate character. For example, an interpretation of the dominant slope was made within each grid cell and the entire cell was assigned that slope value.

It should be noted that no detailed site explorations were used in the study except where these data were available from previous jobs. In addition, much of the published geologic data is on small scale maps that cannot be accurately projected to larger scales.

After characterizing each block or grid cell and assigning a susceptibility category, the boundary lines were sketched between cells within different susceptibility categories. Because physical characteristics naturally follow landforms rather than arbitrary cell patterns, the final boundaries were smoothed out to follow topographic or geologic trends using the 250 x 250 foot grid cells as a guide.

## Use of Maps

The maps were developed as a reconnaissance and planning tool for City use only and were not intended to be site specific. Although the maps were based on the planimetric maps (scale 1"=200') provided by the City, the boundaries of the relative susceptibility categories could not be interpreted to the same degree of accuracy as the planimetric data. The limits of prudent interpretation were controlled by the accuracy to which it was possible to analyze the presence of the critical factors. This analysis required transferring data provided at much smaller scales and of a more general intended use onto a map of a much larger scale. In addition, the necessary simplification of area evaluation by the use of the grid cell method, specified the scale of the smallest feature analyzed and, therefore, the basis of boundary locations.

## Summary

The study found evidence that the colluvial and/or residual soil overburden in some areas of the north face of Red Mountain is slowly creeping downslope under the influence of gravity and groundwater. However, with the exception of these very slow creep movements, the study found that the mountain in its natural, undeveloped state does not contain significant areas that are actively undergoing landsliding. In almost all cases the landslides were a direct result of man-made slope modifications.

The study also found that steepness of the preslide slope (either man-made or natural) was the most critical factor in the occurrence of landslides. Most landslides developed where the slopes were steeper than 35 percent (about 3 Horizontal to 1 Vertical - 3(H):1(V)), and no landslides were identified on slopes flatter than 25 percent (about 4(H):1(V)). Other characteristics common to areas judged to be highly susceptible to landsliding included shallow rock, shaley bedrock and the presence of colluvial soil.

Based on the observations and findings, it was possible to subdivide the study area into areas of "low", "moderate" and "high" susceptibility to landsliding. The various susceptibility areas were presented on a series of maps. Areas that appeared to be disturbed by surface mining activity or which were thought to be underlain by mine shafts were also noted on the maps.

## Acknowledgements

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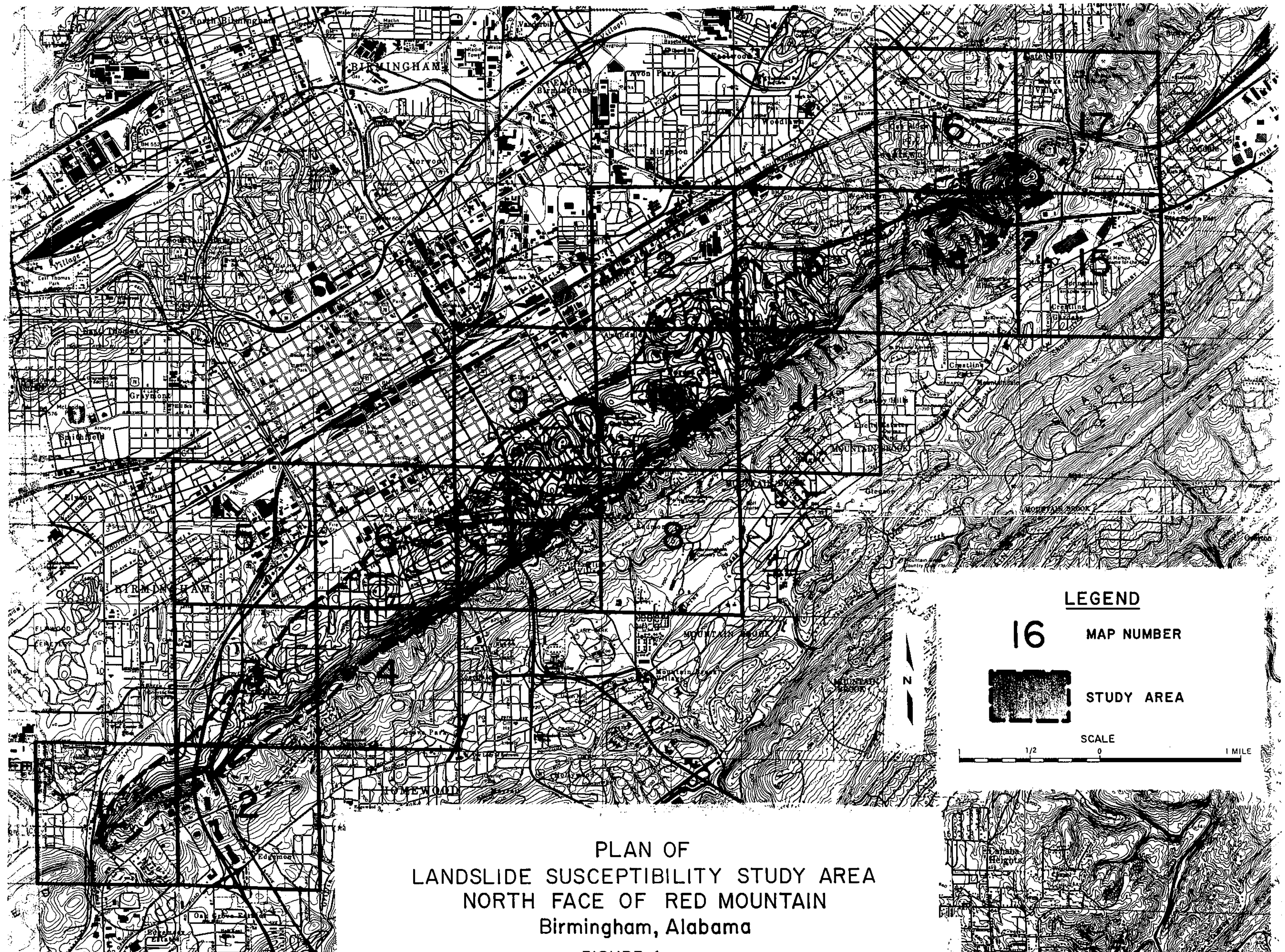
TABLE I

RED MOUNTAIN LANDSLIDE SUSCEPTIBILITY STUDY
INFORMATION SOURCE LISTING

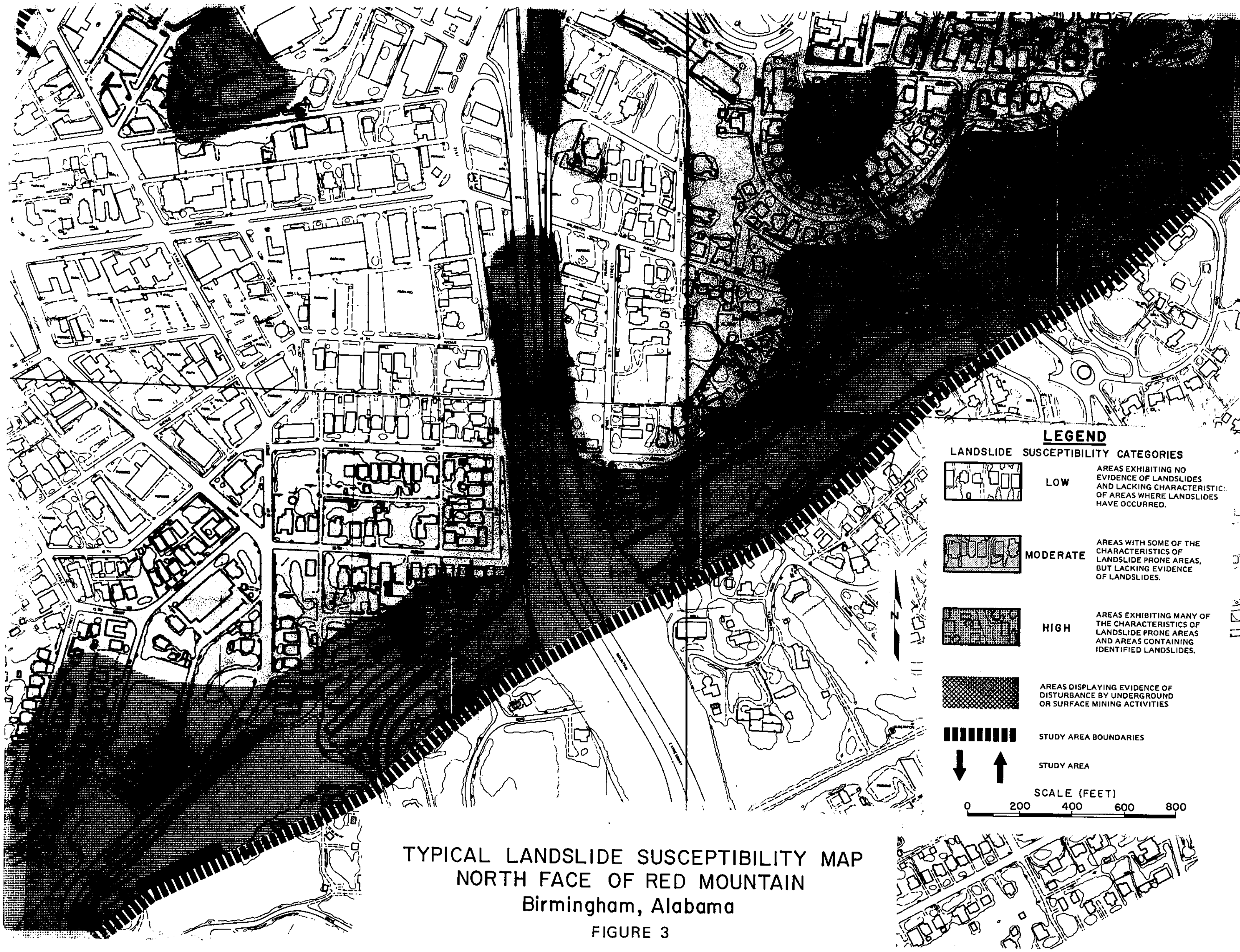
| Agency Contacted | Information |
|---|---|
| City Inspection Services | Yes |
| County Inspection Services | Yes |
| Planning & Zoning | Yes |
| Traffic Engineer | Yes |
| Water Works Board | No |
| Civil Defense | No |
| Streets & Sanitation Dept. (includes sewer) | No |
| Alabama Geological Society | Yes |
| Soil Conservation Service | Yes |
| Highway Department | No |
| Board of Industrial Relations (Mining Data) | Yes |
| Revenue Department | No |
| National Cartographic Information Center | Yes |
| Birmingham Library | No |
| Community Development | No |
| Birmingham News | No |
| Post Herald | No |
| Birmingham Eng. Dept. | Yes |

## TABLE II
## RED MOUNTAIN LANDSLIDE SUSCEPTIBILITY STUDY
## TYPICAL LANDSLIDE DATA

| SLIDE # | SLIDE TYPE | PRESLIDE SLOPE | SLOPE DIRECTION | AREA (ft.$^2$) | GEOLOGY | HUMAN DISTURBANCE | SOIL TYPE | REFERENCE | LOCATION | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | QUARTER SECTION | SECTION | TOWNSHIP | RANGE |
| 15 | Rotation | 50% Fill<br>30% Orig. | N 20W | 1,200 | Red Mtn. &<br>Chickamauga | Cut & Fill<br>Chickamauga | Bodine | Field Observ. | NE | 27 | 17S | 2W |
| 16 | Translation | 30% Orig.<br>75% Fill | N 10E | 10,000 | Red Mtn. | Fill | Bodine | Field Observ. | NE | 27 | 17S | 2W |
| 17 | Rotation | 30% Orig.<br>100% Fill | N 85W | 7,500 | Knox | Fill | Fullerton | Field Observ. | SE | 15 | 17S | 2W |
| 18 | Rotation | 100% Fill<br>40% Orig. | N | 400 | Chickamau | Cut & Fill | Bodine | Field Observ. | NE | 6 | 18S | 2W |
| 19 | Rotation | 70% | N40W | 100 | Knox | Cut | Urban | Field Observ. | NW | 32 | 17S | 2W |
| 20 | Rotation | 70% | N40W | 3,000 | Knox | Cut | Urban | Field Observ. | NW | 32 | 17S | 2W |
| 21 | Rotation | 35% | N40W | 1,200 | Chickamauga | Cut | Bodine | Field Observ. | NE | 6 | 18S | 2W |
| 22 | Rotation | 70% | N60W | 2,700 | Chickamauga | Fill & Cut | Bodine | Field Observ. | NE | 6 | 18S | 2W |
| 23 | Not Confirmed | 55% | N35W | 1,500 | Chickamauga | Cut | Bodine | Field Observ. | NE | 6 | 18S | 2W |
| 24 | Translation<br>(Creep) | 45% | ---- | 9,200,000 | Red Mtn &<br>Chickamauga | ---- | Bodine | Field Observ.<br>Field Observ. | -- | Various | 17S | 2W |
| 25 | Rotation | 40% | N60W | 10,000 | Chickamauga | Cut | Bodine | LETCO Job<br># B-2161 | NW | 5 | 18S | 2W |
| 26 | Rotation | 70% | N33W | 4,000 | Knox | Cut | Urban | LETCO Job<br># B-2387 | NW | 32 | 17S | 2W |
| 27 | Rotation | 75% | S | 6,000 | Fill/Red. Mtn. | Fill | ------ | Field Observ. | SE | 32 | 17S | 2W |
| 28 | Rotation | 50% | N10W | 900 | Chickamauga | Cut | Bodine | Field Observ. | SW | 6 | 18S | 2W |
| 29 | Translation | 120% | N35W | 600 | Chickamauga &<br>Red Mtn. | Cut | Bodine | Field Observ. | SW | 6 | 18S | 2W |
| 30 | Rotation | 55% | N65W | 40,000 | Chickamauga | Fill | Bodine | Field Observ. | SW | 6 | 18S | 2W |
| 31 | Rotation | 35% | N30W | 12,000 | Chickmauga | Cut & Fill | Bodine | LETCO Job<br># B-3145 | SE | 1 | 18S | 3W |




PLAN OF
LANDSLIDE SUSCEPTIBILITY STUDY AREA
NORTH FACE OF RED MOUNTAIN
Birmingham, Alabama
FIGURE 1




TYPICAL LANDSLIDE SUSCEPTIBILITY MAP
NORTH FACE OF RED MOUNTAIN
Birmingham, Alabama
FIGURE 3

# LANDSLIDES IN THE COLLUVIAL SOILS OF SOUTHWESTERN DAVIDSON COUNTY AND NORTHERN WILLIAMSON COUNTY, TENNESSEE

by

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Abstract. Blankets of colluvial soil cover major portions of hillside properties within southwestern Davidson and northern Williamson counties in Tennessee. In recent years, residential development around Nashville has encroached upon the colluvial slopes, and numerous landslides have occurred as a result of the instability of the colluvium, particularly during periods of unusually wet weather. Many of the landslides are believed to be ancient slides, reactivated as a result of modern development. This paper discusses the results of numerous investigations directed at establishing remedial treatments for these landslides. Laboratory-derived strength parameters for the colluvium are compared with strength data obtained by "back calculating" the shear strength for slopes which are known to have failed. These data point out some of the difficulties of estimating shear strength and stability based on the laboratory tests. Several examples of remedial treatments recommended are described and some of the basic problems associated with practical slope stability analysis and remedial construction for landslides in residential areas are discussed.

## Introduction

During the past ten to fifteen years, numerous landslides have occurred within the southwest portions of Davidson and northern Williamson Counties in Tennessee as residential and commercial development surrounding Nashville have extended in the southwest direction and have encroached upon colluvium-covered hillside properties. The colluvial soil in those areas has developed from the weathering and mass-wasting of argillaceous limestones and shales that cap the higher hills. Because of its method of development as a gravity deposited mass of loose soil and rock fragments, the colluvial soil typically possesses minimal stability.

The occurrence of the colluvial soils within that geologic setting is directly related to the geology, specifically the stratigraphy, of the area; therefore, its presence can be rather confidently predicted based on a review of available geologic data and from site inspections. Despite the apparent ease in identifying those areas where potentially unstable colluvium exists, construction has often proceeded indiscriminately onto the colluvial slopes. Presumably, while the potential for slope instability within areas of colluvial soil is understood by geotechnical engineers and geologists, house builders and developers often fail to recognize the difficulties and inherent risks associated with construction within that setting. There are several reasons why builders have been slow to recognize the potential for landslides within the colluvial slopes. Among these are:

1) Nashville has not had a long history of slope instability problems. Until residential development extended onto the colluvial slopes, most of the construction in the Nashville area was in more level terrain where the soils typically

consist of stiff, residual clays and where subgrade conditions are generally competent. Therefore, contractors and builders are not accustomed to working with these adverse soil conditions.

2) There are no grading code restrictions that require contractors and builders to investigate hillside properties prior to development.

3) Geotechnical studies or professional consultation are generally not provided for the residential type of construction that has experienced a majority of the landslide problems.

4) The nature of the colluvial soil is such that, if construction is done in a dry season of the year, the soil may appear stable and competent. Any problem with slope instability may not be apparent until months, or years, following construction, when the property is subjected to heavy precipitation or other changes take place in subsurface drainage characteristics, such as the addition of a septic tank and absorption field.

Although it appears that home builders and developers are gradually becoming more cognizant of the risks associated with building on colluvial slopes, still, every year, and particularly during periods of above-average precipitation, new landslides are reported, most of them involving new construction in areas that have not been investigated prior to development. The number of problems associated with unstable colluvial slopes could be greatly reduced if both home builders and buyers were better informed of the risks. Also, because construction will inevitably continue within that geologic setting, engineers need to develop an improved understanding of the properties of the colluvial soil and of those factors which influence the stability of the colluvial slopes. Such are the purposes of this presentation.

## Geologic Setting

The City of Nashville is located near the western edge of a broad topographic depression which is referred to as the Central Basin of Tennessee. From a structural geology viewpoint, the Central Basin is a breached anticlinal dome (The Nashville Dome), which is the southern extension of a broad regional upwarp known as the the Cincinnati Arch. The Nashville Dome/Cincinnati Arch extends through South-Central Ohio, Central Kentucky, and Middle Tennessee. Historical geologists generally agree that this regional structure began to form during Middle Ordovician time and that various transgressions and regressions of those and subsequent seas are in part responsible for the variations in the types of sedimentary rocks that comprise the local stratigraphic section. It is evident that the Nashville Dome has maintained its structural character since Ordovician time. Today the structure is revealed by the outcrop pattern of the Paleozoic sedimentary rocks. The Nashville Basin physiographic province coincides with the central area of the dome where Middle Ordovician rocks have been exposed by erosion. Surrounding the Nashville Basin is the Highland Rim physiographic province. The Highland Rim coincides with the outer flanks of the Dome where younger strata are preserved and where relatively resistant silicious limestones of Mississippian age cap higher hills and plateaus. Figure 1 illustrates a typical cross-section through the Nashville Basin and Western Highland Rim. Also, a typical stratigraphic section is illustrated to show the various bedrock units that occur within the study area.

## The Development of Colluvial Slopes

The colluvium results from the weathering and mass-wasting of the argillaceous limestones, calcareous shales, and silicious limestones that crop out along the escarpments of the Highland Rim and its outlying, erosional remnant hills. The occurrence and distribution of the colluvial soils are closely related to the stratigraphy of the area, and the distribution of the colluvium is predictable based on an understanding of the areal geology and topography. Basically, there is more likely to be significant amounts of colluvium on those slopes where the thicker sections of calcareous shales and argillaceous limestones are preserved. Miller and White, 1975, and Weber and Wilson, 1983, discuss the relationship between the geology and the distribution of the colluvial soils within the study area.[1,2]

Close observations and subsurface explorations of the colluvial slopes have led the writers to suspect that many of the recent landslides are the result of a reactivation of ancient landslides which formerly occurred during natural processes of slope development. The presence of hummocky topography, indicating that the soil has been ruffled and disturbed, as well as apparent healed scarps and cirque-like indentations in the upper portions of the colluvial slopes, suggest that landslides have occurred in the past, even in areas where development has not occurred. Also, subsurface studies typically encounter shear zones within the subsurface soils. These zones usually are in the form of thin layers of soft, light-gray or greenish-gray clay. Also, some display slickensides indicating that movement of the colluvium has occurred. These features suggest that, even in their natural state, colluvial slopes often go through periods of reduced stability,

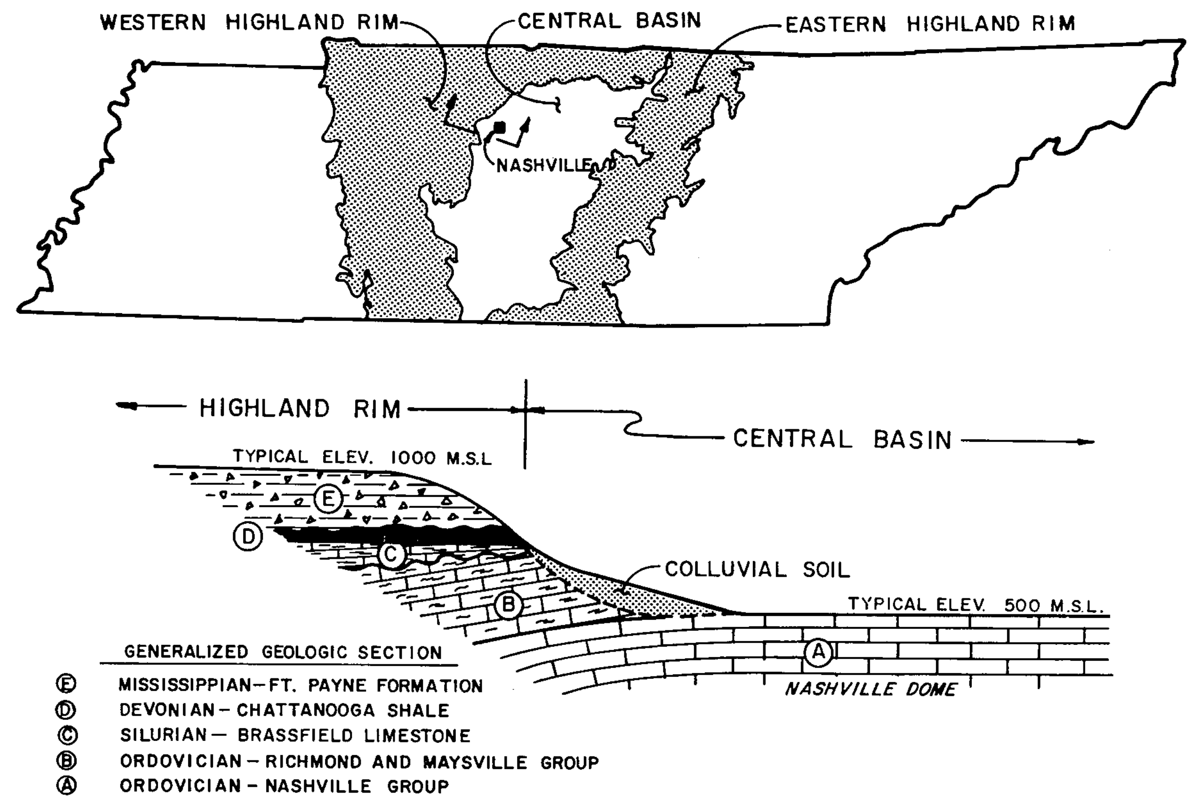


FIGURE 1. LOCATION MAP AND GEOLOGIC CROSS-SECTION THROUGH STUDY AREA

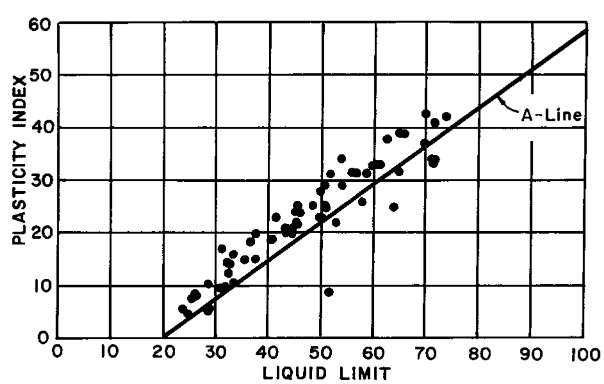


FIGURE 2. ATTERBERG LIMITS FOR COLLUVIAL SOILS

during which time landslides or accelerated soil creep occurs.

Testing and Analysis

The colluvium developed from the argillaceous and siliceous rocks of the Highland Rim can be generally described as a light brown, silty clay containing varying amounts of small, angular, shaly, rock fragments. Numerous in-house investigations directed at evaluating landslides within the colluvial soils have

| SITE | % MOISTURE CONTENT | LIQUID LIMIT | PLAS-TICITY INDEX | UNCONFINED COMPRESSIVE STRENGTH PSF | DRY UNIT WEIGHT PCF |
|---|---|---|---|---|---|
| B | 35.5 | 70 | 43 | 1,650 | 95.1 |
| | 26.2 | 45 | 24 | 2,300 | 91.4 |
| C | 22.8 | 50 | 22 | 2,970 | 101.2 |
| | 28.8 | - | - | 1,720 | 95.8 |
| | 21.3 | - | - | 6,610 | 109.0 |
| D | 25.0 | 60 | 33 | 9,200 | 95.7 |
| E | 17.9 | - | - | 2,700 | 114.1 |
| F | 27.7 | 46 | 25 | 9,900 | 95.0 |
| | 20.5 | 44 | 21 | 8,600 | 107.4 |
| | 25.0 | 63 | 38 | 7,000 | 99.8 |
| G | 24.9 | - | - | 7,100 | 99.6 |
| | 28.2 | 65 | 32 | 8,200 | 97.0 |
| J | 29.9 | 72 | 41 | 4,400 | 93.9 |
| | 24.5 | 66 | 39 | 7,200 | 101.8 |
| K | 17.0 | 51 | 25 | 3,900 | 109.0 |
| N | 35.0 | 64 | 25 | 6,000 | 84.7 |
| | 31.0 | - | - | 9,100 | 89.0 |
| | 30.6 | - | - | 2,100 | 90.1 |
| O | 17.5 | 51 | 25 | 3,900 | 109.0 |
| P | 16.9 | 26 | 8 | 1,700 | 113.2 |
| | 19.5 | 24 | 6 | 2,000 | 115.7 |
| | 31.3 | 53 | 31 | 5,300 | 100.3 |
| | 20.9 | 38 | 20 | 4,300 | 106.3 |
| | 24.4 | 53 | 22 | 1,400 | 99.1 |
| | 26.8 | 55 | 30 | 6,400 | - |
| Q | 29.3 | - | - | 2,400 | 96.3 |

TABLE I. TEST RESULTS FOR COLLUVIAL SOILS yielded a considerable amount of data which serves to illustrate the highly variable characteristics of the colluvial soils. Figure 2 illustrates a plot of seventy-six Atterberg limits determined from representative samples gathered throughout the study area. The diagram shows that the colluvial soil covers a broad range of plasticity but the majority of the data points plot along a narrow band following the "A" line on a plasticity chart. Table I illustrates the equally wide range in strength characteristics determined by unconfined compressive strength tests and compares those strengths with moisture contents and unit weights. As can be seen from the data, the strength of the colluvial soil is highly variable and, though the strength of the colluvium is known to be greatly influenced by moisture content, such trends are not readily discernible from the results of these laboratory tests. Fewer samples have been subjected to triaxial tests, but the majority of the triaxial tests have been unconsolidated- undrained tests without pore pressure measurements. These tests are not well suited for evaluating effective stress parameters and long-term states of stability. More recently, samples of the colluvial soil have been subjected to consolidated-undrained triaxial tests with pore pressure measurements being made in an effort to better evaluate the effective-stress strength characteristics of the colluvium.

In addition to performing unconfined and triaxial shear tests on undisturbed samples, strength parameters for the colluvial soils have been calculated for slopes which are known to have failed by using a back calculation process. This approach allows for various trials under a range of combinations of cohesion and angles of internal friction for the colluvial soil. Provided that the geometry of the sliding mass is well defined, the calculations can be used to determine what the strength of the soil has to be for the slope to have a safety factor of unity. Generally, these back calculation trials show that, in order for the slopes to exist in an unstable condition, the colluvial soil must have a very low strength, much lower than the results of most unconfined and triaxial strength tests performed on the colluvium.

To provide better insight into the variations in soil strengths determined by the various methods of assessment, it is possible to compare the results of unconfined compressive strength tests and quick triaxial tests with strengths determined by back calculations for a given site, and then also compare the back-calculated strength with results obtained by consolidated-undrained triaxial tests with pore pressure measurements. One such example involves data obtained for a landslide which occurred in the spring of 1983 following a period of extended wet weather. The slope failure was located in the front lawn of a residence in northern Williamson County and included an area measuring approximately 200 feet across and about 250 feet from crest to toe. A cross section of the slope and the results of field tests performed for the exploration are shown in Figure 3. Soon after first symptoms of a major slope failure became evident, the owner contracted for engineering services to examine the situation and provide recommendations for remedial treatment. Subsurface exploration was conducted and those tests included Standard Penetration tests and sampling with thin-wall Shelby tubes. Borings were extended to refusal and water-level measurements were recorded. The drilling and sampling were performed as the slope was continuing to move at a very slow rate. The width of tension cracks at the crest of the slope increased at a rate of about a one-fourth of an inch per day. Upon completion of the drilling, some tension cracks and scarps of up to a foot or more in height marked the upper limits of the slope failure and large bulges and heaving of soil was evident at the toe of the slope. Selected undisturbed samples of the colluvium were tested for unconfined compressive strength, unit weight and Atterberg limits. Based on the visual classifications of the samples, as well as a general change in the consistency of the soil as observed by variations in N values, the limits of the sliding mass were defined as shown on the Profile.

Back calculations were performed for the failed slope using a sliding wedge analysis. Several trials were performed using an assumed cohesion of zero and with various angles of internal friction, ranging from 12° to 23°. A plot of friction angle versus factor of safety for the slope is shown in Figure 4. These data indicate that, for the slope to exist with a safety factor of 1, strength parameters for the soil must be about $\emptyset = 16°$ and $C = 0$. A similar analysis using a $\emptyset = 0°$ indicates that for the slope to have a safety factor of less than 1, the cohesion can not exceed approximately 250 PSF. These low values for soil strength are interesting in the light of the results of the unconfined compressive strength tests which yielded unconfined strengths of 2.2 KSF, 2.0 KSF and 3.2 KSF. Also, the results of the Standard Penetration tests, as shown in Figure 3, generally would be considered indicative of soil having a higher strength than that shown by the back calculation trials. The type of laboratory test that compares most favorably with the back calculated strength data is the unconsolidated undrained triaxial corrected for pore pressures. One such triaxial test for a representative sample of colluvial soil indicated a $\emptyset$ of 24° and $C = 0$.

A major difficulty in assessing the stability of the colluvial slopes resides in the inability to sample and test the soil strength along the slickensided shear zones that occur within the sliding mass of soil. These well-established shear

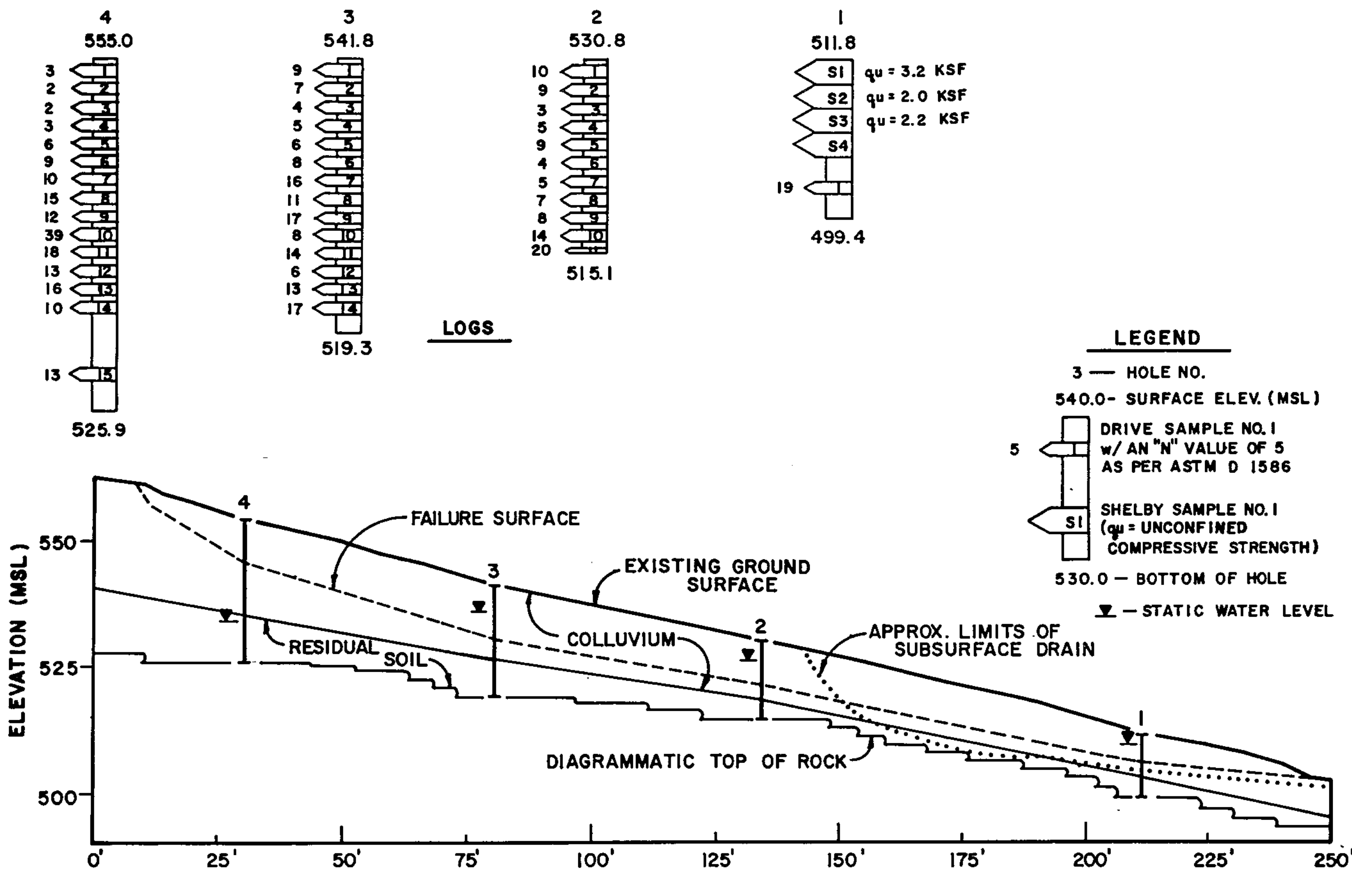


FIGURE 3. PROFILE OF LANDSLIDE ON COLLUVIAL SOIL SLOPE AND LOGS OF EXPLORATORY HOLES

zones presumably have strength characteristics that more nearly coincide with residual strengths rather than peak strengths for the soils. Further, the nature of the clays in which the shearing surfaces occur may be considerably different than those of the overall soil

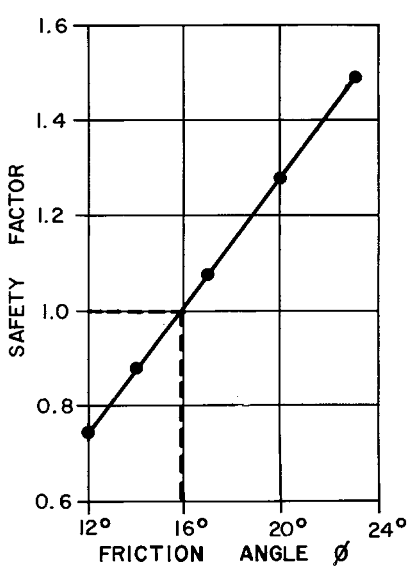


FIGURE 4. FRICTION ANGLE VS. SAFETY FACTOR FOR LANDSLIDE EXAMPLE ASSUMING ZERO COHESION

mass. Consequently, samples obtained at random within the colluvial soil may not have the same properties as the soil within the narrow zone where the failure surface exists. Because the low strengths obtained by the back calculation process more typically characterize the residual strength of the colluvial soils within the shear zone, rather than the peak strengths of the soils comprising the majority of the soil profile, the back calculation process appears to be the best approach at assessing the soil strength along the established shear zones, provided, of course, that the geometry of the slope is well-defined. Unconfined compressive strength tests and triaxial strength tests for soils comprising the general mass of colluvium are useful in establishing general levels of soil integrity but will likely not provide reliable data for defining the soil strength that governs slope stability. The results of field tests, such as N values obtained during standard penetration testing, could be very misleading because they are a reflection of the overall integrity and consistency of the colluvial soil rather than of the strength characteristics along failure surfaces. The standard penetration test is useful, however, as a means of comparing relative soil strengths within a slope and to provide samples for visual assessment, laboratory classification tests, and moisture content determinations. In essence, any method of assessing soil strength and stability for the colluvial slopes which fails to take into account the presence of pre-existing shear zones and failure surfaces of reduced strength is likely to grossly overestimate the long-term state of stability for the slope.

