TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle	5. Report Date
"A Survey and Evaluation of Remedial Measures for Earth Slope Stabilization"	August 1974
	6. Performing Organization Code
7. Author(s) Rudolph J. Schweizer and Stephen G. Wright	8. Performing Organization Report No. Research Report 161-2F
9. Performing Organization Name and Address The Center for Highway Research The University of Texas at Austin Austin, Texas 78712	10. Work Unit No.
	11. Contract or Grant No. Research Study 3-8-71-161
	13. Type of Report and Period Covered Final September 1970 - August 1973
12. Sponsoring Agency Name and Address Texas Highway Department Planning & Research Division P. O. Box 5051 Austin, Texas 78763	14. Sponsoring Agency Code

15. Supplementary Notes

Work done in cooperation with the Federal Highway Administration, Department of Transportation.
Research Study Title: "Stability of Earth Slopes"

16. Abstract

The results of a literature survey undertaken to identify remedial measures which have been used to stabilize earth slopes are presented. In this review attention is directed to specific case histories and field conditions where the remedial measures were actually used.

The remedial measures reviewed include drainage of surface and subsurface water, restraint structures, elimination and avoidance of the slide area, benching and slope flattening as well as a number of special procedures including electro-osmosis, thermal treatment and addition of stabilizing admixtures. Of the procedures reviewed drainage of surface and subsurface water appears to be the most widely and successfully used technique. However, the success of each measure depends to a large degree on the specific soil and groundwater conditions for the slope and the degree to which these are correctly recognized in an investigation and design.

17. Key Words	18. Distribution Statement
literature survey, remedial measures, earth slopes, stabilization, case histories, drainage, restraint structures, benching, slope flattening	

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	137	

Form DOT F 1700.7 (8-69)

A SURVEY AND EVALUATION OF REMEDIAL MEASURES FOR EARTH SLOPE STABILIZATION

by

Rudolph J. Schweizer
Stephen G. Wright

Research Report Number 161-2F

Stability of Earth Slopes
Research Project 3-8-71-161

conducted for

The Texas Highway Department

in cooperation with the
U. S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

August 1974

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PREFACE

This report is the second and final report on the findings of Research Project 3-8-71-161, "Stability of Earth Slopes." Included herein are the results of a literature survey of remedial measures which have been employed for the stabilization of earth slopes. The types of remedial measures used, the soil and slope conditions where these have been used, the procedures for investigation, analysis, design and construction, and the success of the measures are reviewed and evaluated on the basis of available data.

The authors wish to acknowledge the support of the Texas Highway Department and the Federal Highway Administration for their interest and support of this study.

Rudolph J. Schweizer
Stephen G. Wright

August 1974

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

LIST OF REPORTS

Report No. 161-1, "A Survey of Earth Slope Failures and Remedial Measures in Texas," by Timothy G. Abrams and Stephen G. Wright, gives results of a survey of earth slope failures along Texas highways and the remedial methods employed.

Report No. 161-2F, "A Survey and Evaluation of Remedial Measures for Earth Slope Stabilization," by Rudolph J. Schweizer and Stephen G. Wright, includes the results of a literature survey of remedial measures employed for the stabilization of earth slopes.

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

ABSTRACT

The results of a literature survey undertaken to identify remedial measures which have been used to stabilize earth slopes are presented. In this review attention is directed to specific case histories and field conditions where the remedial measures were actually used.

The remedial measures reviewed include drainage of surface and subsurface water, restraint structures, elimination and avoidance of the slide area, benching and slope flattening as well as a number of special procedures including electro-osmosis, thermal treatment and addition of stabilizing admixtures. Of the procedures reviewed drainage of surface and subsurface water appears to be the most widely and successfully used technique. However, the success of each measure depends to a large degree on the specific soil and groundwater conditions for the slope and the degree to which these are correctly recognized in an investigation and design.

KEY WORDS: literature survey, remedial measures, earth slopes, stabilization, case histories, drainage, restraint structures, benching, slope flattening.

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

SUMMARY

A survey of the remedial measures employed for earth slope failures, the soil and groundwater conditions at the site, and the performance of the remedial measures is presented. The remedial measures include:

(1) drainage, consisting of
   (a) surface water control,
   (b) horizontal drains,
   (c) vertical drains and well systems,
   (d) stripping of unsuitable soils and backfilling with a select free-draining material,
   (e) transverse and longitudinal drainage trenches, and
   (f) tunnels;
(2) restraint structures, consisting of piles, piers and retaining walls;
(3) elimination and avoidance of the slide area by excavation or relocation;
(4) benching and slope flattening by regrading; and
(5) special procedures, including
   (a) electro-osmotic stabilization,
   (b) addition of stabilizing additives and chemical treatment,
   (c) thermal treatment,
   (d) slope planting,
   (e) use of reinforced earth, a patented process, and
   (f) freezing.

This review of remedial measures has shown that a number of remedial measures have been used and, depending on the site conditions, all have enjoyed some degree of success. The information presented in this report should be useful in establishing preliminary selection of remedial measures for slide stabilization.

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

IMPLEMENTATION STATEMENT

The results of this research indicate that a relatively large number of types of remedial measures has been used for stabilization of earth slopes and all have enjoyed varying degrees of success. The success of any one of these measures depends to a large extent on the use of proper and thorough field and laboratory investigation procedures and employment of established principles of geotechnical engineering for evaluation and design of remedial measures.

The results of this research are intended to aid the field engineer in recognizing possible alternatives for the repair of earth slopes and making preliminary qualitative evaluations of their feasibility. In addition the stability charts presented in Chapter 8 provide a means for making some quantitative predictions of the potential effectiveness of either flattening or benching an earth slope to improve its stability.

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

TABLE OF CONTENTS

CHAPTER 1. INTRODUCTION

Virtually every highway department in the United States has been troubled by problems of landslide stabilization and prevention (Smith, 1958). The Texas Highway Department has spent considerable time and effort to correct slide problems in several Districts. This problem is expected to increase as years of average or above average rainfall occur, and as new construction proceeds the problem of preventing and controlling landslides will become increasingly important.

One of the problems associated with slides has been the lack of dissemination of available information regarding the techniques and various applications of these techniques which have been successfully applied in the prevention and correction of landslides. The purpose of this report is to present, in one volume, a number of the means available to the engineer for the control of landslide problems. In doing so, various remedial measures are discussed, case histories are presented to illustrate the use of these methods, and case histories found in the literature are summarized.

In addition to the review of the literature concerning remedial measures, stability charts are presented which may be used to determine the effects of slope flattening and benching on the stability of a given slope. These charts are shown to be more accurate than existing charts and to result in more economical slope designs.

This page replaces an intentionally blank page in the original.
-- CTR Library Digitization Team

CHAPTER 2. ECONOMIC IMPORTANCE AND GENERAL INVESTIGATIONS

Economic Significance of Highway Related Landslides

On May 11, 1969, one of the main commuter routes to San Francisco, Interstate Route 80, near Pinole, California, was closed to traffic. Within hours a section of the embankment 400 feet in length slid out leaving a single lane of the six-lane interstate highway in place. Because adequate safety precautions had been taken, no lives were lost; however, the economic significance of this single highway landslide is of consequence. Because of the necessity of a high capacity commuter artery to serve the San Francisco area, construction of a six-lane detour was started immediately. The cost for the temporary detour was approximately $350,000. By April of 1970, when the stabilizing work was complete, the total cost of this slide was more than $1,250,000 (Smith, et al, 1970).

Reliable estimates as to the yearly costs of highway related landslides are difficult to obtain. The nationwide questionnaire issued to state highway departments in 1956-57 by the Committee on Landslide Investigation revealed the following data: one state reported annual costs in excess of $1,000,000; three between $500,000 and $1,000,000; one between $250,000 and $500,000; five between $100,000 and $250,000; six between $25,000 and $100,000; and eleven less than $25,000 (Highway Research Board, 1958). Smith (1958) states that these figures are probably low because many highway department accounting methods fail to fully disclose maintenance costs that are directly related to landslide problems. In addition most highway departments state that costs of numerous small landslides which are handled as routine maintenance problems and not reported as cost for slide correction, should be added to the reported figures.

The above costs indicate only direct costs for the correction of active landslides. Smith (1964) estimates that 10 percent of the original construction cost of interstate highways in mountainous terrain is for control of subsurface water for landslide prevention. Contract change orders have typically increased this percentage to between 14 and 19 percent of total construction costs.

Symons (1970) has recently quantified information regarding the magnitude and cost of minor slides on major roadways in England. For this study, Symons examined a 250-mile length of motorway (similar to interstate highways) and 90 miles of major trunk road. Three factors were found to considerably influence the frequency and cost of instability problems along the roadways studied: the size of the earthwork (both in cut and fill sections), the soil types through which the road passes, and the age of the slopes.

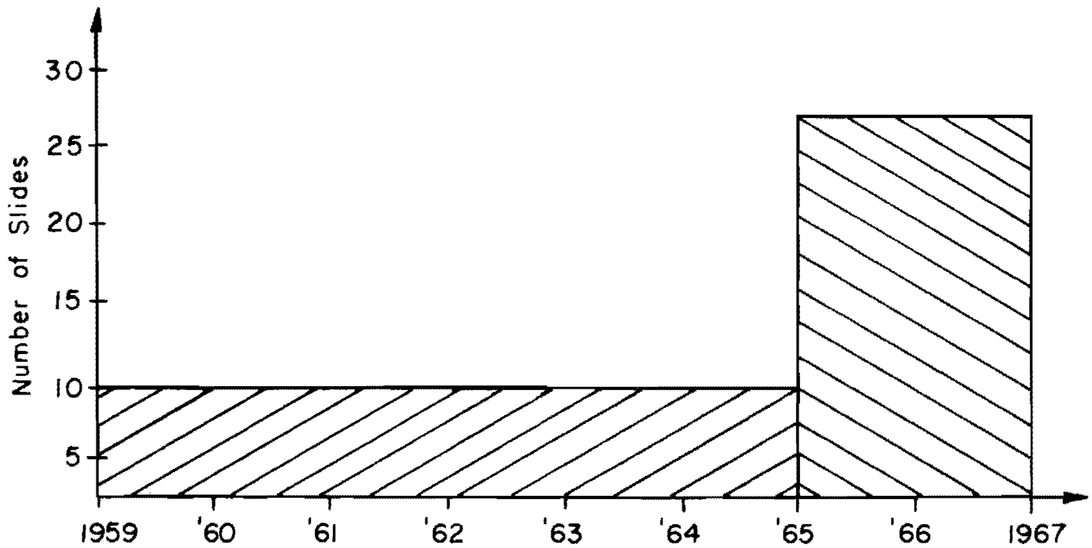
Of the 75 miles of motorway which opened in 1959, sections of instability were confined to short lengths which represented less than seven percent of the length of the road. Standards of construction were generally the same for all counties and it therefore seems probable that the instability problem is governed by soil types through which the road was constructed. Areas of instability were limited to embankments and cuttings greater than 16 feet. Two main factors were considered to have contributed to the problem: inadequate surface and subsurface drainage and construction using unsuitable borrow material.

The comparison of age of the slope versus magnitude and cost of the problem is illustrated in Fig 2.1. Of the roads which were opened in 1959, approximately 50 percent of the cost and 60 percent of the failures occurred during 1965 and 1966, suggesting that the problem increased with the age of the slope.

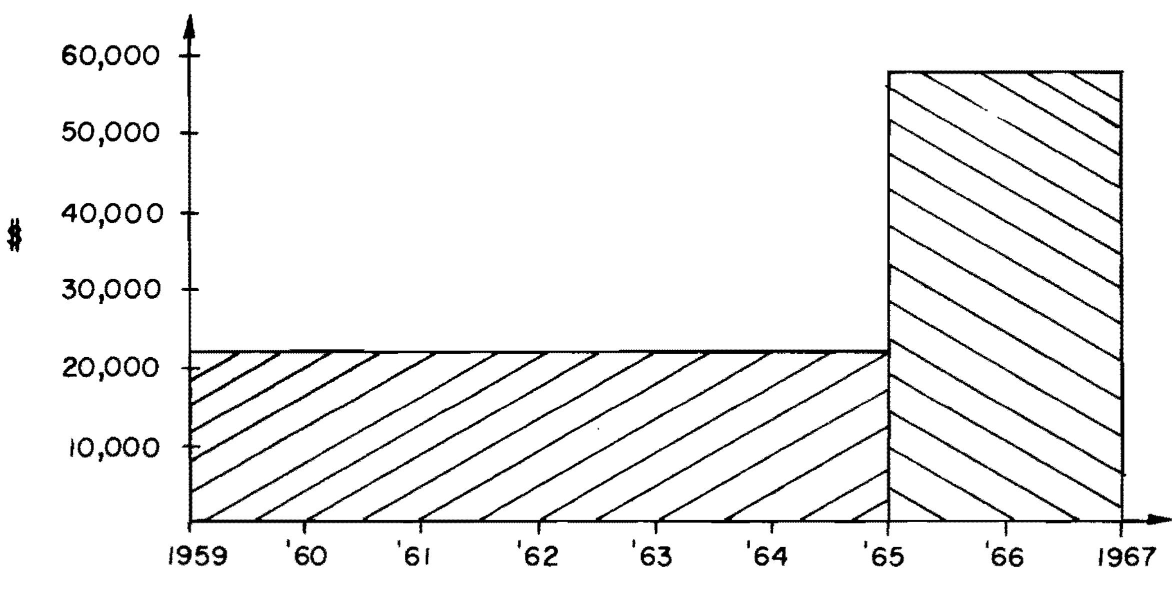
Annual costs for correction of minor instability problems on the sections studied averaged approximately $600 per mile of roadway constructed prior to 1960. Where side slopes were flatter than 2:1 this cost was less than $200 per mile. These figures represent approximately 10 percent of the annual maintenance budget of $4000 per mile.

While the study by Symons indicated that the problem was not sufficiently serious to warrant fundamental design changes, it illustrated that particular care is necessary to insure adequate subdrainage in slide susceptible areas, and that control of embankment material and construction must be increased as deeper cuts and larger embankments become more necessary.

Even though isolated sliding may represent but a small amount of the overall maintenance budget, large highway landslides and the overall problem are of considerable economic significance. The Committee on Landslide



b.) Average Number of Failures Per Year.



a.) Average Yearly Cost for Minor Instability Problems.

Fig 2.1. Summary of costs and frequency of occurrence for slope failures surveyed in England (Symons, 1970).

Investigation (Highway Research Board, 1958) estimated that over $10,000,000 are spent annually for highway landslide prevention and correction within the United States.

Investigations and Analysis

Because of the magnitude in the variation of types of slides, it is extremely difficult to establish meaningful procedures for soil investigations and analysis in order to determine what type of corrective measure might be most applicable to a given slide. The extent of such investigation or analysis will be governed by such items as engineering experience with similar slides in the area, the potential danger or economic loss from repeated occurrence, and the cost of the investigation versus benefit that may be derived. Every slide is different and accordingly will be treated somewhat differently. However, when dealing with slides that are large enough to warrant office and field investigations, general patterns exist, and, consequently, general guidelines should be made available for the engineer to apply. These guidelines may indicate what type of procedures has been used in case histories reported in the literature and are intended to illustrate a general pattern that has been followed with some degree of success in the past.

As in most types of geotechnical engineering work the first step in the investigation and analysis process should be a preliminary office investigation of all available information on the area in the vicinity of the slide. This should include such information as local geology, general groundwater conditions, rainfall in the previous years, rainfall in the period immediately preceding the slide, general soil type, aerial photographs, and any plans or cross sections which may be available for the area in question. In addition, correction methods which have been previously used in the area should be examined. This stage should be combined with a preliminary site investigation.

The principal objectives of the initial field and office studies are to classify the slide movement as to type, to determine the extent of movement, to determine the need and depth of additional studies, and to determine the probable methods of correction which may prove feasible. The alternative methods of correction should be generally compared for economy. In some cases it will prove more economical to eliminate the slide either through avoidance of the slide area or removal of the sliding mass. If such is the case further field studies may not be warranted. The advantages of these initial investigations lie in the savings that may be realized in future field and office analysis.

The second phase in the solution of the problem should be a detailed field and office investigation directed toward understanding the mechanics of the slide. It is important that this investigation be conducted by field personnel who are familiar with the local landslide problem and are aware of the various measures available for the correction of the problem. Baker (1952) suggests that this information include the extent of the slide; the type and topographic description of the underlying soil, both in and adjacent to the slide; the type, character, and topographic description of the underlying soil and bedrock; the location of any groundwater; and location of any possible seepage strata.

In this stage any information is collected which will aid in the design of any remedial measure which is being considered. The details to be obtained from this field study will depend on whether a complete analysis has been deemed necessary by the preliminary office and site investigations. During this stage of the investigation, depending on the types of corrective measures being considered and the extent of the investigation decreed necessary by the preliminary studies, samples should be taken and borings logged. This subsurface work will generally provide such information as detailed soil classification, groundwater and moisture conditions including Atterberg Limits, soil density, and shear strengths in the slide area. In many cases, the area under investigation will not be conducive to classical, theoretical methods of analysis; however, in most cases, application of soil mechanics principles will provide a means to a somewhat rational comparison of the various methods of treatment. The final analysis and interpretation of the data is very much a function of local engineering experience, and steps in this procedure have been suggested (Baker, 1952; Root, 1955a; and Smith, 1964) and are evident in the literature.

As part of the detailed investigations and analysis, typical cross sections of the slide area should be prepared. All available data uncovered by the preliminary or detailed investigations should be appropriately noted on the cross section. This has proven the most effective way to organize slide information (Root, 1955a; Smith, 1964). The location of the slip plane should be reliably determined and marked on the cross section. Means available for this include interpretation of field data (Root, 1955a), visual examination of test pits or bore holes (Baker, 1952), use of the geometry of known points both before and after sliding (Philbrick and Cleaves, 1958; Toms and Bartlett, 1962), or use of an inclinometer (Toms and Bartlett, 1962; Wilson, 1962). Accurate location of the failure surface is required in order to properly evaluate the influence of the proposed corrective measure on the slope in question.

The last phase in the analysis and investigation procedure is to determine what corrective action(s) will produce the desired result. If drainage is considered, the potential effectiveness of a reduction of seepage or groundwater levels should be determined. The effects of buttressing or retaining walls should be analyzed by means of an appropriate stability analysis. It should be noted that the correction of an existing slide or the prevention of a pending slide is a function of a reduction in the driving forces, an increase in the available resisting forces, or avoidance or elimination of the slide. Any remedial measure used must provide one or more of the above results.

CHAPTER 3. DRAINAGE FOR LANDSLIDE PREVENTION AND CORRECTION

Introduction

Drainage of surface and subsurface water appears to be the most successful remedial measure for the correction of active landslide problems. California (Baker, 1953; Smith, 1964) reports that subdrainage used in combination with excavation (slope flattening or benching) has been the single most successful method for treatment of large slides. Downs (1930) indicates that subsurface drainage has successfully treated several slipouts in West Virginia. Ladd (1928), Downs (1930), and Root (1958) state that West Virginia provides surface runoff protection for all highway related cuts or embankments. In addition to this, all areas where the flow of groundwater may be altered by new construction are protected by a surface and subsurface drainage system. Smith (1964) states that in areas of complex groundwater conditions groundwater control costs represent from 10 to 15 percent of the total contract cost, and that excellent returns result from this expenditure.

Catastrophic slides have occurred when adequate drainage provisions have not been provided for in the initial design. The slide on Interstate Highway 80 near Pinole, California (Smith, 1970), numerous slides in the San Francisco area (Forbes, 1947), and several West Virginia slides (Ladd, 1928; Downs, 1930; Parrott, 1955) give testimony to this. If groundwater problems initiated the slide activity, adequate groundwater control will frequently produce a stable slope. However, many variables influence the problem of groundwater control and as a consequence other methods of correction are usually used in conjunction with drainage to insure a stable slope.

Many different techniques for surface and subsurface drainage have been used for the prevention or correction of landslide problems. These methods have not all been successful; the same methods in different types of materials have not all been successful. Many have been used in conjunction with slope excavation and their effect alone is not readily determined. However, in most cases, it is evident that drainage has substantially increased the stability of a given slope.

Surface Drainage

Although no case histories have been found which indicate surface drainage as the only corrective measure for landslide stabilization, all reports strongly indicate that the first step in correction should be to insure that all surface runoff is prevented from entering the slide area. The primary use of surface drainage is for prevention of slides in potentially unstable areas and it is equally applicable to both cut and fill sections. Root (1958) states that "any sags, depressions or ponds above the slope line of either an embankment or cut should be drained to minimize the possibility of surface water percolating into a potentially weak or unstable area."

Surface drainage has almost always been employed to aid in the solution of active slide problems. In the case of potential slides, where no movement has occurred, surface drainage may result in greater returns for the investment than any other preventive treatment (Forbes, 1947; Root, 1955a). Techniques which have been used to improve surface drainage including reshaping of slopes, construction of lined ditches, seeding or sodding, treatment with bituminous material, thin masonry or concrete walls, and installation of flumes or conduits.

Mencl and Zaruba (1969) recommend that the first step in correction of an active landslide include drainage of surface water flowing into the slide area, dewatering of all drainless depressions, and filling and tamping of all open cracks which could be pervaded by surface water. Baker and Marshall (1958) recommend that open ditches should be constructed to completely surround the slide area and intercept runoff from higher ground and that care should be taken to locate such runoff trenches so that they will not become blocked by slope debris. Caution must be employed if a ditch is to be constructed within the active slide area. It should be sloped to provide a rapid flow, or its base should be sealed with an impervious material. If not, it may become a device for feeding water into the slide rather than draining the area. Open ditches are often employed to drain ponds or springs in the area.

The sealing of surface cracks in slide areas will often reduce the amount of slide movement by preventing the entrance of surface water and the subsequent buildup of hydrostatic pressures in the slide mass (Baker and Marshall, 1958). Clay, bituminous materials, cement grout, dry lime, and lime slurries have been used to fill individual cracks. If surface cracks are extensive, reshaping of the slope may be more economical than individual filling and sealing.

Slope reshaping or paving has been used to provide a better surface runoff pattern for the area. In the Ventura Avenue oil field many acres of land were paved with asphalt to promote runoff and reduce infiltration (Kerr, 1969). This technique was used in conjunction with horizontal drains, vertical drains, and retaining walls. Surface drainage may provide adequate protection to insure future stability, but the techniques are most commonly used in conjunction with retaining structures or subsurface drainage to provide a more complete solution to the problem.