Equally important in the assessment of slope stability for the colluvium is the recognition of the variation in soil strength dependent upon the degree of saturation and the level of any phreatic surfaces developed within the soil mass. Typically, the colluvial soils are relatively porous and permeable and, because they occur at the base of hillsides, they receive considerable runoff from adjacent hills. The soils are easily saturated by both surface water and migrating shallow ground water. Following periods of extended wet weather and heavy precipitation, ground water levels can rise within the soil, saturating shear zones and softening and weakening the soil. Also, increased water content of the soil causes the overburden to be heavier, which can increase the driving force for the landslide. Consequently, slopes that may be stable under dry or average moisture conditions can become unstable during periods of greater soil saturation and higher water levels within the slope.

For the landslide example described previously, a stability calculation was performed using chosen strength parameters, which resulted in a safety factor of .99 with the water levels as shown by the boring logs in Figure 3. Upon lowering the water levels to below the base of the failure surface and recalculating the stability using the same strength data, the reduction in water levels resulted in an increase in safety factor to 1.09, or an increase of about 0.1. Although this increase in stability is minimal, it is significant because it reflects only the influence of the water table upon the stability of the slope and allows for no change in strength characteristics of the soil. In actuality, if sufficient drainage can be provided to prevent water levels from rising to any critical level within the colluvium, we expect that the soil strength along the shear zones can be preserved at a higher level, which will result in an even greater degree of stability.

## Remedial Treatments

Remedial treatments for the colluvial slope landslides have centered around three principal approaches with the recommendations often relying upon one or more of these approaches to improve the state of stability. The three basic approaches are:

1) Improving surface and subsurface drainage in order to minimize the degree of saturation of the colluvial soil, thus preserving as great a soil strength as possible and limiting the driving forces from heavy, saturated soils.

2) Redistributing driving and resisting forces. This approach includes the construction of counterweight berms at the toe of the landslide or the removal of soil or other surcharge influences from the crest of the slope.

3) Increasing the shear strength resistance along the failure surface by replacing the unstable colluvial soil with higher strength materials or by interrupting and destroying slick surfaces within the slide area. Practical applications employing this technique include construction of shear keys within the slope and/or the complete removal and replacement of colluvial soil with compacted, engineered fill.

Most all remedial treatments rely on one or more of the above approaches. For the example given in a preceding section of the paper and shown in Figure 3, that slope was treated by constructing subsurface drains within the lower third of the slope. The drains extended below the base of the sliding colluvial soil and were constructed by backfilling trenches to near finished grade with free-draining, three-quarter-inch, sized crushed stone. Before backfilling, the sides of the trenches were covered with filter fabric to prevent contamination of the crushed stone with clay. In addition, a four-inch diameter perforated plastic pipe was embedded at the base of each drain in order to promote rapid drainage. Each drain was daylighted at the toe of the slope, and the site drainage was improved at the base of the slope to make sure that no surface water ponded near the toe of the slide. To prevent further infiltration of surface water into the slope, all tension cracks and depressions within the area surrounding the landslide were sealed with compacted clay. Also, roof drains from the residence were piped to the base of the slope, rather allowed to discharge at the crest of the landslide area. Drainage ditches extending across the landslide were arranged to facilitate more rapid runoff and were lined with a mixture of soil and cement in order to prevent infiltration of surface water through the bottom of the drainage ditch. Incidentally, this treatment provided a very durable trench bottom that resisted erosion on the steep slope and was considerably more economical than alternative designs, such as a concrete or asphalt-lined drainage ditch. A minor amount of fill was removed from the crest of the slope and redistributed at the base of the hillside and, although this amounted to a removal of only about two feet of soil over a small area, it was accomplished with minimal effort and expense and further served to improve surface drainage around the crest of the slope.

The improvement in the stability of the slope following these remedial treatments relies primarily on the effects of the toe drains to minimize the level of saturation within the colluvial soil. Also, overall shearing resistance of the slide is increased because the drains also serve as shear keys which interrupt the failure surface and provide a higher strength medium in the form of crushed stone.

Such remedial treatments are typical of those specified for most of the landslides and they are usually quite effective. On some landslides, an abrupt ceasing of movement has been noticed as toe drains were excavated into the unstable mass of soil. Furthermore, for some slopes, exploratory trenches have been excavated in the soil and then simply backfilled with the same soil, with no further movement occurring in the landslide. Presumably, even this minor improvement in drainage and the disruption of failure surfaces by excavation and replacement of remolded soil has been sufficient to increase the safety of the slope to something slightly greater than 1.

Any design for remedial treatment of these landslides must take into account numerous factors, such as accessibility of the slope with construction equipment, the size of the area, the economics of the proposed remedial construction, and the level of risk that the client is willing to accept. Obviously, a landslide which threatens the integrity of a structure must be treated with more deference than a slope failure located in a lawn, where only the landscaping is threatened. Moreover, because most of the landslides involve residential structures, with homeowners as clients, seldom are those homeowners able or willing to fund the detailed geotechnical investigations and extensive remedial treatments that would be necessary to redesign the slope and construct it to a high level of stability consistent with normal engineering standards. Therefore, there are many compromises which must be made and, in the end, the owner must be willing to accept only the best information and the amount of remedial construction that his money is able to purchase. More often than not, these landslides are treated with the idea of improving the state of stability sufficient to provide only a marginal degree of assurance against future movement. Also, because of the high costs associated with detailed, comprehensive geotechnical investigations, it becomes important to be able to utilize data and experience gathered from studying numerous, similar slope failures and from evaluating the effectiveness of various types of remedial treatments, in order to best serve these clients with minimal expense and realistic plans for relatively inexpensive, effective remedial treatments.


## Acknowledgements

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# A CASE STUDY OF SLOPE STABILITY IN NEW PROVIDENCE SHALE

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Abstract. A landslide in south central Jefferson County, Kentucky, on the New Providence shale caused widespread disruption in a subdivision area, including destruction of three homes. Field measurements and observations were used to determine the physical extent of the landslide. In-situ vane shear tests were performed to aid in the identification of the failure surface. Piezometric levels were monitored in several borings at the site of the slide. Laboratory direct shear tests were performed to obtain drained shear strength perameters. Historical information was collected to establish the time sequence of the events leading to the slide. A stability analysis based on the wedge method was utilized to evaluate the stability of the slope. For the failure surface identified from field tests, the drained shear strength parameters obtained in the laboratory indicated a factor of safety of one for the natural slope prior to subdivision construction, for saturated conditions. Subdivision construction included redirection of drainage which increased flow into the slide area; increased porewater pressures in the soil triggered the slope failure.

## Introduction

The landslide which is described in the following paragraphs occurred on the west side of Manslick Road, south of Gagel Avenue in Jefferson County, Kentucky. The site is underlain by members of the Mississippi Borden Formation, particularly the New Providence shale which forms most of the slope from elevation 580 to elevation 650, and the Kenwood siltstone which forms the top of the hill, from elevation 650 to elevation 670. The landslide occurred on the edge of the Knobs Physiographic Province. The New Providence shale is a clay shale with very minor limestone lenses. In the unweathered condition, the shale is olive-gray to grayish-green and weathers to light greenish-gray to yellowish-gray. Ironstone concretions are abundant along bedding planes in the upper 70 feet of the unit. The New Providence shale is silty, micaceous and illitic, and becomes plastic when wet.[1] The Kenwood Siltstone consists of interbedded layers of siltstone and shale. The siltstone forms the crest of the hill on which the slide occurred. The siltstone is the parent material in weathered, residual form for the hillside soil near the crest of the slope which failed.

Most of the hillside involved in the slide is covered with residual soil derived by weathering of the New Providence shale. Weathering occurs in two ways: first, erosional removal of load is responsible for the formation of stress relief joints in the shale. The action of water in the opened joints gradually breaks the shale into fragments, with the size of the fragments depending upon the degree of bonding in the shale. Weathering in the shale also takes place as a result of seasonal variations of moisture content and temperature, as well as the action of roots and organisms. Generally, there is a gradual transition between the weathered top layer of soil and the jointed shale; additionally, the transition to hard shale is not abrupt.[2] This gradual transition in material properties complicated the analysis of this slide because it was not possible to assume, with any degree of certainty, that the failure plane in the slope was located at the boundary between residual soil and jointed rock. Rather, it was necessary to locate this surface in the field between the residual soil and the unweathered shale.

## Sequence of Events

The subdivision where this slide occurred was developed in stages beginning in the 1960's and ending in 1975. To obtain information on the pre-construction conditions at the site, aerial photographs were obtained and analyzed. An April 1963 stereopair showed the slide area in undeveloped state, with wooded slopes and a single foot path or bike path along the crest of the hill. The hillsides were covered with mature trees with the exception of an area approximately 100 feet square on the south of the hill. This area was much more exposed and bare of vegetation than any of the immediately surrounding hillside areas. This area coincided exactly with the location of the subsequent landslide. Because of the existence of this bare area in the 1963 photograph, it was distinctly possible that this slope had experienced signifi-

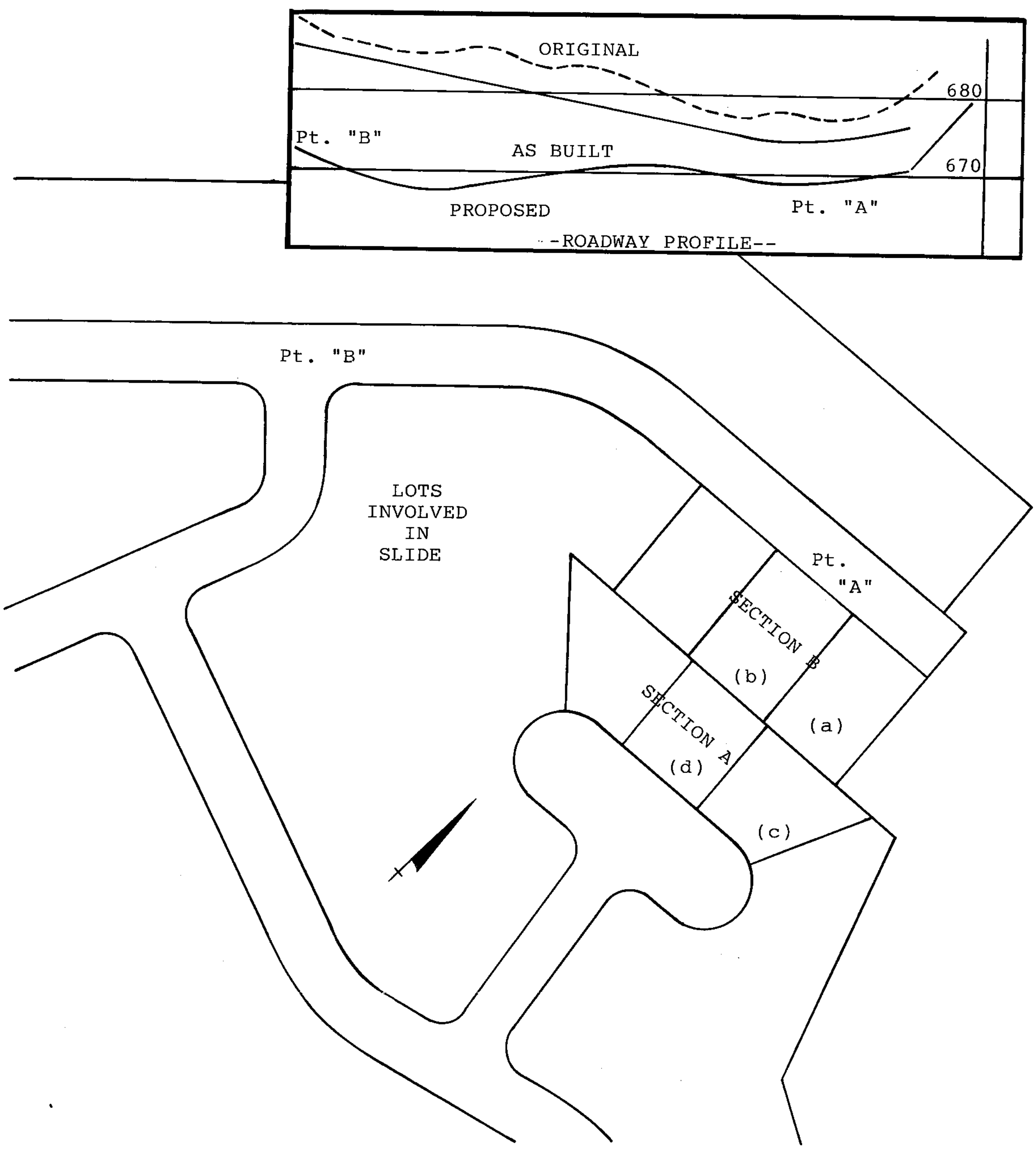



FIG. 1. Location Map and Profile, Slide Area.

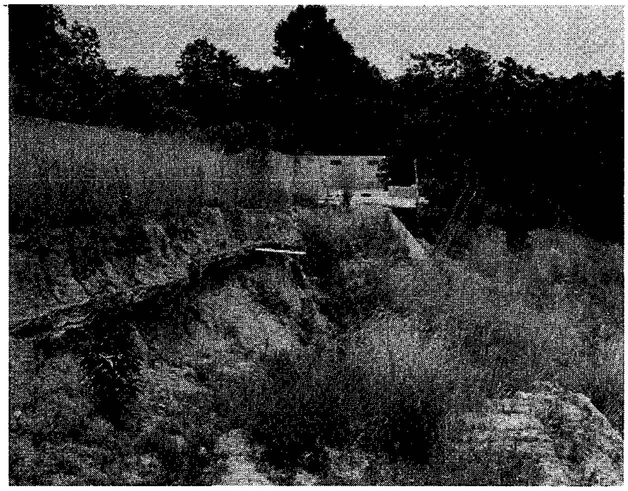


FIG. 2. Scarp of Landslide, Demolished House.

cant distress before that date.

Plans for the subdivision construction included a preliminary survey prepared in 1967. These plans included topographic maps with 2-feet contour lines, which could be used in later analysis. Additionally, proposed drainage provisions also appeared on these plans. Of particular interest in this investigation was the placement of two 15-inch-diameter culverts at low points along the crest of the hill where a road was to be placed in the subdivision development.

The subdivision development was built in stages. First, streets were constructed and utility services were installed. Then lots were laid out and each property was sold to a builder. The builders were responsible for house construction and lot landscaping. In the section where the landslide occurred, construction was completed in 1971. Steep slopes flank the section of subdivision where the slide

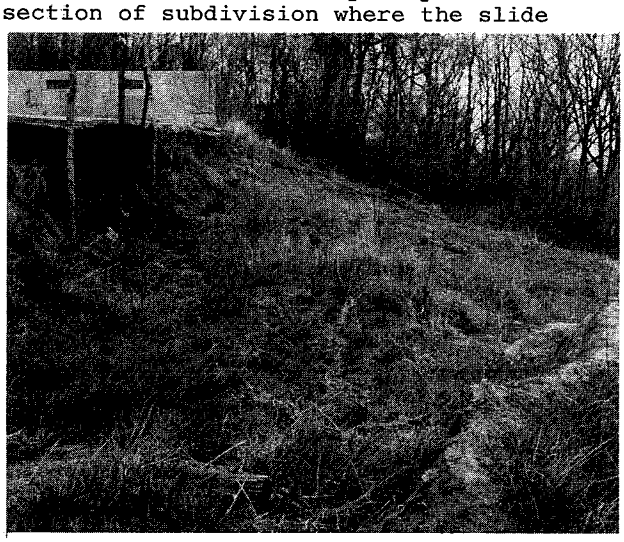


FIG. 3. Retreat of Scarp Under Demolished House.

occurred and the hill crests rise 60 to 80 feet above the bases of the hillsides, over horizontal distances of 250 to 300 feet.(See Fig. 1) All of the homes in Section A of the subdivision were occupied by the end of 1972. However, no development had taken place on the hillside above this section of the subdivision. Homeowners at the bottom portion of the slope which later moved, reported that the slope extending uphill from the rear of their homes was clear of trees when they moved in during August 1972. This clear area was identical to the open area seen in the aerial photographs taken in 1963. No problems with the hillside had been observed by the homeowners when contruction of an adjacent Section B of the subdivision, farther up the slope, began.

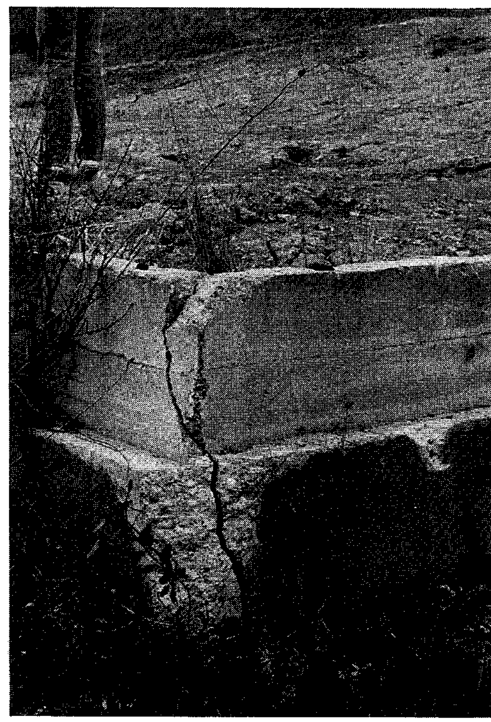


FIG. 4. Foundation of House at Toe of Slope Cracked from Lateral Earth Pressure.

Plans for this upslope section of the subdivision included the roadway along the crest of the hillside. The proposed profile on this roadway originally included two low areas separated by a high, with as much as 12 to 14 feet of excavation at the upslope end of the profile to the west of the slide area, where one of the low points in the profile was located. In addition to the two 15-inch-diameter concrete culverts, the plans also called for unpaved ditches along the roadway alignment to carry surface runoff to the culverts. A survey of the actual as-built profile of the roadway disclosed a very different profile.(See Fig. 1) The as-built road had a continuous downhill grade from its western end to the easternmost depression in the roadway, located immediately above the area of the landslide. Additionally, further to the east for approximately 150 feet, the roadway slope increased upward to the end of the ridge line. Approximately 1,200 feet of roadway thus drained to a single 15-inch culvert located immediately upslope from the bare area seen in the 1963 photographs. Further investigation located a set of revised construction plans that showed the same centerline profile which was measured at the time of the stability investigation.

Construction in this upslope section of the subdivision was begun in 1974. Clearing for the roadway, which ran along the crest of the hill north of the landslide location, was begun in the summer of 1974. The subgrade for the roadway was inspected by a county official in July 1974 and was accepted, except for the area immediately upslope from the landslide location where the subgrade was considered "soft". Later in the summer of 1974, all of the roadway was approved for placement of dense graded aggregate, and in October 1974, the roads in this section of the subdivision were paved with concrete. Three houses in this section of the subdivision were affected by the later landslide. Two of these houses were constructed between spring and autumn, 1976. The third house was not constructed until the summer of 1978. By May of 1979, two of the houses (a and b in Fig. 1) in the upslope section of the subdivision had been condemned and demolished (See Figs. 2 and 3), and one house at the base of the slope in the lower section of the subdivision had also been condemned and demolished (c in Fig. 1; see also Fig. 4). A second house on an adjacent lot at the base of the hillside had also been severely damaged by May of 1979 (d in Fig. 1). Following the occurrence of the landslide, further development of the subdivision was suspended.

Although the aerial photographs suggest the possibility of slope movement as early as 1963, the first reported observation of movement in the slide area was in March 1975 behind one of the downslope lots. The house on this lot was a 1-story structure with a basement, and in the rear, a small patio extended 10 feet toward the hillside. From the end of this patio, the ground surface had a continuous uphill grade to the lots adjacent to the roadway along the crest of the ridge. Following a rainstorm in March 1975, the owners on the downslope lot reported what they described as "mud sliding downhill" toward their home. During this period of movement, survey stakes on lots uphill from their home moved as much as 12 feet downslope. By the time the movement ended, the patio behind this home was buried and soil was piled 12 inches deep against the rear foundation of the house. The residents waited for the "hillside to dry out" and then removed 16 truckloads of soil from behind their house between June and July of 1975.

The hillside then was inactive for a period of almost two years. In March of 1978, while the slope was covered with snow, residents at the top of the hill noticed cracks in the ground behind their house and observed that the ground was sinking in spots. After the snow melted, they were able to observe that the ground surface adjacent to their foundation had dropped two to three feet, and that soil again covered the patio of the home at the base of the hill.

In May 1978, the owners of the two lots a and c undertook a joint effort to restore their property. Fill was imported and a bulldozer was used to grade the slope into two terraces. The owners of the upslope house reported that their yard fell almost immediately when the next rains began, and soil again covered almost one half of the patio behind the downslope house. By June of 1978, the slide had expanded laterally westward to include two additional lots and homes. The owners of the four affected lots quickly combined their efforts and again regraded the hillside into a continuous slope. They sodded the hillside and attempted to provide drainage by installing plastic pipes on the surface in gullies and wherever water was seen to collect. Shortly afterward, cracks again were observed on the hillside. During June and July of 1978, the ground behind the upslope house most seriously affected continued to drop, eventually sinking more than four feet down along the foundation of the house. One resident reported observing 15 feet of lateral movement downslope during this episode. At this time and during the fall of 1978, the residents described the appearance of the hillside as undulating and hummocky, with occasional movements measured within the space of hours.

No movements were observed during the winter of 1978 to 1979, but renewed movement became apparent in early March 1979. A "wave" of soil one to two feet thick moved down the slope and pushed against the two houses on the lots at the base of the hillside. Cracks opened up in the basement walls of the home at the top of the hill. In the final days of March 1979, the slope experienced large movements, and according to one of the residents, "...within a week, the entire hillside came loose." Soil pushed against the base of the house immediately below the landslide and cracked the foundation, shoving the house forward off the foundation. At the top of the slope, the slide scarp was 10 to 12 feet high and coincident with the rear walls of the two homes closest to the slide area at the top of the slope. At the base of the slope, a second house was damaged by soil pressure against the basement wall of the home, and trees on the western portion of the slide area were displaced. In April 1979, the two houses a and b at the top of the hillside above the slide area, and the house c

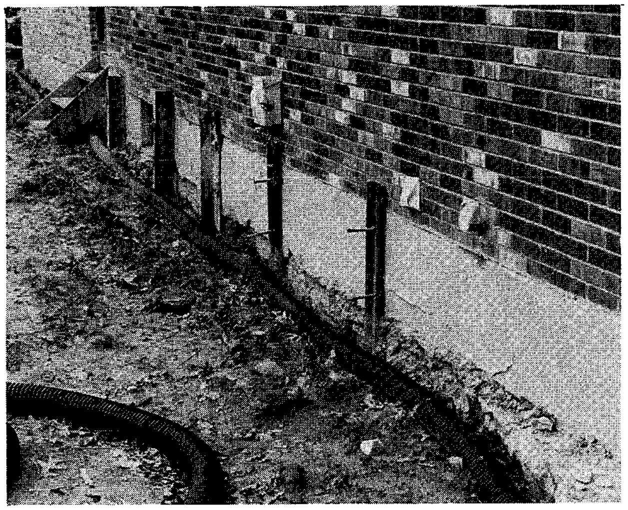


FIG. 5. Homeowner's Remedial Measures at Toe of Slide Area.

immediately below the slide, were condemned and demolished. Shortly afterward, the County Department of Public Works installed drainage ditches on the slope, but slope movements continued to occur.

Two years later, in fall of 1981, the owners of the remaining house at the base of the hillside began additional remedial efforts. A layer of soil one to two feet thick which had moved against the rear wall of their house was excavated. The owners built a retaining wall consisting of railroad ties placed horizontally approximately 10 feet from the rear wall of the house. A ditch to catch surface runoff from the hillside was constructed at the north end of the lot about midway up the hillside and was extended down along the western limits of the lot. In March of 1982, the railroad tie wall was distorted and pushed over, and the house began to experience structural distress. During early 1982, the rear basement wall of the house began to crack and displace inward, and water entering through the cracks flooded the basement. The owner then jacked the wall back into position, strengthened the wall using horizontal and vertical steel channel sections bolted together (on both sides of the wall, See Fig. 5), and sealed the wall with fiberglass to prevent leakage. Ownership of this house changed in fall of 1983. The new owner removed the remains of the railroad tie wall which had been pushed up against parts of the rear wall of the house by this time, and excavated the two to three feet of soil that had pressed up against the back of the house. Broken and damaged studs in the rear wall of the house were replaced, as was the warped and cracked brick veneer. The owner then decided to install a retaining wall of his own design. Using a ditch excavator, the owner cut a four foot deep trench all along the rear of the house, 10 feet from the rear wall (See Fig. 6). As the trench was being advanced, the upslope wall of the cut was seen to move downslope. For a period of time, workers stood in the trench with shovels trimming the advancing wall in order to maintain the desired width. This effort was abandoned and the trench was allowed to close on itself.

Continued observations on the site showed that the area of instability gradually extended to the west and began to affect a third house at the top of the slope and a third house at the bottom of the hillside (See Fig. 7). At the top of the slope, the scarp continued to extend westward and the ground surface began to drop behind the house located there. At the base of the hillside, heaving of a driveway pavement indicated that the south limit of the slide had progressed down to the level of the house located at the base of the hillside. Movements slowed during 1984-1986, but did not cease.

## Preliminary Analysis

It has long been known that one of the principal controlling factors in hillside instability is water.[3] Water adds weight to the sliding mass, while

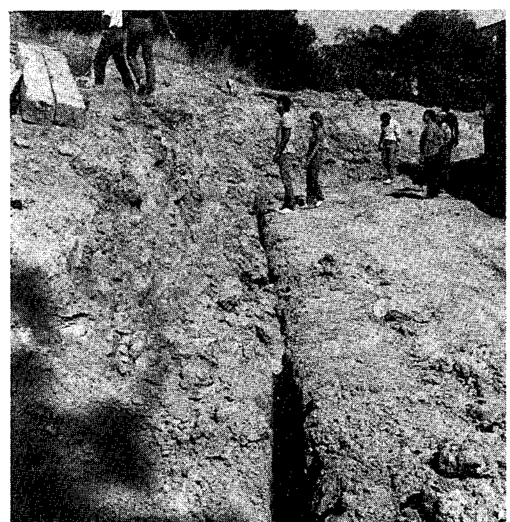


FIG. 6. Trench for Retaining Wall at Toe of Slope Squeezed Shut by Slope Movement.

water pressures decrease shear strength. In the slide area under investigation, water could enter the zone of instability as surface runoff or as groundwater. To investigate the possibility of an increase in surface runoff, as well as the possibility of increased surface infiltration, precipitation data were collected from the National Weather Service. These data are shown in Table 1. The actual precipitation for each month in the years from 1971 through 1983 is shown, and the departures from the average precipitation for each month and year are given. The greatest amount of annual precipitation, 50.8 inches, occurred in 1979. Additionally, during the winter of 1977-78 slightly over 22 inches of snow fell in the area of the slide. Much of the snow remained on the ground until the spring melt in mid-March 1978. The year with the next greatest amount of precipitation was 1975 with 56.3 inches measured.

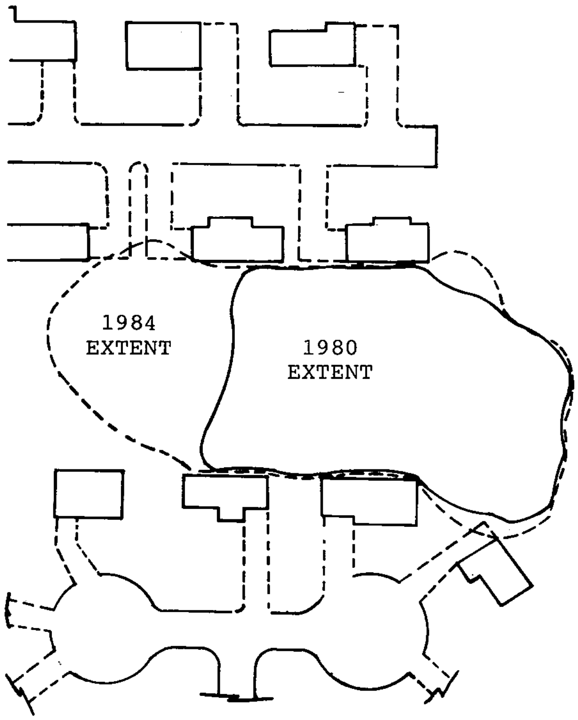
The information on hillside development and slope movement was collated with precipitation data. This collation indicated that movements of the hillside were linked directly to the development of the roadway along the crest of the ridge, and periods of instability were seen to coincide with periods of precipitation. The lower section of subdivision was developed in 1971 and all the houses in that area were occupied by the end of 1972. Precipitation during 1973 was greater than normal. Construction of the ridge line roadway was begun in summer of 1974 and completed in October of that same year. Precipitation for 1974 was very close to average. As of January 1975, no problems with the hillside had been observed. Heavy rains during March of 1975 (4.6 inches above normal) occurred just before the first observed movement of the slope. At that time, the landslide affected only one house at the base of the hill and one lot at the top of the hill, although horizontal movements downslope reached as much as 12 feet. Nevertheless, in July 1975 the owner of the downslope lot removed 16 truckloads of displaced soil from the toe of the slide area, without causing any additional movement. Obviously, between spring and summer of 1975, porewater pressures in the hillside must have dissipated. Nevertheless, the removal of the material from the toe area undoubtedly reduced the overall safety of the slope and created conditions which led to further movement after another wet weather period.

In spring of 1976, the houses at the top of the hillside were built. The lot immediately above the slide area was regraded in the summer of 1976, with material from the house excavation used as fill near the crest of the slope. This obviously added surcharge to the top of the unstable slope.

As mentioned previously, 1978 was a very wet year. Following the snow melt in March of 1978, the residents at the uphill lot saw that at the top of the hill, the ground surface adjacent to the rear of their house had dropped two to three feet. At this time, the landslide was still limited to one lot at the top of the hill and one lot at the bottom of the hill. The owners of these houses joined efforts to recontour the slope. Fill was placed at the top of the slope in an effort to replace the fallen area at the scarp. Failed material farther down the slope was pushed upslope to form two terraces. Once again, the tendency of the hillside to come to equilibrium had been cancelled by the actions of the landowners. The toe support provided by displaced soil was removed and the filling at the scarp added to the driving forces in the slide. Following a rainfall in late spring, the ground surface at the scarp fell again. By June of 1978, four lots on the slope were affected. The landslide continued to be active through July, with a total downward movement of more than four feet near the scarp. The landslide remained active during the winter of 1978-79 with occasional movements. Following heavy rains in early April 1979, massive movements in the slope occurred. Precipitation for the month of April measured 7.3 inches-3.2 inches above average. During this time, the scarp moved downward 10 to 12 feet. When the houses were condemned and demolished, the number of observers of the landslide was correspondingly reduced. For the year 1980, no stability problems were reported. It is significant to note that annual precipitation for 1980 was more than five inches below average. Precipitation during 1981 was almost nine inches below average. During this time, some soil moved almost imperceptibly against the rear wall of the remaining house at the base of the hillside. This type of movement appeared to be an accelerated creep. During most of the 1980s, the pattern of movement on the hillside has been episodes of significant movement during winter and spring months, with little movement, or very slow creep, during the remainder of the year.

The fact that no movement took place in this slope prior to construction of the roadway along the ridge line, even during the wet year of 1973, indicated that construction of that roadway played a significant part in the later instability. From April 1975 through September 1984, the hillside remained essentially unstable, although decreases in porewater pressure during dry periods gave temporary stability to the slope during the summer and fall months. The action of the landowners on the site in removing the support of the toe by excavation and by placing a surcharge of fill at the top of the slide decreased the overall factor of safety in the slope. Whenever an increase of porewater pressure followed a significant rainfall event, the slide once again became active. This preliminary analysis indicated that the critical period for

FIG. 7. Change in Extent of Slide With Time.



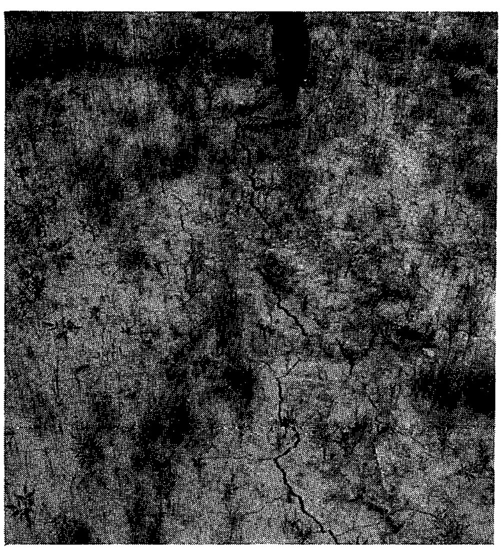



FIG. 8. Tension Crack in Failed Slope (typical).

FIG. 9. View Southeast Across Slope (note water ponded in depression).

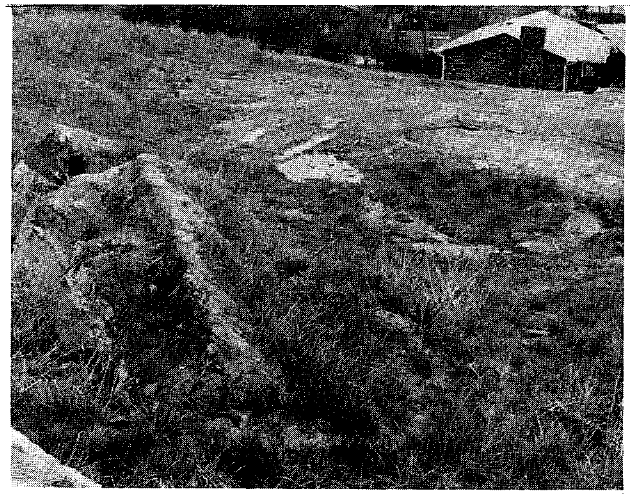


FIG. 10. Groundwater Exit Seep in Toe Area of Slide.

FIG. 11. Seep Near East End of Toe.