Subsurface Drainage

Horizontal Drains. There is more qualitative information on horizontal drains as a method for the solution of slope stability problems than any other remedial measure. This may be attributed to the fact that horizontal drains were introduced as a novel solution to landslide stability problems, and consequently much attention has been given to the description of the equipment and examples of techniques which have been used. The literature review has uncovered only one case history in which horizontal drains were used and were not successful. In the following discussion horizontal drains are explained, problems are identified, uses are outlined, case histories to explain general uses are described, and all case histories found in the literature are summarized.

A horizontal drainage system usually consists of 2-inch to 4-inch-diameter steel pipes installed in the face of the slope. Although described as horizontal drains, the pipes usually vary in inclination from 2 to 20 degrees above the horizontal. The pipe is usually perforated with 3/8-inch holes on approximately 3-inch centers. Depending on the slope geometry some type of collector system may be required to prevent the water from reentering the slide area, thus causing future stability problems. The collector pipes range from open trenches to large diameter precast concrete pipes.

The first reported use of horizontal boring equipment for installing horizontal drains was by the California Division of Highways, to place utility pipes under pavements without disrupting the flow of traffic on the road surface (Hellesoe, 1941). In 1939 this equipment was used to drain the area in and around an active landslide (Forbes, 1947; Stanton, 1948). At this time the equipment consisted of a rotary type drill mounted on a light portable frame. Water, forced by compressed air through the drill rod, was used to cool the bit and to wash the cuttings from the boring. The literature describes this machine, illustrated in Fig 3.1, as the "hydrauger drill." Various modifications of the hydrauger drill were used by the California Division of Highways, the West Virginia Road Commission, and the Bureau of Public Roads between the years 1939 and 1953. The California Division of Highways equipment and subsequent improvements have been described by Smith and Stafford (1957) and Root (1955b). Other Pacific coast states, the Bureau of Public Roads, and West Virginia all report using similar equipment.

The horizontal drain has seen limited use elsewhere in the United States. Toms and Bartlett (1962) report success in stabilizing railway cuts and embankments by jacking a closed and perforated pipe into the toe of the slope. In these cases no drilling equipment was used. Zaruba and Mencl (1969) report success using horizontal drain equipment in stabilizing active landslides in central and eastern Europe.

The purpose of the horizontal drain is to remove excess water from a hillside, cut slope, or embankment. Downs (1930) states that critical groundwater conditions are the cause of well over 50 percent of the slides in West Virginia. Baker (1952) states that water is a critical factor in nearly all highway related landslides.

Horizontal drains have been used in three ways to remove excess water from the slope. They may be used to divert water from its source, to lower the groundwater table in the slide area or in adjacent areas, or to drain a pervious or artesian stratum. However, they have been used only after site investigations indicate the presence of a high groundwater table, unfavorable seepage forces, or possible locations of pervious strata.

Horizontal drains have been successfully used on a wide variety of slope profiles and in soils of markedly different engineering characteristics. They are applicable both as a preventive and as a corrective measure. Generally, horizontal drains are least applicable to cohesionless soils (Smith and Stafford, 1957) and most applicable when used to drain water sources in deep seated slides (Baker, 1953) or to intercept pervious water-bearing strata (Eager, 1955). On occasion, horizontal drains have been installed at various levels on benches in cut slopes during construction or while the excavation is being deepened (Cedergren, 1962). In cuts less than 30 feet in depth

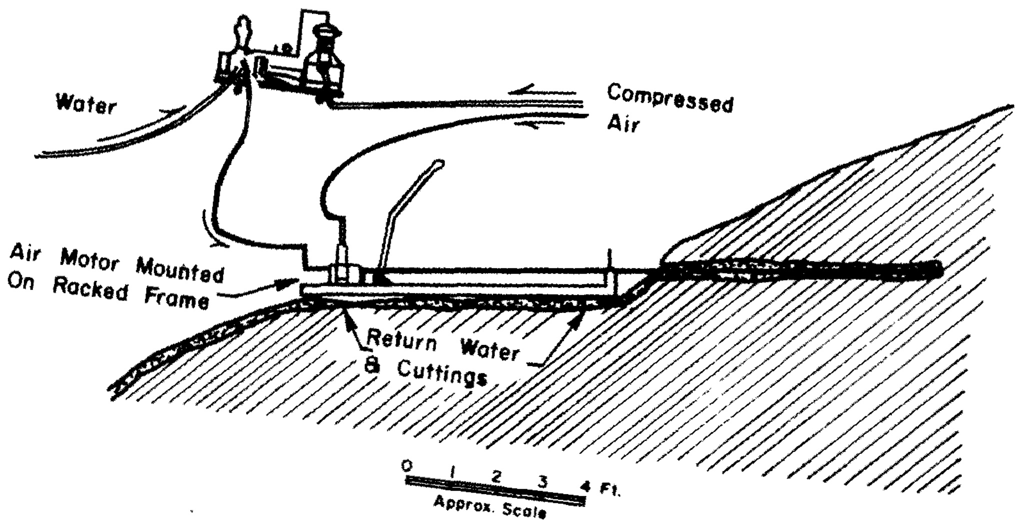


Fig 3.1. Schematic of hydrauger drill for drilling horizontal drains (after Smith and Stafford, 1957).

horizontal drains are usually installed at roadway level. Although used more often to stabilize slopes, horizontal drains have been used for improving the stability of fill and embankment foundtaions (Root, 1958; Smith, 1964).

Case Histories Involving Horizontal Drains

The following paragraphs describe case histories involving horizontal drains; important applications and restrictions are illustrated. A summary of the data found in the literature is presented in Table 3.1.

Willits Slide. The use of horizontal drains to correct stability problems in cut slopes is commonly combined with some type of slope grading. There are two possible reasons for this. Firstly, the increase in stability with each method may be small and relatively indeterminate, and, secondly, benching or slope flattening is frequently required in order to make room for the equipment used to install horizontal drains. In addition to slope grading, vertical wells are often used in conjunction with horizontal drains to form an integrated drainage system for the permanent correction of the slide. Such corrective measures were used to correct the Willits Slide in California (Smith and Stafford, 1957). This slide is the earliest slide found in the literature for which both vertical wells and horizontal drains were used to form a complete drainage system.

The slide occurred in a cut on U.S. Highway 101 approximately 2-1/2 miles south of Willits, California, as illustrated in Figs 3.2 and 3.3. Seepage was noted during construction in 1947 but was not considered alarming. Maximum height of the cut was approximately 60 feet and 2:1 side slopes were used. The slide occurred in 1950, following three winters of unusually wet weather. The material in the cut slope consisted of 20 feet of stiff blue clay overlain by layered terrace gravel and clay. Failure was on the interface of the blue clay and the overlying strata. The head scarp was approximately 220 feet perpendicular to the highway centerline and was about 300 feet wide at the toe of the slope.

Site investigation consisted of approximately 20 vertical exploratory borings and indicated a high groundwater table. Seepage toward the face of the cut was much greater than observed during construction. No stability analyses were performed. On the basis of the location of the failure surface and high groundwater table it was decided to correct the situation using

TABLE 3.1. SUMMARY OF CASE HISTORIES IN WHICH HORIZONTAL DRAINS WERE EMPLOYED FOR REMEDIAL MEASURES

Slope Designation	Site Conditions			
Height and Inclination	Soil	Hydrologic	Comments	Remedial Measures
Los Gatos-Santa Cruz Highway slipouts; 200' fill; 1-1/2:1; Hellesoe (1941).	Uniform fill foundation was in an active fault zone; also earthquake shattered fine found and fill material.	Two unusually wet seasons caused a rise in GWT. Excessive hydrostatic pressures developed in the foundation.	10,000 GPD seepage present after stripping. Caving problem in drill holes. Circular failure through fill.	All loose material stripped from slide area. This material used as a toe buttress. H.D. used on 10-20' c-c. Perforation at 2-3/4" c-c. 2" ϕ pipe hydrauger installed. Permanent wells driven along the roadway to monitor the drainage system.
Grapevine grade slide; 150' cut/fill; $\approx$ 2:1; Scott (1941, 1942).	Sandy clay shale overlain by alternate layers of silt, sand and plastic clay.	High GWT and seepage toward the face of the slope.	Average flow 150 GPH per pipe; 2" ϕ pipes; hydrauger installed.	(1) Interception and drainage of water from the hillside above the roadway (H.D.). (2) Buttress across canyon at toe of slope and drainage (H.D.) to drain fill. (3) Vertical drains to interconnect pervious and impervious layers.

(continued)

TABLE 3.1. Continued

Slope Designation Height and Inclination	Site Conditions Soil	Site Conditions Hydrologic	Comments	Remedial Measures
Lookout Point Slide (Oregon); originally 2:1 Unk. height; cut; Contractors and Engineers (1952).	Volcanic material clay lava rock, talus with water seams.	Excessive hydrostatic pressure.	5,000,000 CY slide; trouble penetrating talus with conventional bits; maximum length = 360'.	(1) Slope flattened to 4:1. This did not stop the slide. (2) Hydrauger installed H.D. using "rat" bits and "N" rods. (3) 4 × 6 tunnel H.D. were used as lateral extensions of the tunnel.
Lookout Point Slide (California); 1:1; 200' cut; Smith and Stafford (1957).	Extensive faulting zone geologically active landslide area. Material chiefly graphitic shist with fractured quartz seams.	During construction springs discovered on face of cut. Waterbearing strata confined to lower 1/3 of slope; perched water table.	Sidehill cut toe of slide 1000' long; 100' cut but total height greater benched slope every 60'.	H.D. used as a prevention. Initial flow 140,000 GPD until perched water table drained out; then 5,000 GPD steady. 10,000 GPD from exploratory H.D. H.D. at roadway level 6-20^{o} above horizontal.

(continued)

TABLE 3.1. Continued

Slope Designation	Site Conditions			
Height and Inclination	Soil	Hydrologic	Comments	Remedial Measures
BPR Case History; Wyoming; 40'; 3:1; (cut); Bohman (1955).	Geologically altered volcanic and carbonaceous clay shales.	High GWT. No presence of free water was indicated.	Cracks in head of slope water W/W 12" of surface; horizontal drains solved the problem.	First failure during construction 30,000 CY of soil removed at head. Second failure 150' of roadway lifted 4'. Four H.D. placed at 30' c-c.; average length 205'.
Sears Point Slide; 125'; 2:1; (cut); Smith and Stafford (1957).	Impervious highly plastic clay.	High GWT.	300' along roadway head 400' right angle to ℄ ; 5' c-c. spacing because of impervious clay.	First slide: 10 drains 200'; another larger failure occurred. Second slide: 3:1 slope and 49 more H.D. at various slope levels.
Orinda Slide; 350'; $\approx$ 2:1 (cut); Herlinger and Stafford (1952).	Clay and clay shale over bedrock.	High GWT.	E 1/2 mudflow, W 1/2 mudflow with broken rock and shale.	Extensive bending used to reach critical locations for H.D. 10,000' of 2" ϕ H.D. installed in 11,700' of bore holes. 135,000 GPD during winter.