TABLE 1

MONTHLY PRECIPITATION

Actual and Departure from Normal*

| YEAR | | J | F | M | A | M | J | J | A | S | O | N | D | ANNUAL |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1971 | Actual | 2.64 | 6.28 | 2.12 | 2.16 | 6.15 | 2.64 | 6.74 | 1.83 | 4.72 | 1.96 | 2.06 | 2.98 | 42.28 |
| | Dep. | -1.46 | 2.99 | -2.47 | -1.66 | 2.25 | -1.35 | 3.38 | -1.14 | 2.09 | -.29 | -1.14 | -.24 | .96 |
| 1972 | Actual | 2.87 | 3.94 | 4.07 | 8.48 | 4.46 | 1.08 | 3.64 | 2.45 | 4.24 | 2.55 | 6.31 | 5.29 | 49.38 |
| | Dep. | -1.23 | .65 | -.52 | 4.66 | .56 | -2.91 | .28 | -.52 | 1.61 | .30 | 3.11 | 2.07 | 8.06 |
| 1973 | Actual | 1.96 | 1.60 | 6.26 | 5.77 | 7.04 | 6.20 | 9.38 | .91 | 2.34 | 2.28 | 7.59 | 2.64 | 53.79 |
| | Dep. | -2.14 | -1.69 | 1.67 | 1.95 | 3.14 | 2.21 | 6.02 | -2.06 | -.29 | .03 | 4.39 | -.58 | 12.65 |
| 1974 | Actual | 4.38 | 1.64 | 5.41 | 2.74 | 3.86 | 2.58 | 2.04 | 8.79 | 3.52 | 2.09 | 3.03 | 2.85 | 42.93 |
| | Dep. | .85 | -1.83 | .36 | -1.36 | -.39 | -1.47 | -1.72 | 5.08 | .58 | -.26 | -.30 | -.49 | -.18 |
| 1975 | Actual | 4.78 | 4.53 | 9.65 | 6.47 | 4.50 | 3.15 | 1.91 | 3.89 | 2.64 | 6.12 | 3.69 | 4.89 | 56.31 |
| | Dep. | 1.34 | 1.06 | 4.60 | 2.37 | .30 | -.90 | -1.85 | .90 | -.30 | 3.77 | .36 | 1.55 | 13.20 |
| 1976 | Actual | 3.85 | 3.13 | 2.87 | 0.76 | 5.09 | 4.71 | 2.10 | 3.18 | 3.10 | 3.99 | 0.72 | 0.65 | 34.15 |
| | Dep. | .32 | -.34 | -2.18 | -3.34 | .89 | .66 | -1.66 | .19 | .16 | 1.64 | -2.61 | -2.69 | -8.96 |
| 1977 | Actual | 2.33 | 1.45 | 4.69 | 3.40 | 1.37 | 7.59 | 3.29 | 6.12 | 3.67 | 4.76 | 6.11 | 4.32 | 49.10 |
| | Dep. | -1.20 | -2.02 | -.36 | -.70 | -2.83 | 3.54 | -.47 | 3.13 | .73 | 1.41 | 2.78 | .98 | 5.99 |
| 1978 | Actual | 5.90 | 0.76 | 3.76 | 3.33 | 4.76 | 2.67 | 3.55 | 5.61 | 1.17 | 3.61 | 5.94 | 4.07 | 45.13 |
| | Dep. | 2.37 | -2.71 | -1.29 | -.77 | .56 | -1.38 | -.21 | 2.62 | -1.77 | .26 | 2.60 | .73 | 2.02 |
| 1979 | Actual | 3.81 | 4.49 | 2.71 | 7.32 | 3.59 | 3.03 | 10.05 | 2.37 | 10.49 | 2.27 | 5.85 | 3.82 | 59.80 |
| | Dep. | .28 | 1.02 | -2.34 | 3.22 | -.61 | -1.02 | 6.29 | -.62 | 7.55 | -.08 | 2.52 | .48 | 16.69 |
| 1980 | Actual | 1.71 | 1.09 | 4.80 | 2.63 | 4.58 | 3.70 | 5.41 | 3.76 | 3.17 | 3.37 | 2.42 | 1.25 | 37.89 |
| | Dep. | -1.82 | -2.38 | -.25 | -1.47 | .38 | -.35 | 1.65 | .77 | .23 | 1.02 | .91 | -2.09 | -5.22 |
| 1981 | Actual | 0.45 | 3.23 | 1.54 | 4.44 | 4.63 | 3.23 | 3.98 | 3.21 | 3.22 | 1.60 | 2.40 | 2.02 | 33.95 |
| | Dep. | -3.08 | -.24 | -3.51 | .34 | .43 | -.82 | .22 | .28 | -.75 | -.93 | -1.32 | -9.16 | |
| 1982 | Actual | 5.28 | 1.55 | 5.89 | 3.05 | 2.96 | 3.86 | 3.72 | 3.74 | 3.46 | 1.26 | 5.50 | 5.11 | 45.38 |
| | Dep. | 1.75 | -1.92 | .84 | -1.05 | -1.24 | -.19 | -.04 | .75 | .52 | -1.09 | 2.17 | 1.77 | 2.27 |
| 1983 | Actual | 1.63 | 1.52 | 2.16 | 7.10 | 10.58 | 4.42 | 0.99 | 2.39 | 1.13 | 6.47 | 5.03 | 3.96 | 47.38 |
| | Dep. | -1.75 | -1.71 | -2.57 | 2.99 | 6.43 | .82 | -3.11 | -.92 | -2.22 | 3.84 | 1.54 | .48 | 3.82 |
| 1984 | Actual | 1.04 | 1.68 | 4.41 | 5.53 | 6.78 | 0.49 | 6.94 | 5.08 | | | | | |
| | Dep. | -2.82 | -1.55 | -.32 | 1.42 | 2.63 | 3.11 | 2.84 | 1.77 | | | | | |

*Units are in inches.

further study was the months immediately before the first movement in March 1975. The original ground surface profile could be obtained from a preliminary survey made in 1967. However, field investigations were needed to locate the failure surface, and it was necessary to determine shear strength parameters for the slide area through field and laboratory testing.

## Detailed Investigations

Between May 1983 and November 1984, further investigations of the landslide area were undertaken. Included in this work were a topographic survey of the slope and roadway, exploratory borings, in-situ vane shear tests, piezometric monitoring, and a drainage analysis. Frequent visual inspections were made throughout the period of this investigation. The extent of the landslide was defined by the presence of scars or features associated with movement. The main scarp at the top of the hillside coincided with the location of the remains of rear foundation walls of the two houses which had been demolished. The eastern flank of the hillside was clearly delineated because of the presence of a densely wooded, undisturbed zone adjacent to the failed slope along that flank (See Fig. 3). A low scarp two to three feet high was oriented at approximately 105 degrees from the scarp at the top of the hillside along this east flank. The top scarp was over 200 feet in length and was approximately eight feet high. Along this boundary, the slide area was characterized by a zone where failed soil lay over the natural ground surface. Fence posts and small trees had been displaced in this zone and the failed soil had "rolled" over the natural soil downslope in a thin zone usually less than one foot thick. Because of the actions of the landowners near the base of the slope, the toe of the landslide could not be identified by any visible intersection of failed soil and the natural ground surface. The extent of the slide was identified by the evidence of damage in the downslope structures. Tension cracks were present throughout the upper part of the slope (See Fig. 8). The cracks were vertical and followed the contours of the hillside, and extended from several feet to more than five feet below the surface.

Examination of the ridge line above the failed area disclosed the pattern of drainage on the slope. Before construction of the roadway along the crest of the ridge, the ground surface at the crest undulated, and surface runoff moved north and south down the hillsides. Construction of the roadway rerouted surface runoff through unpaved ditches along the crest of the ridge to a culvert located beneath the road immediately above the landslide area. The purpose of the culvert was to transport water from the south side of the roadway north beneath the road to an open transverse ditch which was intended to carry the water downslope northward. However, when this culvert was inspected, it was found clogged with debris. During heavy rainfalls, water was observed to be ponding at the location of the culvert causing the culvert eventually to be submerged. As the south drainage ditch continued to feed water to the inlet of the clogged culvert, the depth of the ponded water increased until overland flow began spilling southward down the lots and driveways to the slide area. Once this overflow elevation had been reached in the ditches, all subsequent runoff drained down the south hillside into the slide area. The main scarp of the landslide area coincided with the rear foundation walls and the pavement edges of the driveways on the upslope lots. It was obvious that the major source of water to the landslide consisted of runoff which had been directed to the culvert but which flowed down the south slope as a result of the inadequate size and malfunction of the culvert.

In the landslide area, a number of borings were made to investigate the properties and condition of the soils there. Because of the numerous regrading efforts which had taken place on the hillside, it was recognized that the findings of the subsurface investigation had to be evaluated with care. Prior to the investigations made during 1983 and 1984, four borings had been made in the hillside. One of the borings was made just north of the scarp and the other three extended down the slope in a line. The locations of the subsequent borings were selected to supplement the information gained from the prior borings. Three additional borings were made in August 1983 and two final borings were made in June 1984. At the base of the main scarp, intact shale was reached at a depth of 2.5 feet, with no observed change in soil with depth. In a second boring made approximately 75 feet downslope from the top of the scarp, refusal to sampling operations was reached at a depth of about 6.5 feet. In a third boring approximately 120 feet downslope from the scarp, resistance to penetration was much less than in the upslope borings. Shelby tubes were utilized for obtaining samples. When sampling extended to a depth of approximately six feet, the tube sampler was cold and wet when it was withdrawn from the boring. The summer of 1983 was extremely hot and dry, and no rain had fallen on the hillside for at least three weeks before the borings were made. Since groundwater obviously had been encountered in the third boring made at that time, the hole was left open to permit qualitative observations of groundwater conditions. In June 1984, two additional borings were made in an attempt to locate the failure surface of the landslide. These borings were advanced by means of a hand auger, and vane shear tests were attempted every six inches of depth in order to obtain a shear strength profile of the slope. It was possible in these borings to differentiate between fill soil and the natural ground surface by the presence of a zone of organic soil at the top of the natural ground.

The failure surface of the landslide was inferred from the information gathered in the subsurface exploration. The minimum undrained shear strength had been obtained in a thin gray clayey soil layer adjacent to the surface of the unweathered New Providence shale. Based on these observations, it was concluded that sliding probably had occurred along the interface between the residual soil and the shale. The elevations of the shale surface were obtained wherever the borings had reached the shale. The location of the remainder of the shale surface was estimated, based on available information. Because of the observed lateral displacements on the surface, the lack of rotation in the slide mass, and the shallow location of the interface between soil and shale, the movement was considered to be primarily translational. The failure surface then was approximated by a broken straight line between the elevations of the shale surface in the borings. The original ground surface profile and the failure surface thus defined were utilized in the slope stability analysis. The sliding wedge method of slope stability analysis was selected to suit these conditions.

The information available on conditions prior to the initial failure in 1975 contained no mention of groundwater conditions. Although the slope area was considerably reworked between 1975 and 1984, there was little apparent change in the overall drainage of the hillside. For this reason, it was assumed that groundwater drainage was not significantly altered by the landslide. Therefore, groundwater conditions observed during the investigation of 1983-84 were used to estimate the conditions which had been present at the failure in 1975. There was considerable evidence of surface groundwater flow in the form of seeps and springs, and this information together with the observations of groundwater in the borings, was used to evaluate the overall drainage characteristics of the slope. High levels of groundwater were virtually certain to have occurred at the period of failure. During the very dry and hot weather of August 1983, water was found in a depression on the level portion of the hillside about half way down the slope (See Fig. 9), even though no precipitation had occurred for at least three weeks prior to the site visit. This water obviously represented an outflow of groundwater. Therefore, the depression marked the lowest possible position of the piezometric surface at that location. The first observations of groundwater conditions by the authors followed an extended wet period in March 1984. At that time, numerous seeps were seen on the inclined portions of the hillside, but groundwater outlfow was most obvious on the upper slope just below the scarp. In the lower portions of the hillside, the number of seeps was much less than in the upper hillside, but the quantity and velocity of groundwater outflow from the downslope seeps was much greater than those in the seeps near the scarp. Flow in these lower seeps was sufficient to cause subsurface erosion and to create voids below the ground surface (See Figs. 10 and 11). As mentioned previously, in one of the borings made in 1983, groundwater was first encountered at a depth of approximately seven feet. This uncased boring was left open so that the level of water in the hole could be monitored. In less than eight hours, the water had risen to within one inch of the ground surface and remained there for more than a week until the uncased hole collapsed. It is important to note that this occurence took place during a prolonged period of dry weather. The investigations of the colluvial soil on this hillside indicated that the permeability characteristics of the soil changed with depth. The colluvium was more permeable with depth, particularly near the shale surface. Tighter, less permeable soil had formed at the surface due to weathering processes. Obviously, these results indicated that groundwater was present under pressure and that the soil overlying the water-bearing layer had a sufficiently low permeability to confine the groundwater to the more permeable zone near the shale surface. In the upper portion of the hillside, tension cracks penetrated the less permeable upper soil and provided direct access for water to the more permeable zone against the shale surface. During subsequent stability analyses, a variety of assumptions were made concerning groundwater conditions to determine the sensitivity of the slope to groundwater effects; the results of the analysis were compared to the conditions which were thought to have existed at the time of failure.

Laboratory tests were carried out on samples of the soil taken from the hillside. Index tests indicated that the soil was a clay of low plasticity, with more than 45 percent of the material consisting of clay size particles and a plasticity index of 19 (liquid limit of 40). These results were obtained for samples of soil taken from the soft zone which was considered to be the location of the failure surface. Samples obtained near the soil surface contained much less fine-grained material. Only 30 percent of the soil from the surface passed the number 200 sieve, compared to 97 percent for the failure zone soil. The surface soil was classified ML/CL, with a liquid limit of 36 and a plasticity index of 17. Much of this surface material appeared to be contaminated with fill which had been placed on the hillside.

Observations of slides in colluvial soils have indicated that the resistance along the failure surface may approach the residual strength of the soil.[4] Moreover, the aerial photographs taken in 1963 indicated the possibility of a preexisting slide on this site. For these reasons, a program of laboratory shear tests was carried out. A direct shear apparatus was used to determine the residual shear strength of the soil taken from the failure zone. The tests were conducted on remolded samples and the rate of shear was adjusted to guarantee adequate drainage of water to relieve any porewater pressures generated during shearing. The large deformation necessary for the determination of residual shear strength was obtained by reversal of the shearing direction, until the 2.5-inch-long specimens had been sheared more than one inch in cumulative displacement. A value of angle of internal friction of 21 degrees (consolidated drained results) was obtained for the samples taken from the failure zone. The unit weight of the soil from the failure zone was 128 pcf. These parameters were used in the stability analysis, with the results of the field investigations, to define the slope conditions thought to have existed at the time of failure in 1975.

The stability analysis which was utilized to investigate this slope was the sliding wedge method as presented by Drnevich.[5] The assumption that the failure surface is a plane in this method coincided with field observations. Moreover, the method was easily programmed. The cross-section of the hillside was assembled from historical and field investigations. The laboratory shear strength parameters were utilized with this cross-section to form a "base case" for the analysis. The base case represented, to the best of the authors' ability, the conditions in the hillside at the time of the failure in March 1975. In this analysis, the water table was assumed to coincide with the ground surface, and a tension crack extending to the shale surface was assumed to exist at the scarp. These assumptions were based upon field observations. However, field observations of the seeps in the downslope portion of the site indicated that artesian conditions could exist near the toe of the slide. In this way, the assumption that the groundwater surface coincided with the ground surface was considered to be conservative.

The stability analysis for the base case yielded a factor of safety equal to 1.01, indicating the slope was very near failure in 1975. This result indicated that the slope geometry, groundwater conditions, and shear strength parameters used in the analysis were reasonable. The influence of individual parameters on the stability of the hillside was assessed further through a sensitivity analysis. Essentially, base case conditions were held constant except for one parameter which was varied to investigate its influence. The parameters examined were: the effective angle of internal friction, $\phi$; the effective unit weight, $\gamma$; the undrained shear strength, $S_u$; and the porewater pressure coefficient, or $r_u$. Additionally, the effect of water in a tension crack was examined. In the analyses, the effective unit weight of the soil was varied from 62 to 70 pcf, and the effective angle of internal friction was varied from 15 to 25 degrees. The influence of each parameter on the factor of safety was found to be essentially linear. Increasing effective unit weight or effective angle of internal friction increased the shear resistance of the soil. The factor of safety against sliding increased for increases in effective unit weight even though the driving forces also were increased. A change in effective unit weight of 1 pcf caused a change in the factor of safety of less than 0.01. On the other hand, a change in the effective friction angle of one degree caused a change in the factor of safety of approximately 0.05. Another influential parameter was found to be the porewater pressure coefficient for the slope. When the piezometric surface was placed at or below the failure surface, a factor of safety of 2.15 was obtained. For the water table at the ground surface, as in the base case, the corresponding porewater pressure coefficient was 0.48, and the factor of safety was 1.01. Increasing the porewater pressure coefficient to a value of 0.51 decreased the factor of safety to 0.99. The presence of water in a tension crack at the scarp was modeled by lowering the level of water in the crack while maintaining the surface-saturated condition throughout the remainder of the slide area. For every foot that the water level was lowered in the tension crack, the factor of safety increased by slightly more than 0.01.

Use of the undrained shear strength in stability analyses of slopes in weathered shale has been reported to lead to an overestimate of the factor of safety of the slope.[6,7,8] For this reason, the undrained shear strength values which were obtained in the vane shear tests in the field were utilized to evaluate the effect of using undrained shear strength in stability calculations. First, the effective angle of internal friction was set equal to zero and a back calculation was used to determine that an undrained shear strength value of 250 psf would have been necessary for the landslide to have occurred. Using the values of undrained shear strength measured at the failure surface in the vane shear test (900 psf), a safety factor of approximately 3.6 was calculated. Obviously, the in-situ shear strength was much lower than the shear strength measured in the undrained test. The reason for this difference is that the porewater pressures existing during the undrained tests were not equal to the in-situ pore pressures during the actual failure.

The sensitivity analysis indicated that the stability of slope was least sensitive to the effective unit weight of the soil and most sensitive to the effective angle of internal friction. These values were considered to be fairly constant throughout the slope because of the geologic history of the site. The base case values of both of these parameters had been obtained from a single laboratory direct shear test because of the great difficulty in obtaining and testing an undisturbed sample. However, shear strength values for similar soils reported in the literature supported the result of the direct shear test[9,10] as did back calculation of shear strength from the sliding wedge analysis. All of these conditions led to the conclusion that the parameter values used in the base case were sufficiently accurate for the analysis. The most variable parameter in the analysis was considered to be the porewater pressure ratio. Seasonal variations in infiltration and associated changes in the water table in the slope would be expected at this site. For a completely dry slope, the safety factor was 2.2, while the factor of safety for the base case, with the water table coincident with the ground surface, was 1.01. If the water pressure was increased near the toe of the slope because of the indications of artesian conditions there, the factor of safety quickly dropped below 1.0, indicating that failure of the slope would have occurred. It was concluded that the formation of excess porewater pressure in this hillside was the most important factor in the failure of the slope.

## Conclusions

It is seldom possible to attribute a landslide to a single definite cause. In most cases, a number of causes exist simultaneously until some triggering action sets the soil mass in motion. The trigger action cannot be regarded as the only cause even though it was necessary for failure.

The landslide which has been described in this paper obviously could be considered a result of the characteristics of the geologic formation, the New Providence shale, which produced the soil on the hillside. The factor of safety against sliding in such a natural hillside is seasonally low after periods of wet weather. Aerial photographs taken prior to construction in the area of the slide indicated that a landslide may have occurred at this location prior to any clearing operations on the site. Alteration of the natural drainage system by construction of a roadway along the ridge at this site undoubtedly diverted runoff along the entire crest of the hill and concentrated it at a point above the hillside. When the inadequate culvert at that point clogged, water spilled over the ground surface into the area of instability. Saturation of the soil in the slide area increased the weight of the soil, and increasing porewater pressures decreased the shear strength of the material. The formation of excess porewater pressures in a permeable zone along the soil/shale contact further reduced the forces resisting movement. Water in a tension crack at the top of the slope provided another driving force, particularly with the runoff from the inadequate culvert at the top of the hill.

The combination of the factors described above produced a landslide in March 1975. Remedial measures taken by the landowners affected by the slide did not address the source of water coming to the slope. Rather, their efforts made conditions worse since they removed failed soil from the toe of the slide, thereby reducing the resisting forces, and placed fill material near the scarp, increasing driving forces. These alterations were performed during dry periods when the stability of the slope was higher because of the decrease in water pressure in the slope. When the water pressures in the slope increased after significant subsequent rainfall events, inevitable movements occurred.

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# METHODS OF ANALYZING THE STABILITY OF NATURAL SLOPES

by J. M. Duncan*

Notes for a lecture to be presented at the 17th Annual Ohio River Valley Soil Seminar, at Louisville, Kentucky, on October 17th, 1986

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# METHODS OF ANALYZING THE STABILITY OF NATURAL SLOPES

J. M. Duncan

## INTRODUCTION

The stability of natural slopes depends on a number of factors, including slope configuration, soil strength, and groundwater conditions. Accurate analysis of slope stability requires use of a suitable method of analysis, and accurate representation of the strength of the soil within the slope, the slope geometry, and the pore pressure conditions.

Several different methods of slope stability analysis have been developed. Choosing among these requires an understanding of the conditions under which they provide accurate results, and an appreciation of their advantages and shortcomings.

One effective means for evaluating soil strengths for slope stability analyses is to use "back-analysis." This procedure can be applied wherever landslides have occurred. By considering that the slope had a factor of safety equal to unity when it failed, strength parameters can be determined by trial and error. Back-analysis provides strength values that are often more reliable than values measured in laboratory tests.

In the following sections of these notes, procedures for analyzing the stability of natural slopes and for back-analysis are discussed.

## METHODS OF ANALYZING SLOPE STABILITY

A number of different methods of slope stability analysis have been developed and used in practice. From the viewpoint of a practicing engineer, it is desirable to use the simplest method of analysis that provides acceptably accurate results. In a paper entitled "The Accuracy of Equilibrium Methods of Slope Stability Analysis," Stephen Wright and the writer have described studies undertaken to determine the basic accuracy of a number of methods that are widely used for slope stability analysis. A copy of the paper is included at the end of these notes. The principal conclusions of the study were:

(1) The Ordinary Method of Slices is an accurate method for analysis for total stresses.

(2) The Ordinary Method of Slices is very inaccurate when applied to effective stress analysis of flat slopes with high pore pressures.

(3) Bishop's Modified Method is an accurate method of analysis for all conditions. Its limitations are that, like the Ordinary Method of Slices, it is only applicable to circular slip surfaces, and it is sometimes subject to numerical convergence problems.

(4) Force equilibrium procedures (methods that do not consider moment equilibrium, such as wedge methods, the Corps of Engineers Modified Swedish Method, and Lowe and Karafiath's Method) can be used to analyze either circular or non-circular slip surfaces. Their drawback is that the calculated factors of safety are affected by the assumed inclinations of the side forces between slices.

(5) Methods that satisfy all conditions of equilibrium (such as Morgenstern and Price's Method, Janbu's Method, and Spencer's Method) can be used for circular and noncircular slip surfaces. Their drawbacks are that they sometimes do not converge, and they may take an appreciable amount of computation time, depending on the speed of the computer on which they are run.

These conclusions provide guidance for selecting methods of analysis of slope stability, to achieve the best combination of accuracy and ease of analysis.

In addition to the methods mentioned above, the use of slope stability charts is worthy of consideration. For many conditions stability charts provide factors of safety that are as accurate as those that can be calculated by detailed computer analyses, and they can be performed in a fraction of the time required for computer analyses. The main requirement for achieving good accuracy with slope stability charts is use of systematic procedures for averaging soil strengths and unit weights, as explained by Duncan and Buchignani (1975). In many cases the accuracy with which factors of safety can be calculated using charts is equal to or better than the accuracy with which soil conditions and strength values are known. In the writer's opinion, slope stability charts are under-appreciated and under-used.

BACK-ANALYSIS

Back-analysis to determine soil strengths has a number of advantages as compared to conventional sampling and laboratory testing procedures. Use of field data provides the average strength of a much larger mass of soil than can be tested in the laboratory, in an undisturbed condition. It also accounts for progressive failure, anisotropy, and strain rates effects.

The slope geometry and groundwater conditions at the time of failure must be known or assumed for back-analysis. In addition, only one strength parameter (c or phi) can be calculated. The other must be assumed. Frequently what seems like a reasonable value of one of the parameters requires an unreasonable value of the other for a factor of safety of unity. As a result, a number of trials may be required to determine realistic values of c and phi.

Of even greater importance in back-analysis is that the values of c and phi determined should be consistent with the shape of the slip surface. Skempton (1945) showed that, all other things being equal, the critical slip surface is shallower for a large value of phi, and deeper for a small value of phi. This is shown in Fig. 1. Fig. 1 was developed by Stark (1984) using data derived from Janbu (1954). The parameter $\lambda_{c\phi}$ is defined by the equation:

$$\lambda_{c\phi} = \frac{(\gamma H - \gamma_w H'_w) \tan \phi'}{c'}$$

This dimensionless parameter is a measure of the relative contributions of c and phi to the shear strength. When $\lambda_{c\phi}$ is small, the strength is mostly cohesive. When $\lambda_{c\phi}$ is large, the strength is mostly frictional.

Fig. 1 shows that the depth of the critical slip surface depends on $\lambda_{c\phi}$. For large values of $\lambda_{c\phi}$, the ratio d/l is small, corresponding to a long, flat slip surface. For small values of $\lambda_{c\phi}$, the value of d/l is larger, corresponding to a slip surface that extends deeper in relation to its length.

The relationships illustrated in Fig. 1 can be used to determine values of c and phi that are consistent with the actual slip surface. This is important if the results of the back-analysis are to be as accurate as possible, and useful for further evaluations of slope stability.

To determine values of c and phi consistent with both a factor of safety of unity and the observed shape of the slip surface, the following procedure can be followed:

(1) Estimate the value of $\phi'$ based on the characteristics of the soils. Fig. 2 shows a correlation between $\phi'$ and PI that can be used to estimate $\phi'$ for this purpose.

(2) Estimate the value of $\lambda_{c\phi}$ based on the slope angle and the value of d/l using the relationships shown in Fig. 1.

(3) Estimate the value of c' using this equation:

$$c' = \frac{(\gamma H - \gamma_w H'_w) \tan \phi'}{\lambda_{c\phi}}$$

(4) Calculate the factor of safety of the slope using the value of $\phi'$ from step (1) and the value of c' from step (3). Call this value of safety factor $F_1$.

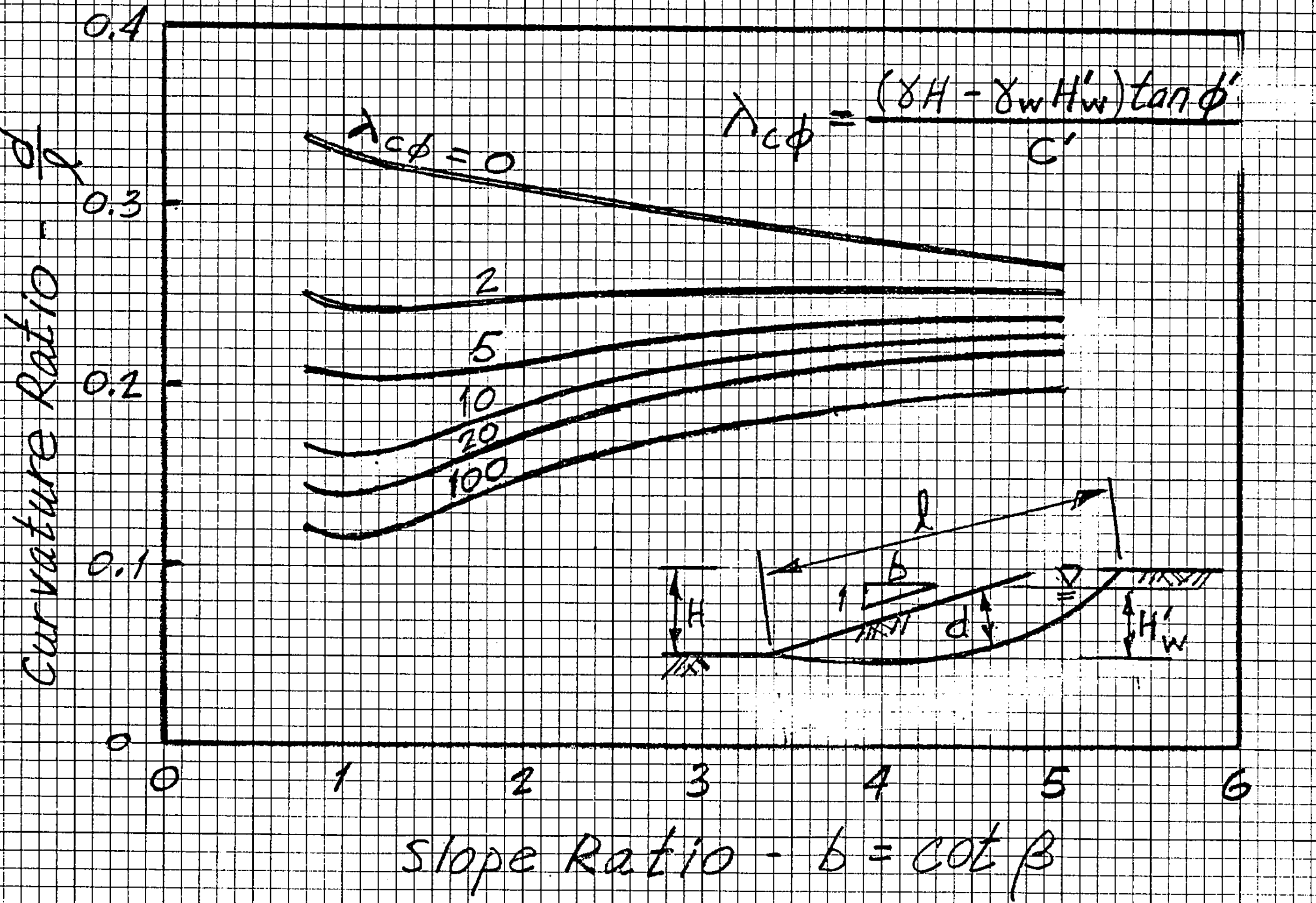


FIG. 1 - Relationship between Slope Ratio, Curvature Ratio, and $\lambda_{c\phi}$ (after Stark (1984))

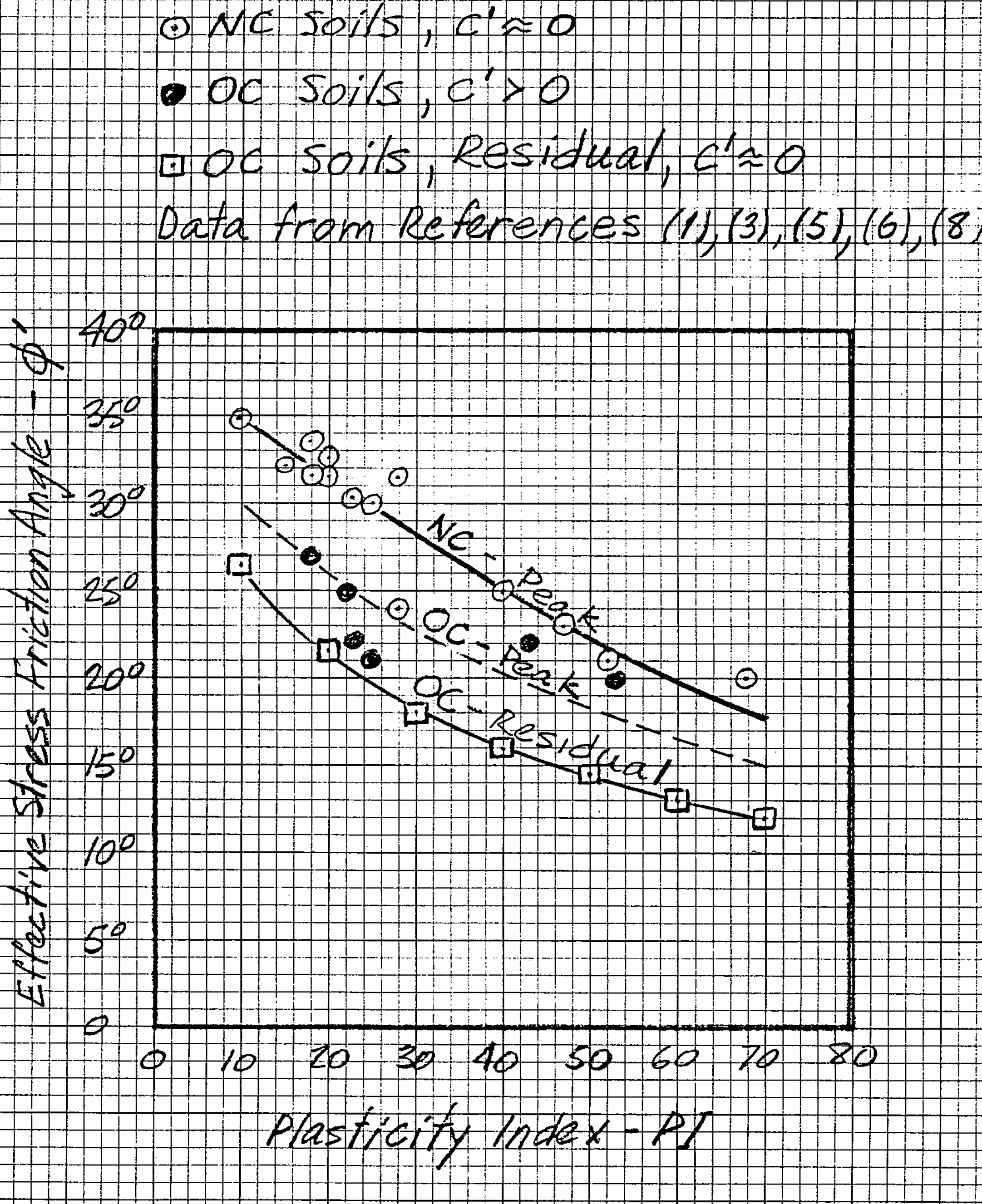


FIG. 2 - Correlation of $\phi'$ with PI (after Stark (1984))

(5) Calculate the values of c' and $\phi'$ corresponding to F = 1 as follows:

$$c' = \frac{c' \text{ from step(3)}}{F_1} \qquad \tan\phi' = \frac{\tan\phi' \text{ from step (1)}}{F_1}$$

This procedure for back-analysis has two important advantages:

First, the values of c' and $\phi'$ are consistent with the shape of the slip surface. Thus if a search for the critical slip surface was performed using these values, the actual slip surface would be found to be more critical than any other. This would not be true for other values of c' and $\phi'$, corresponding to other values of $\lambda_{c\phi}$. Thus other values would not be valid for use in a forward analysis of the stability of the same slope.

Second, the balance between the cohesive and frictional components of the shear strength is consistent with the actual field behavior of the soil. This is important for purposes of evaluating the effects of changes in groundwater conditions. One example would be for evaluating the effectiveness of drainage to improve stability. Another would be for evaluation of the stability of another slope in the same soil, subject to different groundwater seepage conditions than the slope where the back-analyzed failure occurred.

## CONCLUSION

Analyses of stability of natural slopes requires careful consideration of the method of analysis, and the slope geometry, groundwater conditions, and soil strength. Consideration of the accuracy, advantages, and limitations of the various methods of slope stability analysis provides a rational basis for selecting a suitable method of stability analysis. For many conditions slope stability charts provide an accurate and efficient method of analysis.

Back-analysis of slope failures provides an effective procedure for determining soil strengths under field conditions. Determination of strengths that are fully consistent with field behavior requires consideration of the shape of the slip surface as well as the slope geometry and groundwater conditions.

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# THE ACCURACY OF EQUILIBRIUM METHODS OF SLOPE STABILITY ANALYSIS

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## ABSTRACT

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Equilibrium methods of slope stability analysis all involve assumptions because the number of equilibrium equations available is smaller than the number of unknowns involved. Furthermore, a number of widely used methods do not satisfy all conditions of equilibrium, and thus do not employ all of the available equations of equilibrium. This paper discusses the inaccuracies which arise from these inevitable assumptions and these neglected conditions of equilibrium.

Comparative analyses have shown that all of the methods which satisfy all conditions of equilibrium result in the same value of safety factor with an accuracy no worse than ± 5%, which is perfectly acceptable for practical purposes. Furthermore, within this range of accuracy, this may be considered to be the 'correct' answer. Bishop's modified method, which does not satisfy all conditions of equilibrium, has been found to be as accurate as the methods which do so, and thus to be an effective and simple means of analyzing circular slip surfaces.

The ordinary method of slices, which satisfies only one condition of equilibrium, gives factors of safety which may be as much as 50% smaller than the correct value for flat slopes with high pore pressures. Force equilibrium procedures with ill-chosen side force assumptions may give factors of safety which are 30% larger than the correct value for slopes in cohesive soils.

The results of the study show how equilibrium methods may be selected which avoid significant errors arising from the mechanics of the analysis, and thus allow the engineer performing the analysis to devote his attention and effort to correct evaluation of shear strength.

## INTRODUCTION

Equilibrium analyses of slope stability are widely used in design of excavation and embankment slopes, and extensive experience has demonstrated their effectiveness and reliability. The accuracy of an equilibrium analysis of slope stability depends on the accuracy with which the strength properties and geometric conditions can be defined, and on the inherent accuracy of the method of analysis.

In most cases the uncertainties related to definition of geometry and soil properties are greater than those which arise from the approximations involved in the analytical technique, and the most accurate possible evaluation of shear strength is a critical aspect of all analytical studies of stability. Uncertainties in slope stability calculations also arise from the approximations made in developing the methods of analysis, and in some cases these are very significant.

The characteristics of the ordinary method of slices (Fellenius, 1927), Bishop's modified method (Bishop, 1955), Morgenstern and Price's method (Morgenstern and Price, 1965), Janbu's generalized procedure of slices (Janbu, 1957), Spencer's method (Spencer, 1967), the log spiral method (Wright, 1969), and the force equilibrium methods (Lowe and Karafiath, 1960) are reviewed in the following sections. It is shown that in some cases the errors resulting from inherent inaccuracies in the analyses may be very significant. However, if the method of analysis is suitable for the condition analyzed, the inaccuracy in the factor of safety engendered by the method of analysis need be no more than a few percent.

## EQUILIBRIUM METHODS

All equilibrium methods of slope stability analysis have four characteristics in common:

(1) They all use the same definition of the factor of safety ($F$):

$$F = s/\tau \tag{1}$$

in which $s$ = shear strength and $\tau$ = shear stress required for equilibrium. The use of this definition of $F$ is appropriate because the greatest uncertainties in most practical problems are related to evaluation of the shear strength. Thus defining $F$ in terms of a factor on shear strength associates the factor of safety directly with what is almost always the most significant uncertainty in practical problems.