(continued)

TABLE 3.1. Continued

Slope Designation Height and Inclination	Site Conditions Soil	 Hydrologic	Comments	Remedial Measures
Half Moon Bay Slipout; 240'; 2:1 (fill); Smith (1957).	Unk.	Seepage through embankment.	Large amount of water noted in gravel subdrain below embankment. 200' of roadway involved.	12 drains at toe at grades of 8 - 25%. Average L = 200'. 14 at roadway grade to intercept water before it reached the embankment. Initial flow 13,000 GPD. Steady flow 8,000 GPD in wet seasons.
Bedrock Failure, Youghiogheny River Reservoir; 300'; 1:4 to 1:1; Root (1958).	Ames shale and limestone sliding on top of water bearing sandstone.	Water found in sandstone (artesian).	No comment.	H.D. 50 to 100' apart beneath the upper band of Saltzburg sandstone. H.D. as a preventive measure.
Willits Slide; 2:1 (cut); Smith (1957).	Stiff blue clay overlain by alternate layers of brown silty clay and terrace gravel. Under blue clay was another gravel layer.	High GWT caused by heavy rains.	Seepage evident at face of slope.	10 of 17 H.D. placed on the clay. Overburden interface 10-30' apart. Average length = 180'. 7 drains at roadway level.

(continued)

TABLE 3.1. Continued

Slope Designation	Site Conditions			
Height and Inclination	Soil	Hydrologic	Comments	Remedial Measures
Towle Slide; 2:1 to 3:1; 100± (cut); Cedergren (1962).	Very weak ground material. Bedrock available 40-50' below surface.	Soil with very high W%, high PI, weak, high GWT.	Part of Trans Sierra Freeway.	Four transverse stabilization trenches and horizontal drains.
Priluky Landslide; 300'±; 1:1 to 2:1 (cut); Zaruba and Mencl (1969).	Palaeogene sandstone shales overlain by loam.	High GWT. GWT lowered to elevation of H.D. Observed through vertical borings.	Holes stabilized during drilling by drilling mud in some cases.	Three horizontal drains. 4" ϕ perforated pipes used as permanent casing for drill rods. Bit remains in the hole. 600' long slide stabilizer. 40 GPM.
Nevada City Slide; 35'; 3:1 (cut); Smith and Stafford (1957).	Silty sand with clay binder.	Seepage on face of benched slope. High GWT.	Combination corrected the problem.	First bench to correct small slide during construction; 10' wide, 20' deep. Second - 12 H.D., L = 80', grade = 15-20%. Initial flow = 890 GPD.

(continued)

TABLE 3.1. Continued

Slope Designation	Site Conditions			
Height and Inclination	Soil	Hydrologic	Comments	Remedial Measures
Pinole Slide; 50-80'; 2-1/2:1 (fill); Smith et al (1970).	Rolling hills with rocky out-crops; 2-3' of black plastic clay over brown silty to sandy clays. UU at field W% = 2-6 TSF. UU at $S_r = 1$ 0.7-1.5 TSF.	Embankment caused a natural drainage basin. A culvert was planned and no problems were anticipated. GWT 23-28' below ground level. Artesian head of 10-12'. Failed because heavy rains caused $S_r = 1$ and the subsequent loss of strength.	11 years old at time of failure. For use in analysis lab testing reported C = 0.25 TSF for the black surface clay and 1.0 TSF for the more silty clay. Total cost in excess of $1,250,000. Note: stripping and a pervious blanket would probably have prevented this slide.	Three sets of horizontal drains were used to stop the slide while a temporary detour was constructed. Vertical drain cut-off trench was the ultimate remedial measure. (See text and vertical well section.)

(continued)

TABLE 3.1. Continued

Slope Designation	Site Conditions		Comments	Remedial Measures
Height and Inclination	Soil	Hydrologic		
Dyerville Cut; H = 480'; 1:1 benched slope (cut); Cedergren and Smith (1962).	Thick interbedded sandstone layers with some shale and conglomerate. Badly fractured and upper 50' of jointed material was very weathered.	GW accumulation was evident in fractures. GW evident in all exploratory holes. Exploratory horizontal drains produced water in the lower 50' of the cut.	20' benches at 60' intervals were used. All action taken was a preventive measure.	3000' of horizontal drains were installed on several of the lower benches. Pervious blankets and underdrains were installed below the roadbed to protect the structural sections from excessive groundwater.
Santa Rosa Slide; $\approx$ 20'; 1 to 1-1/2:1; (cut); Smith and Stafford (1957).	Very silty clay.	Seasonal variations in GW level. High GWT each winter which caused sloughing and local slides.	Approximately 3 years old when horizontal drains were tried. First slide was during construction in 1947.	Horizontal drains were used to stabilize the cut. Because of the sandy material most of the drains caved in and the slide is still moving.

(continued)

TABLE 3.1. Continued

Slope Designation Height and Inclination	Site Conditions Soil	 Hydrologic	Comments	Remedial Measures
Carquinez Cut; 2:1; H = 350'; Cedergren and Smith (1962).	Proposed cut passes through active fault zones. Sedimentary deposits, highly fractured, ranging from hard sand-stone to soft friable sands to soft clays and clay shales.	High GWT with evidence of seepage toward the face of the cut.	9,000,000 cu. yds. total cut volume. A small amount of cracking and local sloughing occurred during construction but caused no major problems.	Horizontal drains and benching used as a preventive measure. Only local sloughing has occurred since the cut appeared.
Togwotte Pass Slide; H = 25'; 2:1 (fill); Eager (1955).	A-7-6 (13); foundation material talus and glacial till with inter-spersed slope debris. High PI soil.	High GWT.	Horizontal drains success-fully stabilized this slide.	1200' of 2" perfor-ated pipe was installed. The long-term discharge has been 15,000 GPD.

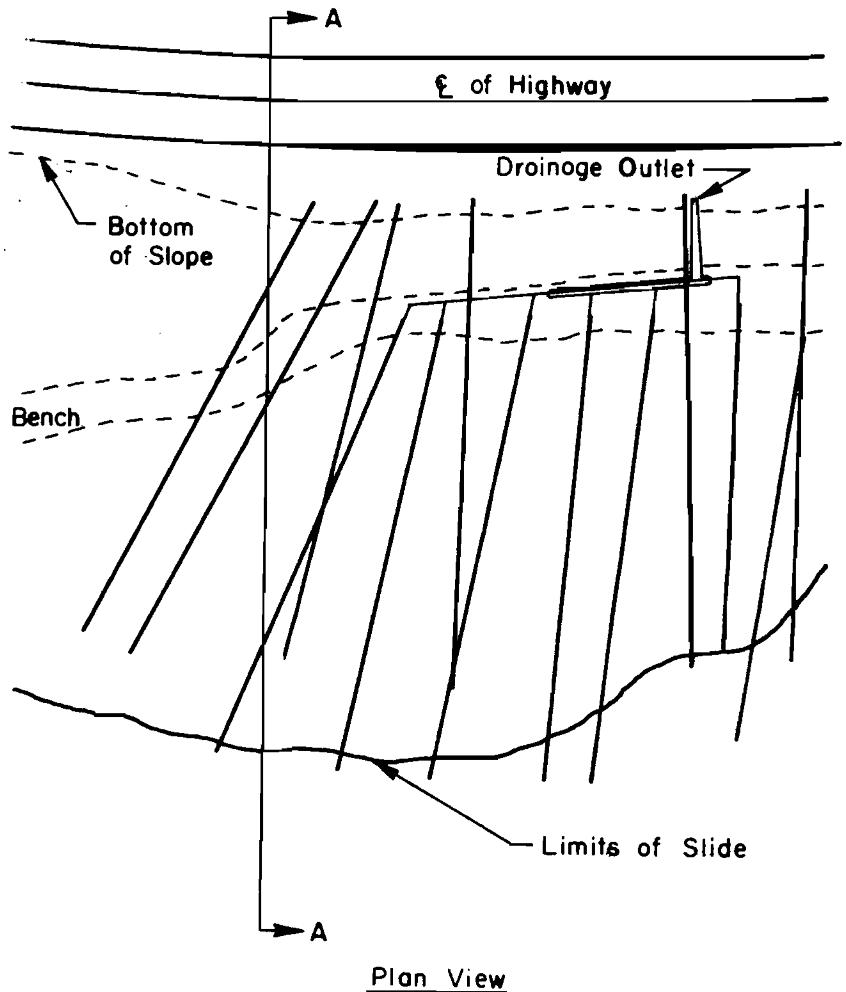


Fig 3.2. Plan view of slide area and horizontal drainage system - Willits Slide (Smith and Stafford, 1957).

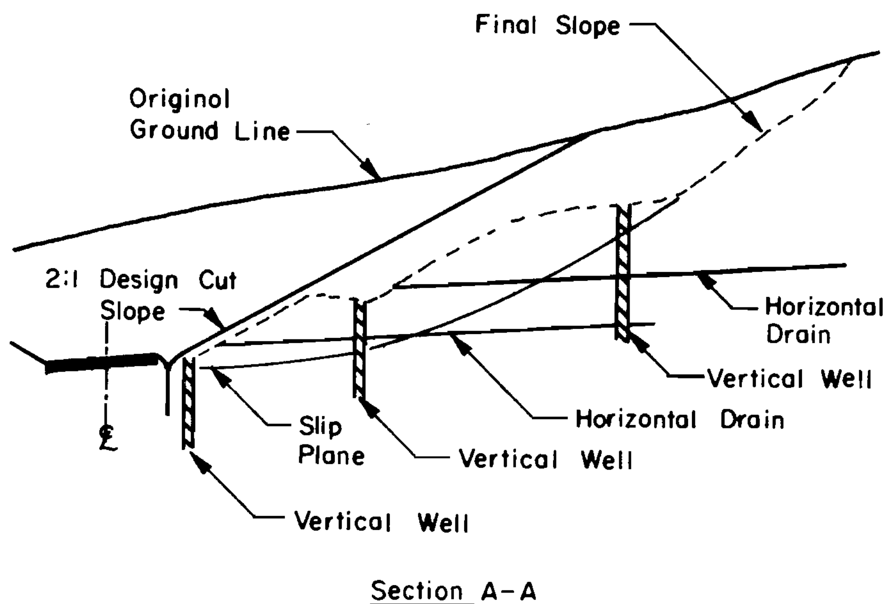


Fig 3.3. Cross section of slide and drainage system - Willits Slide (Smith and Stafford, 1957).

horizontal drains and vertical wells. The vertical wells were used to interconnect the gravel layers which were separated by the imprevious clay of the surface deposit, and the horizontal drains were used to drain the water from the area in the vicinity of the slide.

Ten of the 17 horizontal drains were placed along the interface between the blue clay and overlying layers by excavating a bench 20 feet above roadway grade, and the others were installed at roadway grade in the stiff clay layer. The upper drains were spaced from 10 to 20 feet apart while the spacing of the roadway drains varied from 10 to 50 feet. The average length of the drains was 130 feet and total initial flow was 13,600 gallons per day. Sustained flow (to 1956) was approximately 1000 gallons per day.

The subdrainage of the slide was sufficient to stabilize the slope. No sliding took place in the area of the slide after the horizontal drain project was completed, while adjacent areas not stabilized by horizontal drains were plagued with continued landslide problems.