(2) They all involve the implicit assumption that the stress—strain characteristics of the soils forming the slopes are non-brittle, and that the same value of shear strength ($s$) may be mobilized over a wide range of strains along the slip surface. This assumption is necessary because there is no consideration of strains or deformations in these methods, and no assurance that the strains may not vary significantly from point to point along the slip surface. Thus, strictly speaking, these methods are not applicable to analysis of slopes in soils such as stiff-fissured clays and shales, which have residual strengths appreciably lower than their peak strengths. In practice this problem is overcome by using strengths lower than the peak in such cases, as indicated by experience with slopes which have failed (Skempton, 1977).

(3) They all use some or all of the equations of equilibrium to calculate the average value of $\tau$, and to calculate the normal stress on the slip surface ($p$) which is required to determine the shear strength using eq.2:

$$s = c + p \tan\phi \qquad (2)$$

in which $c$ and $\phi$ are the Mohr-Coulomb strength parameters.

(4) They all involve explicit assumptions to supplement the equations of equilibrium. Since the number of equilibrium equations is smaller than the number of unknowns in the problem, all methods employ assumptions to make up the balance.

## EQUATIONS, UNKNOWNS AND ASSUMPTIONS

Methods of analysis which are applicable to practical problems must be able to accommodate conditions where the slip surface is curved and the soil properties and pore pressures vary with location through the slope. For this reason, most of the equilibrium methods divide the freebody bounded by the slip surface into a number of vertical slices. The forces acting on a typical slice are shown in Fig.1. These forces are $W$ = weight of slice, $S$ = shear force on base of slice ($S = \tau$ multiplied by $l$, where $l$ is the length of the base), $P'$ = effective normal force on base, $U$ = water pressure force on base, $X$ = vertical side force, and $E$ = horizontal side force. $c'$ and $\phi'$ are the effective stress shear strength parameters and $F$ is the factor of safety.

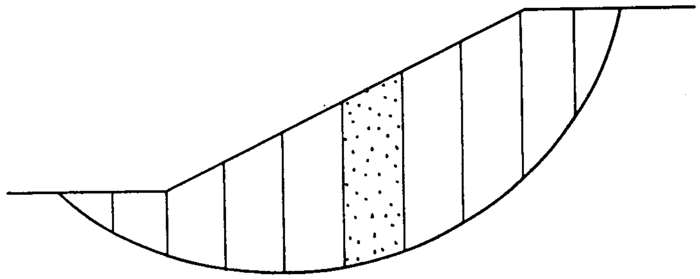
### *Methods satisfying all conditions of equilibrium*

Methods which use both force and moment equilibrium (see Table I) have three equations of equilibrium for each slice (horizontal force, vertical force, and moment). For $N$ slices, this is a total of $3N$ equations. The unknowns are: $N$ values of $P'$ (one for each slice), $N-1$ values of $X$ (one for each inter-slice boundary), $N-1$ values of $E$, $N-1$ locations (moment arms) of $E$, $N$ locations of $P'$, and one value of $F$. In total there are $5N-2$ unknowns. Thus for more than one slice, the equations exceed the unknowns and assumptions are required to make up the balance.

The location of $P'$ on the base of the slice is not a critical unknown. Assuming that $P'$ acts in the center of the base or below the center of gravity introduces very little uncertainty, especially when the slices are narrow, and each of the methods which satisfy all conditions of equilibrium make one of these two assumptions, reducing the number of unknowns to $4N-2$. This leaves $N-2$ non-trivial assumptions required to make the equations balance the unknowns. The assumptions involved in various methods are summarized in Table II.

### *Force equilibrium methods*

Methods which use only force equilibrium have two equations for each slice (horizontal force and vertical force equilibrium). For $N$ slices, this is $2N$ equations. The unknowns are: $N$ values of $P'$, $N-1$ values of $X$, $N-1$ values of $E$, and one value of $F$. In total there are $3N-1$ unknowns. Thus, as for



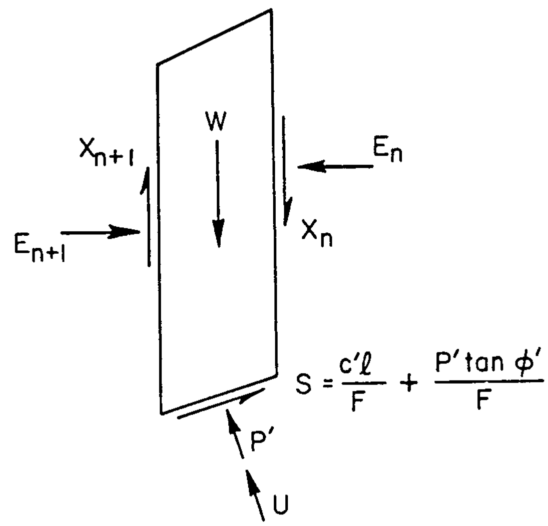



Fig.1. Forces on a typical slice.

methods using both force and moment equilibrium, the number of unknowns exceeds the number of equations if there is more than one slice, and assumptions are required to make up the difference between the number of equations and the number of unknowns.

*The ordinary method of slices*

Some methods do not employ some of the equilibrium equations discussed above. For example, the ordinary method of slices satisfies only one condition of equilibrium, which is moment equilibrium around the center of the circular slip surface. (The method is only applicable to circular slip surfaces.) The method assumes that the resultant of all side forces acting on any slice acts parallel to the base of the slice. Then, by resolving forces normal to the base, the following equation is derived:

$$P = P' + U = W \cos\alpha \tag{3}$$

in which $\alpha$ = inclination of the base of the slice. Because the direction in which forces are resolved varies from slice to slice (and because equilibrium is not satisfied in the direction parallel to the base of each slice), the ordinary method of slices does not satisfy either horizontal or vertical force equilibrium

TABLE I

Characteristics of equilibrium methods

| Procedure | Equilibrium conditions satisfied | | | | Equations and unknowns* | Shape of slip surface | Practical for: | |
|---|---|---|---|---|---|---|---|---|
| | overall moment | ind. slice moment | vert. force | horiz. force | | | hand calc. | computer calc. |
| Ordinary method of slices | yes | no | no | no | 1 | circular | yes | yes |
| Bishop's modified method | yes | no | yes | no | $N+1$ | circular | yes | yes |
| Janbu's generalized procedure of slices | yes | yes | yes | yes | $3N$ | any | yes | yes |
| Morgenstern and Price's method | yes | yes | yes | yes | $3N$ | any | no | yes |
| Spencer's method | yes | yes | yes | yes | $3N$ | any | no | yes |
| Force equilibrium | no | no | yes | yes | $2N$ | any | yes | yes |
| Log spiral | yes | — | yes | yes | 3 | log spiral | yes | yes |

*$N$ = number of slices.

TABLE II

Assumptions in equilibrium methods

| Procedure | Assumptions employed |
|---|---|
| Ordinary method of slices | Resultant of side forces is parallel to base of each slice. |
| Bishop's modified method | Resultant of side forces is horizontal (no vertical side forces). |
| Janbu's generalized procedure of slices | Location of side force resultants on sides of slices (location can be varied). |
| Morgenstern and Price's method | Pattern of variation of side force inclination ($\theta$) from slice to slice: $\theta = \lambda f(x)$. The value of $f(x)$ is assumed at each interslice boundary, and the value of $\lambda$ is an unknown. |
| Spencer's method | Side forces are parallel ($\theta$ = constant). Corresponds to $f(x)$ = constant in Morgenstern and Price's method. |
| Force equilibrium methods | Inclinations of side force (value of $\theta$) at each interslice boundary. |
| Log spiral | Shape of slip surface is a logarithmic spiral. |

for the mass above the slip surface. Thus the method involves only one equation (overall moment equilibrium around the center of the circle) and one unknown (the factor of safety).

The value of $P$ given by eq.3 is a conservative (low) approximation, which leads to a conservative value of $F$. In cases of flat slopes with high pore pressures, the error in the value of $F$ may be as much as 50%. In total stress analyses the error is not more than about 10%.

*Bishop's modified method*

The simplified method developed by Bishop (1955) satisfies $N + 1$ conditions of equilibrium. These are: overall moment equilibrium around the center of the circle (the method is applicable only to circular slip surfaces), and vertical equilibrium for each slice. Thus the method has $N + 1$ equations and $N + 1$ unknowns, the unknowns being the $N$ values of $P$ on the base of each slice and the factor of safety. The method does not satisfy horizontal force equilibrium or individual slice moment equilibrium. In spite of the fact that it does not satisfy all conditions of equilibrium, Bishop's modified method has been found to be an accurate method of analysis for circular slip surfaces, including flat slopes with high pore pressures.

*Force equilibrium procedures*

These include: (1) the method described by Lowe and Karafiath (1960) which follows the work of Taylor; (2) the method developed by Seed and Sultan (1967); (3) various methods (commonly known as 'wedge' methods) which divide the slip surface into an active uphill wedge, a straight central section, and a passive downhill wedge; and (4) any other method which satisfies only force (not moment) equilibrium. In all such methods the analysis can be accomplished by trial-and-error graphical procedures wherein a value of $F$ is assumed and a trial force polygon is drawn for each slice or wedge; if the last slice is in equilibrium, the assumed value of $F$ is correct. The analyses may also be accomplished by a numerical equivalent of this graphical procedure.

In all force equilibrium procedures, the assumed quantities are the $N - 1$ values of the side force inclinations. The factor of safety calculated using these procedures is very significantly affected by the assumed side force inclination. For example, an embankment on a soft clay foundation which the authors studied had a factor of safety equal to 2.27 as calculated by methods which satisfied all conditions of equilibrium. Detailed studies indicated that this value could be considered 'correct', at least within a range of ± 15%. However, a force equilibrium analysis, in which the side force was assumed to be parallel to the ground surface, resulted in a value of $F = 2.98$, about 30% higher than the correct value. Although this difference was not practically significant in the particular case studied because all methods indicated the embankment was stable, it could have been highly significant if the strengths of the embankment and the clay foundation (and thus the value of $F$) had been lower. For example, for strength values low enough so that the correct value of $F$ was 1.0, the force equilibrium method with side forces assumed parallel to the ground surface would have shown $F = 1.3$. On the basis of the force equilibrium analysis the embankment would have been judged to have a margin of safety, whereas it was actually on the verge of instability.

Of all the possible assumed inclinations for the side forces, the one suggested by Lowe and Karafiath (1960) appears to be the most generally applicable. They proposed that the side force inclination at each interslice boundary should be assumed to be the average of: (a) the inclination of the ground surface at the top of the interslice boundary; and (b) the inclination of the slip surface at the bottom of the interslice boundary. This assumption leads to values of $F$ which are about 10% too high for $\phi = 0$ analyses. For larger values of $\phi$ or $\phi'$, the assumption leads to more accurate values of $F$.

### *The log spiral method*

Unlike the methods discussed previously, the log spiral method is not a method of slices. By assuming that the slip surface has the shape of a logarithmic spiral, the equilibrium of the mass bounded by the slip surface can be satisfied completely without further assumption. The method involves three equations (overall moment, horizontal and vertical force equilibrium) and three unknowns (the magnitude and the direction of the resultant of the normal and the frictional shear forces, and the factor of safety). Because the log spiral method is applicable only to homogeneous conditions, it is not particularly useful for practical purposes, which nearly always involve greater complexity. The method is useful as a basis for comparison with the other equilibrium methods, all of which employ slices and thus involve fundamentally different types of assumptions.

## COMPARISONS OF FACTORS OF SAFETY

Although Tables I and II show that the various methods listed satisfy different conditions of equilibrium, and that they employ different assumptions to make up the balance between equations and unknowns, it remains to be determined whether these differences have large or small effects on the factors of safety calculated by the various methods. A number of comparative analyses have been made to evaluate these differences. Before these results are discussed, it is useful to consider a number of factors which influence factors of safety for slopes.

### *Minimum values of F*

In making comparisons of the various methods it is essential that the values of $F$ which are compared are the minimum values for each method. Table III illustrates why this is so. In general, if a number of methods are used to analyze circular failure surfaces in the same slope, they will be found to have different critical circles and different values of $F$ for the same circle. Thus, as shown in Table III, if a comparison was made for circles which are selected arbitrarily, the comparison might support any conclusion regarding which of the methods results in the lower factor of safety, i.e., it might be concluded that $F_1 < F_2$, $F_1 = F_2$ or $F_1 > F_2$. Obviously, only one of these can be correct in general. The only factor of safety for a given slope and

TABLE III

Illustration of the necessity of comparing minimum factors of safety

| | | Factor of safety calculated by: | |
|---|---|---|---|
| | | method 1 | method 2 |
| Circle A (critical for method 1) | | $F_1 = 1.5$ (min) | $F_2 = 1.7$ |
| Circle B (not critical for either method) | | $F_1 = 1.6$ | $F_2 = 1.6$ |
| Circle C (critical for method 2) | | $F_1 = 1.8$ | $F_2 = 1.4$ (min) |
| Comparisons: | circle A: | $F_1 < F_2$ | |
| | circle B: | $F_1 = F_2$ | |
| | circle C: | $F_1 > F_2$ | |
| | min. values: | $F_{1min} > F_{2min}$ | |

method of analysis which has any general significance is the minimum factor of safety. Thus, in comparing factors of safety for various methods, the minimum values of $F$ should be compared. This procedure has been followed in developing the comparisons discussed in the following paragraphs.

### *Definition of* $\lambda_{c\phi}$

One problem in comparing methods of slope stability analysis is that quite a large number of parameters are involved in the problem. These include the slope height ($H$), the slope angle ($\beta$), the values of the strength parameters ($c$ and $\phi$), the unit weight of the soil ($\gamma$), and the magnitudes of the pore-water pressures within the slope. The number of parameters is so large that making a systematic comparison of values of $F_{min}$ for the methods discussed previously would be a very formidable task. Fortunately, Janbu (1954) defined a dimensionless parameter ($\lambda_{c\phi}$) which combines four of the variables involved in the problem, and reduces the number of parameters which must be investigated from six to three. The definition of $\lambda_{c\phi}$ is:

$$\lambda_{c\phi} = \gamma\, H(\tan\phi)/c \qquad (4)$$

It may be seen that for a slope in soil with only cohesive shear strength ($\tan\phi = 0$), $\lambda_{c\phi}$ is equal to zero. As the value of $c$ decreases, the value of $\lambda_{c\phi}$ approaches infinity. The examples listed in Table IV show values of $\lambda_{c\phi}$ for various conditions.

For values of $\lambda_{c\phi} < 1$, the critical slip circle passes beneath the toe of the slope, as shown in Fig.2, provided the slope angle is flatter than about 50°. Theoretically, the critical circle would extend infinitely deep in a layer of

TABLE IV

Values of $\lambda_{c\phi}$ for various slopes

| $\gamma$ (t/m³) | $H$ (m) | $\tan\phi$ (dimensionless) | $c$ (t/m³) | $\lambda_{c\phi}$ (dimensionless) |
|---|---|---|---|---|
| any value | any value | 0 | any value | 0 |
| 1.6 | 10 | 0.5 | 4 | 2 |
| 1.6 | 10 | 0.5 | 2 | 4 |
| 1.6 | 10 | 1.0 | 2 | 8 |
| 1.6 | 20 | 1.0 | 2 | 16 |
| 2.4 | 20 | 1.0 | 2 | 24 |
| any value | any value | any value | 0 | $\infty$ |

$\lambda_{c\phi} = \lambda H(\tan\phi)/c$

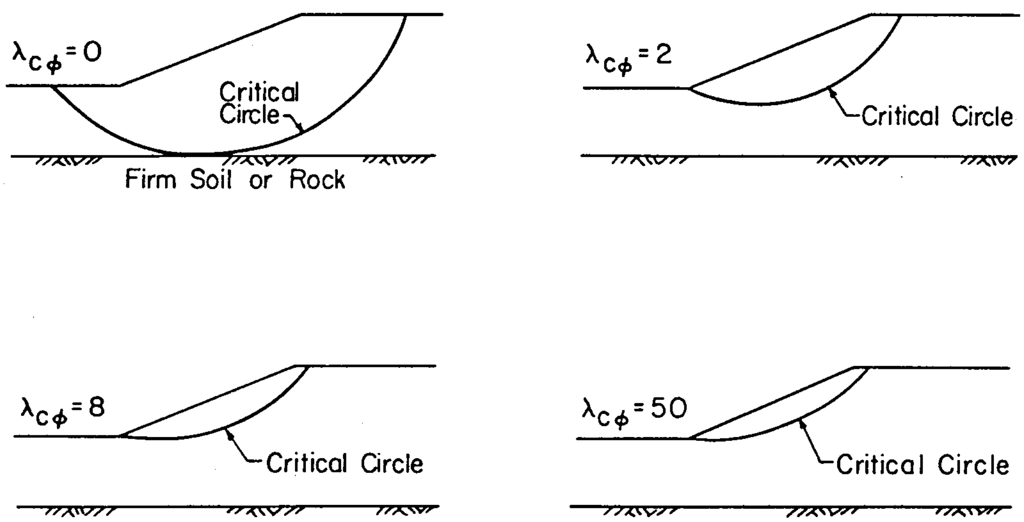



Fig.2. Critical circle locations for four values of $\lambda_{c\phi}$.

constant shear strength with $\lambda_{c\phi} < 1$. Practically, the critical surface will extend to the top of firm soil or rock.

For values of $\lambda_{c\phi} > 2$, the critical slip circle passes through the toe of the slope, becoming more and more shallow as the value of $\lambda_{c\phi}$ increases as shown in Fig.2. For $\lambda_{c\phi}$ approaching infinity (c = 0), the critical slip surface is a plane parallel to the surface of the slope. This plane parallel to the surface of the slope, and passing through the toe of the slope, may be thought of as the shallowest possible slip circle, which has infinite radius. For this special condition all of the methods described in Tables II and III give the same value of $F$, which may be shown to be:

$$F_{(c=0)} = \tan\phi/\tan\beta \qquad (5)$$

for a slope with no pore pressures.

*Definition of $r_u$*

The pore pressures within a slope affect its stability, and they also affect the relative values of factor of safety calculated by various methods. For purposes of examining the effects of pore pressure, it is convenient to examine conditions in which the ratio of the pore pressure to the overburden pressure is constant throughout the slope. This ratio, denoted by the parameter $r_u$ (Bishop, 1954) is defined as:

$$r_u = u/\gamma h \tag{6}$$

in which $r_u$ = pore pressure ratio, $u$ = pore pressure at a point, $\gamma$ = total unit weight, and $h$ = depth of overburden at the point. Assuming that $r_u$ = constant (or that the pore pressure is a constant fraction of the overburden pressure throughout the slope) is a simple and reasonable means of approximating the distribution of pore pressures in embankments at the end of construction, or within natural slopes or excavations for some conditions of seepage. Comparing factors of safety calculated by various methods for $r_u$ = constant thus provides results which are relevant to practical conditions involving non-zero pore pressures.

*Comparison of minimum values of $F$*

A number of homogeneous slopes varying from 1.5 on 1 (horizontal on vertical) to 3.5 on 1, with values of $r_u$ varying from 0.0 to 0.6 were analyzed using each of the methods described in Tables I and II. The results for the 3.5 on 1 slopes are summarized in Table V (for $r_u$ = 0) and Table VI (for $r_u$ = 1). In each case the values of $c$, $\phi$, $\gamma$ and $H$ used in the analyses were adjusted so that $F$ = 1.00 for the log spiral method. Because the log spiral method is not a method of slices, and because it satisfies all conditions of equilibrium, it provides a convenient basis for comparison.

On the basis of these studies, a number of conclusions have been reached regarding the accuracy of these equilibrium methods of slope stability analysis:

(1) Methods which satisfy all conditions of equilibrium (log spiral, Janbu's, Spencer's, and Morgenstern and Price's methods) all give essentially the same value of $F$. Studies of non-homogeneous slopes and dams, and non-circular slip surfaces, show a slightly wider disparity in the values of $F$ calculated by these methods. These studies indicate that for any practical slope stability problem, any method which satisfies all conditions of equilibrium will give a value of $F_{min}$ which differs by no more than ±5% from what may be considered the 'correct' answer. Thus, although there is no mathematical proof that the values of $F$ calculated by Janbu's, Spencer's, and Morgenstern and Price's methods are rigorously correct, from a practical point of view there is no doubt that they may be considered to be correct for all practical purposes.

(2) Bishop's modified method, which does not satisfy all conditions of equilibrium, gives virtually the same value of $F$ as methods which satisfy all

TABLE V

Minimum values of $F$ for a 3.5 on 1 (horizontal on vertical) slope with $r_u = 0$

| Analysis procedure | $\lambda_{c\phi}$ | | | | | |
|---|---|---|---|---|---|---|
| | 0 | 2 | 5 | 8 | 20 | 50 |
| Log spiral | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ordinary method of slices | 1.00 | 0.94 | 0.94 | 0.95 | 0.96 | 0.98 |
| Bishop's modified method | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Force equilibrium (Lowe and Karafiath's assumption) | 1.09 | 1.02 | 1.01 | 1.00 | 1.00 | 1.00 |
| Janbu's generalized procedure of slices | 1.00 | — | 1.00 | — | 1.00 | 1.00 |
| Spencer's procedure; also Morgenstern and Price's procedure with $f(x)$ = constant | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |

TABLE VI

Minimum values of $F$ for a 3.5 on 1 (horizontal on vertical) slope with $r_u = 0.6$

| Analysis procedure | $\lambda_{c\phi}$ | | | | | |
|---|---|---|---|---|---|---|
| | 0 | 2 | 5 | 8 | 20 | 50 |
| Log spiral | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Ordinary method of slices | 1.00 | 0.91 | 0.75 | 0.68 | 0.57 | 0.50 |
| Bishop's modified method | 1.00 | 1.00 | 1.00 | 1.00 | 0.99 | 0.99 |
| Force equilibrium (Lowe and Karafiath's assumption) | 1.09 | 1.03 | 1.02 | 1.01 | 1.00 | 1.00 |
| Janbu's generalized procedure of slices | 1.00 | — | — | — | — | — |
| Spencer's procedure; also Morgenstern and Price's procedure with $f(x)$ = constant | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |

conditions of equilibrium. Thus, for analyses of circular slip surfaces, no more elaborate method need be used to insure that the errors arising from the mechanics of the method will be small.

(3) The ordinary method of slices, which is also applicable only to circular slip surfaces, gives values of $F$ which are lower than those calculated by more accurate methods. For $r_u = 0$ conditions (in practical terms these would be total stress analyses), the inaccuracy is no more than a few percent, which is certainly tolerable for practical purposes. For effective stress analyses with high pore pressures, as shown by the results for $r_u = 0.6$ in Table VI, the inaccuracy may be as much as 50%. Thus, while the ordinary method of slices may be applied to total stress analyses, it should not be used for effective stress analyses with high pore pressure.

(4) For $\phi = 0$ conditions ($\lambda_{c\phi} = 0$ in Tables V and VI), any method which satisfies moment equilibrium around the center of a circular slip surface will give the correct value of $F$ for this condition, regardless of what other equilibrium conditions it does or does not satisfy. This is true because: (a) for $\phi = 0$, shear strength is independent of the normal stress on the slip surface (see eq.2); and (b) the average value of shear stress required for equilibrium of a free body bounded by a circular arc is determined completely and uniquely by the equation of moment equilibrium around the center of the circle. Thus the ordinary method of slices, Bishop's modified method, Janbu's GPS, Spencer's method, and Morgenstern and Price's method all give exactly the same value of $F$ for circular slip surfaces and $\phi = 0$ conditions.

(5) Values of $F$ calculated using procedures which satisfy force equilibrium only are sensitive to the assumed inclination of the side forces between slices. The assumption proposed by Lowe and Karafiath (side forces inclined at the average of: (a) the ground surface slope; and (b) the slope of the slip surface) appears to be quite accurate over a wide range of conditions. As shown in Table V and VI, this assumption is least accurate for $\lambda_{c\phi} = 0$, in which case the value of $F$ is about 10% larger than the correct value. The more steeply inclined the side forces are assumed to be for $\phi = 0$, the more inaccurate the calculated value of $F$. If, for example, the side forces are assumed to be parallel to the ground surface, the value of $F$ may be as much as 30% too high.

## SUMMARY

All equilibrium methods of slope stability analysis involve assumptions, because there are fewer equations of equilibrium than there are unknowns in the case of either a circular or non-circular slip surface, with the mass above divided into slices. Furthermore, some of the methods which are frequently used for practical purposes (the ordinary method of slices, force equilibrium procedures, and Bishop's modified method) do not satisfy all of the conditions of equilibrium.

Under some circumstances methods which do not satisfy all conditions of equilibrium may be highly inaccurate. The ordinary method of slices may give values of $F$ which are 50% smaller than the correct value if used for effective stress analyses of slopes with high pore pressures. Force equilibrium procedures may give values of $F$ which are 30% larger than the correct value if used for $\phi = 0$ analyses with steeply inclined side forces between slices.

However, methods which satisfy all conditions of equilibrium give accurate results for all practical conditions. Regardless of the assumptions they employ, these methods (Janbu's, Spencer's, and Morgenstern and Price's methods) give values of $F$ which differ by no more than ±5% from the correct answer. Bishop's modified method is also equally accurate, even though it does not satisfy all conditions of equilibrium.

Based on these findings, equilibrium methods of stability analysis can be selected which can be relied on to produce results which involve no more than ±5% inaccuracy as a consequence of the approximations made in treating the mechanics of the problem. When this is the case, the engineer performing the analysis is justified in considering the factors of safety he calculates to be 'correct' in terms of the mechanics of the problem, and he can devote his attention and concern to accurate evaluation of the properties of the soils.

## ACKNOWLEDGMENTS

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# REPAIR OF SMOKEY LANDSLIDE USING A TIED-BACK WALL

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Abstract. In 1984, a landslide occurred in Scott County, Tennessee which caused a county road and a railroad to be temporarily blocked with debris. The landslide occurred in weathered shale which is transected by at least three high-angle joint sets. The ground movement appeared to have been initiated by a high ground water level which was being recharged by surface infiltration from an area that was disturbed by previous coal strip mining activities. In response to this emergency, The Office of Surface Mining Reclamation and Enforcement (OSMRE) initiated steps to stabilize the landslide and repair the county road.

Presented herein is a case history of the Smokey Landslide and the tied-back wall which is currently being constructed to repair the slide. As part of this construction, horizontal drains are being installed to lower the phreatic level within the hillside and thereby reduce the pressure that must be resisted by the tied-back wall. The construction of the tied-back wall and horizontal drains will enable the landslide to be stabilized while maintaining traffic on the county road.

## Introduction

In 1977, Public Law 95-87 (Surface Mining Control and Reclamation Act) was enacted by Congress to regulate coal mining in the United States. Moreover, the Office of Surface Mining Reclamation and Enforcement was established to administer the requirements set forth by Public Law 95-87. Specifically, any company engaged in active mining performed after that time was required to have an adequate bond whereby reclamation of the specific site could be assured. However, thousands of mine sites were abandoned prior to the enactment of Public Law 95-87. At many of these sites, little or no reclamation was performed prior to abandonment. In addition to regulating active coal mining, OSMRE was charged with reclaiming abandoned mine lands that present an emergency condition. Funding for the reclamation of these previously abandoned sites will be obtained from a reclamation fee paid by companies actively mining coal.

One abandoned mine site which qualified for reclamation under Public Law 95-87 was the Smokey Landslide. Based on the available historic data, County Road 191 and a railroad were constructed in the early 1900's between Norma and Smokey Junction, Tennessee. Due to the steep terrain and the proximity of the New River, the county road and the railroad had to be partially excavated into a natural hillside composed primarily of weathered shale. In the early 1970's, an area situated several hundred feet uphill from these existing roads was disturbed by coal strip mining activities. In the course of the mining operations, overburden was blasted and surface drainage was rerouted. In 1984, a landslide occurred in the area between the surface mined area and the county road and this slide temporarily blocked the county road and the railroad.

This paper presents a case history of a permanent tied-back, soldier beam and concrete lagging wall which is currently being constructed to stabilize the Smokey Landslide. Record low precipitation during the design phase of the project required that some conservative assumptions be made with regard to identifying the failure mechanism and assessing the required capacity of the tieback anchors. However, the number of deep joints in the weathered shale in the hillside, as revealed during construction, indicates that this conservative approach was prudent.

## Site Conditions

The Smokey Landslide is located approximately one mile southeast of Norma, in Scott County, Tennessee. As shown on Figure 1, the Southern Railway System main line and Scott County Road 191 converge at a bend in the New River at this location. Both the Scott County Road and the Southern Railroad were constructed by a combination cut/fill; excavations for the northern side of County Road 191 and the northern side of the railroad exposed weathered shale bedrock.

The average ground slope in the area of the slide is at an inclination of approximately 1 horizontal to 1 vertical, but some areas have near vertical rock faces up to 30 feet in height. Also, a large isolated pinnacle of bedrock existed in the slide area near the county road as shown on Figure 2. The drainage area above the limits of the landslide is about 3 acres. Near its upper limits, the area was disturbed by previous strip mining activities which consisted of an access road and pits which were probably excavated for coal exploration purposes.

## Subsurface Conditions

### Geologic Setting

The area of the landslide is located within the Cumberland Plateau physiographic province. Generally, the bedrock of the Cumberland Plateau consists of a thick sequence of interbedded sandstone and shale with thin beds of bituminous coal. Although some portions of the Cumberland Plateau have been folded and faulted in the geologic past, the majority of the region consists of tectonically undeformed, relatively flat-lying strata such as those found in the area of the Smokey Landslide.

### Soil

The soil cover in the area uphill from County Road 191 averages less than five feet in thickness. However, rockfill debris exists to depths of up to 20 feet at a few locations. The natural soil in the slide area consists of a mixture of brown silt and shale fragments, with the percentage of shale fragments increasing with depth. The area immediately downhill from County Road 191 in the area of the slide is underlain by as much as 20 feet of both soil fill and rockfill overlying a thin layer of residual soil.

### Bedrock

The cored holes and geologic mapping at the site indicate that the bedrock consists mainly of shale with minor sandstone beds and traces of coal. The published geologic map shows that the bedrock is a portion of the Slatestone Formation of Pennsylvanian Age. The bedrock has a slight dip to the north and east of about 1%, with no evidence of faulting shown on the published geologic map or in the bedrock cores.

Weathering of the bedrock occurs along bedding planes, high-angle fractures (joints) and the surface portions of the bedrock. Generally, only the upper two to three feet of the bedrock interval cored contained numerous weathered bedding plane weaknesses. However, weathering along high-angle fractures (joints) is more extensive; several of the exploratory holes transected such features and show that weathering along these joints extends to depths of at least 50 feet below the bedrock surface. These high-angle fractures are probably remnants of submarine slumping which occurred prior to complete rock lithification, wherein the partially lithified sediment was fractured and separated slightly. These fractures were then filled with sediment subsequent to complete lithification.

As shown on the generalized geologic cross-section through the slide area (Figure 3), the orientation of the near vertical fractures within one of these joint sets is parallel to the face of the slide. Also, one joint set is oriented perpendicular to the face of the slide. Such features are a likely focus for lateral movement to occur, because the shear strength in these near-vertical weaknesses is much lower than the shear strength of the unfractured bedrock.

### Ground Water

Ground water levels recorded during the period from August, 1984 to August, 1986 showed large variations with time. Further, there were marked differences

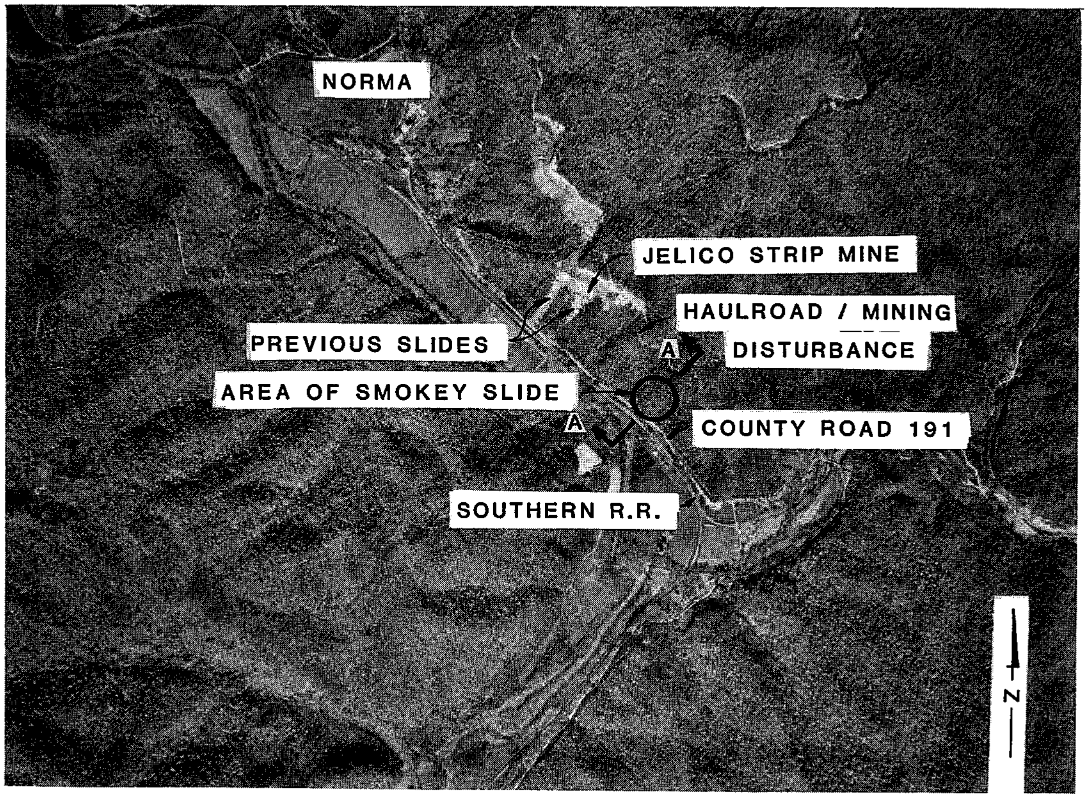


Figure 1. Aerial photograph taken in November 1971 during strip mining of coal in area above eventual Smokey Landslide.

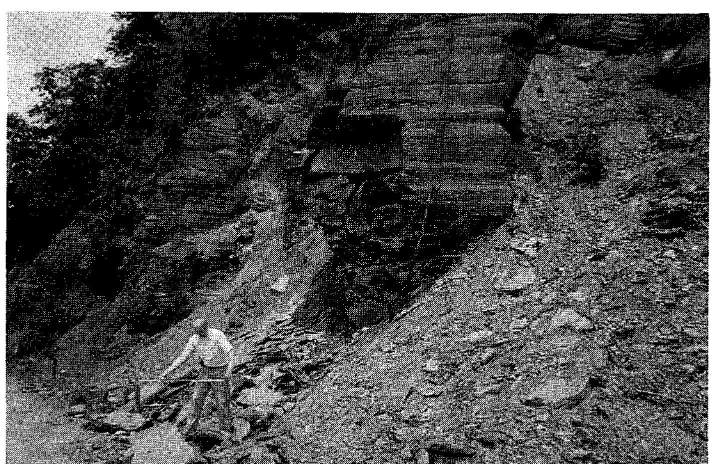

Figure 2. Photograph of Smokey Landslide uphill from County Road 191.