Pinole Slide. One of the most successful uses of the horizontal drain method of slope stabilization was the Interstate 80 slide near Pinole, California described by Smith et al (1970). In May of 1969, 400 feet of Interstate 80 embankment slid out, leaving only one lane of the six-lane interstate open to traffic. Embankment heights in the area where failure occurred were 60 feet at the roadway centerline, 84 feet at the north crest of the slope, and 46 feet at the south crest of the slope. The original embankment was 106 feet wide at roadway grade with 2:1 side slopes. The embankment was completed in 1958 and traversed a natural drainage course.

Site investigations for the original embankment indicated 2 to 3 feet of black, plastic clay overlying brown, silty, and sandy clays. Borings in the drainage course revealed a general pattern of wet plastic surface clay to varying depths, which would require removal before embankment construction could proceed; however, this material was not entirely removed prior to construction. Water was generally not encountered along the alignment, and no special foundation treatment was recommended.

Thirty-six vertical borings were taken during the investigation of the slide, and inclinometers were used to determine the depth of subsurface movements. Groundwater was discovered in large quantities at depths ranging from 20 to 25 feet. Upon release of the overburden pressure this water rose 10 to 15 feet in the vertical borings. On the basis of the preliminary data, engineers decided that the use of horizontal drains would be an expedient to any permanent correction. Twelve horizontal drains, ranging in length from 550 to 830 feet, were placed in fan patterns from three different locations near the toe of the slide. Total flow produced was 12,000 gallons per day. The groundwater table was lowered 7 feet at the toe of the proposed detour upslope of the slide, 3 feet at the center of the sliding mass, and 1 foot at the toe of the slide. It was felt that the drains increased the stability enough to allow the construction of a six-lane detour at the southern edge of the embankment without further movement taking place. The plan and cross section of the remedial measures taken at the Pinole slide are illustrated in Figs 3.4 and 3.5, respectively.

It should be noted that horizontal drains were successfully employed with a minimum of investigation and analysis to maintain a sufficient factor of safety to allow construction of a large detour directly above and behind the failure zone, thereby opening the roadway for traffic. The remainder of the discussion pertaining to corrective action taken on this slide is reserved for the later section of this report describing vertical well systems.

Grapevine Slide. When sidehill fills or cut/fill sections are being stabilized, horizontal drains have often been used with toe buttressing to insure a permanent solution to the problem. The widening of Grapevine Grade on U.S. 99 in California eliminated a slide problem which had plagued the area for years by using such a combination method of stabilization (Scott, 1936, 1941).

Excessive seepage forces threatened to slowly move the highway into Grapevine Creek at the bottom of the canyon (Fig 3.6). The material on the southern side of the highway was a sandy clay/shale, overlain by thin layers of silt, sand, and plastic clay. The stabilization consisted of two distinctly separate operations:

(1) the interception and drainage of water from the hillside above the highway, and

(2) construction of a toe buttress, and drainage of the fill beneath the roadbed.

The first operation consisted of placing 24-inch-diameter vertical sand filled drains in the area most highly stratified with alternate layers of impervious clay and water bearing sand and gravel. Horizontal drains were then installed from 2 to 3 feet above roadway grade into the side slope at

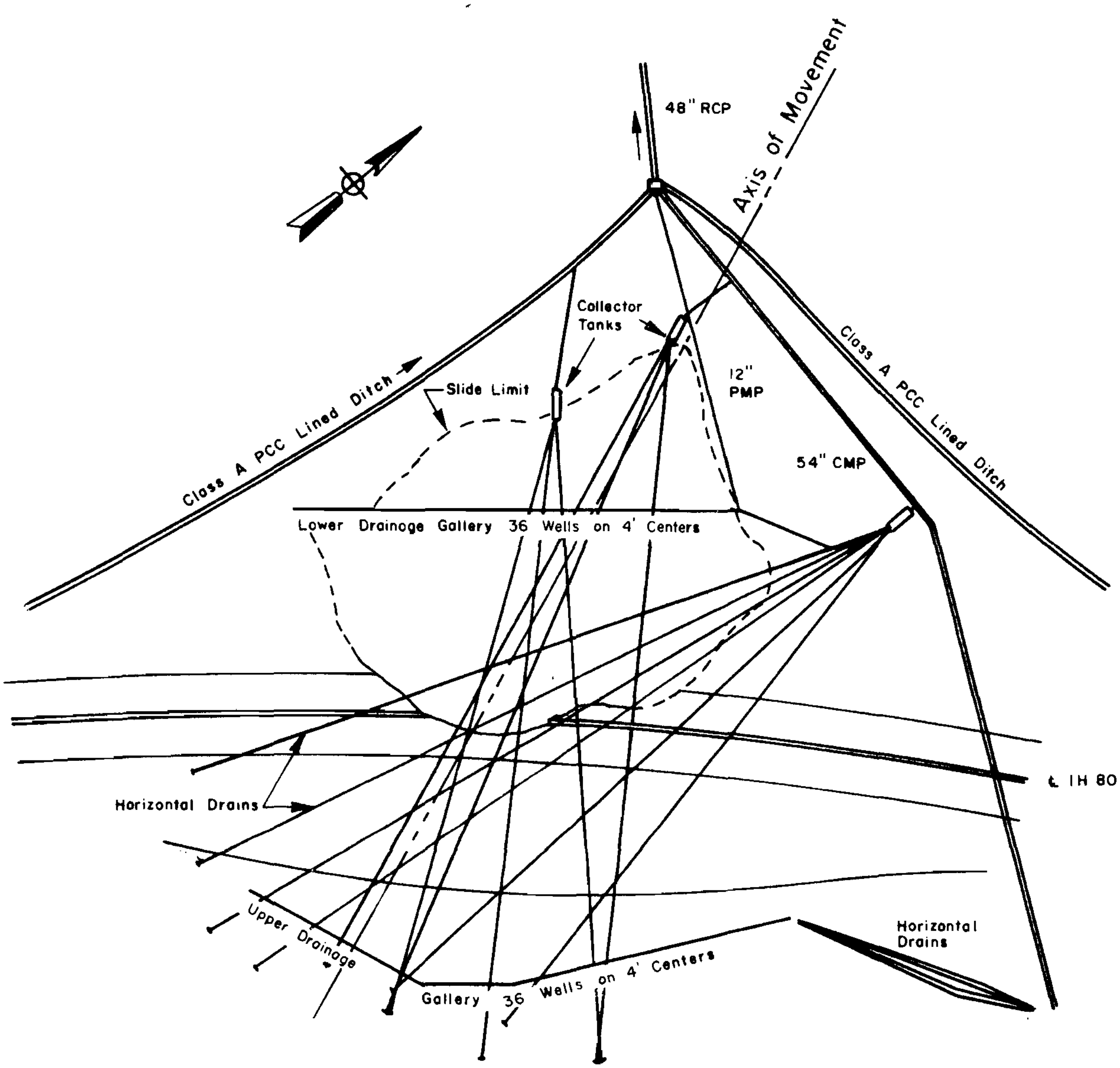


Fig 3.4. Plan view of drainage system - Pinole slide (Smith et al, 1970).

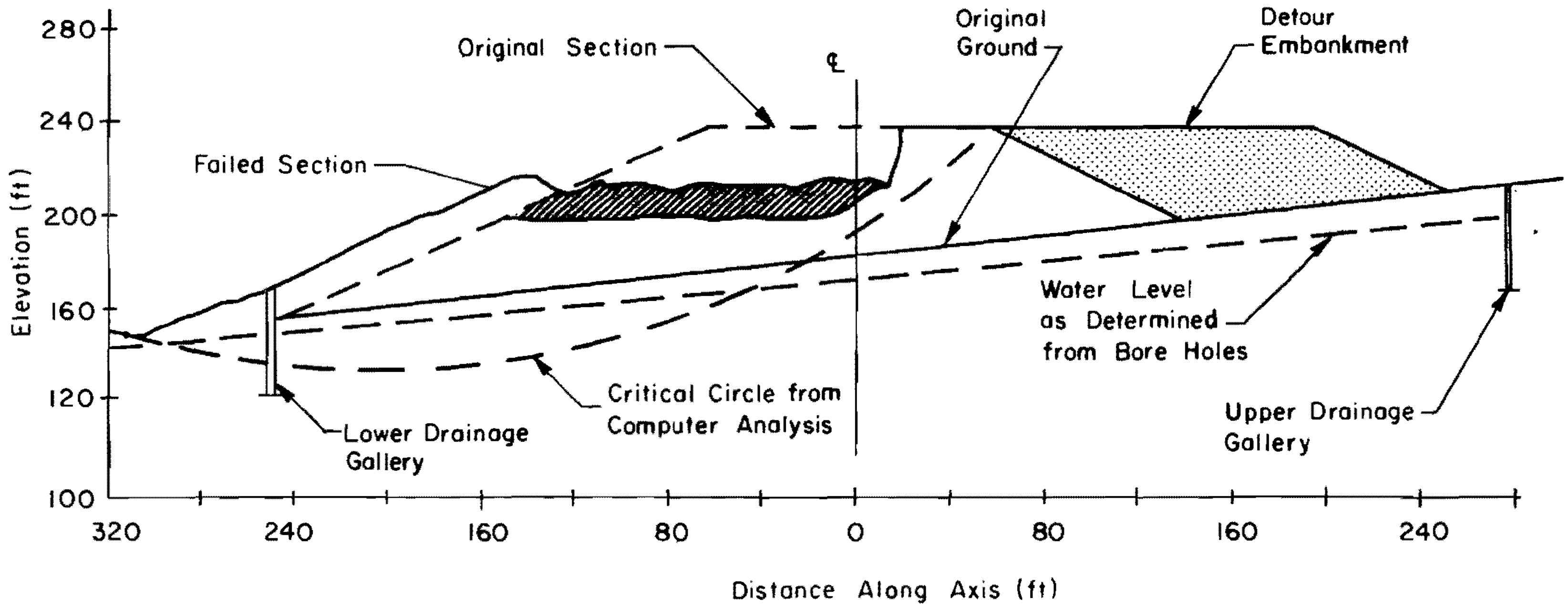
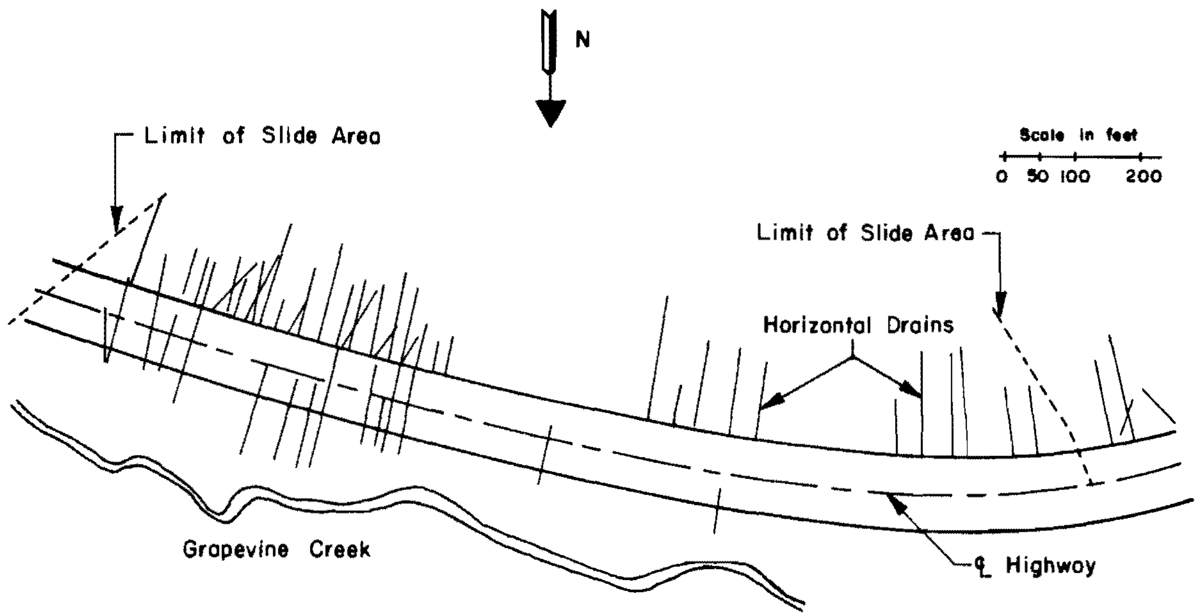
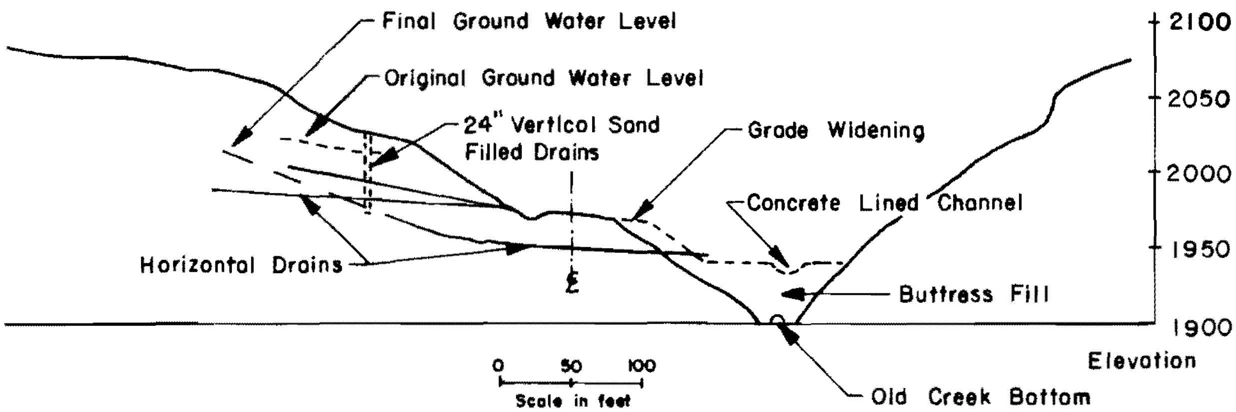


Fig 3.5. Cross section of slide along axis of movement - Pinole slide (Smith et al, 1970).



a) Plan



b) Typical Section

Fig 3.6. Drainage system and remedial measures - Grapevine slide (Scott, 1936, 1941).

angles ranging from 2 to 20 degrees above the horizontal. Lengths of the drains averaged 100 feet; the maximum length was 170 feet. Two-inch-diameter perforated pipes were installed in the 4-inch hydrauger holes. A 200-gallon per hour maximum flow was reported in some of the horizontal drains.

The second stage of the correction procedure was the construction of a 40-foot hill along 2000 feet of the highway in the Grapevine Creek bed. The elevation of the creek bed was raised 40 feet by placing a concrete lined channel on top of the earth buttress. While the buttress was being constructed, horizontal drains were being used to reduce seepage forces and lower the groundwater table in the cut/fill section. The plann and cross section in Fig 3.6 illustrates the measures used in the stabilization of this slope. No further problems have been reported since completion of the remedial measures in 1942.

Santa Rosa Slide. In order to improve the alignment of U.S. 101 north of Santa Rosa, California, it was necessary to excavate a cut in the crest of a ridge composed of silty sand. The maximum height of the cut was approximately 20 feet and it was approximately 200 feet in length. Minor sloughing occurred during construction in 1947 and became progressively worse in the following winters. Maintenance crews reported that it was affecting property outside the highway right-of-way and in 1951 the California Division of Highways decided that steps to correct the situation should be undertaken (Smith and Stafford, 1957).

Vertical borings were made in the slide area. The information from these borings together with evidence of seepage on the face of the cut indicated that a considerable quantity of groundwater was present in the vicinity of the cut. Horizontal drains appeared to be the most logical and economical means of correction. Thirteen horizontal drains were installed from two general locations. Five drains were fanned into the slide area from one end of the slide, while the remaining eight were fanned into the slide area from the other end. All drains were installed at approximately roadway level.

A great deal of difficulty was encountered in installing drains in the sandy material which existed within the slope. This material continually sloughed and blocked the holes so that the casing could not be advanced. In 2708 feet of drilled hole, only 1575 feet of casing could be installed. The combined initial flow of the drains was 5000 gallons per day; however, sustained flow was negligible. The flow produced was much smaller than estimated, and the groundwater table, as determined from water levels in vertical borings, had not been substantially lowered. The sustained flow of the horizontal drains was not great enough to appreciably lower the groundwater table in the area.

Movement continued in this area until at least 1957. The interception of groundwater was not great enough for the installation to be considered a success, thus illustrating that problems may be encountered when using horizontal drains to lower the groundwater table even in a slope which is predominately sand.

Vertical Drains and Well Systems

Well systems have been employed in slope stabilization to control adverse groundwater conditions in both cut and embankment sections. When used to control hydrostatic pressures in conjunction with earth dams or levees these well systems have commonly been referred to as "relief wells." When used for highway related slope stabilization they have been referred to as vertical sand drains, sand drains, vertical wells, and, in some cases, relief wells (Parrott, 1955; Holm, 1969; Smith, 1964; Smith and Stafford, 1957). As a landslide prevention or correction measure, well systems are most commonly used in conjunction with horizontal drains to provide relief of hydrostatic pressure and gravity discharge of the subsurface water, respectively. When employed for slope stabilization vertical drains have been used for three basic purposes:

(1) to provide a drainage path between lenses or strata of water-bearing material which are separated by impervious layers (Palmer, 1950; Parrott, 1955);

(2) to relieve artesian conditions which may develop at or below the surface of rupture (Holm, 1969; Smith, 1964; Smith, 1969); and

(3) to relieve excess hydrostatic pressures in slopes of saturated clay and therefore expedite consolidation and increase the shear strength of the soil (Holm, 1969; Fellenius, 1955).

In addition to the above mentioned "drainage methods," interconnected vertical wells have often been used in place of cut-off trenches where pervious water-bearing strata lie beyond the reach of conventional trenching equipment.

Root (1958) states that vertical drainage wells are equally applicable as corrective or preventive measures. He also indicates that vertical drainage wells have been used with a greater degree of success than horizontal drains in the correction and prevention of slides. However, the statement that vertical drains have been more successful than horizontal drains as a stabilization procedure may be misleading in that, according to the literature, vertical drains are most often used in conjunction with horizontal drains to form an integrated drainage system, rather than as a remedy in themselves. This is illustrated by case histories which follow.

Vertical drains reported in the literature have ranged from 24 to 36 inches in diameter and were reported varying between 25 and 80 feet in length. They have commonly been installed using a disk type auger, with backfill material varying from standard filter to coarse gravel. For successful installation the pervious material should have two characteristics, which are somewhat contradictory. Firstly, the material must be many times as permeable as the surrounding material from which the water is to be drained, and secondly, the permeable material should not contain voids sufficiently large to permit the migration of the soil surrounding the vertical well into the drainage material (Cedergren, 1967; Bertram, 1940). For large slides requiring extensive subdrainage (other than horizontal drains) the criteria developed by Bertram and revised by the Corps of Engineers have been successfully applied to meet these requirements. However, for smaller slides where the consequences of future instability do not greatly endanger life or property, standard drainage material specifications, such as those developed by the California Division of Highways, provide a proper balance between added costs and reliable drainage facilities.

In some cases it has not been possible to design a backfilled vertical drain with sufficient capacity to remove the estimated seepage, and in these cases perforated or slotted pipes have been installed. Care should be taken to insure that the filter-drainage material surrounding the pipe has sufficient permeability to permit seepage to freely enter the pipe. Cedergren (1967) has developed design charts which may be used either to estimate the rates of seepage into pipes through filter materials of known permeabilities, or to determine the required permeabilities of filter materials to permit given rates of seepage to enter the vertical pipes surrounded with such material. These charts may be used to gain an estimate of the general capabilities of filled wells for handling flow and as an aid in designing well systems.

A summary of case histories where well systems were employed as a remedial measure is presented in Table 3.2.

Case Histories Involving Vertical Drains

U.S. 220, Virginia. Although several early instances appear in which some type of vertical drain was incorporated into the overall landslide stabilization scheme (Forbes, 1947), the first reported use found of vertical sand drains alone to correct an active landslide was during the relocation and widening of U.S. Highway 220 in Alleghany County, Virginia, in 1947 (Parrott, 1955). The relocated highway lies parallel to and 70 feet downhill from the original U.S. 220 with the Jackson River located 50 feet further down the slope, as indicated by the cross section in Fig 3.7.

Excavation for the new route in the area of interest showed that the road would pass through a thick mantle of talus, mostly sandy soil interspersed with large boulders. This soil was classified as an A-2 sandy silt, and it was in this material that a slide first showed evidence of developing. The area between the new grade and U.S. Highway 220 was described as the potential problem area, as illustrated by the cross section in Fig 3.7.

The first attempt to stabilize the area was a 4-foot-wide $\times$ 2-foot-high masonry rubble wall keyed into place at the bottom of the cut slope for the new route. It was reported that this wall proved successful in stabilizing local movements in the area; however, excess groundwater caused further creep movements, and seepage toward the face of the cut caused the bank to slough off and encroach on the pavement of the original highway.

A detailed field investigation of the face of the cut revealed a nearly horizontal bed of highly plastic varved clay about 35 feet above the grade of the new excavation. Water draining through the mantle of talus would reach the bed of clay and break out onto the cut slope. During consultation with the Engineering Branch of the United States Geological Survey, several methods of correction were discussed, including slope flattening, benching, and chemical grouting. The final decision was to use vertical sand drains to bypass the impervious clay that impeded the natural drainage within the slope. The objective of the Virginia Department of Highways was to discharge the drained water into porous material in the bottom of the drain. This would allow all water within the slope to drain into the Jackson River well below the new

TABLE 3.2. SUMMARY OF CASE HISTORIES IN WHICH WELL SYSTEMS WERE EMPLOYED AS A REMEDIAL MEASURE

Slope Designation	Site Conditions			
Height and Inclination	Soil	Hydrologic	Comments	Remedial Measures
Parker Ave. Slide (1935); 110'; 2:1; Forbes (1947).	Highly stratified. Weathered serpentine, some bedrock at greater depths.	Complicated GW system; pervious layer in the weathered serpentine.	Slope cut in 1919. 1931, constructed at head of slope. 1933, a large building placed at head of slope. Water introduced to slope through an extensive sprinkling system. Slide occurred in 1933-34.	A combination of vertical relief wells and a tunnel extending into the slope from the toe. Discharge of tunnel was 1,000,000 G/D for the first 2 years.
Guntrop Landslide (1953); H = 45'; 2:1 slope; Fellenius (1955).	Quick clay; $\phi = 35^o$. Interbedded sand gravel and silt.	Water infiltration at head of slope causing artesian conditions to develop lower in the slope profile.	The artesian pressures in the pervious layers caused a reduction of σ in the clay and the subsequent loss of strength. This was a railroad slide.	(1) The railroad was moved down the slope to counterbalance the toe. (2) The relocated railroad was founded on a pile foundation (330 rails and a 2' R-C slab). (3) 9" ϕ vertical drains were used to reduce the hydrostatic pressures in the sand and gravel layers.