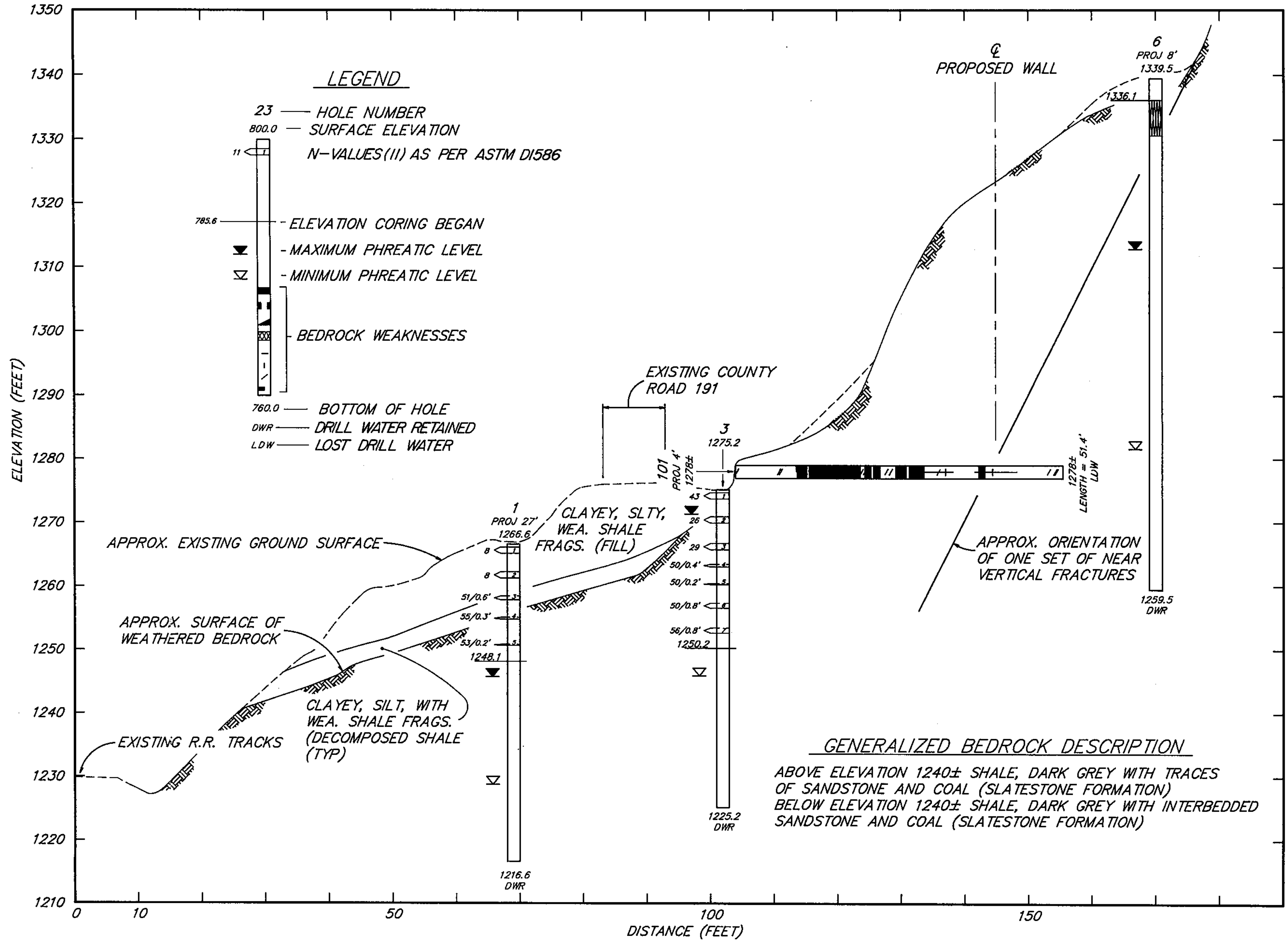


Figure 3. Generalized Geologic Section A-A through slide area.

in ground water elevations between adjacent borings on the same date. Specifically, ground water levels within holes drilled along the existing roadway ranged from about five feet to greater than 50 feet in depth beneath the ground surface. Also, following periods of intense rainfall, water levels in some of the holes increased significantly and water was observed to be issuing from fractures exposed in the bedrock above the road. The borings which had large fluctuations in water levels probably penetrated more permeable zones (i.e. - bedding plane weaknesses and joints), whereas those borings that showed little variation in water levels probably penetrated bedrock with few weaknesses.

## Cause Of Failure

The Smokey Landslide appears to be two separate landslides, each with a different mechanism of failure. The section of the landslide situated uphill from the county road is moving along one of the high-angle fractures which is oriented parallel to the face of the slide as shown on Figure 3. Based on the monitoring of several inclinometers, piezometers, one extensometer and subsequent stability analyses, the instability is due primarily to:

(1) the road excavation being made in an area where the shale strata have pre-existing, high-angle weaknesses which are oriented in the direction of the slide; and

(2) high ground water levels which develop during extended periods of rainfall.

To a lesser degree, vibrations from the previous mining operations (i.e. - during blasting) could have enlarged the natural, near-vertical weaknesses in the bedrock, thereby making the subject area more susceptible to water infiltration. Regardless of whether or not the vibrations enlarged the fractures, the quantity of ground water within the slide appears to have increased significantly due to previous mining activities uphill from the slide.

The section of the landslide situated downhill from the county road appears to be moving along the contact between the fill and natural soil as shown on the previous Figure 3. The relatively steep outslope downhill from the county road is probably due to rockfill debris from the upper section of the slide being removed from the county road at various times and being dumped over the outslope. High ground water levels which develop during extended periods of heavy rainfall probably contributed to the movement in the area downhill from the county road.

To assess the respective effects of ground water infiltration and the fracturing of the strata on the instability, a series of stability analyses were performed for a variety of conditions. In the first analysis, the factor of safety of the slide was assessed based on measured shear strength data for the weathered shale, the orientation of joints within the bedrock, and published shear strength data for the materials encountered along the probable failure zone. Using: 1) an effective angle of internal friction of 32° for both the natural, fractured shale strata (uphill of the county road) and the combination of spoil-residual soil (beneath the county road), and 2) the maximum phreatic levels recorded in the piezometers, a factor of safety of about 1.2 was calculated. Considering that the factor of safety of the slide had to be less than 1.0 for failure to occur, there must have been an additional contributing factor to the instability such as a periodic high phreatic level. Unfortunately, the piezometric levels were recorded during one of the dryest periods of record, whereas the landslide occurred during an earlier period of intense rainfall. As evidenced by the large quantity of water which was observed to be emerging from seeps in the bedrock during the occasional periods of intense rainfall when the landslide was being studied, it is likely that a high phreatic surface could have developed during extended heavy rainfall events. Moreover, the phreatic level could have risen to a level which resulted in instability. This assessment was verified by the instrumentation which showed movement only after the occasional intense periods of rainfall.

## Potential Remedial Measures

Because the landslide was caused, in part, by past coal mining activities, the Office of Surface Mining Reclamation and Enforcement initiated remedial design and construction measures. In developing these remedial measures, the following criteria were used:

(1) the remedial measure must have a factor of safety of at least 1.5 for static conditions and 1.2 for earthquake conditions;

(2) the remedial measure must be able to be implemented while maintaining traffic on the county road and the Southern Railroad;

(3) as much as practical, the selected remedial measure should require minimal or no future maintenance; and

(4) if the above criteria are met by more than one remedial measure, then the option with the least cost should be selected.

Accordingly, several potential remedial measures were examined for the Smokey Landslide, including the following:

(1) relocation of the county road above or below the slide;

(2) subsurface drainage improvements;

(3) slope reduction with subsurface drainage improvements;

(4) gravity buttress with subsurface drainage improvements;

(5) tied-back reinforced gunite facing with subsurface drainage improvements; and

(6) tied-back soldier beam and concrete lagging wall with subsurface drainage improvements.

Alternative 2 was eliminated because the required factor of safety could not be achieved. Alternative 4 was eliminated because an excavation would be required which would present a hazard to the county road before the gravity buttress could be constructed. Alternatives 1, 3, 5, and 6 met the initial requirements for the remedial measures; however, the tied-back soldier beam and concrete lagging wall with subsurface drainage improvements was selected based on cost assessments of the various options. Details of the selected alternative are included in the following sections.

## Design Of Permanent Tied-Back Wall

One of the significant advantages of intalling a tied-back wall to repair a landslide is that the stabilization can begin at the top of the slide and then progress in a downward direction. In this manner, only a relatively small excavation is required before installation of tieback anchors begins. Moreover, the tieback anchors can be post-tensioned during construction to verify their capacity before the excavation continues downward to the next level of tieback anchors.

As shown on Figure 4, the Smokey Landslide will be repaired by constructing a tied back wall according to the following construction sequence:

(1) excavating a bench into the hillside near the top of the slide to provide access for construction equipment;

(2) pre-drilling holes on eight feet centers and installing wide-flanged steel soldier beams to their ultimate depths;

(3) installing precast reinforced concrete and/or cast-in-place reinforced shotcrete lagging between the flanges of the soldier beams;

(4) installing high capacity tieback anchors above the level of the excavated bench;

(5) backfilling behind the wall and then post-tensioning the upper tieback anchors;

(6) excavating down to the next level of tieback anchors and repeating the steps outlined above until the wall is completed; and

(7) installing horizontal drains and weep holes at various levels during the construction.

After the tied-back wall is completed, the county road will be relocated and the steep outslope downhill of the county road will be regraded as shown on Figure 4.

The first step in the design of the permanent tied-back wall was to develop an earth pressure diagram. Based on the small magnitude of movement recorded in the inclinometers, a definite failure plane within the bedrock was difficult to delineate. This small magnitude of recorded movement was probably due to the record low precipitation levels during the monitoring period. Therefore, the decision was made to develop the pressure diagram based on the conservative assumption that the wall would retain weathered shale which was essentially a cohesionless material with an effective angle of internal friction of 32°. Considering the numerous high-angle joint sets in the weathered shale, this conservative approach seemed prudent.

In the design of the wall components, a rectangular apparent earth-pressure envelope was used based on the following formula suggested by Terzaghi and Peck for open cuts in sand:

$$p = 0.65\, K_A\, \gamma H \qquad (1)$$

where:

$p$ = lateral earth pressure
$K_A$ = the Rankine active earth pressure coefficient
$\gamma$ = unit weight of the material behind the wall
$H$ = height of the wall.[1, 2]

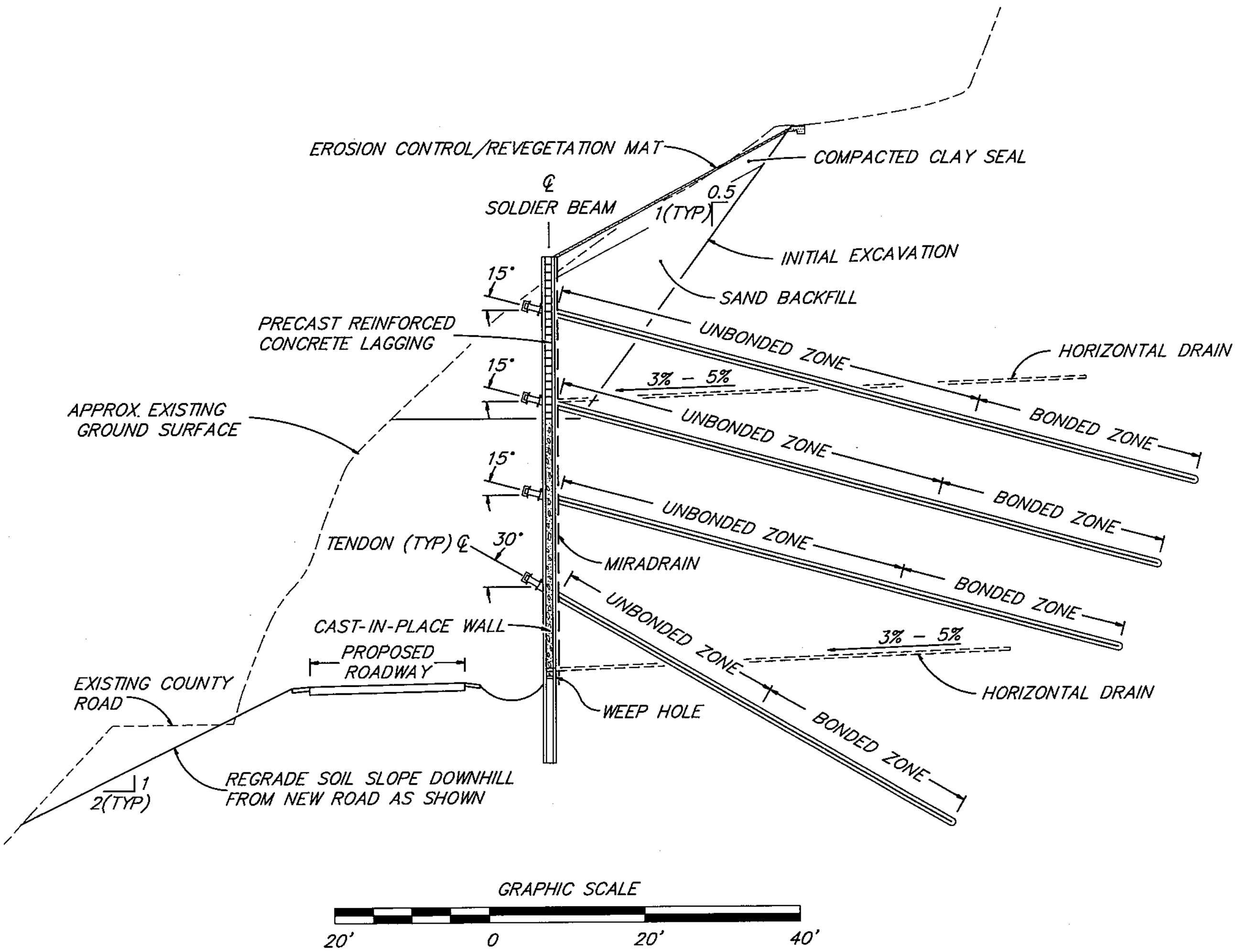


Figure 4. Typical tied-back wall section.

The lateral pressure due to the surcharge of the sloping backfill was also added to the pressure diagram. Horizontal drains will be installed into the bedrock to control the ground water level behind the wall. Also, a synthetic vertical drain will be installed behind the wall to control seepage forces caused by percolating water during rainstorms. Consequently, no hydraulic pressure was added to the pressure diagram used to design the wall.

The height of the wall ranges from six to 52 feet and the required working capacity of the tieback anchors ranges from 51 to 241 kips. The allowable working capacity of a specific tieback was determined by the equation:

$$P = \pi D \, l_a \, \tau_{ult} \div FS \qquad (2)$$

where:

- $P$ = allowable working tieback capacity
- $D$ = anchor diameter
- $l_a$ = anchor length
- $\tau_{ult}$ = ultimate rock-grout bond stress
- $FS$ = Factor of Safety of 2.

An ultimate rock-grout bond stress of 50 psi was used in the design which was based on the anchor being installed in soft shale.[3] Due to the potentially corrosive environment (i.e. - the shale contains some pyrite) in which the tiebacks will be installed, the strand tendons will be completely encapsulated by grouting them into corrugated plastic tubes. The unbonded length of each tieback will also be greased and sheathed for additional corrosion protection.

After determination of the required capacity for the various tieback anchors, several stability analyses were performed using the tieback subroutine of the STABL4 computer program which was developed by Purdue University.[4] The overall factor of safety of the wall was assessed based on: (1) a block failure mode using various joint orientations and failure planes, and (2) random circular failure surfaces. The minimum calculated factors of safety for static and earthquake failure modes were about 1.6 and 1.4, respectively.

In preparing the construction drawings and specifications for the wall, specific details were provided for each component of the system. However, provisions were made in the contract documents to allow the selected contractor to propose modifications provided that the modifications met certain performance standards and were approved by the designers. In this manner, the contractor will be able to use any patents or innovative techniques which he has developed. Finally, although minimum requirements for the tieback anchors were contained in the specifications, the contractor will be held responsible for replacing, at his own expense, any anchors which fail upon stressing.

## Construction

The initial phase of construction consisted of the excavation of a bench to accommodate the construction equipment. The excavation was made along one of the high-angle fractures oriented parallel to the face of the slide and resulted in a relatively smooth, rock face as shown on Figure 5. After the bench was excavated, a test tieback anchor was installed and tested to verify the estimated rock-grout bond stress used in the design. The anchor was stressed to 200% of its design load without failure, using two 150 ton hydraulic jacks as shown on Figure 5.

Boreholes for the soldier beams were then drilled ten feet below the proposed finished grade elevation of the new roadway as shown on Figure 6. The soldier beams were then installed and the bottom ten feet of each borehole was backfilled with concrete having a minimum compressive strength of 3000 psi. The remainder of each borehole was backfilled with concrete having a minimum compressive strength of 500 psi to provide a suitable bearing surface behind the soldier beams. This lower strength concrete above the embedded section will enable the concrete between the flanges of the beams to be removed as successively lower levels are excavated for the wall.

Following installation of the soldier beams, drilling was started for the tieback anchor holes as shown on Figure 7. Also, precast reinforced concrete lagging panels were installed between the flanges of the soldier beams as can be seen on Figures 8 and 9. The wall segment below the construction bench shown on Figure 9 will be constructed using cast-in-place reinforced shotcrete lagging.

Installation of the production tieback anchors has been complicated by two factors. First, blockage of the drilled anchor holes has occurred. This blockage was probably caused by caving of the holes within the weathered joints. Second, large volumes of grout have been required in installing the tieback anchors which is indicative of the extensive network of open cavities or fractures within the bedrock. The presence of the numerous openings in the bedrock verifies that the conservative design approach, as previously described, was prudent.

Figure 5. Photograph of stressing of test tieback anchor to 200% of its design load.

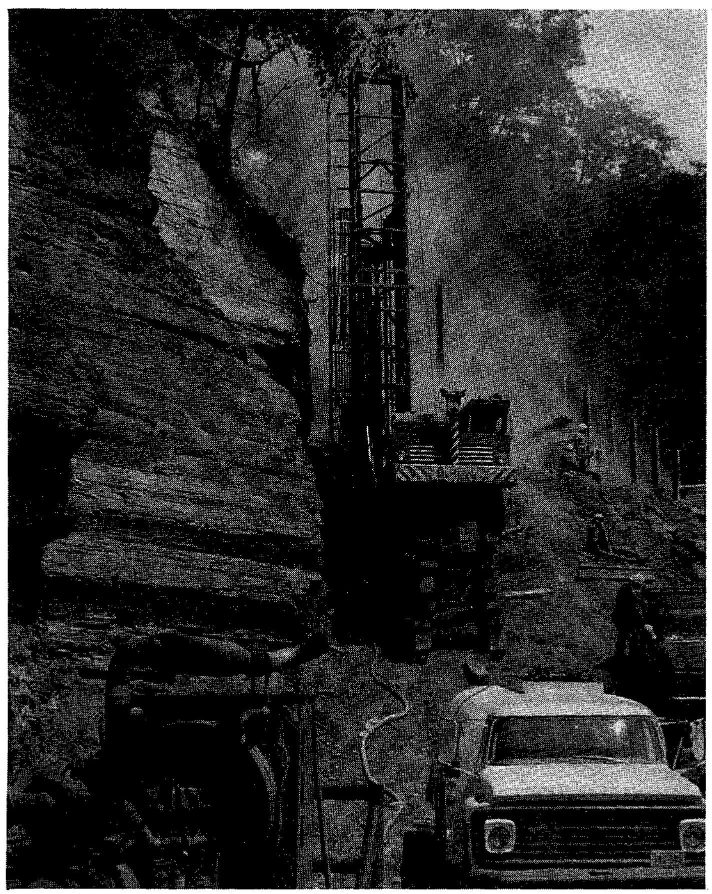

Figure 6. Photograph of drilling of holes for soldier beams.

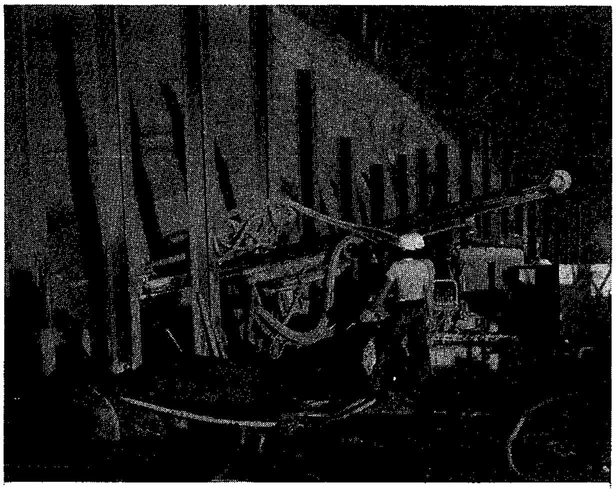

Figure 7. Photograph of drilling of production tieback anchor.

Figure 8. Photograph of grouting of production tieback anchor.

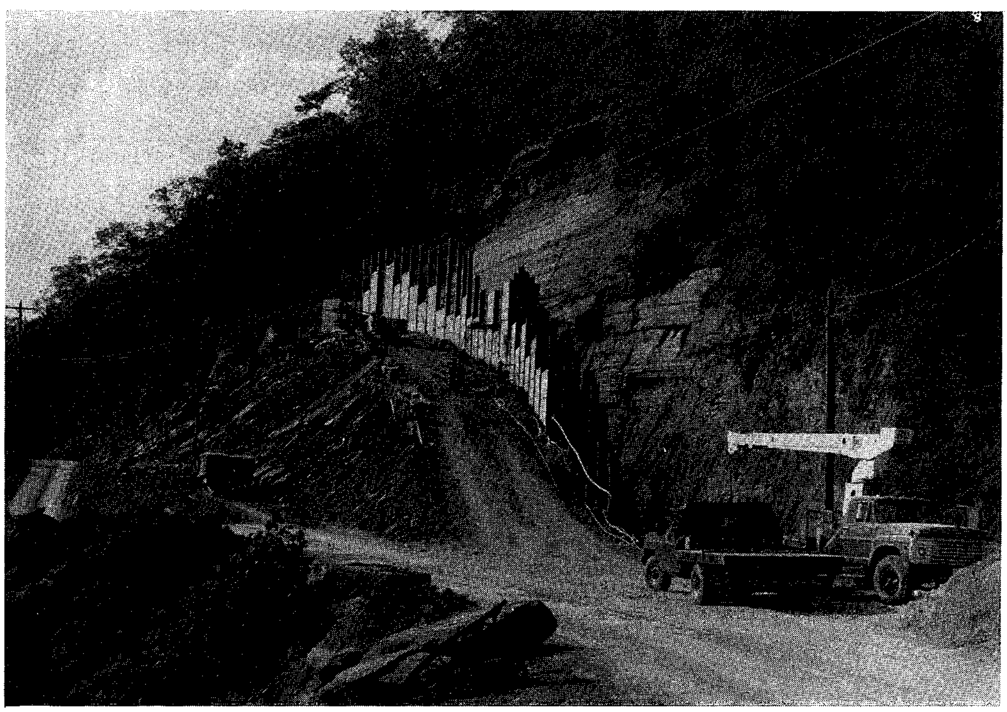

Figure 9. Photograph of partially completed tied-back wall being constructed while maintaining traffic on County Road.

At the time of this writing, none of the production tieback anchors had been stressed. However, after the production tieback anchors have been grouted in place, and after the wall has been backfilled with sand to an elevation above the tiebacks, each anchor will be stressed to at least 133% of its design load. Five percent of the production tieback anchors will be stressed as test anchors to 150% of their design loads. Finally, all of the anchors will be "locked off" at 80% of their design working loads.

## Conclusions

The construction of a tied-back wall can be a cost effective, permanent remedial measure for stabilizing a landslide. In areas where highways or railroads are involved, the construction of a tied-back wall has the added advantage of allowing the road to remain operational while the remedial stabilization is being performed. As with any earth retaining structure, the construction specifications should contain provisions to enable modifications to be made by the designers based on conditions that are revealed during construction.


## Acknowledgements

The tied-back wall at the Smokey Landslide is being constructed using Abandoned Mine Land Reclamation funds as administered by the U.S. Department of Interior, Office of Surface Mining Reclamation and Enforcement. Geologic Associates, Inc., of Knoxville, Tennessee, serves as the design and construction quality control consultant for the project. Kerr Construction, Inc., of Elmwood Park, New Jersey, is constructing the tied-back wall.


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# DESIGN OF HIGHWAY EMBANKMENTS ON UNSTABLE NATURAL SLOPES

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## ABSTRACT

Sections of the new Alexandria-Ashland (AA) highway in northeastern Kentucky must be located on and through natural slopes of the Kope and Crab Orchard Formations that consist mainly of shales. Numerous highway embankment and cut slope failures have occurred in these two geologic formations in past years. The shales of these formations have very poor and undesirable engineering properties. Many stability problems encountered with the Kope and Crab Orchard shales are caused primarily by the tendency of those shales to breakdown when exposed to water and produce clays and clayey silts of relatively low shear strengths. Construction of highway embankments with, through, and on the shales of the Kope and Crab Orchard Formations has been necessary because of the vast aerial presence of these shales and the lack of more suitable and economical alternate construction materials. In the design of the AA highway, geotechnical engineers were faced with three difficult problems. First, in forming embankments, there was a question of how the shales should be compacted and what constituted proper compaction. Second, the most difficult problem, perhaps, was the selection of appropriate shear strength parameters of the overconsolidated clays and clayey shales of the natural slope foundations and the embankments constructed of the shales. Numerous landslide studies involving overconsolidated clays and clay shales show that use of peak strengths from triaxial tests may lead to underconservative designs while use of residual shear strengths may lead to uneconomical designs. A third factor complicating the design of shale embankments on weathered shale slopes is the seepage of water into the embankment. Exposure of the lower portions of the embankments composed of Kope and Crab Orchard shales to seepage and rapid-drawdown conditions created by the Ohio River make the shales susceptible to breakdown and swelling and may produce a progressive "softening" and decrease in shear strengths. The design of certain sections of the AA highway passing through the Kope and Crab Orchard shales was further complicated by the fact that some natural slopes and existing highway fills having slopes of 3 horizontal to 1 vertical are failing. The design problem became one of placing new highway embankments on existing failing natural slopes. This paper presents case histories to document existing failures and discusses the treatment of the three factors -- compaction, shear strength, and seepage. A discussion of the different aspects of the geotechnical design for new fill placement is presented. Emphasis is placed on the selection and justification of design parameters for foundation and embankment materials as well as the construction procedures and compaction specification that were finally adopted.

## INTRODUCTION

The stability of natural slopes, cut slopes, and embankment slopes is a major concern to any highway designer. The Alexandria-Ashland (AA) Highway in northern Kentucky will cross two areas that historically have been difficult areas for both highway construction and maintenance activities due to slope stability and settlement problems (1-16). Weak clayey shales that comprise the bedrock in these two areas are the major contributor to the problem. Major problems which roadways have experienced in the two areas are catastrophic landsliding and significant settlements of several inches to several feet within embankment sections.

The purpose of the AA Highway is to provide a direct link between the Covington-Cincinnati area and the Ashland-Greenup area in northeastern Kentucky as shown in a plan view in Figure 1. The AA Highway begins in Campbell County at the interchange of I

* American Engineering Company is providing overall supervision of design and construction of the AA Highway for the Commonwealth of Kentucky. Geotechnical overview was provided by Mr. Bishop for American Engineering Company.

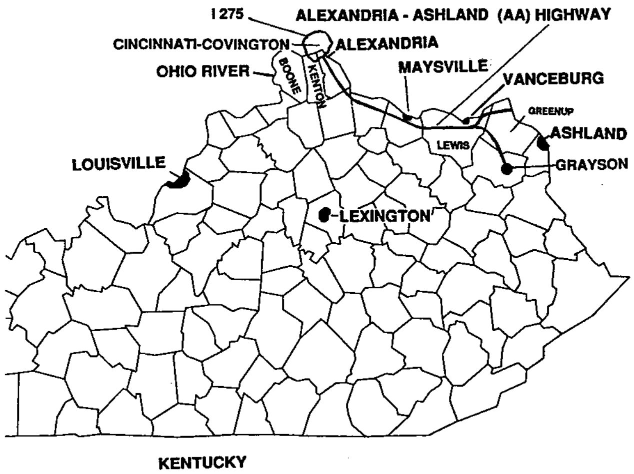


FIGURE 1. Location of the Alexandria-Ashland (AA) Highway in Northeastern Kentucky.

275 and KY 9 near Alexandria and extends eastward paralleling the Ohio River to Maysville and then to Vanceburg. Typically, the AA Highway is located about one to five miles south of the Ohio River. Near Vanceburg, the AA Highway splits, with one spur traversing due east to the Greenup Dam bridge, and the other spur traveling southeast to Grayson. Total project length is approximately 136 miles, of which 125 miles are new construction. The AA Highway will be a two-lane facility with truck passing lanes where required. In areas of heavy traffic, a four-lane roadway will be constructed.

Numerous large scale failures of shale embankments along major interstate routes in the early seventies prompted major research efforts by the Federal Highway Administration (FHWA), universities, and other governmental agencies. Kentucky experienced its share of failures on Interstates 64 and 75. The focus of the research was to develop guidelines for design and construction of new shale embankments as well as remedial measures for existing failing embankments. Incorporation of existing experience and research data into the design and construction of the AA Highway was a major project objective.

In the design of cut slopes and embankments the following factors must be considered:

* subsurface profile
* settlement properties
* shear strength properties
* surface and subsurface drainage
* construction specifications

The engineering performance of clayey shales in highway embankments and cut slopes has been demonstrated to be a function of the type of construction procedures used and a function of time. When first excavated, clayey shales appear and perform as rock. After exposure to weathering, the clayey shales break down, becoming a soil of low shear strength and high deformational properties. Overcoming the poor engineering properties of the clayey shales presented a design and construction challenge for engineers working on the AA Highway project.

## TOPOGRAPHIC AND GEOLOGIC SETTING

The alignment of the AA Highway crosses two areas of Kentucky where slope instability problems have severely affected highways and other structures. Northern Kentucky and southern Ohio counties surrounding Covington and Cincinnati have experienced slope stability problems since development began in that area. These problems are directly related to the rock types (the Kope and Fairview Formations) outcropping in the area and the topography. A second area to be crossed by the alignment where severely unstable natural slopes are present is in Lewis County. Between the communities of Tollesboro and Vanceburg, the Crab Orchard Formation outcrops and consists of clayey shale. Numerous natural slope failures and embankment and cut slope failures exist in this area.

The Kope and Fairview Formations outcrop in the Outer Bluegrass Physiographic Region of Kentucky. The area is a maturely dissected upland of rolling hills and small stream valleys. Elevations range from 500 feet above mean sea level adjacent to the Ohio River to 900 feet along ridge tops. Local relief ranges typically from 200 to 300 feet. Erosion has reduced the land surface to essentially an all-sloping landscape. Wide stream valleys are scarce.

The Kope and Fairview Formations are upper Ordovician in age and range in combined thickness from 260 feet to 370 feet. The Fairview overlays the Kope and comprises the ridge tops in many areas. The Kope Formation consists approximately of 80 percent shale and 20 percent limestone. The shale is gray to bluish or greenish gray in color, and laminations range from thick to thinly bedded. A few beds reach thicknesses of 6 feet. Slake durability indices (17-20, 8) are typically less than 50 percent and average in the 30's. Two types of limestone are present in the Kope Formation. About two-thirds of the limestone is medium gray in color, finely to coarsely crystalline, and bioclastic and occurs in indiscrete regular to irregular beds measuring as much as 12 inches in thickness. Generally, the beds are less than 8 inches in thickness. About one-third of the limestone is medium gray to dark gray in color, fine grained, argillaceous, and silty. This portion of the Kope occurs in beds measuring as much as 8 inches in thickness and commonly is laminated or cross-laminated. The limestones are interbedded with the shale throughout the unit, but interbeds occur somewhat more frequently near the top (21, 22).

The Fairview Formation consists of interbedded limestone and shale. Limestone comprises approximately 40 percent of the Formation near the base of the unit and about 65 percent near the top of the unit. The limestone is coarsely crystalline, sparry, and bioclastic and occurs in beds measuring as much as 3 feet in thickness. Generally, the beds of limestone are less than 8 inches in thickness. The shale is light bluish gray to medium gray in color and weathers olive gray to yellow. The formation is laminated to thinly bedded in beds less than 2 feet in thickness. Generally, the beds are less than 10 inches thick. Slake durability indices for the shale are typically less than 50 percent and average in the 30's. When exposed to weathering, shales of the Kope and Fairview Formations usually break down rapidly into soils that are plastic clays and silts with low shear strength (21, 22).

The Crab Orchard Formation outcrops at the juncture between the Outer Bluegrass and Knobs Physiographic Regions of Kentucky. The outcrop of Crab Orchard shales located in western Lewis County is approximately five miles wide. Topography of the area is a maturely dissected upland and consists of well-developed stream valleys. Elevations range from 600 to 850 feet above mean sea level near Tollesboro and 600 to 1100 feet at the eastern edge of the outcrop. Wide flat ridge tops are present at Tollesboro. Progressing eastward, the landscape becomes more mountainous and the local relief increases. Hillside slopes in the area are irregular and hummocky. Evidence of natural slope failures is numerous and indications of continual creep movements are prevalent. Two embankment landslide failures and numerous cut slope failures are present along (existing) KY. 10.

The Crab Orchard Formation is Middle Silurian in age and is differentiated into an upper and lower unit. The upper unit consists entirely of clayey shale and is the source of most slope instability problems. Thickness of the upper Crab Orchard ranges from 130 feet to 160 feet as mapped on the Tollesboro's and Charters' Geologic Quadrangle Maps (23, 24). This portion of the Crab Orchard consists totally of clayey shale. The upper unit is mainly greenish gray in color, but thin red and brown zones are present. The unit is laminated to thinly bedded, fissile to chunky, and when exposed to water expands and becomes very plastic. Slake durability indices are typically less than 40 percent and average in the low twenties and upper teens. A prominent characteristic of the lithologies associated with the Crab Orchard Formation is the overlying Bisher Limestone. It is a dolomitic limestone, light grey in color, fine to coarse grained in texture with irregular bedding. Thickness of this unit ranges from 0 to 70 feet. Many natural landslides occur on slopes just below the outcrop of the Bisher Limestone.

Soils along the alignment where the Kope and Fairview outcrop are primarily residual in origin. They are clays and silty clays that classify as ML, MH, CH, and CL by the United Soil Classification System. Thicknesses of these soils range from 1 foot or less on steep slopes to 10 or more feet on relatively flat terrain. Alluvial soils are present along major stream valleys. Thickness of the alluvium in the stream valleys was directly related to the distance from the Ohio River. The thicker alluvium is located immediately adjacent to the river.

Random and erratic areas of glacial soils exist in Campbell County on the alignment of the AA Highway. Thickness of these deposits range from a few feet to as much as 60 feet. The glacial deposits are the result of both Illinoian and Wisconsin glaciation. Glacial outwash, drift, lacustrine, and alluvial deposits are present.

Soils along the alignment in the area where the Crab Orchard outcrops are residual silts and clays that classify as ML, MH, CL, and CH by the United Soil Classification System. Thicknesses of these soils range from a few feet to more than 20 feet in some areas. In some areas, it is difficult to establish a soil-rock interface due to weathering of the shale. Only in major stream valleys is alluvium present.

## ENGINEERING PROPERTIES

### INDEX PROPERTIES

Typical engineering characteristics of the residual overburden soils and weathered clayey shales along the roadway foundation of the AA Highway are generally poor. Based on an analysis of 290 tests, average values of liquid limit and plasticity index of the residual soils and weathered clayey shales of the Kope and Fairview Formations were about 46 (±10) percent and 22 (±6) percent, respectively. Natural water contents of soils and the clayey shales in the rock disintegration zone generally averaged about 27.3 (±2.0) percent based on an analysis of 46 tests. However, natural water contents of the unweathered Kope shale typically are about 8 to 9 percent (8). Liquidity indices of the weathered materials are typically about 0.15 and range from about 0.5 to 0.0. Liquidity indices of the intact unweathered clayey shales are typically -0.7. Based on the Unified Soil Classification System and the AASHTO Classification System, the weathered soils and clayey shales predominantly classify as CL and CH and A-7-5 and A-7-6, respectively. Hence, the weathered soils and clayey shales of the Kope and Fairview Formations are generally moist to wet, firm, highly overconsolidated, plastic clays.

Based on an analysis of 63 tests performed on weathered overburden soils and clayey shales of the Crab Orchard Formation and geological formations similar to the Crab Orchard, the liquid limit and plasticity index of the weathered materials along the roadway foundation typically averaged 51 (±14) percent and 26 (±10) percent, respectively. Natural water contents of the weathered materials averaged 24 percent (based on 194 tests). A typical average value of liquidity index was 0.0. Natural water content of the unweathered intact clayey shales of the Crab Orchard Formation is typically 7.4 percent (8). Liquidity index of the unweathered shale is about -0.65. Average values of activity were 0.7 (±0.2). Percentages of clay particles finer than the 0.002-mm size averaged 38.4 (±11). Based on the Unified Soil Classification System and AASHTO Classification System, the weathered materials of the Crab Orchard and associated geologic formations generally classified as CL and CH and A-7-5 and A-7-6. Hence, the weathered materials were typically moist to wet, firm, highly overconsolidated, plastic clays.

## BEARING RATIO

Bearing ratios (CBR) of soaked specimens of the weathered soils and clayey shales of the Crab Orchard and Kope Formations, and similar formations, were generally very low. Soaked bearing ratios of weathered materials of the Kope Formation averaged about 3.4 (±1.0) percent. Soaked CBR values of the weathered materials of the Crab Orchard and formations similar to the Crab Orchard averaged about 6.6 (±4.5) percent. Frequently, CBR values were less than 3.0. CBR values of compacted shales from the Crab Orchard and Kope Formations, and reported elsewhere (25), were 1.9 and 2.0 percent, respectively. CBR values before soaking were 23 and 32 percent, respectively. Moreover, high swelling strains occurred during those tests. Maximum vertical strains due to swelling during soaking were 12.4 percent and 10.6 percent, respectively, for the Crab Orchard and Kope shales. The large reduction in bearing strengths and large increase in volume of these shales when exposed to water has caused many failures of pavements and embankments and illustrates the poor engineering properties of these shales.