(continued)

TABLE 3.2. Continued

Slope Designation Height and Inclination	Site Conditions Soil	Hydrologic	Comments	Remedial Measures
U.S. 220 (Virginia); 1947; H = 70'; 3/4:1; Parrott (1955).	Sandy talus interspersed with large boulders. A thin seam of highly plastic clay was discovered ≈ 35' above grade line.	GW drained freely through the talus.	Slide occurred during relocation of the highway 70' downhill from the original C_L. Before discovering the clay layer, a rabble retaining wall was used to correct the slide. This was unsuccessful in halting movements along the clay seam.	Ten 6" ϕ vertical drain wells 50' long filled with well-graded concrete sand were used to allow GW to bypass the clay layer. Blasting was necessary to provide free draining through the bottom of the holes.
Naval Station Slides (1948); 1:1 to 2:1; H = 35' (cut); Palmer et al (1950).	Highly overconsolidated glacial till of non-plastic clay sized particles with isolated sand, silt and gravel lenses.	Excess rainfall in the preceding winter; also several springs were known to exist in the area.	Four slides occurred in the spring of 1948. Cause was excessive hydrostatic pressure build-up in the sand and gravel lenses. Retaining wall was considered but would be too expensive.	Vertical drain wells were placed at the top of the slope, located to intercept the sand lenses discovered by the site exploration. Horizontal drains used as outlet pipes for the vertical drain wells.

(continued)

TABLE 3.2. Continued

Slope Designation	Site Conditions			
Height and Inclination	Soil	Hydrologic	Comments	Remedial Measures
MTN Blvd. Freeway; 1-1/2:1; H = 30-40'; depressed freeway section; Nordfelt (1956).	Saturated hardpan clay interspersed with serpentine. 0_r 1.	The GWT in the vicinity of this freeway was exceptionally high. Seepage toward the face of the cut slope.	Disc augers used to drill 30" ϕ holes below subgrade depth (40-45'). Horizontal pipes used at the bottom of each vertical drain to transport water to a central pumping station (vertical wells belled at bottom).	Two rows of vertical wells staggered on 10' centers on each side of the freeway ($\approx$ 800' each side). A pumping station on each side of the freeway was used to pump water to existing stone sewers.
Oslofiord (Norway); H = 35'; $\approx 20^o$ slope; Holm (1969).	Sand and gravel overlying quick clay. $\approx$ 30-50' in depth. Bedrock lies below the quick clay layer.	High GW level and excess pore water pressure near the bottom of the clay layer.	It was thought that the proposed construction of a retaining wall at the top of the slope would cause excess pore pressures that would render a F.S. for the slope of less than unity.	As a preventive measure, bleeder type vertical drains were installed to provide a relief system for any increase in excess pore water pressure. Excellent results were obtained (see text).

(continued)

TABLE 3.2. Continued

Slope Designation Height and Inclination	Site Conditions: Soil	Site Conditions: Hydrologic	Comments	Remedial Measures
Pinole Slide (California); H = 50-80'; 2-1/2:1 (fill); Smith (1970).	Rolling hills with rock outcrops. 2-3' of black plastic clay over brown silty to sandy clays. Embankment material well compacted of the above material UU at field W% 2-6 TSF; UU at S_r = 1 .7 to 1.5 TSF	This embankment crossed natural drainage basin. GWT 22-28' below surface. Artesian head 10-12'. Failure occurred after excessive rains.	11 years old at failure after 3 seasons of excessive rains. Horizontal drains were the expedient to the permanent correction (see horizontal drain section).	36" ϕ vertical wells were installed on 4' centers. These were belled at the bottom to form a continuous drainage trench. These were installed along each toe of the slope. Horizontal drains used as an outlet for the water trapped in these wells.

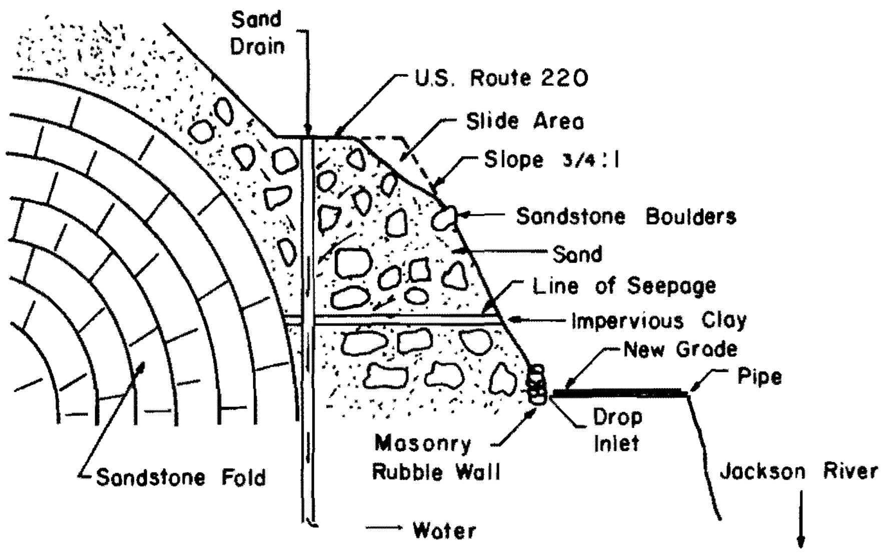


Fig 3.7. Remedial drainage - U.S. 220, Alleghany County, Virginia (Parrott, 1955).

grade. Upon completion of the first hole it was discovered that the water would not drain freely from the bottom of the holes. As a result of this finding, explosive charges were set in the bottom of each hole to provide a freely draining path for the water within the slope. Ten 6-inch-diameter vertical sand drains, filled with a well-graded concrete sand and sealed with a bituminous cap, were installed in this manner. This group of ten vertical drains allowed seepage to drain vertically into the alluvial sand and gravel and discharge freely at the base of the 80-foot holes into the Jackson River. No other drainage was necessary, and the stabilization procedure remains effective to date.

<u>Naval Station Slide</u>. Following an unusually wet winter of 1948, several landslides developed along the west boundary of the Naval Station in Seattle, Washington (Palmer et al, 1950). The slope in the active landslide area was a highly overconsolidated glacial till composed predominately of nonplastic clay sized particles or rock flour. Isolated pockets or lenses of sand, silt, and gravel existed within the slope, and although basically a natural slope, several areas had been regraded to provide adequate surface runoff. In regrading the slope, the fine grained soils could not be compacted to the density of the undisturbed material, and, consequently, the reconstructed portions were less stable than the natural material. The average height was approximately 35 feet and slopes varied from 1:1 to 2:1.

Four separate slides were precipitated by excessive hydrostatic pressures developed in the isolated sand and gravel lenses. Engineering experience in the area indicated that these pressures could not effectively be resisted by gravity walls or concrete cribbing. In isolated spots along the landslide area, field investigations indicated hydrostatic uplift pressures exceeding the overburden pressure, and active springs were reported in the area.

Even though seemingly unpredictable groundwater conditions existed within the slide area it was felt that an extensive exploratory investigation would provide sufficient data for adequate subdrainage of the area. The exploratory program involved extensive vertical borings to locate the sand and gravel lenses. With the location of the pervious strata known, it was possible to design a drainage system to intercept these pockets and conduct the water away from the embankment.

The first step in the construction procedure was to install large diameter (12-inch) horizontal drains at the locations of the water as indicated by the exploratory boring data. These were installed at the toe of the slope and were designed to be intercepted by vertical drains to be installed later. The 12-inch diameter pipes were installed by jacking, and excavation was accomplished from inside the pipe by jetting.

After the drain pipes were installed, vertical drain wells were placed by driving and jetting 24 or 36-inch-diameter casings to a depth slightly above the elevation of the horizontal drains. At the intersection of the horizontal and vertical drains, the vertical drain was enlarged to form a cavity with a minimum diameter of 4 feet, using hand labor. Filter material was placed in the enlarged cavity by an operator working inside the shaft. A double filter layer of gravel (inner filter) and sand (outer filter) was first placed to the invert elevation of the horizontal drain. Following this operation, a well screen was installed and placement of the sand and gravel was continued to the top of the enlarged cavity, with extreme care taken to insure that the layer of gravel between the well strainer and the layer of sand would not become impaired by any infiltration of sand. The remainder of the vertical well was then filled with the dual gravel and sand filter drain and covered with several thicknesses of tar paper. The space above the tar paper was backfilled with the natural materials and compacted in place.

The performance of the vertical drains in the area was excellent. No further sliding occurred in the stabilized area, while adjacent slopes continued to cause problems. This case history illustrates the manner in which vertical well systems may be used to provide a drainage path between lenses of water-bearing material which are separated by imprevious layers and to relieve an artesian or hydrostatic condition which may precipitate instability.

Pinole Slide. In a previous section, the Pinole Slide was used to illustrate how horizontal drains were used with a minimum of investigation to maintain a stable slope during the construction of a detour. In this section the same case history is used to illustrate the manner in which a large highway slide was successfully stabilized on a permanent basis using an integrated system which incorporates both horizontal and vertical drains.

Evaluation of data based on visual observations, borings, field measurements, and laboratory tests led to the conclusion that failure resulted from a loss in strength of foundation soils due to an unprecedented rise in the groundwater table. This rise followed a series of severe winter rainstorms that exceeded all seasonal normals. A second factor that contributed to the failure was the construction in 1964 of two 25-foot-high embankments downstream in the natural drainage course over which the highway fill was placed. These fills appeared to have restricted natural drainage aquifers and aggravated the increase in water elevation. Inspection of the highway embankment indicated a dry, well-compacted fill of good quality.

Before reconstruction of the embankment, an extensive subdrainage system was designed to be effective under the worst possible groundwater conditions. Thirty 6-inch-diameter vertical gravel filled wells, on 4-foot centers, were installed along 600-foot lines parallel to the toe of both the east and west slopes of the embankment. These wells were enlarged at the bottom by belling to form a continuous drainage gallery, and the watter collected by this drainage gallery was drained laterally by means of a 6-inch-diameter steel pipe supplemented by horizontal drains, as shown in Fig 3.4. From this point, the flow from the upper drainage gallery was conducted beneath the embankment to a drainage disposal area by means of a 54-inch-diameter, 900-foot corrugated metal pipe (CMP). A 12-inch-diameter, 350-foot perforated metal pipe (PMP) drained the lower vertical well cut-off trench to the master junction box.

The 12 horizontal drains initially installed to relieve groundwater pressure were also incorporated into the permanent drainage system. Three 32-foot lengths of 120-inch pipe with closed ends were utilized as tanks to collect the flow from the three groups of horizontal drains. Water collected in the tanks was discharged into either the 54-inch metal drain pipe on the east or the concrete lined ditch on the west.