## SLAKE DURABILITY

Slake-durability indices (SDI) (17-20, 8) of unweathered shales of the Kope and Crab Orchard encountered along roadway are typically in the middle of the intermediate range while indices of the weathered shales in the rock disintegration zone are typically less than 50 percent and classify as "soil-like." Typical jar slake numbers of these shales are 1 or 2, that is, the shales slake rapidly and breakdown immediately into soil when exposed to water.

## MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT OF NATURAL OVERBURDEN SOILS

Average values of maximum dry density and optimum moisture content as determined from AASHTO T-99 (26) of the weathered soils from the Kope and Fairview Formations were 104.3 (±3.2) pounds per cubic foot and 19.3 (±2) percent (based on an analysis of 42 tests). Average values of maximum dry density of the weathered soils of the Crab Orchard Formation (based on an analysis of 21 tests) were 102.5 (±5) pounds per cubic foot and 21 (±3.6) percent. In both cases, the weathered overburden soils of the Kope, Fairview, and Crab Orchard Formations frequently had natural water contents larger than optimum moisture contents. Hence, aeration of the overburden soils was required before compaction in many cases. Since the overburden soils were generally very thin and shallow (varying generally from 2 to 15 feet in thickness), major portions of the embankments will be constructed of unweathered shales of the Kope, Fairview, Crab Orchard, and similar Formations.

Results of standard compaction tests -- AASHTO T99 -- (reported elsewhere (27)) performed on shales from the Kope and Crab Orchard Formations yielded values of maximum dry density of 116.3 and 118.6 pounds per cubic foot and optimum moisture contents of 13 and 11.4 percent, respectively. Since natural water contents of unweathered shales of the Kope and Crab Orchard shales are typically about 7 to 8 percent, then additional water will be required to obtain the optimum moisture content. The addition of water to the shales before compaction, in areas of Kentucky where water is not plentiful, was one major economical factor that had to be considered and strongly influenced the formulation of embankment compaction specifications. The addition of water to the unweathered shales was desirable since the water would cause the shales to slake, breakdown, and facilitate compaction.

## SHEAR STRENGTH OF NATURAL OVERBURDEN SOILS

Based on 62 triaxial failure envelopes from tests (isotropically consolidated-undrained triaxial compression tests with pore-pressure measurements) on weathered, overburden soils and shales of the Kope Formation, average peak effective stress parameters, $\phi'_p$ and $c'_p$, were 28.7 (±3) degrees and 232 (±125) pounds per square foot (psf), respectively. Based on the percentage ($P_{0.002mm}$) of clay particles finer than 0.002-mm size, and a correlation ($\phi'_r = 68.2 - 30.2 \log(P_{0.002mm})$) (9), the estimated residual shear strength parameter, $\phi'_r$, for weathered soils derived from the Kope is 22 degrees. Based on one standard deviation, the estimated $\phi'_r$-values may range from about 19 to 27 degrees. Average peak effective stress parameters for the weathered overburden soils derived from the shales of the Crab Orchard Formation (based on four triaxial failure envelopes) were about 28 (±5) degrees and 43 (±51) psf, respectively. Based on the percentage of particles finer than 0.002 mm, and the correlation presented elsewhere (9), the estimated residual strength parameter, $\phi'_r$, is about 20 degrees. The $\phi'_r$-value, however, is estimated to range from 17 to 25 degrees.

## SHEAR STRENGTH OF COMPACTED SHALES

Shear strength parameters, $\phi'_p$ and $c'_p$, of compacted or remolded specimens of shales from the Kope and Crab Orchard Formations are shown in Figures 2 and 3 (27). In these series of tests, the shales were compacted at three different densities and compactive energies using modified compaction (ASTM D 1558-78), standard compaction (ASTM D 698-78), and a low-energy compactive effort. The specimens were compacted to maximum dry density and optimum moisture at each level of compactive energy. Isotropically consolidated-undrained triaxial tests with pore-pressure measurements were performed. Effective stress parameters, $\phi'_p$, obtained for the compacted shales of the Kope Formation, corresponding to the three different compactive energies, were 25.4, 26.6, and 26.2 degrees, respectively. Values of $c'_p$ were 958, 272, and 020 psf, respectively. Values of $\phi'_p$ obtained for the shales of the Crab Orchard Formation were 23.0, 23.9, and 24.5 degrees, respectively. Corresponding values of $c'_p$ were 1210, 768, and 550 psf. Variations of the parameters $\phi'_p$ and $c'_p$ with dry density are illustrated in Figures 2 and 3. While the $\phi'_p$-value changes only

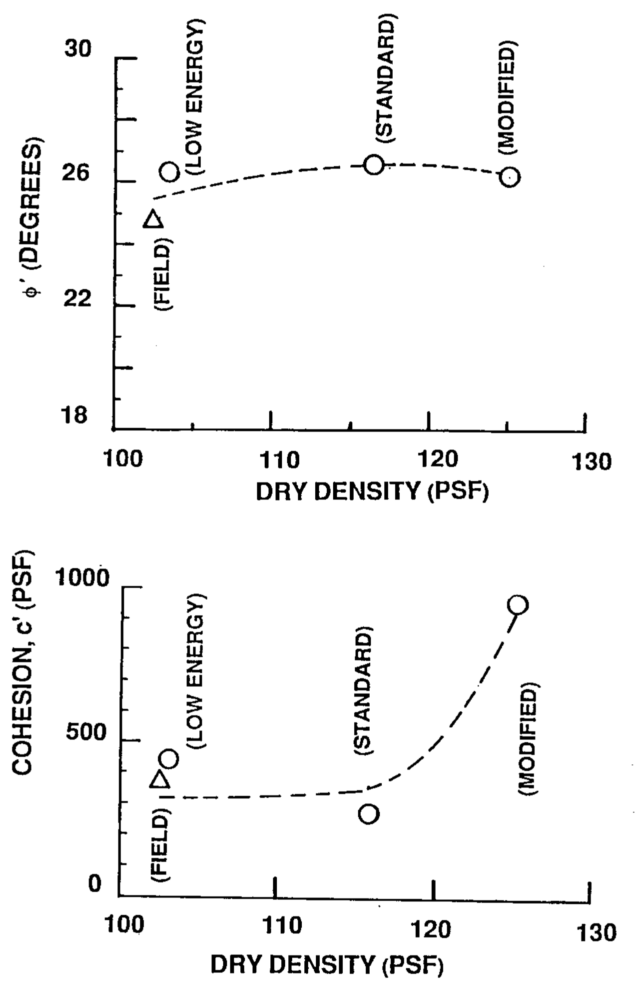


FIGURE 2. Variation of Effective Stress Parameters, $\phi'_p$ and $c'_p$, for Compacted Kope Shales as a Function of Dry Density (27).

slightly with an increase in dry density and compactive effort, the $c'_p$-value increases significantly as the compactive energy increases.

In 1981, extensive studies of numerous highway embankment failures on 75 in Boone, Grant, and Kenton counties were conducted (10). Embankments in these well-documented studies were constructed of materials from the Kope and Fairview Formations in the late 1950's and 1960's and have been in place some 20 to 25 years. Numerous settlement and stability problems have occurred. During the studies, some 336 consolidated-undrained triaxial compression tests with pore-pressure measurements were performed on undisturbed samples from numerous embankments experiencing settlement and stability problems. The average peak effective stress parameters, $\phi'_p$ and $c'_p$, obtained from the triaxial tests were 24.7 (±5.3) degrees and 368 (±252) psf. These parameters are similar to parameters obtained from triaxial tests (27) performed on remolded samples of the Kope shale compacted at standard compaction (ASTM D 698). Those tests yielded values of 26.6 degrees and 272 psf. Moreover, the average dry density obtained from 78 measurements of undisturbed samples from embankments on I 75 was 102.5 psf. Based on 139 tests, the average water content of the embankment (Kope) shales was 21.9

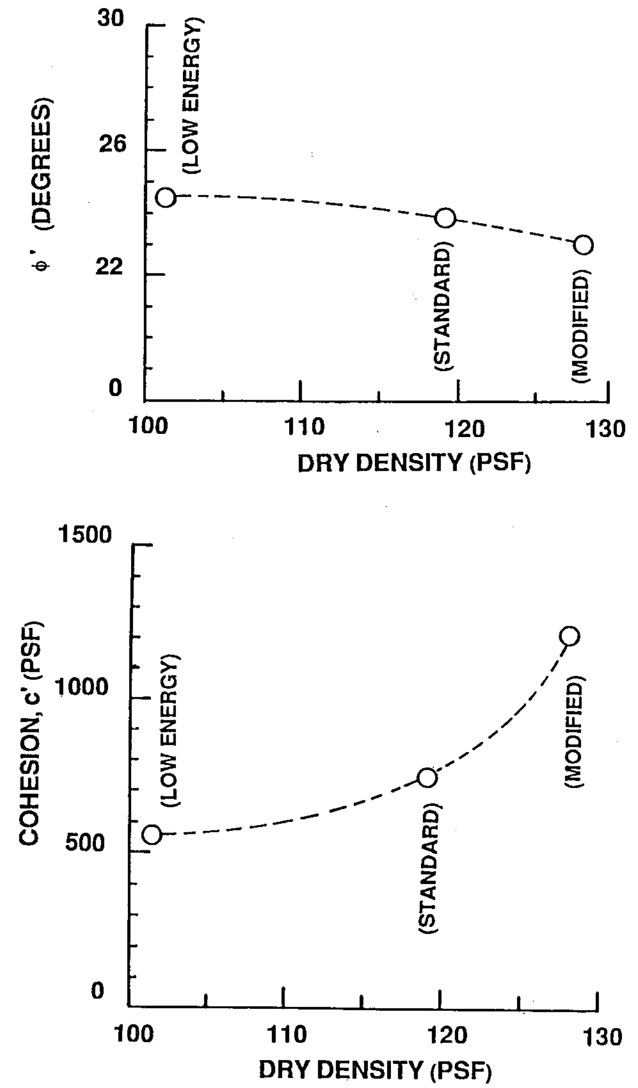


FIGURE 3. Variation of Effective Stress Parameters, $\phi'_p$ and $c'_p$, for Compacted Crab Ochard Shales as a Function of Dry Density (27).

percent. That value is similar to average values of maximum dry density and optimum moisture contents (ASTM D 698) obtained from tests performed on natural overburden soils in the Kope areas -- 102.5 (±5) psf and 19.3 (±2) percent. Hence, the long-term (>20 years) average dry density and optimum moisture content of embankments constructed of shales from the Kope and Fairview are about the same as the maximum dry density and optimum moisture content (AASHTO T 99) of the natural overburden soils. However, the in situ average dry density and moisture content are lower than the maximum dry density and optimum moisture content obtained from standard compaction tests on Kope shales. Based on those data, the relative compaction of the I-75 embankments was about 90 percent.

The low value of relative compaction is probably partly due to the fact that lift thicknesses up to 3 feet were permitted 20 years ago and partly due to swelling that occurred when the shales were subjected to seepage and surface infiltration of water. Also, the compactive effort was insufficient to breakdown mixtures of limestone and shales. Insufficient compaction left voids in the fill. As illustrated in Figures 2 and 3, increasing the compactive effort above standard compaction increases the c'-value and therefore increases the factor of safety and stability.

Although greater compactive effort may be used in the field, the question that arises is whether the cohesion, c', will decrease over a long period of time as relaxation and creep of the embankment occurs and seepage and surface water infiltrate into the embankment, which tend to negate high negative pore pressures imparted by compaction.

Past studies (28, 9) have shown that, when clays of highway embankments and foundations (natural overburden soils) have liquidity indices less than about 0.4, stability analyses based on either a total stress analysis or an effective stress analysis and based on peak shear strength parameters, $S_u$ or $\phi'_p$ and $c'_p$, respectively, usually yields unreliable results. Triaxial tests overestimate the in situ shear strength of the overconsolidated materials. This observation is based on a review of 75 national and international, well-documented geotechnical failures. For example, stability analysis (10) of several embankment failures on I 75 using the peak strength parameters, $\phi'_p$ and $c'_p$, obtained from an analysis of 336 triaxial tests yielded factors of safety ranging from 1.7 to 2.0. Liquidity indices of those embankment clays were less than 0.4. Consequently, because of uncertainties that arise in performing stability analyses of slopes and foundations (LI < 0.4) composed of overconsolidated plastic clays and clayey shales and based on past experience, the concept of reducing the peak shear strengths from triaxial tests was adopted in designing embankments on the AA Highway.

Guidelines based on slake-durability classifications for selecting design shear strength parameters, $\phi'_d$ and $c'_d$, contained in the Kentucky Geotechnical Manual (20) and NAVFACS (29) suggest the following strength parameters for shale embankments:

| | |
|---|---|
| Soil-like Shale (SDI < 50) | $S_u$ = 1,000 to 1,500 psf |
| | $\phi'$ = 20° to 25° |
| | $c'$ = 200 psf |
| Intermediate Shale (50<SDI<95) | $S_u$ = 1,000 to 1,500 psf |
| | $\phi'$ = 26° to 30° |
| | $c'$ = 200 psf |

Based on the past, poor performance of Kope and Fairview shales, and considering that these shales generally classify as soil-like (SDI < 50), design parameters of $\phi'_d$ equal 20 degrees and $c'_d$ equal to 200 psf were selected. These values are smaller than values for $\phi'_p$ and $c'_p$ obtained from compacted specimens of Kope shale (26.6 degrees and 272 psf) and average parameters ($\phi'_p$ equal to 24.7 degrees and $c'_p$ equal to 368 psf) obtained from an analysis of 336 triaxial tests performed on undisturbed specimens from several embankments constructed of Kope and Fairview shales, or

$$\tan \phi'_d \approx 3 \tan \phi'_p/4 \qquad (1)$$

and

$$c'_d \approx 3\, c'_p/5. \qquad (2)$$

To check the selection of the design strength parameters, two embankment failures on I 75 (slides identified in the report by Munson, et al. (10) as 90 and 95) were reanalyzed. Those embankments were constructed of shales from the Kope Formation. The embankment failures were reanalyzed using the ICES LEASE computer program (Bishop model (30)) and HOPK-I (a new stability model and computer program (31)), the shear surfaces as indicated from slope inclinometers and observed scarps, monitored pore pressures, and the reduced shear strength parameters. Factors of safety of 1.07 and 1.08 were obtained for Slide 95 from the ICES LEASE and HOPK-I programs, respectively. For Slide 90 factors of safety of 0.89 and 0.90 were obtained. Hence, the reduced shear strength parameters for the Kope shale embankments appeared to be a reasonable choice since the factors of safety were near 1.0.

Various other analyses were performed using average parameters obtained from 336 triaxial tests and parameters obtained from specimens compacted at different densities. In all cases, both stability programs gave nearly identical results. Differences were less than 0.5 percent. The various factors of safety based on the different shear strength parameters are shown in Figure 4 as a function of the effective stress parameter c'. Use of effective stress parameters obtained from modified compaction yields factors of safety that range from 2.3 to 3.6 and illustrates the potential benefits of using high compactive efforts.

Considering the past poor engineering performance of Crab Orchard shales and since these shales generally classify as soil-like (SDI < 50), effective stress parameters, $\phi'_d$ and $c'_d$, selected for design purposes were 18 degrees and 100 psf. Also, these parameters were selected on the basis of back-calculations of two highway failures and specimens remolded at different relative compactions. The first massive sidehill highway embankment failure occurred in 1972 on 64 (Milepost 118). Details of that failure are given elsewhere (4). Pore pressures (observation wells) and lateral movements (five slope inclinometers) were monitored over a period of 4 years prior to failure. The 100-foot sidehill embankment was constructed of Crab Orchard shales and rested on a 20-foot thick foundation of residual soils derived from Crab Orchard shales.

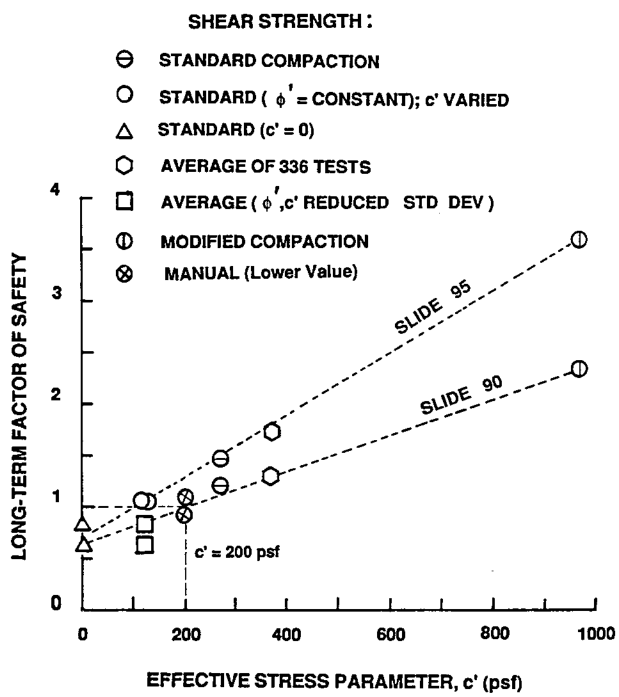


FIGURE 4. Factors of Safety as a Function of $c'_p$ -- Reanalyses of Slides 90 and 95 (Kope Shale Embankments) on I 75.

Based on inclinometer data, the unstable mass was wedge-shaped. A major portion of the shear surface passed through the foundation soils. To test the selected design parameters, a stability analysis was performed using the HOPK-I stability computer program. A factor of safety of 0.93 was obtained. Analyses also were performed using the ϕ'-value (equal to 23.9 degrees) obtained from compaction tests and varying c' until a factor of safety of 1.0 was obtained. A cohesive value of zero yielded a factor of safety of 1.12. Details of the stability analysis of the second slope failure, which occurred on KY Route 10 along the AA Highway alignment, are given in the Section entitled "CASE HISTORIES."

Although minimizing settlements of shale embankments was a major consideration, major slides in the Kope and Crab Orchard areas have generally resulted from overstressing of weak clays and shales at or below the bases of embankments on sidehill locations (1-9). Numerous embankment failures have occurred at sidehill locations due to a bearing capacity failure of the foundation. In most cases, back analyses show that laboratory triaxial shear strengths overestimate field shear strengths. Consequently, special considerations were given to formulating design criteria for sidehill locations.

## SETTLEMENT

At many locations on the AA Highway, embankment foundations consist of relatively thin (less than 10 feet), overconsolidated soils. Overconsolidated soils usually settle (32) a small fraction of settlements predicted from Terzaghi's (cf 32) consolidation theory and initial and primary consolidation occur rapidly. Hence, these types of settlements will occur before the placement of the pavements. Although secondary compression may be appreciable over a long period when thick foundation deposits are present (5, 33), this type of settlement was not considered significant because of the thinness of many of the embankment foundations. Except in thick deposits or in alluvial areas, foundation settlement due to primary and secondary compression was not considered significant.

Settlement of embankments constructed of shale is a common problem and is difficult to predict and control. Use of conventional consolidation testing to define deformational properties generally are not applicable because of large particle sizes of the shales. Consolidation rings are usually too small to accommodate the large particles. In a relatively dry state, compacted shales exhibit small compression. However, as shown by Shamburger, et al. (15) and Drnevich, et al. (34), compacted shales (using large molds) when soaked may exhibit large and excessive settlements (creep or secondary compression and shear strain) with increasing time. Based on an approximate correlation (15) of slake-durability indices (20) and compression of soaked shale specimens, gross estimates of settlement of an 80-foot embankment (soaked) constructed of either Kope or Crab Orchard shale is about 0.4 to 0.6 percent of fill height, or 4 to 8 inches for SDI's ranging from about 20 to 40 percent.

Secondary compression and settlement due to shear strain may occur even for well compacted fills. These settlements may amount to approximately 0.3 to 0.6 percent of the fill height over a period of 15 to 20 years (according to NAVFAC (29)). As shown in Figure 5 (33), estimated settlements due to secondary compression and shear strain for embankments constructed of fine-grained plastic soils (CL, CH, OL, MN) become significant for embankment heights greater than about 50 feet. Long-term settlements from field measurements and reported elsewhere (33) are compared to the criteria from NAVFACS in Figure 5. Embankments constructed of clays and clayey shales and greater than about 50 feet in height exhibited the most settlement. Large embankment settlements (10) observed on I 75 also confirm these observations. Many of these fills in excess of about 50 feet had settled several inches in a 20- to 25-year period after construction.

As shown by A. W. Bishop (cf 35), an embankment having a factor of safety below about 1.8 will exist in a state of plastic equilibrium. As the factor of safety decreases below this value, the shear strain potential increases. This aspect of embankment settlement is, perhaps, illustrated in Figure 6. Observed long-term settlements (projected to 27.4 years) of several embankments were plotted as a function of the long-term factors of safety (33). Observed pore pressures and slope-inclinometer data obtained over a period of several years were used in stability analyses of those sites. Also, shear strength parameters were obtained from triaxial

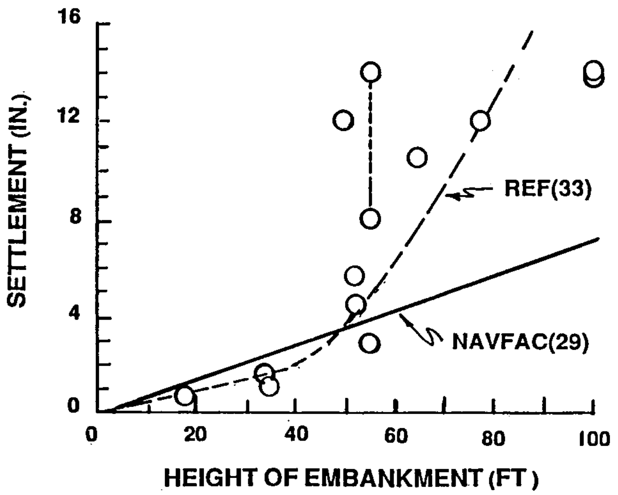


FIGURE 5. Long-Term Settlement as a Function of Embankment Height (after Hopkins (33)).

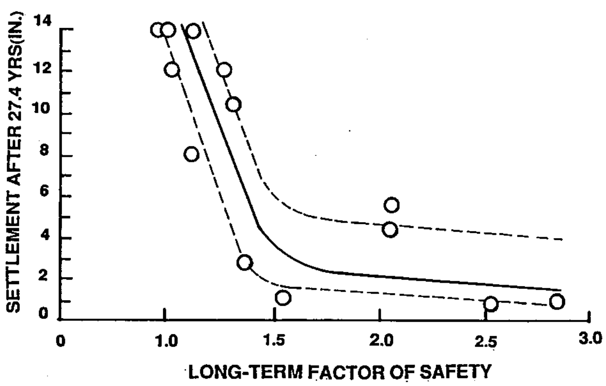


FIGURE 6. Long-Term Settlements as a Function of Long-Term Factors of Safety (after Hopkins (33)).

sites. Also, shear strength parameters were obtained from triaxial tests performed on "undisturbed" samples obtained several years after construction. Settlements of approach pavements and foundation settlements were monitored for several years. Embankment settlement was obtained by subtracting the foundation settlement from the bridge approach settlement. Settlements obtained in this manner were plotted as a function of the logarithm of time. The relationship between embankment settlement and the logarithm of time was linear. Hence, the linear relationship could be projected with time to obtain settlements at some future date. Each case was projected to 27.4 years (10,000 days). Generally, settlements that occur after 27.4 years were insignificant. Although a considerable scatter of data was present, the data in Figure 6 show that a factor of safety of 1.5 or greater tends to decrease long-term settlements.

As one approach to estimating (gross) long-term settlements of embankments constructed of Kope and Crab Orchard shales, the method proposed by Hopkins (33) was used. The slope of the linear settlement-log time relationship is referred to as the coefficient of shear strain and secondary compression, $C_{ss}$ (an empirical coefficient obtained from field measurements). Settlement due to secondary compression and shear strain may be estimated from the following empirical equation:

$$H_{ss} = C_{ss} H_e \log_{10}(t_{ss}/t_c) \qquad (3)$$

and;

$$H_{ss} = (10^{(1.53\log_{10}F_r - 4.676)}) H_e \log_{10}(t_{ss}/t_c) \qquad (4)$$

in which $H_{ss}$ = settlement of the embankment due to secondary compression and shear strain,

$H_e$ = height of embankment,

$t_c$ = time of placement of pavement (the time between the start of construction and placement of the pavement),

$t_{ss}$ = time at the end of significant secondary compression and shear strain of the embankment (assumed), and

$F_r$ = Ratio of $H_e$ and long-term factor of safety.

Equation 4 relates shear strain and secondary settlement to the height of embankment, the factor of safety, and time. The factor of safety, of course, is a function of shear strength ($\phi'$ and c'), geometry, dry density (and compactive effort), and pore pressures. As shown in Figures 2 and 3, the shear strength is a function of dry density. Consequently, as the dry density increases, the factor of safety increases since the cohesion, c', increases with dry density. In Figure 7, the settlement, $H_{ss}$, obtained from equation 4 is plotted as a function of the height of embankment, $H_e$, for various assumed values of factors of safety. The value of log ($t_{ss}/t_c$) was taken to be 1.1 for these curves.

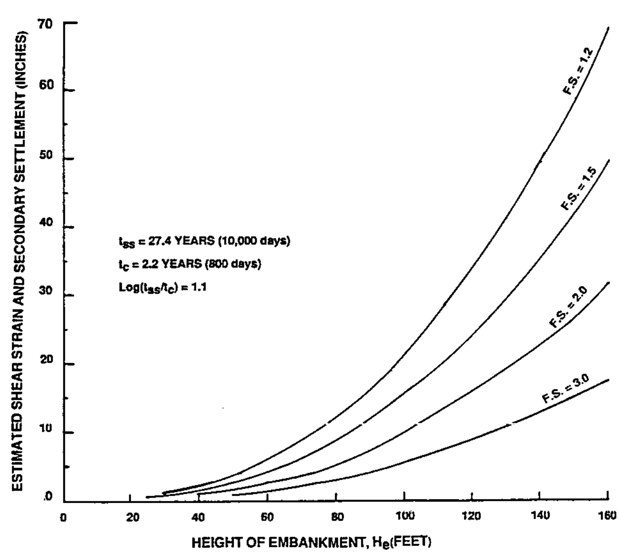


FIGURE 7. Estimated Embankment Settlement as a Function of Embankment Height and the Long-Term Factor of Safety (after Hopkins (33)).

Based on Figure 7, the effect of the factor of safety on settlement is readily apparent. Based on these curves, and assuming a design factor of safety of 1.5, the long-term settlement of a 50-foot embankment (or pavement) is estimated to be 2.7 inches at the end of 27.4 years. Based on criteria given by NAVFAC, the settlement is estimated to be about 3.6 inches. Based on the approximate method by Shamburger, et al. (15), settlements are estimated to be about 4 inches. Consequently, for embankments 50 feet or less in height on the AA highway, special (high compactive efforts) compaction was not specified since settlements of about 2 to 4 inches were considered tolerable and acceptable.

## COMPACTION OF SHALES

Numerous large scale failures of shale embankments along major interstate routes in the early seventies prompted a major research effort into the engineering uses and properties of shale. The research focused on development of guidelines for design and construction of new shale embankments as well as compiling a list of remedial measures that may be used to correct existing embankments in distressed or failure states. A cursory review of existing literature regarding this research and a review of shale compaction procedures of states surrounding Kentucky are discussed below. Finally, a brief discussion of the Special Shale Embankment Construction Specification adopted for design and construction of the AA Highway is presented. A copy of this specification is presented in Appendix A.

## LITERATURE REVIEW -- COMPACTION

Numerous papers have been published regarding the engineering properties and uses of shales in highway construction. Topics range from geologic classification of shales to case histories of embankment failures. The number of reports is too extensive for a thorough review in this paper. Two major works that influenced current shale compaction procedures for Kentucky and adjacent states containing large areas of shales will be discussed. The most comprehensive study is contained in five reports (12-16) prepared by the U.S. Army Waterways Experiment Station for the Federal Highway Administration (FHWA).

### Research Studies Sponsored by FHWA

In 1974, after numerous failures of existing shale embankments, the Federal Highway Administration initiated and sponsored a three-phase study to develop design and construction guidelines for remedial actions on existing failures and for design and construction of new shale embankments. The research was conducted by the U.S. Army Engineers Waterways Experimental Station, Vicksburg, Mississippi. As part of the preliminary work conducted for Phase 1 (12), the researchers made a very important observation that provided the impetus for their, as well as other, research:

> "... The underlying cause of excessive settlement and slope failures in highway shale embankments appears to be deterioration or softening of certain shales with time after construction. Inadequate compaction and saturation are two other primary causes of shale embankment problems."

Phase 1 of the research focused on available information regarding classification and material properties, physical and chemical tests, design guidelines and construction control procedures, and sampling and testing procedures for in situ and compacted shales. As part of Phase 1, the researchers summarized current construction procedures used at that time by some of the state highway departments and found that acceptance or rejection criteria varied considerably among the states. They also found that most of the cited causes of failures could be linked to the lack of tests for predicting shale performance with time. The researchers also determined that "... the major factor is the degree of durability exhibited by the shale material and how this durability can be expected to change with time."

The second phase (13) of the research dealt mainly with the evaluation and remedial treatment of shale embankments that were exhibiting distress. Results of this part of the research provided a process by which a highway geotechnical engineer could assess the current overall stability of an existing embankment and also provided recommendations for correcting shale embankment problems. Perhaps the most important part of their results was the statement that surface and subsurface drainage is a critical part of most remedial measures. Therefore, considerations of surface and subsurface drainage are extremely important in design of new embankments.

Phase 3 (14) was designed to fill gaps identified in the first two phases of the study and to provide a comprehensive manual for design and construction of shale embankments. As part of this phase, information and interviews supplied by 15 state highway departments regarding shale embankment performance was correlated with the slake durability index to provide a guideline for placing shale in embankments. As shown in Figure 8, slake-durability indices were generally correlated with lift thickness and embankment performance.

Also as part of Phase 3, the researchers provided conclusions and recommendations regarding the use of shale in highway embankments. Based on those findings, the primary causes of large settlements and slope stability problems are inadequate compaction, saturation, and shale deterioration. Another conclusion was that classification of shales according to long-term durability was absolutely essential in development of design measures. Recommendations included classifying shales as either soil-like or rock-like and then placing the material in lift thicknesses according to classification.

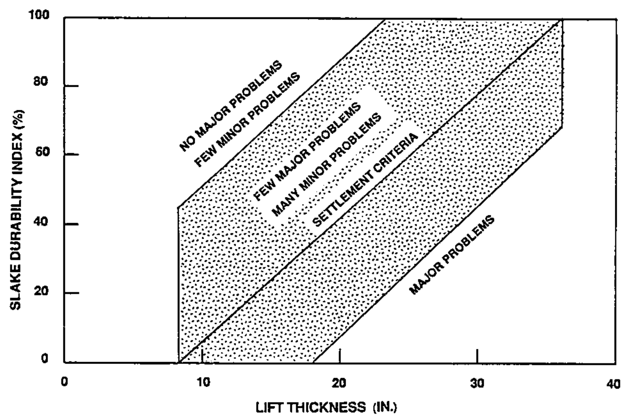


FIGURE 8. Slake-Durability Index as a Function of Lift Thickness.

**Franklin Study**

As an extension of the FHWA study and in an effort to provide additional information as to the long-term durability of shales in highway embankments, Franklin in 1981 devised a shale classification system (36). In Franklin's classification system, a shale rating "R" was determined based on slake durability index and either plasticity or point load strength. Plasticity is used in conjunction with the slake durability index when the slake durability index is less than 80 percent. If the slake durability index is greater than 80 percent, the point-load strength is used. Based on these correlations, a shale rating on a scale of 0.0 to 9.0 is determined for the sample. Based on the FHWA studies (12-16), Franklin devised a tentative correlation, as shown in Figure 9, between the R value, lift thickness, and expected performance of shale embankments.

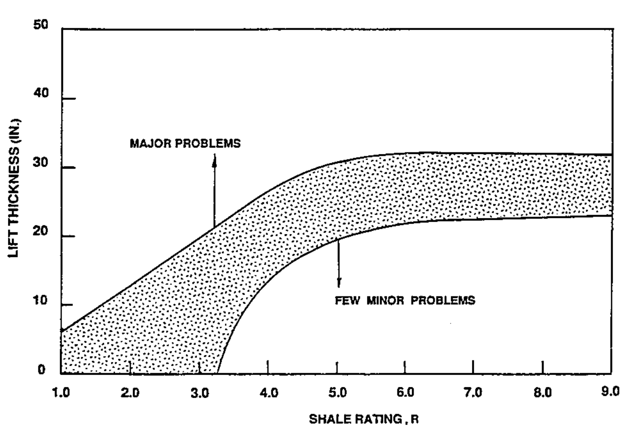


FIGURE 9. Lift Thickness as Function of Shale Rating ("R") and Expected Performance.

**SHALE COMPACTION SPECIFICATIONS OF ADJACENT STATES**

As part of the research effort described above, the Kentucky Transportation Research Program, University of Kentucky, conducted a survey (27) in 1985 of surrounding states with regard to shale compaction specifications. The states of Indiana, Pennsylvania, Tennessee, Virginia, Ohio, and West Virginia were contacted. Specifications of states responding to this survey are discussed briefly below.

**Indiana**

The most detailed set of specifications were those obtained from Indiana (37). Those specifications require classifying the shale according to the Franklin system. Any material with a R-value below 5 is considered to be a shale or "soft rock." For embankments constructed of these materials, the Indiana specifications require that the embankment materials be placed in maximum 8-inch loose lifts. After placing, the shales are brought to within -2 percent to +1 percent of optimum moisture content by adding water and "uniformly incorporating" the water with a minimum 24-inch gang disk. The shales are then compacted using three passes of a 60,000-pound static tamping-foot roller, and then using two passes of a 55,000-pound dynamic tamping-foot roller. The newly placed material is bladed between the static and dynamic rollers. Roller speed is restricted to 3 mph.

Indiana also has special requirements when the geologic formation consists of a combination of limestone and shale. The compaction requirements include items mentioned above, but also limits the size of limestone rocks. Limestone fragments of 6 inches in thickness and 1.5 feet in any other direction are prohibited. Moreover, the specifications require encasing the slopes with 10 feet of non-shale, non-erodible material. The number of passes of the static roller is also increased to six from three.

**Pennsylvania**

The Pennsylvania specifications are not nearly as detailed as the Indiana's, and yet they do require a maximum lift thickness of 8 inches. Shales are placed approximately at optimum moisture content and at 97 percent of the maximum dry density. However, the specifications do not give procedural specifications on how this density is to be obtained. The Pennsylvania specifications also require placing coarse material on the out slopes while requiring that finer material must be placed on the inside of the embankment. Additionally, the specifications require "large" pieces of rock to be broken down until "most" of the voids are filled.

**Tennessee/Virginia**

Neither Tennessee nor Virginia had special provisions for shale embankments at the time the research survey was conducted. Tennessee handles shale as unclassified material and Virginia treats shale as rock. The Virginia Department of Highways did, however, state that their method of handling shales has created some performance problems.

**Kentucky Standard Specifications**

Current Kentucky "standard" specifications, adopted in 1975, make distinctions between durable and nondurable shales and places some requirements on the manner in which shales are to be compacted. Shales having SDI's less than or equal to 95 percent are considered nondurable and are placed in 12-inch loose lifts. According to "standard" procedures, shales are then compacted to 95 percent of maximum dry density. For durable shales (SDI greater than 95), placement is permitted in maximum loose layers up to 3 feet with compaction achieved by blading or dozing so that voids, pockets, and bridging will be minimized. Dimensions of boulders are limited to 3 feet vertically and approximately 4.5 feet horizontally. Recognizing the limitations of the standard compaction specification when applied to the compaction of shales, Kentucky adopted a special compaction provision in 1984. Statewide application of this provision was approved by the Federal Highway Administration in 1985. The provision was modeled after the Indiana shale compaction specifications.

## SPECIAL SHALE EMBANKMENT CONSTRUCTION SPECIFICATION

Comparison of Kentucky "standard" specifications to FHWA and Franklin correlations for shale embankment performance indicates a major portion of these embankments will exhibit some form of distress. Based on these correlations and past experience on I 75 and other major highways, a "Special Shale Embankment Construction Specification" was formulated for use in the design and construction of the AA highway. A complete copy of the Special Shale Embankment Construction Specification is presented in Appendix A.

The "Special Shale Embankment Construction Specification" for the AA Highway was patterned after specifications currently used by Indiana. Shales that exhibit SDI's less than 95 are subject to the special compaction criteria as determined by the designer. These materials are to be placed in maximum 8-inch loose layers. After placing and blading to a uniform thickness, water is added to obtain a moisture content of -4 to +2 percent of optimum moisture. Compaction is achieved using a 60,000-pound static tamping foot roller and 50,000-pound vibrating tamping foot roller. Each lift must receive three passes of the static roller and two passes of the vibratory roller. Also, the AA special compaction criteria requires large limestone slabs to be either broken down or removed from the fill.

## DESIGN APPROACHES

Three major concerns along the AA highway were slope stability, embankment settlement, and surface and subsurface drainage. Stability and settlement problems were addressed by the selective use of the special shale embankment construction specification. Surface and subsurface drainage considerations are addressed on a site-specific basis using currently accepted and documented practices (38).

During the final design stage of the AA highway, engineers realized that indiscriminate use of the special shale embankment construction specification would be very costly. As a result, general criteria and geometric configurations were developed to aid in determining the applicability of using the special method. Using these general guidelines, consulting engineers were able to design economical embankment configurations for locations where the special criteria were deemed necessary. The following paragraphs describe in more detail how slope stability and settlement were treated.

## SLOPE STABILITY

### Embankments

Slope stability analyses were conducted using a systematic trial-and-error procedure. Considering preliminary analyses and past performance of shale embankments, initial configurations were chosen based on proposed embankment height. Generally, special compaction techniques were not employed at this stage of the analysis. Table 1 presents a summary of the initial embankment geometry, shear strength parameters, and required factors of safety.

TABLE 1. PARAMETERS USED IN INITIAL SLOPE STABILITY ANALYSIS

| EMBANKMENT HEIGHT (ft) | SIDE SLOPES (H:V) | SHEAR STRENGTH PARAMETERS | REQUIRED FACTOR OF SAFETY |
|---|---|---|---|
| <25 | 2:1 | Lower bound* | 1.5 |
| 25 - 50 | 2:1 or 2.5:1 | Lower bound* | 1.5 |
| >50 | 2.5:1 or 3:1 | Lower bound* | 1.4 |

* From Geotechnical Manual, Division of Materials, Kentucky Department of Highways (20)

Based on results of initial stability analyses, embankments that did not meet the required factor of safety were reanalyzed using the special shale compaction specification. In this case, $\phi'$ and $c'$ were set equal to about 23 to 25 degrees and 200 pounds per square foot, respectively. If the special shale compaction criteria were used as a slope stabilization technique, the entire lower portion of the embankment up to a specified elevation was designated to receive special compaction. Generally, designers selected the top elevation based on trial analyses and tried to keep the specially compacted areas in the passive portion of the embankment. Typical configurations for embankments stabilized using the shale embankment construction specification is shown in Figure 10.

At sidehill locations where major highway failures have occurred due to overstressing of weak soils and shales in Kope and Crab Orchard areas and where the soil underlying the embankment and overconsolidated, plastic soils classified as soil-like (SDI < 95 percent), benching techniques were specified. Typical embankment foundation benching used at sidehill locations are shown in Figure

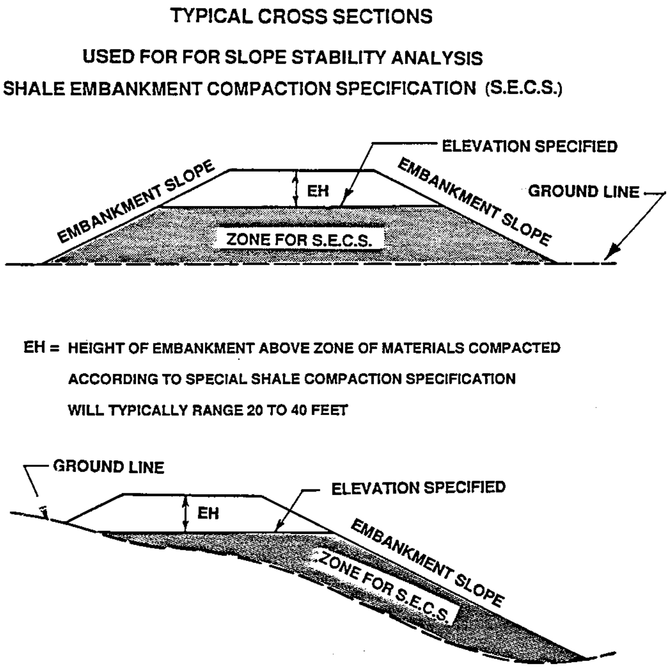


FIGURE 10. Typical Configurations for Embankments Stabilized Using the Special Shale Embankment Construction Specification.

11. In applying this technique, thin overburden soils are removed and benches are excavated into the rock disintegration zone (RDZ). Although it was certainly desirable to locate the bottom of the benches below the lower level of the RDZ material, excavation costs were uneconomical because the RDZ in many cases extended several tens of feet below the bottom of the overburden soils.

### Cut Slopes

Cut slopes along the AA Highway alignment are predominantly in rock. Only in areas where glacial materials were present and where the Crab Orchard Formation outcrops were special design considerations given to cut slopes. Rock cut slope designs followed standard practice for the lithologies encountered

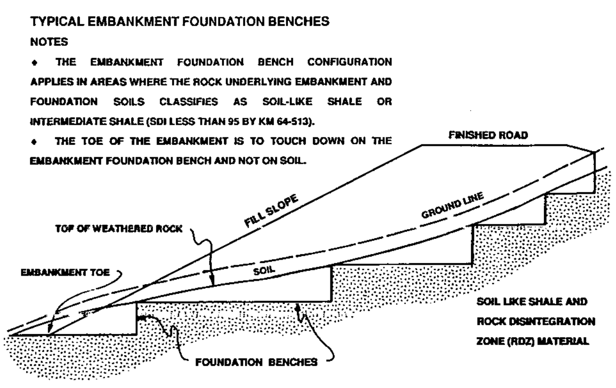


FIGURE 11. Typical Configuration Showing Benching Scheme for Sidehill Embankment Locations.

(20). Typically, slopes of one horizontal to one vertical with 18-foot wide intermediate benches spaced at 30-foot lift intervals were used in the Kope Formation. Where overburden soils were thin, typically less than 10 feet, cut slopes through the RDZ materials and soils were typically constructed on slopes at two horizontal to one vertical.

Cut slopes through glacial soils presented problems in some areas due to high ground-water tables. At several sites along the alignment, the existing watertable was above the roadway grade. In those areas, slopes of three horizontal to one vertical and subsurface drainage provisions were specified. Also, surface water diversions were placed around the heads of the slopes.

Cut slopes in the Crab Orchard Formation presented unusual problems. The boundary between the overburden soils and the shale bedrock was difficult to establish due to the softness of the shale. Also, soils on existing hill slopes are unstable and are slowly creeping down slope. Cut slopes of two horizontal to one vertical were typically used and extra wide benches were specified.

Slope stability analyses were performed on all cut slopes involving overburden soils of the Crab Orchard Formation greater than 10 feet in thickness. Shear strength parameters used in the analysis were obtained from consolidated-undrained triaxial tests with pore-pressure measurements. However, $c'_p$ was set equal to zero in the analysis to account for "softening" that may occur when a slope is excavated. Generally, ground-water levels obtained during the corridor studies were used in the stability analyses.

### Settlement

Settlement of shale embankments due to compression within the fills is a documented problem. Along the AA Highway alignment, numerous embankments greater than 50 feet in height will exist. The tallest embankment on the project is 160 feet in height measured from the toe of the embankment to the top of the embankment. To minimize settlements expected in embankments of this height, increasing the density was the only practical and economical approach. The special shale embankment construction specification was used in these cases to achieve densities higher than those normally obtained from standard compaction (AASHTO T 99).

Settlement was considered to be a problem for all embankments over 50 feet in height and at all bridge approaches. Ideally, it was desirable to construct the entire embankment following the special specification. However, because of increased construction costs, it was not economical to compact the entire embankment using the special compaction specification. The next best alternative was to compact just the core zone of the embankment under the roadway using the special compaction techniques. Figure 12 presents typical cross sections showing the zoned embankment configuration where special compaction techniques were applied. The configuration is a flat-topped pyramidoidal zone within the center of the embankment. The top

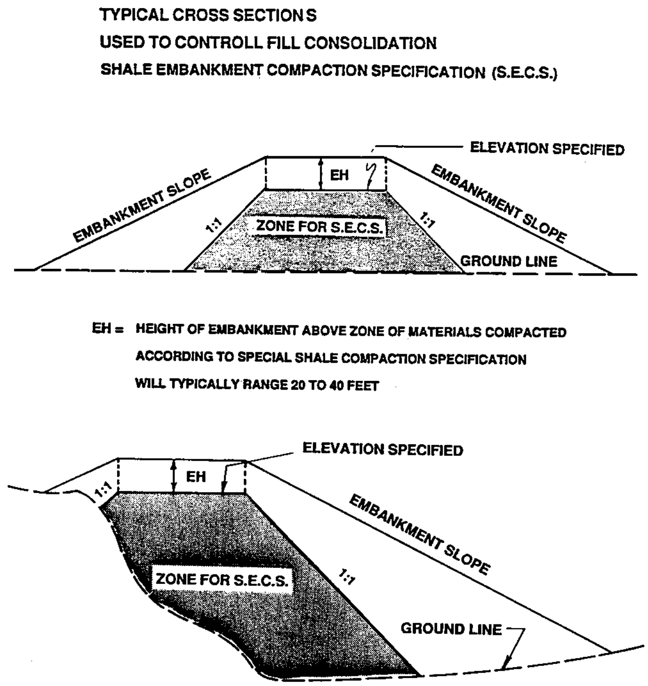


FIGURE 12. Typical Cross Sections Illustrating Zoned Embankment Configurations.

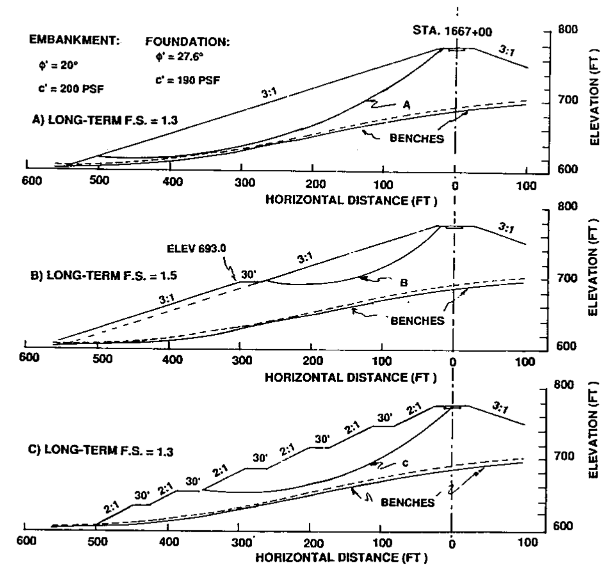


FIGURE 13. Cross Section of Embankment Located at Station 1667+00.

elevation of the zone is typically 25 to 50 feet below roadway grade. The width of the zone at the specified elevation is the width of the roadway template, as measured from shoulder to shoulder. Below the specified elevation, the zone widens on both sides on one horizontal to one vertical slopes. The pyramidoidal zone in the center of the embankment was used because it will provide high densities in the core of the embankment where the highest stresses exist. All bridge approach embankments were compacted according to the special compaction techniques.

## CASE HISTORIES

### STATION 1667+00

The highest embankment on the AA Highway project is located in Bracken County between Stations 1664+00 and 1672+50. Total height of the embankment at Station 1667+00, measured from the toe of embankment to the shoulder or crest of the embankment, is 166 feet (see Figure 13). The maximum depth of the fill measured along the centerline, which occurs at Station 1668+25, is 133 feet. The embankment is a cross-valley fill and the centerline of the roadway is perpendicular to the existing valley and stream.

Both the Kope and Fairview Formations outcrop in the area of the embankment. Foundation soils average 5 feet in thickness. The Kope Formation outcrops over the entire area. Fill material for this embankment will consist of both Kope and Fairview shales. Foundation soils classified as CL and A-7-6. Triaxial and unconfined compression tests were performed on the foundation soils. Undrained shear strengths were 4,060, 3,940, 6,920 and 3,800 psf. Results obtained from triaxial tests were $\phi'$ equal to 27.6 degrees and c' equal to 190 psf. Slope stability analyses were performed using effective stress parameters of the embankment and foundation materials.

Stabilities of three slope configurations were evaluated. Due to the large volumes of material required to construct the embankment, the application of the special compaction specification to the entire embankment was not economical. Considering the great height of this fill, the primary problem was to determine an embankment configuration having stable slopes while attempting to minimize embankment settlements. Fortunately, there were no right-of-way restrictions to limit the location of the embankment toe and large volumes of excavated materials were available to construct flatten slopes. For these reasons, slope stability calculations, based on a $\phi'$ equal to 20 degrees and c' equal to 200 psf (embankment), were performed to find a stable embankment configuration having an adequate factor of safety. Results of the analyses are given in Figure 13. The selected configuration consisted of a three horizontal to one vertical slope with a berm (middle section in Figure 13).

The special shale embankment construction specification was applied to the core zone to control settlement. Gross estimates of settlement of this embankment based on the approximate correlation given by Shamburger, et al. (15) were 12 to 16 inches. Based on criteria given in NAVFAC (29), estimated settlements of 12 to 15 inches may occur over a period of 15 to 20 years after construction. Using the method by Hopkins (33), a factor of safety of 1.5, and Equation 4 gross estimates of settlements of 35 inches may occur over a period of 25 to 30 years. However, if dry densities in the field approximate dry densities obtained from modified compaction, which may result from the special compaction specification, then factors of safety may be nearer to values of 2 to 3. Constructing drains at the base of this fill also will minimize ground-water seepage into the fill. By minimizing seepage, the factor of safety will remain at a high level. In this case, long-term settlements of the fill are estimated (Equation 4 and factors of safety equal to 2 and 3, respectively) to range from 21 to 11 inches. Approximately half of the embankment height in the core zone located under roadway centerline will be constructed according to the special compaction specification.

The embankment at Station 1667+00 is an example of a situation where lower shear strength parameters were used in the slope stability analyses to obtain a stable embankment configuration. The lack of right-of-way restrictions and the availability of fill material allowed placement of the toe of the embankment some 540 feet left of roadway centerline. Constructing embankments of Kope shales to heights such as the embankment at Station 1667+00 presents a dilemma to highway designers since design stabilities and tolerable settlements obtained from analyses must be balanced against acceptable economical considerations. Obtaining acceptable stabilities and tolerable settlements for such large embankments are difficult because of the uncertainties involved in defining appropriate shear strength and settlement parameters of compacted shales.

## HERRON HILL EMBANKMENT

Herron Hill is located east of Tollesboro in Lewis County near the community of Ribolt. The Crab Orchard Shale outcrops throughout the area. Kentucky 10 presently crosses the area. On the east side of Herron Hill, embankment slope failures are affecting the existing roadway. The alignment and grade for the AA Highway requires placement of an embankment on the existing slope below KY 10. Station 1826+00 is located in the center of the existing landslide area and was selected as the critical cross section for sampling, instrumentation, and analysis. Figure 14 presents the cross section at Station 1826+00.

The subsurface investigation consisted of obtaining four soil samples and installing slope inclinometers in two of the borings. Only one triaxial compression test with pore-pressure measurements was performed on an undisturbed sample because of difficulties in obtaining good quality soil samples. The triaxial test yielded effective shear strength parameters of $\phi'$ = 22.0 degrees and c' equal to 0 psf.

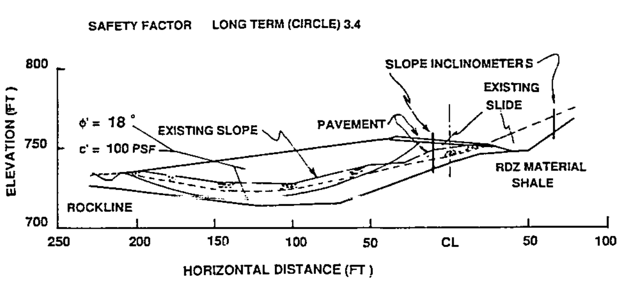


FIGURE 14. Cross Section of Embankment Located at Herron Hill, Station 1826+00.

Slope inclinometer data were used to establish the configuration of the failure surface. A wedge method of analysis (31) was used to calculate effective stress strength parameters along the failure surface. In the back calculations, c' was assumed to be zero since the failure mass had moved along the failure surface (38) and $\phi'$ was varied until a factor of safety of unity was obtained. The resulting shear strength parameters were $\phi'$ = 18 degrees and c' = 0 psf.

To evaluate the shear strength parameters of the compacted fill, samples of Crab Orchard Shale were remolded and tested. The samples were remolded at relative compactions of 90 and 95 percent and at different moisture contents. Results of the triaxial testing contained appreciable scatter. Compaction data and effective stress parameters are given in Table 2. Based on the results of these tests

TABLE 2. SHEAR STRENGTH PARAMETERS OF SPECIMENS OF CRAB ORCHARD SHALES COMPACTED AT DIFFERENT WATER CONTENTS AND VALUES OF RELATIVE COMPACTION

| | MOLDING CONDITIONS | | | | |
|---|---|---|---|---|---|
| TEST SET NUMBER | WATER CONTENT (%) | DRY DENSITY (pcf) | RELATIVE*** COMPACTION (%) | $\phi'$ (degrees) | c' (psf) |
| 1* | 11.0 | 118.5 | 100 | 23.9 | 768 |
| 2** | 23.6 | 104.5 | 93.3 | 17.7 | 0.0 |
| 3** | 19.8 | 95.7 | 90.5 | 15.9 | 0.0 |

* Research Study in Progress (27) -- Raw Shales
** Weathered Overburden Clayey Soils and Shales
*** Standard Compaction (AASHTO T99):
Set 1: Opt. M.C.= 11.4%; Max. Dry Density = 118.6 psf
Set 2: Opt. M.C.= 21.4%; Max. Dry Density = 112.0 psf
Set 3: Opt. M.C.= 20.5%; Max. Dry Density = 105.8 psf

and the back calculations of the existing landslides, effective stress shear strength parameters of $\phi'$ equal to 18 degrees and c' equal to 100 psf were used for both the foundation soil and embankment materials. Based on these values, a factor of safety of 0.93 was obtained for the embankment at Milepost 118 on I 64 as described previously in the section entitled "Engineering Properties." Hence, these values appeared to be a reasonable choice for design purposes.

Slope stability analyses were performed using embankment slopes of five horizontal to one vertical and six horizontal to one vertical. Factors of safety of 2.1 and 3.4 were obtained for these slopes, respectively. The extremely flat embankment slopes were selected since right-of-way restrictions were not a problem and large volumes of waste materials existed on the project.

In the design, a subsurface drainage blanket consisting of a 5-foot layer of dolomite or a hard durable shale (SDI > 95 percent) will be constructed at the base of the embankment. The drainage layer will extend from the uphill shoulder to the toe of the embankment. Perforated pipes will be placed in all existing surface drainage channels prior to placement of the drainage blanket and embankment.

The Crab Orchard shale is one of the weakest and most troublesome clayey shales in Kentucky (8, 27). By using waste materials to construct relatively flat slopes, extra compactive effort to densify the embankment shales, and subsurface drainage measures, the embankment at this site should remain stable.

## SUMMARY

An overview of geotechnical criteria used in the design of the Alexandria-Ashland Highway was presented. Formulation of design criteria relied heavily on past experiences, local studies, and performances of cut slopes and embankments constructed through and with clayey shales of the Kope and Crab Orchard Formations. In past years, lift thicknesses up to 3 feet were permitted, and slopes of two horizontal to one vertical frequently were used. However, local experiences have shown that the past criteria have resulted in numerous settlement and instability problems and costly maintenance and remediation. Past experiences and case histories also have shown that peak shear strengths of the overconsolidated clays and clayey shales of the Kope and Crab Orchard Formations obtained from triaxial tests frequently overestimate the shear strengths available in the field. Criteria adopted for the design of the AA Highway were specifically formulated to minimize settlements and slope instability of embankments. Based on economical considerations, embankments measuring 50 feet or greater in height were zoned to minimize settlements. In the core zones, procedural specifications required a loose lift thickness of 8 inches and compactive efforts greater than compactive efforts normally used or required. To prevent slope instability, shear strengths smaller than peak shear strengths from triaxial tests were used in stability analyses. The selected design shear strength parameters were based on local experience and analyses of several case histories. One of the major objectives of this paper was to document the geotechnical criteria used in designing the AA Highway so that future evaluations of the criteria may be made by observing the performance of the cut slopes and embankments on the AA Highway.

The problem of predicting stabilities of earth structures was recognized by Terzaghi (39) when he stated the following:

> "Comparing the results of shear tests performed in the laboratory with the shear values computed from shear failures on slopes in the field, I realized that the shear strength of cohesive soils in the field may depend on several factors which cannot be adequately reproduced in the laboratory. Hence in 1933 I suggested at the International Congress on Large Dams in Stockholm that the shear values obtained from laboratory tests should be divided by an empirical number which I called the "instability number." It may range from unity for materials which were found to perform under field conditions in accordance with our forecast and five for those, like the Bearpaw shale, which start to move as soon as the shearing stresses exceed a small fraction of the laboratory strength ..... the need for purely empirical "instability numbers" continues to exist (1960) ..... However, our knowledge of the conditions which require the use of "instability numbers" or, more appropriately "coefficients of ignorance" is steadily increasing, and that is a great asset for the practicing engineer."

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## APPENDIX A

## SPECIAL SHALE EMBANKMENT CONSTRUCTION SPECIFICATION

Embankments or portions of embankments, as designated in these plans and cross-sections which are to be constructed of soil-like shales (SDI 50 or less by KM (Kentucky Methods) 64-513, slake durability index test) or intermediate shales (SDI greater than 50 but less than 95 by KM 64-513), shall be constructed according to the following construction techniques.

The embankments shall be constructed in successive uniform layers not to exceed 8 inches in loose depth thickness to the full width of the cross-section. Excavation and blasting procedures shall accommodate the selective placement of these materials. The material shall be bladed as required prior to compaction to insure uniform layer thickness.

Large rock fragments or limestone slabs having a thickness greater than six (6) inches and/or a dimension greater than two (2) feet shall be removed from the layer to be compacted, or broken down in place by hand or mechanical means and then incorporated into the layer. Large rock fragments removed from a layer are to be broken down and incorporated into that layer or subsequent layers. Where waste is present on a project the large rock fragments may be wasted at the contractors expense.

Water shall be added to each layer as required to obtain a moisture content near optimum (-4.0 to +2.0 percent of optimum moisture as determined by KM 64-511, moisture-density test) and to accelerate the slaking (breakdown) of the shales. Water shall be added using a spray bar on a truck or other methods of spraying that produce a uniform application as approved by the engineer. The water shall be uniformly incorporated throughout the entire layer by a multiple gang disk with a minimum disk wheel diameter of 24 inches.

Compaction shall be accomplished with a vibratory tamping-foot roller in conjunction with a static tamping-foot roller. The minimum weight for the static tamping-foot roller shall be 60,000 pounds. The minimum total compactive effort for the vibratory tamping-foot roller shall be 50,000 pounds in accordance with the manufacturer's specifications. Larger rollers will be permitted as required to obtain density. Each tamping-foot on the static roller shall project from the drum a minimum of seven (7) inches. Each tamping-foot on the vibratory tamping-foot roller shall project from the drum a minimum of four (4) inches. The surface area of the end of each foot on both tamping-foot rollers shall be no less than five (5) square inches.

Unless otherwise approved in writing, each embankment lift shall receive a minimum of three passes with the static roller and a minimum of two passes with a vibratory roller. The rollers shall not exceed three (3) MPH during these passes. Each embankment layer shall be compacted to a density of at least 95 percent of maximum dry density as determined by KM 64-511, moisture-density test. The number of passes will be adjusted upward if necessary to obtain 95% of maximum dry density. No additional compensation will be allowed for additional passes as specified herein, the cost of which shall be included in the contract unit prices for roadway excavation, borrow excavation, or embankment in place, whichever applies.

The in-place density will be determined by KM 64-512 (rubber balloon method) and by utilizing a nuclear density gauge. Tests will be conduced at such a frequency as deemed necessary to assure that an entire layer is compacted to the specified density. The layer shall not vary from the optimum moisture content as determined by KM 64-511 (moisture-density test) by more than -4.0 percent to +2.0 percent. This moisture content requirement shall have equal weight with the density requirement when determining the acceptability of a layer. Multiple determinations of moisture content will be required as with density measurements.

The upper surface of a layer shall be shaped so as to provide complete drainage of surface water.

The above described requirements are in addition to sections 207.05, 208.02, and 208.05 of the 1984 Kentucky Standard Specifications for Road and Bridge Construction.

Payment for all labor, machinery, materials, and any costs associated with the construction of shale embankments shall be included in the contract unit price for roadway excavation, borrow excavation, or embankment in place, whichever applies.

# SOIL RETENTION AND SLOPE STABILIZATION WITH GEOTEXTILE FABRIC - A CASE STUDY

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## Abstract

For two decades, the concept of earth reinforcement has been employed in design of slope stabilization. The resistance to failing slopes by earth reinforcement has been predicated upon development of a gravity structure with the use of metal reinforcing strips and concrete or metal facing for soil retention at the free end of the structure. Contemporary designs of earth reinforcement include geotextile fabric as the medium of the retention system.

The design of geotextile retaining walls is based upon fundamental engineering principles. The structural unit is viewed as a gravity retaining wall and procedures concerning overturning and sliding are addressed accordingly. The internal components of the wall, such as lift thickness, fabric embedment, and fabric overlap are functions of the engineering properties of the geotextile, the soil backfill, and the externally applied stresses.

A geotextile retaining structure was designed and constructed along a critical river embankment in eastern Kentucky. The fabric wall was designed from procedures developed by the U.S. Forest Service and outlined in the Geotextile Engineering Manual prepared for the Federal Highway Administration. Design of the wall was completed to satisfy both internal and external stability requirements.

## Introduction

A commercial shopping plaza was constructed north of Pikeville, Kentucky in 1974. The property is located between Levisa Fork of the Big Sandy River and U.S. 23. Based upon our knowledge of original site development, limited grading of the property was done to achieve design grades. To provide required parking areas, the mall structures were positioned as close as practical to the residual slopes down to Levisa Fork.

Levisa Fork in Pikeville is being rechanneled. This development has included major excavations for the river, railroads, and highways. The original channel through the city of Pikeville is being filled and will be used for future development. The rechannelization has caused changes in the hydraulics of Levisa Fork resulting in property downstream of Pikeville, once above flood stage, now subject to flooding.

The referenced shopping center has been inundated twice in the past few years.

In 1977, the waters of Levisa Fork were about 14 ft. above general finished floor elevations. In 1984, flood waters were about 8 ft. above floor grades.

To minimize the potential of future flooding of the shopping center, the owner decided to construct a flood wall between the shopping center and Levisa Fork. The owner's requirements were that the flood wall be flexible, easily constructed, and of course, be economical.

Limited space was available for construction of the wall between the shopping center and the top of the slope to Levisa Fork. Because of the limited space, conventional earthen dike construction could not be utilized. Sheetpile walls and conventional retaining walls were considered but were found to be uneconomical. A geotextile reinforced flood wall was selected for construction because of its flexibility, ease of construction, and relative economics.

Truck access behind the shopping plaza needed to be maintained. The proximity of the river to the truck access together with the weight of the flood wall created a possibly unstable condition in the river bank. Using materials already chosen for the flood wall, a geotextile retaining wall was constructed beneath the flood wall to satisfy slope stability requirements.

This paper presents the design rationale used for reinforcement of the natural slopes. The actual design procedure is listed as well as descriptions of newly developed procedures based upon updated fabric testing and data analysis.

## Historical Development

The contemporary geotextile earth retaining systems evolved from the concept of Reinforced Earth® developed in 1967 by Henri Vidal in France. Vidal utilizes metal strips to improve the tensile strength of soil requiring support. These are tied to facing material (generally concrete) at the free face of the wall. The strips are manufactured of galvanized steel and are often .12" to .20" thick and 1.7" to 3.4" wide. Lengths of the strips are governed by wall height and backfill properties.[1] Reinforced Earth® is a product currently used in many applications, although it has been determined that geotextile fabric used as the reinforcing material has several distinct advantages, such as a lack of corrosion potential, non-skilled labor for construction, and a reduction in material cost.

Fabric retaining walls were initially designed in the U.S. by Dr. J.R. Bell in 1975 and were constructed by the U.S. Forest Service in Oregon.[2] The design method included wrapping the fabric in an individual wall lift similar to the procedure used in the subject application. Walls have since been constructed at many locations across the United States.

The Colorado Division of Highways constructed a 300-ft. long fabric wall near Glenwood Springs in 1982. The wall was constructed using four different fabrics. Instrumentation was installed to determine many engineering properties including internal and external deformation due to stresses. Sections of the wall incorporated intentionally insufficient designs to promote structural failure in an attempt to monitor such an event. When the wall did not fail, the researchers added 17 ft. of surcharge fill, but the wall still remained intact. The flexible nature of the system was exhibited when approximately 2 ft. of settlement occurred in the foundation soils. Maximum lateral displacement has approached 3" as detected by monitoring of slope inclinometers.[3]

Many other synthetic materials have been developed and are currently utilized in stabilization, such as geogrids and composite sections. Design for these functions often details an open-faced slope without encapsulating the soil by wrapping the fabric. The exposed face is generally protected with plant growth or riprap materials.

## Design Considerations

Geotextile fabrics serve many purposes which are dictated by the specific functions of performance. Examples of the diverse functions are presented below, each with a general application.

- Separation - Minimizing penetration of a fine-grained subgrade into a granular base coarse in a pavement section.
- Filtration - Restricting fine-grained soils to be transported into a subsurface drain and subsequently clogging of the system.
- Reinforcement - Increasing the tensile strength of soils in critical slopes and beneath foundations.
- Drainage - Media for permitting the movement of fluids otherwise restricted by low permeable materials.
- Support over Soft Subgrades - An extension of reinforcement, but

generally approached from a different perspective.

One or more of the various properties of the materials may gain significant importance depending on the required service rendered by the material. Types of properties are often categorized, depending on the function, into mechanical, hydraulic, chemical, fabrication, and durability.[4] The fact that the design and use of geotextiles is so contemporary leads to one drawback. Standardized methods for determination of relevant fabric properties are not currently available. Therefore, present design methods do not utilize all of the relevant material properties and, consequently, the structures are most often very conservative in design. It is beyond the scope of this paper to present and explain all of the engineering applications and fabric characteristics.

The function of a geotextile retaining wall is reinforcement of a critical slope to attain a level of confidence of safety against failure. Mechanical properties are fabric properties most used in design for reinforcement. The ultimate tensile strength of the fabric as measured by a standard laboratory procedure is one of the most important of the mechanical fabric properties. Contemporary design procedures often result in the creation of tables comparing different tensile strengths and such variables as the vertical spacing of the fabric or fabric embedment. The soil-fabric frictional resistance is an engineering consideration when designing for pullout of the fabric in tension.

The long term stress-strain characteristics are a noteworthy consideration, but they are also one of the least understood fabric properties. Current technological methods can not predict time dependent fabric creep to evaluate performance over the design life of the structure. With conservative designs, the magnitude of creep may not significantly affect the performance of previously constructed geotextile retaining walls.

The main emphasis of design of a fabric wall remains in satisfying both internal stability within the reinforced mass and stability of the slope outside of the limits of the wall. Internal stability is a function of the resistance to structural failure provided by the mobilized shear strength of the individual components of the wall.

Limit equilibrium methods are generally used to evaluate the internal stability of the reinforced wall. A failure surface, as defined by $\Theta = 45+\emptyset/2$, is transected by the layers of geotextile along the embedment length. As previously mentioned, the mechanism of increased tensile strength is provided by the strength of the layered fabric. Maximum tensile stress occurs along a locus of points which delineates the critical failure surface. In steep embankments, the maximum compressive strains are horizontal and the maximum tensile strain occurs along the vertical plane. Therefore, the efficiency of fabric reinforcement is high as construction methods lead to horizontal placement perpendicular to the strain plane.

There exists a dichotomy in the determination of the lateral earth pressure coefficient used to develop failure plane stresses within the fabric wall system. Bell, et al., uses the at-rest earth pressure in design which is admittedly quite conservative.[5] Leshchinsky, along with other authors, bases design theory on the Rankine active earth pressure coefficient, $K_a$.[6] Tests performed by Holtz and Bromas indicate that the actual applied pressures may be a combination of the two.[7]

External stability is provided by the mass of the gravity type retaining wall. The composite structure interacting with its foundation soil is under an influence of applied lateral earth pressures produced by the retained earth. The wall, which acts as a unit, must be designed to withstand the same conditions promoting failure as any gravity structure. The four considerations which need to be evaluated are overturning at the toe of the wall, sliding along the base of the wall, bearing capacity of the foundation soils and settlement of the structure. Critical limits of the wall deflections propagated by the external forces are moderately relaxed due to flexibility of the structure.

## Design Procedures

Approximately 2820 ft. of flood wall was constructed along Levisa Fork and Weddington Branch in Pikeville, Kentucky. One section of the wall, about 120 ft. in length, required positioning at the top of the slope which resulted in a 1.5H:1V grade down to the edge of the river. The wall along the section has approximately 13 ft. of free face above the top of slope and applies a strip load of approximately 10 kips per lineal ft. of wall (1380 psf).

The soil profile of the river bank consists of about 13 ft. of silty sand fill with minor amounts of miscellaneous material. The fill is likely a result of grading for the shopping center. Beneath the fill is a recent alluvium composed of loose silty sand with clay. The fine-grained soil has plasticity and has very little cohesive strength. This stratum continues to about 28 ft. below original grade (prior to construction of the flood wall). Dense well-graded sand and gravel lies below the alluvium. Shear strength of this material is much greater than the silty sand. Groundwater was found to be about 19 ft. below the original ground surface at the top of the slope.

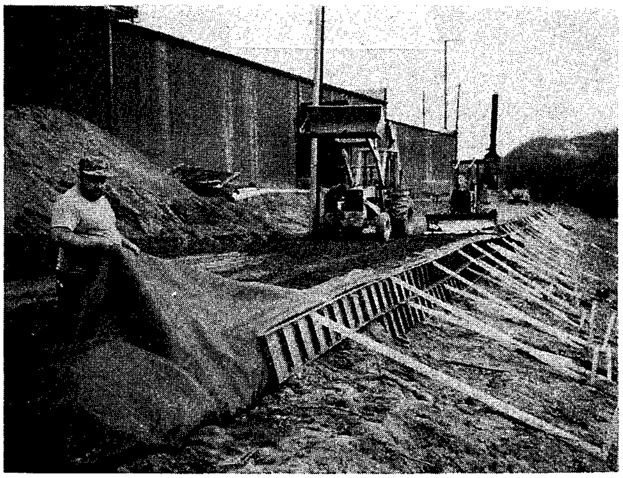

Figure 1. Construction techniques of the geotextile retaining wall.

The reinforced wall stabilizing the critical slope of the river bank was designed by methods outlined by Steward, et al.[8] This method was utilized for design of many of the early retaining walls by the U.S. Forest Service. The following steps are listed as an explanation of this procedure:

1. Determine the angle of internal friction and total unit weight of backfill.

2. Develop the lateral pressure diagram from both earth pressures and live loads or surcharges. $K_o$ is the lateral earth pressure coefficient used in this design.

3. Determine the vertical spacing of the fabric layers from the equation:

   $s = T_{all} / (F.S.)(\sigma_h)$; (1)
   where $T_{all}$ = the allowable tensile strength of the fabric and $\sigma_h$ = the composite lateral pressure.

4. Determine the required fabric embedment length to resist pullout by the equation:

   $L_e = T_{all}/(F.S.)(2d\gamma \tan 2/3\phi)$; (2)
   where d is the vertical distance from the top of the wall to the fabric layer.

   This embedment must transect the Rankine failure plane.

5. Determine the length of fabric overlap at the exposed face of the wall by the equation:

   $L_o = \sigma_h(F.S.)/2 d_f \tan 2/3\ \phi$; (3)
   where $d_f$ is the depth to the overlap.

   The Forest Service method recommends at least 3 ft. for the overlap to provide adequate contact for the soil and fabric.

All of the variables regarding internal structural dimensions are developed in the presented method. The procedures are predicated on assumed fabric characteristics such as tensile strength and the soil-fabric friction angle. One important parameter not included in the procedure is the allowable fabric strain at the mobilized stress.

The design developed for the present application from the Forest Service method is illustrated in Figure 2. It should be noted that the design was based upon an angle of internal friction of 35 degrees for the backfill soil. The fabric is a woven polypropylene with an ultimate strength of 200#/in. as determined by the wide width tensile test method.

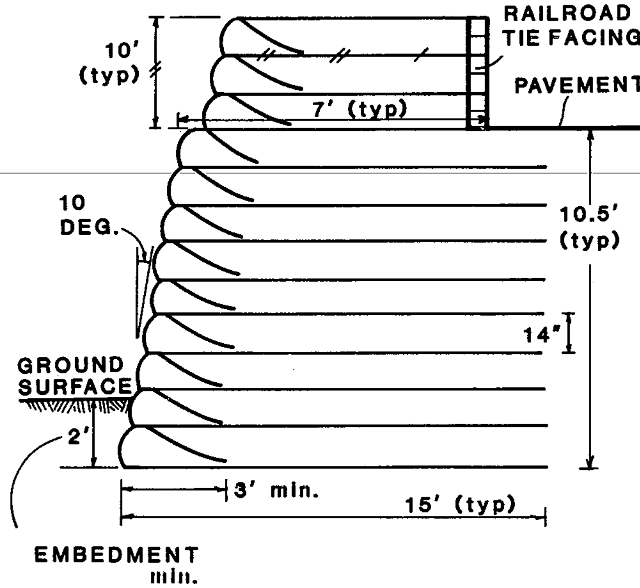


Figure 2. Typical section of the geotextile retaining wall.

The retaining wall was then analyzed to confirm that external stability was satisfied. This was completed by evaluating the four stability requirements listed above. Also included in this analysis was the determination of overall slope stability. The critical slope along the river bank of the Levisa Fork was evaluated using a computer simulation entitled "PCSTABL" developed by C.W. Lovell, et al., at Purdue University in 1976. The critical failure surfaces were determined by the Janbu method and the Modified Bishop's method. To allow a comparison of stability between the slope without and with internal reinforcement, the two profiles in Figures 3 and 4 illustrate the subsurface conditions and critical slip surfaces. The safety factors increase from an average of 0.8 for the slope without reinforcement to a value of 1.4 with the incorporation of the reinforced slope section.

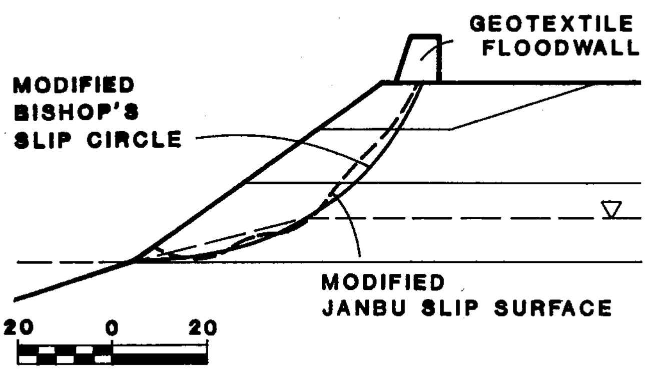


Figure 3. Profile of embankment illustrating slope stability analysis without geotextile retaining wall.

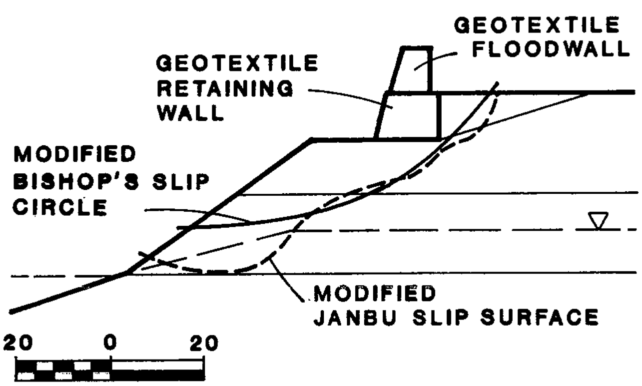


Figure 4. Profile of embankment illustrating slope stability analysis with geotextile retaining wall.

The final consideration which needs to be addressed is that degradation of the exposed geotextile is propagated by ultraviolet light. The time rate of deterioration due to UV light is a function of the type of polymer and the possible inclusion of carbon black in the product. There are many possible methods of protection including gunnite or bitumen coating and timber facing. The subject wall was coated with AC-10 bitumen at an application rate of approximately 0.25 gal./s.y. and sand broadcast on the asphalt.

Contemporary methods allow the designer to geometrically create the retaining wall based upon the external influences and physical constraints. Lateral pressures must still be determined to develop background conditions affecting the wall components. The remainder of the design parameters may be developed from a parametric analysis. The relative impact of backfill material properties, geotextile stress-strain characteristics, fabric embedment, spacing and overlap, and friction properties, may be logically arranged in a series of comparative analyses to evaluate the best solution for the required external conditions. As an example, if a granular material with a known angle of internal friction is a less expensive backfill than a higher strength select material, the difference in required material characteristics and wall geometry may be determined for both materials. This provides a tangible comparison for economic and constructibility considerations. Further analyses may be completed by iteration after varying certain internal properties. The relative importance of changing a variable is easily determined as some properties will have a greater effect on the overall design.

Figure 5. Completed section of the fabric reinforced wall.

Leshchinsky provides an extremely useful design method in which graphs are provided to obtain parameters for subsequent design steps.[9] The main point to be emphasized is that following this and other contemporary design methods, the physical properties of the geotextiles are the final parameters developed. This is a different approach from the earlier design methods where the fabric properties were initially assumed. Therefore, the design is not a function of geotextile manufacturers' specifications, but rather a development of the engineer.

## Summary

The use of geotextile fabric as an integral component in a retaining wall has been demonstrated. The owner/constructor has found construction to be relatively simple and that he has been able to accomplish it with local materials (except geotextile), equipment, and labor. Construction has been more economical than other types of slope reinforcements considered.

The design has been presented for the applied situation. Although it is understood that, with increased knowledge of fabric properties and performance, more exact design procedures will be developed. References to more current methods have been made to allow the reader to review.

Testing methods are presently being developed to permit the engineer to formulate design criteria independent of the geotextile manufacturers' specifications. Certain fabric properties are not well understood, such as time dependent fabric creep. It is believed that, with time and continued use and instrumentation of fabrics in geotechnical applications, the engineering community will gain confidence and understanding.

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# INVESTIGATION OF FOREST CITY LANDSLIDE IN SOUTH DAKOTA

by

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Abstract. The Forest City landslide is one of hundreds of ancient landslides along the Missouri River that had been apparently stable prior to the construction of Oahe Reservoir. Many of these slides have been reactivated due to the impoundment of the reservoir water and are creeping toward the reservoir at various rates. The described landslide, approximately 1 mile wide and 0.75 mile long, is currently threatening the structural integrity of the bridge crossing and its approach roadway. The overall mass is traveling toward the bridge at a speed of several inches per year. Enclosed are the description of the nature of the landslide, the results from the geotechnical investigations, the movement analyses, and the stability analyses, and the proposed corrective measures to stabilize the landslide and/or to avoid the structural failure of the bridge crossing.

## Introduction

U. S. Highway 212 crosses the Oahe Reservoir (Missouri River) via the Forest City Bridge, and connects Cheyenne River Indian Reservation and western cattle and agriculture areas with eastern markets and commerce centers (Fig-1). This bridge is currently threatened by a large active landslide. Structural integrity of the bridge and its southern approach roadway are rapidly failing. The closure of the bridge would result in an 85 mile detour over lower quality roadways with a substantial increase in travel time. Economic and social conditions of a large portion of northwest South Dakota would then be gravely affected.

The specific area of movement that is affecting the bridge and approach roadway is approximately 1 mile in width and 0.75 mile in length from head to toe (Fig-2). The depth of unstable soil ranges from 60 to 200 ft. depending upon local relief. The average depth of the slide is 95 ft. This volume of moving material is approximately 75 million cubic yards. The sliding mass itself is complex and comprised of many large blocks and structureless soils. The overall mass is traveling toward the bridge at a speed of several inches per year. Soils in the immediate vicinity of the bridge foundation are currently moving approximately 0.2 inches per year.

The Forest City Bridge was originally funded by the U.S. Corps of Engineers, and was a replacement structure required because of Oahe Dam impoundment water. Design was done by the State of South Dakota with subsurface investigation, location and design reviews performed by the Corps of Engineers. Construction was essentially completed in 1958 and the roadway remained stable until about 1968 when Oahe reservoir water first reached its normal operation level. Geotechnical problems were first recognized in 1968 in the 5,000 foot long southern approach roadway. Distress problems in the 4,600 foot long bridge apparently were first observed in 1962 with minor shortening of the bridge deck. Changes in the approach and the structure have been continuous to date. Though major repairs were made to the southern end of the bridge in 1976 when the concrete abutment supports had to be replaced, maintenance to the approach roadway has been continuous. Landslide movement has again caught up with the repaired bridge and stresses are beginning to accumulate on the new abutment. Structural integrity of the bridge is again gravely threatened.

This paper describes the nature of the landslide, its soil investigations, the ground movement characteristics, the results of stability analyses, and the possible solutions to correct

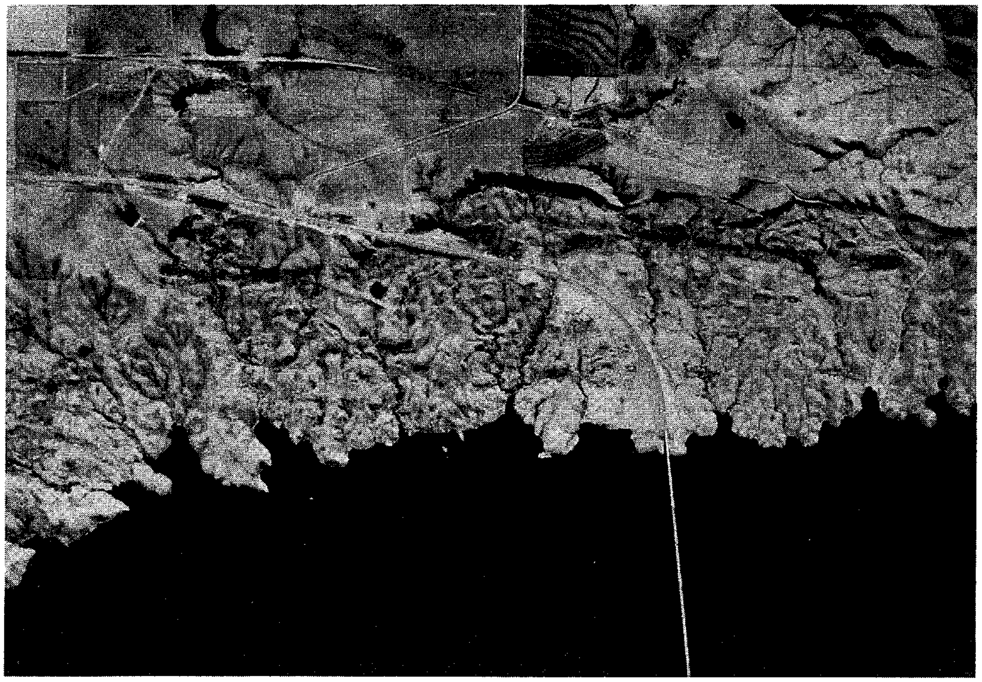

Figure 1. Aerial photo.

the problem. The soil investigations include the results from field boring, long term and short term soil tests, and ground water monitoring. The stability analyses include the overall sliding mass and local masses of bridge approach and fills. The considered corrective solutions include redesigning the bridge structure, lowering of hydraulic head, stabilizing the mass by excavation, and constructing slurry trench walls. These possible corrective measures are individually evaluated and compared in economical and technical aspects.

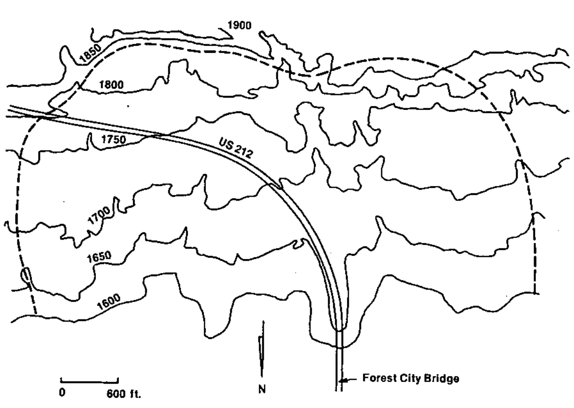


Figure 2. Area of landslide movement.

## Geotechnical Investigations

The landslide mass predominantly consists of hard clay (Pierre shale formation). Landslides in the Pierre shale deposits are usually associated with the deterioration of the soil resulting from combinations of unloading, chemical weathering and subsequent innundation. This deterioration at Forest City is very complex involving numerous components and rates of movement. To determine the stratigraphy at the site, the subsurface model for stability analyses, the slip plane and the material properties, following geotechnical investigations have been made.

### 1) Drilling and Sampling

An NQ-3 triple-tube wire line core barrel was used for the drilling and sampling. The samples from the NQ-3 barrel (1.75 inches) were used for the determination of water content, density and residual shear strengths. A total of twenty five slope inclinometer casings for the measurement of ground movements has also been installed using a heavy duty Acker Model SP-68 core drill combined with 4.5 or 6 in. continuous auger and a barge mounted Acker Hillbilly rotary drill.

Core samples were obtained using a California-type Retractable Plug Sampler. This thick walled sampler was 2-7/8 in. in diameter (PK rod) and was driven with a 490 pound drop hammer falling 30 inches. This sampling method is rugged, dependable and is used without casing. Though this sampler has definite limitations when used in soft soils, it does provide adequate samples for testing hard shales.[1] The obtained samples were used to locate the slip plane and to determine the unconfined compressive strengths, mositure contents, densities, soil classifications and residual strengths. Fig-3 shows a typical cross section of the bore hole as well as the unconfined strengths and moisture contents of the soil.

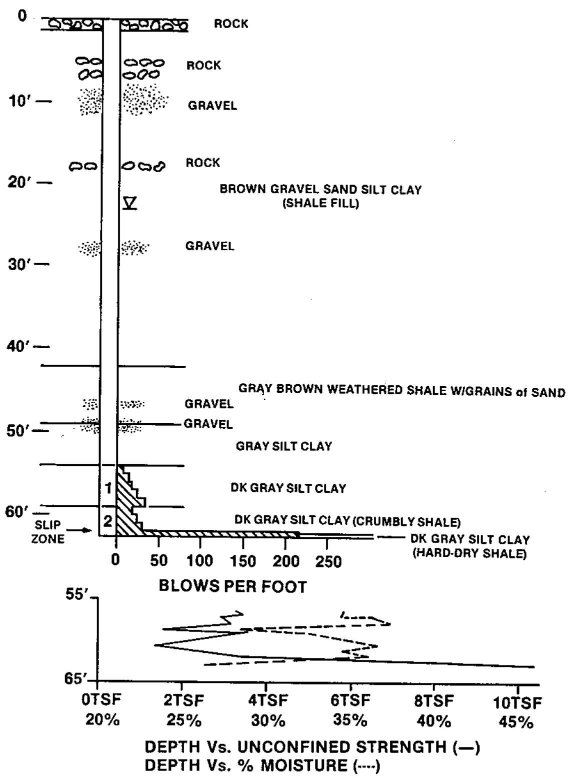


Figure 3. Typical borehole cross-section.

2) Material Testing

Tests run on samples from the Forest City site can be broken into two categories; simple rapid tests and long term direct shear tests. The simple tests were used to obtain data on densities, water contents, grain size distributions and unconfined strengths. The long term tests were used to determine the residual strengths of the shale.

For intact shale, the maximum value of the wet density was 124 pcf with a corresponding water content of 22.7% and the minimum value of the wet density was 107 pcf with a water content of 48.3%. The value chosen for the stability analyses was 120 pcf of the wet density. In weathered shale the wet densities varied from 106 pcf to 123 pcf, and 115 pcf was chosen for the stability analyses. These values correspond with the values found by Scully [1] and Fleming et al. [2] The value of 135 pcf chosen for the silty clay was the result of analysis of soils from the site and experience with eastern South Dakota glacial soils. Unconfined strengths varied from 0 to 11.4 tsf for the sound shale, 0 to 6.6 tsf for the weathered shale and 0 to 1.3 tsf for the silty clay. Grain size distribution showed that the shale was primarily clay. Glacial soils vary from silty gravels to silty clay tills.

The variations in blow count, water content and unconfined strength can, along with slope inclinometer data, all be used to locate the slip plane. The shale above the slip plane is softer as reflected both by the blow count and unconfined strength and the water contents are higher than those in the shale below the slip plane.

Two soil samples were selected for reversible long term direct shear tests. The first sample was taken from a shale deposit within the zone of sliding and the second was taken from a bentonite layer also within this zone. The residual angle of friction for the shale samples was found to be between $6.4^{0}$ and $8.0^{0}$ and for the bentonite layer between $2.8^{0}$ and $4.0^{0}$. These values are consistent with the values obtained by other investigators [2, 3, 4]. The facts that these values nearly coincide with the values found in other studies and that the staility analyses using these values give reasonable factors of safety eliminate the necessity of more extensive testing.

3) Groundwater

Three types of groundwater data collection systems were used at the site; open observation wells, continuous water level recorders (Stevens Model 68, Type F) and pneumatic piezometers (Petur Model P-1000). Utilization of slope inclinometer casings for observation wells has been difficult, because most of them are sealed plastic pipes that are grouted into the bore holes.

The data have indicated that the phreatic surface at the site is located between 20 and 40 ft. below the present ground surface as shown in Fig-4, and approximately independent of the reservoir water elevation. This finding indirectly implies that the Oahe pool-derived-pore water pressures are negligent compared to the pore water pressures apparently derived from the groundwater located in the higher reaches of the site.

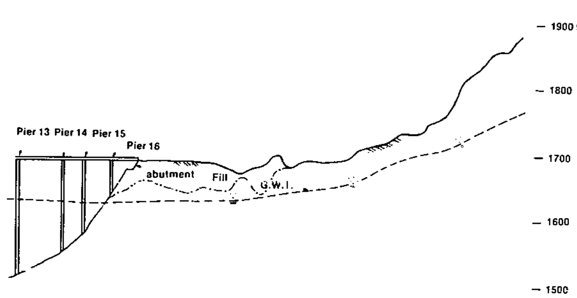


Figure 4. Ground water table.

## Ground Movement

The ground movements associated with the Forest City landslide are very complex. In reality, the mass is a combination rotation-translational failure with many large graben block features that are capable of joint and/or independent movement. Directions and rates of individual block movements are variable. In effect, the Forest City sliding mass is a large volume of numerous blocks and weathered soil matrix, capable of independent movements, moving as one general translational block on a roughly linear, noncircuilar plane. It is probable that the present day geometry of the sliding mass has evolved from complex past rotational and slump failures. In recent history, movement began along rejuvenated previous instabilities.

Independent local slides are also recognized in the roadway approach fill area that was constructed over the preexisting unstable mass. It is thought that all local slides in the approach fill area are controlled by buried preexisting slump blocks. Physical features and aerial photo evidence enhances this concept.

The slip plane of the massive slide runs from scarp cracks at the edge of the plateau, approximate elevation of 1900 ft., to a toe location, approximate elevation of 1580 ft. (Fig-2 and 4). The horizontal distance from head to toe is approximately 3,300 feet and the vertical differential between head and toe is 320 feet. Piers 16 and 17, and abutment are on the moving mass. Depth of the slip plane varies from 180 ft. near the crown to 110 ft. beneath the abutment. The plane is approximately 40 feet below the shoreline when the pool is at its average elevation of 1600 ft. The approximate average elevation of the slip plane in the area of the approach is 1560 ft. Absolutely no evidence exists that would indicate a deeper movement zone - either acting with or independent of the Forest City slide. The slip plane itself is found to vary from zoned slickensided weathered shale to soft nonweathered shale resting on in situ hard shale. No evidence has been found to indicate a failure control by a bentonite seams or similar discontinuity.

On the basis of slope inclinometer data, the thickness of active slip plane zones has been found to vary from about 2 in. to 8 ft. The bottom of the plane is easily defined, whereas the top is more or less gradational and undefined. All the evidence shows that the overall mass is naturally being diverted around the area of the bridge abutment. This is soundly reinforced by the inclinometer shearing directions and movement rate differentials between the upper main mass and the immediate bridge approach area.

Preconstruction aerial photos reveal local landslide features both upstream and downstream perpendicular to the approach roadway. This area would be later covered with up to 40 feet of embankment fill. Part of the embankment failed to the east during the construction; movement was apparently along preexisting slumps. At least 6 small jumbled and weathered slides have been observed in this immediate area. Evidence also exists that some movement took place between the years of 1938 and 1950. It appears that the major drainageway on the east continually eroded the toe(s) of the slides and slumping partially blocked the channel. A berm, presently submerged, was constructed across the valley mouth to stabilize these "local" slides. This area has been the location of the most severe roadway surface damages. Cracks through the asphalt roadway and surface deformations nicely correlate with now-buried slump topography. The present roadway approach problems relate to reactivation of preexisting slides - some of which were temporarily stabilized during construction. Reactivation apparently occurred in 1968.

On the west side of the approach, much of the same preconstruction history is revealed. One large scarp is observed exhibiting past movement perpendicular to the approach. This slide is less jumbled than those on the east and is associated with an incising drainage at the toe. The old head scarp has reflected through the fill near the edge of the roadway where a crescent-shaped scar is found. Underthrusting of material into the approach fill and buried slide mass is presently observed.

## Stability Analyses

Preliminary stability analyses of the Forest City landslide have been made using translatory and rotational slip planes with simplified Bishop's method and Morgenstern and Price's method [5, 6]. These are preliminary in nature, since the material properties have been assumed or back calculated.

A more comprehensive stability analysis has been made by the South Dakota Department of Transportation. The slide geometry and material properties were refined based on more accurate field data. The study included three profiles covering the overall sliding mass as well as the local slides (Fig-5). Two dimensional rotational noncircular planes were considered using Morgenstern and Price's method. [7]

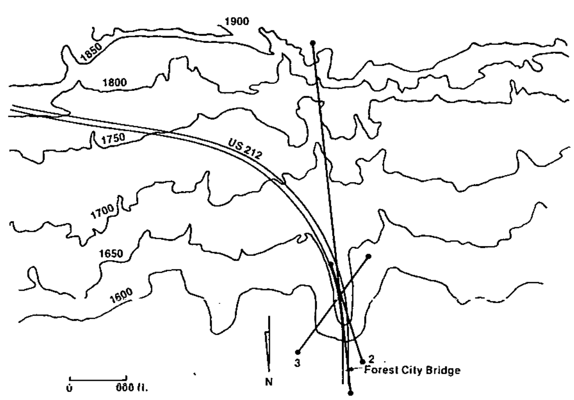


Figure 5. Stability analysis profiles.

Profile 1 pertains only to the overall mass. The most important aspect of this section is that the slide mass is failing from the top of the valley crest to the identified toe of the slide. Profiles 2 and 3 pertain to local slides. Local slides found in the approach are fairly straight forward in nature when compared with the overall sliding mass. The residual internal friction angle of $5.0^{0}$ with no cohesion was used for the analyses. These values were the average values obtained in the laboratory testing. A cohesion of 200 psf was used however for profile 3, where the slip plane was placed through the constructed berm. The stability analysis results of these three profiles are given below.

1) Profile 1

Four potential failure surfaces, A, B, C and D, as shown in Fig-6, were investigated. The surface A was found to be most critical with a resulting factor of safety of 0.97. Stability of this section was then further analyzed by removing 20 ft. of material from the head area. Removal of the overburden, approximately 20 ft. X 280 ft. X 900 ft., resulted in an improved factor of safety of 1.06, indicating that the removal of 187,000 cubic yards of driving material does not increase the stability of the overall slide substantially. A massive removal of 90 ft. of material along the center line tangent to the bridge was then considered. Such a cut, involving 1.5 million cubic yards, raised the factor of safety from 0.97 to 1.6. This would result in a method of stabilization with a factor of safety that would be acceptable. Both investigated methods of reducing driving mass did not address the waste disposition, ground water control, right of way, or the side slope stability of the cut sections.

Placement of a resisting berm near the toe of the sliding mass was also considered. Such a berm was however found to be inconsequential to the stability of the overall sliding mass - increment of the factor of safety from 0.97 to 1.0. A much larger berm could have been considered. However, it would be a monumental effort to place such a berm in a deep water environment.

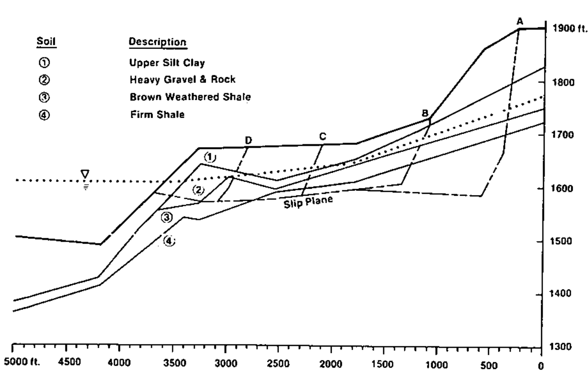


Figure 6. Profile 1.

2) Profile 2

Profile 2 addresses local slides off the abutment to the north. This is the area where the small rip-rap slides have been occurring sporadically since the filling of the reservoir. The calculated factor of safety along the surfaces as shown in Fig-7 ranged from 0.73 to 0.98.

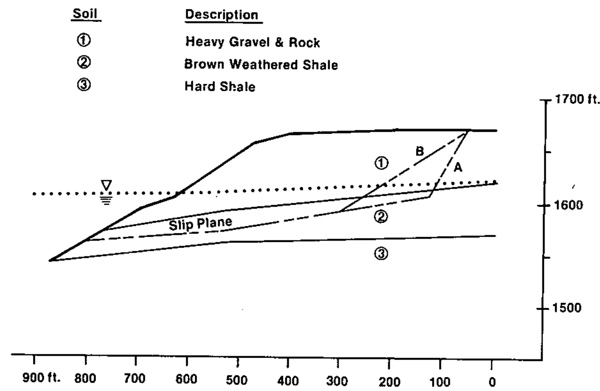


Figure 7. Profile 2.

3) Profile 3

This profile (Fig-8) contains the most active local slide area. The profile runs at the edge of a slide that moved during the construction and has apparently been reactivated. Because the profile is along a very active area, the factor of safety was found very low (0.6 to 0.72).

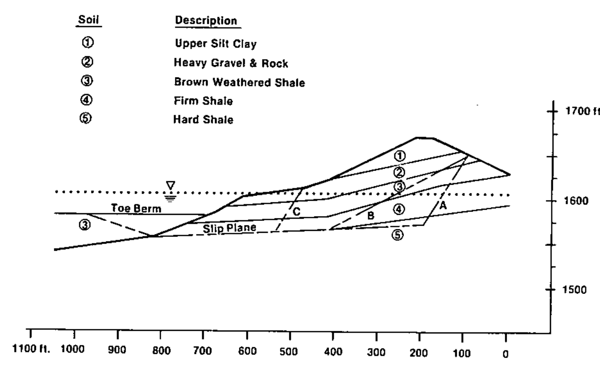


Figure 8. Profile 3.

Corrective Schemes

There are, of course, many different ways to accomplish the possible solutions for the landslide. However, when considering the physical features of the site, the landslide failure mechanisms and the probable costs of implementation, most of the geotechnical engineering options are eliminated or reduced in stature. Several corrective concepts thought possible are presented and discussed below.

1) Bridge Structure Redesign

This concept probably involves replacing at least four spans of a welded plate girder section (550 ft.) with a single truss span or cable stayed span. The new span would have to be fixed at the pier outside the sliding zone and be free at the abutment. An abutment system would have to be designed to allow the slide to continuously move under the structure. Though this concept would result in low to medium cost due to the fact that no need to stabilize the sliding mass is necessary, continual roadway maintenance must be considered.

2) Hydraulic Head Lowering

Lowering of the water table in the area of the overall mass does present a good approach to improve the stability of the sliding mass, especially in light of the hydraulic head found in the sliding mass. The physical accomplishment of such a concept would however be very difficult. Horizontal drains would be almost impossible to install because of the bouldery tills, long drain lengths and unknown locations of the aquifer zones. Lowering the water table by vertical wells could be conceivable, but the geological data indicate that the degree of fracturing and jointing in the shale deposit are extremely erratic, therefore the "area of well influence" may not be estimated correctly. Furthermore, since the fluctuation of the reservoir water should almost immediately be reflected within the approach fill area, the removal of water would have very little effect on improving the stability of the bridge approach fill area.

3) Excavation

As discussed previously, a moderate amount of excavation (187,000 cubic yards) does not increase the stability of the sliding mass substantially. Calculation shows a massive amount of material may have to be removed, approximately 1.5 million cubic yards, to increase the factor of safety within acceptable values. By this reason, the excavation method of corrective scheme is thought to be practically impossible.

4) Diversion

The sliding mass in general is being directed toward the southern abutment of the bridge. However, as discussed previously, a small scale natural diversion of the landslide around the abutment area is currently taking place. Stabilization by diversion is simply an attempt to insure that natural diversion continues in a location controlled by the engineers and to reduce the force components acting on the bridge approach fill in the process.

There are two possible ways of accomplishing the diversion. If the resisting force, larger than the driving force, is applied in the opposite direction of the slide, the sliding mass would then stop or go around. This concept may not be feasible simply because of the magnitude of the force involved. If however the resisting force is reduced in an area adjacent to the bridge approach fill, then the driving force, therefore the moving mass, would follow the path of least resistance and consequently move around the approach fill area. Evidence indicates that natural diversion is currently occurring, but not effectively enough to eliminate the forces on the abutment foundations. To accomplish this, artificial vertical shear walls can be considered around the bridge approach fill area to create diversion paths. The shear wall, approximately 1800 ft. long, 60 ft. deep and several feet wide, would be triangular in shape on the surface and, in effect would be a retaining wall with virtually no strength (Fig-9). A conventional slurry trench wall could be used for the shear wall. Construction of the slurry trench wall could be accomplished by excavating a trench and removing the natural soil, followed by replacing the soil with very low strength material. The back fill material would be a mixture of bentonite, cement, soil and water, forming a thick mud gel that would never set or completely dry out.

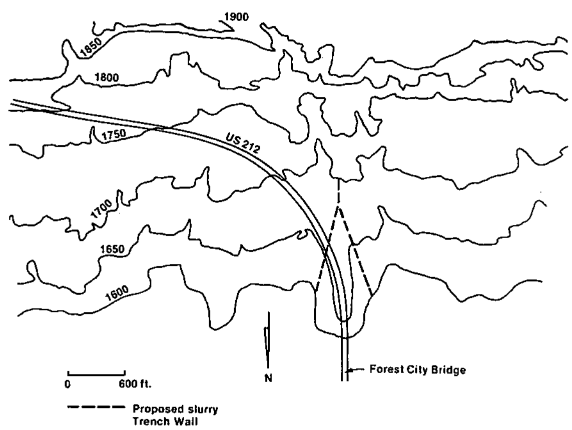


Figure 9. Shear wall location.

To verify the feasibility of utilizing the slurry trench wall for the diversion of the movement, simple three dimensional finite element analyses [8] with and without the slurry trench walls have been performed. Fig-10 shows the magnitudes and directions of the surface movement of the sliding mass without any slurry trench wall. As can be seen the majority of the movement is directed toward the existing bridge structure with additional sideward movements of the approach fill. Fig-11 shows the net differential surface movements of the sliding mass when the slurry trench walls are in place. The figure indicates that the anticipated movements around the approach fill area would be reversed in their directions or reduced in magnitudes. Though this calculation is preliminary in nature - linear isotropic material properties were assumed with simplified geometry due to a large amount of computation involved in three dimensional finite element analysis - it clearly shows that the concept of utilizing the slurry trench wall is technically feasible. This concept would result in relatively low cost and low to medium anticipated construction difficulties, and could solve the bridge instability problem without further local stabilization or at least with less additional works.

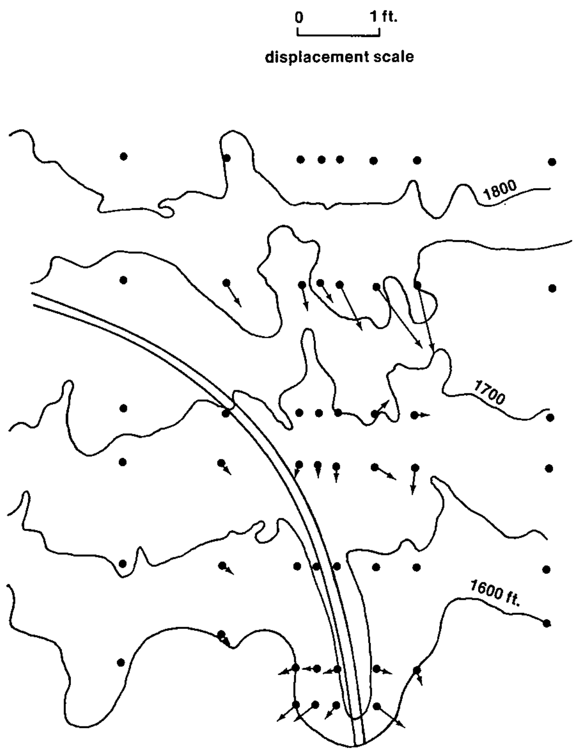


Figure 10. Surface movement without shear wall.

## Conclusion

A massive landslide near Forest City, South Dakota, has been described. The results of geotechnical investigations, ground movement analyses and stability analyses have also been included in detail. Several possible corrective measures are discussed. Among them an innovative technique of diverting the landslide movement, a slurry trench wall construction, has been proposed and its feasibility studied briefly. Though this technique has never been applied for the landslide corrections and/or preventions, the concept has been well known through many other applications of geotechnical structures. The technique however needs to be studied further before it can be implemented in the field, including a more elaborate analytical simulation, a parametric study to evaluate the relative significance of the length, thickness, location and shape of the trench wall, and possibly a combination with other techniques such as grouted anchors or root piles to strengthen the bridge fill area.

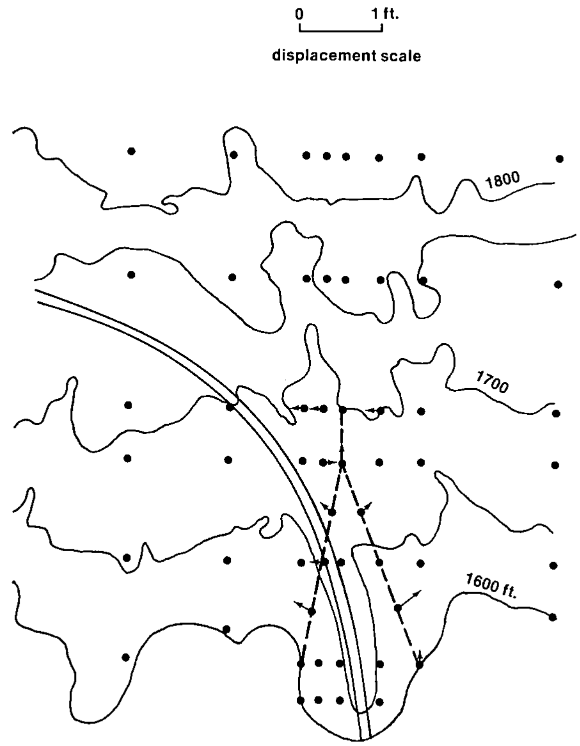


Figure 11. Differential surface movement with shear wall.

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# APPENDIX

Past Proceedings of
Ohio River Valley Soils Seminars

| | |
|---|---|
| ORVSS I: | BUILDING FOUNDATION DESIGN AND CONSTRUCTION, October 16, 1970, Lexington, Kentucky |
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| ORVSS VII: | SHALES AND MINE WASTES: GEOTECHNICAL PROPERTIES, DESIGN AND CONSTRUCTION, October 8, 1978, Lexington, Kentucky |
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| ORVSS XIII: | RECENT ADVANCES IN GEOTECHNICAL ENGINEERING PRACTICE, October 8, 1982, Lexington, Kentucky |
| ORVSS XIV: | FOUNDATION INSTRUMENTATION AND GEOPHYSICAL EXPLORATION, October 14, 1983, Clarksville, Indiana |

ORVSS XV: PRACTICAL APPLICATION OF DRAINAGE IN GEOTECHNICAL ENGINEERING, November 2, 1984, Fort Mitchell, Kentucky

ORVSS XVI: APPLIED SOIL DYNAMICS, October 11, 1985, Lexington, Kentucky

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