Increased stability of the embankment was provided by the placement of a 25-foot berm on the downhill side of the embankment. This berm was 50 feet wide and was keyed into existing subdivision embankments north of the original section. Using the combined system of stabilization, a permanent factor of safety of 1.4 was obtained.

Other Methods of Drainage

Although horizontal drains and vertical relief wells account for the majority of successful uses in the prevention and correction of landslides, several other methods of drainage deserve mention. The use and the degree of success of these methods are a function of the soil and groundwater conditions in the potential slide area. Control of subsurface water in embankment sections has commonly been achieved by stripping the unstable, saturated material and providing a drainage blanket, or, when the unstable material lies at greater depths, by the construction of stabilization trenches. At times these methods are used in conjunction with horizontal drains and vertical relief wells. With the exception of horizontal drains, the most commonly used methods of subsurface drainage in cut sections are underdrains, drainage blankets, and interceptor trenches. In the following paragraphs these methods are discussed, and typical case histories and cross-sections are illustrated. Although discussed separately, in practice subsurface drainage at any single location may combine several of the available methods for prevention or correction.

Stripping Unsuitable Material. If the surface layer of water-bearing material is relatively shallow and is underlain by stable rock or soil the most economical treatment is usually that of stripping the unsuitable material before embankment construction begins (Dennis and Allan, 1941; Baker, 1958; Root and Marshall, 1958). Smith (1964) sets the limits on stripping at 10 to 20 feet. Stripping of the surface material is most commonly followed by placing a layer of pervious material over the stable soil. This procedure serves the dual purpose of replacing the saturated weak material with a compacted material of appreciably higher strength and providing a permeable layer so that groundwater will not become trapped within the embankment. The pervious material may consist of clean gravel, free-draining sand, or other suitable local material. Requirements for the drainage material are basically the same as referred to in previous sections of this report.

The limiting conditions for this type of treatment are the depth of the soft water-bearing material, and the topography of the surrounding area as it relates to the feasibility of providing outlets for the drainage layer. When using this procedure care must be taken to insure that stripping does not merely extend to a zone of stronger material which in turn is underlain by weaker, water-bearing material acting as the basic source of water. A thorough exploration program is required in order to assure success of this method (Smith, 1964).

The primary use of stripping is as prevention of landslides in areas where the construction of highway embankments will endanger the stability of a hillside. The main function of stripping has been to insure stability on hillside fill sections. Figure 3.8 illustrates the use of stripping on a hillside section of the Redwood Highway in Humboldt County, California.

Although stripping is most economically used as a prevention of landslides, several case histories illustrate the successful use of this method as a slide correction. The use of this as a remedial measure is economically limited to embankments of relatively small cross section and slides of shallow depths. This type of slide may be attributed to excessive groundwater and the presence of a soft clay layer immediately beneath the embankment. The procedure for correction of the Castaic-Alamos Creek slides is illustrated in Figs 3.9 and 3.10. Both slides occurred in 1938 and stabilization by stripping has proven successful.

Stabilization Trenches. Where subsurface water or soil of questionable strength is found at such great depths that stripping is uneconomical, deep drainage and stabilization trenches have been successfully applied to prevent landslides. The California Division of Highways first employed this procedure in the early 1930's (Root, 1938) and it has been extremely successful in preventing landslides in areas of poor foundation material. Stabilization trenches have been most commonly used in areas where subsurface water is encountered 10 to 40 feet below the existing ground surface (Smith, 1964). Such stabilization trenches are usually excavated with power equipment, using the steepest excavation side slopes which will remain stable during the construction period. The trenches may be constructed either parallel to the centerline of the highway as longitudinal trenches or perpendicular to the centerline as transverse trenches.

Figure 3.11 illustrates the use of stabilization trenches during the construction of Interstate Highway 5 in northern California described by Smith (1964). The bottom width of the trench, 12 feet, was determined by the minimum convenient width for use of conventional construction equipment. The side slopes ranged from 1:1 to 1-1/2:1. The trenches used along IH 5 were

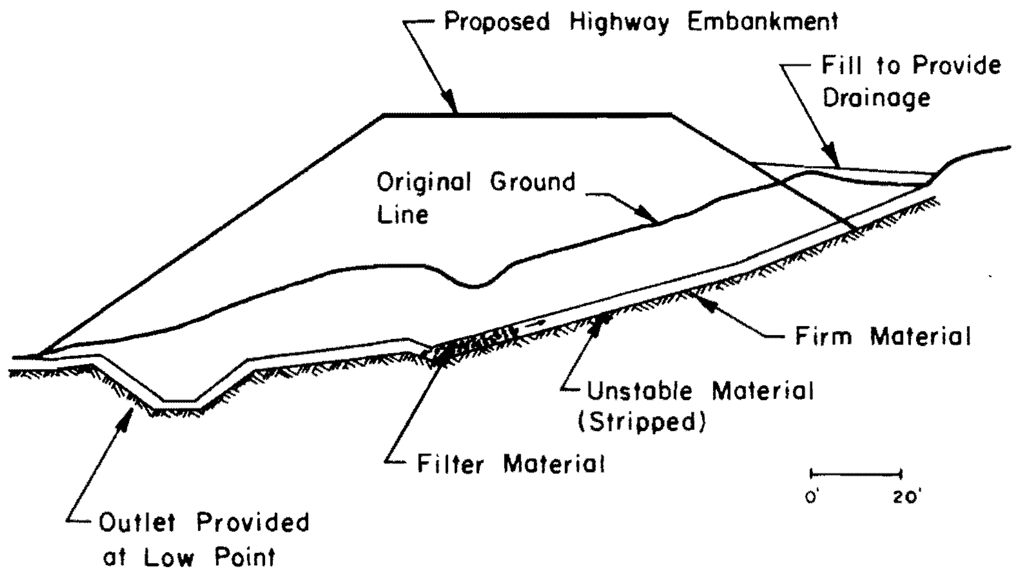


Fig 3.8. Typical cross section of stripping employed for slide prevention - Redwood Highway, California (Root, 1958).

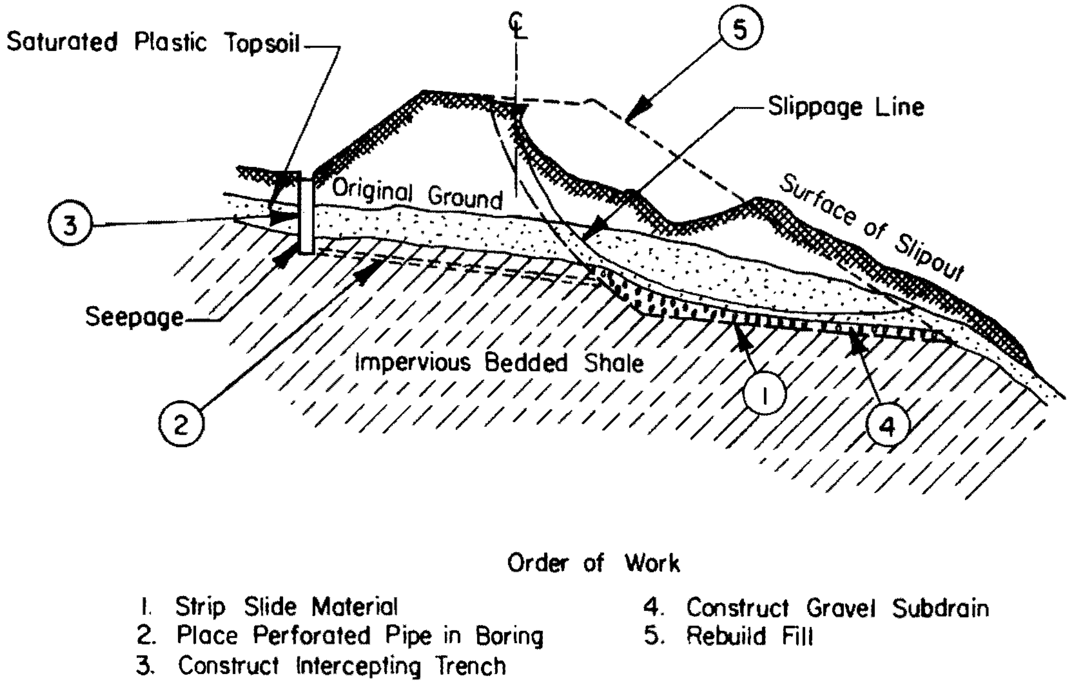


Fig 3.9. Corrective measures - Castaic - Alamos Creek slides (Dennis and Allan, 1941).

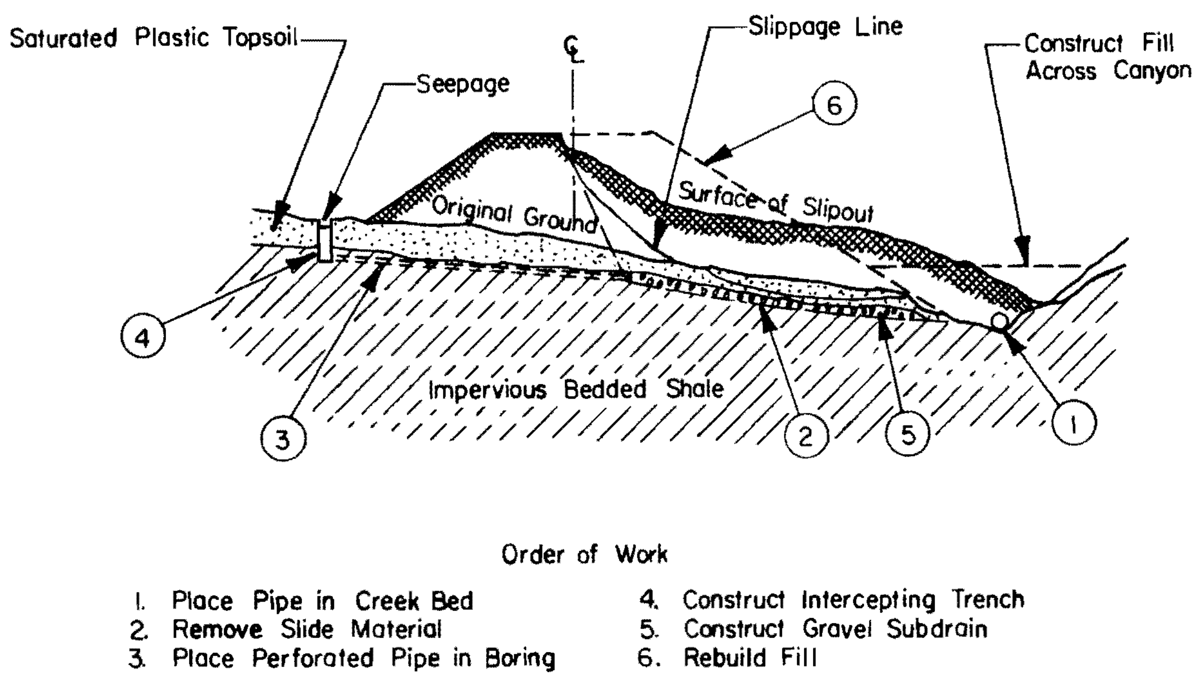


Fig 3.10. Corrective measures - Castaic - Alamos Creek slides (Dennis and Allan, 1941).

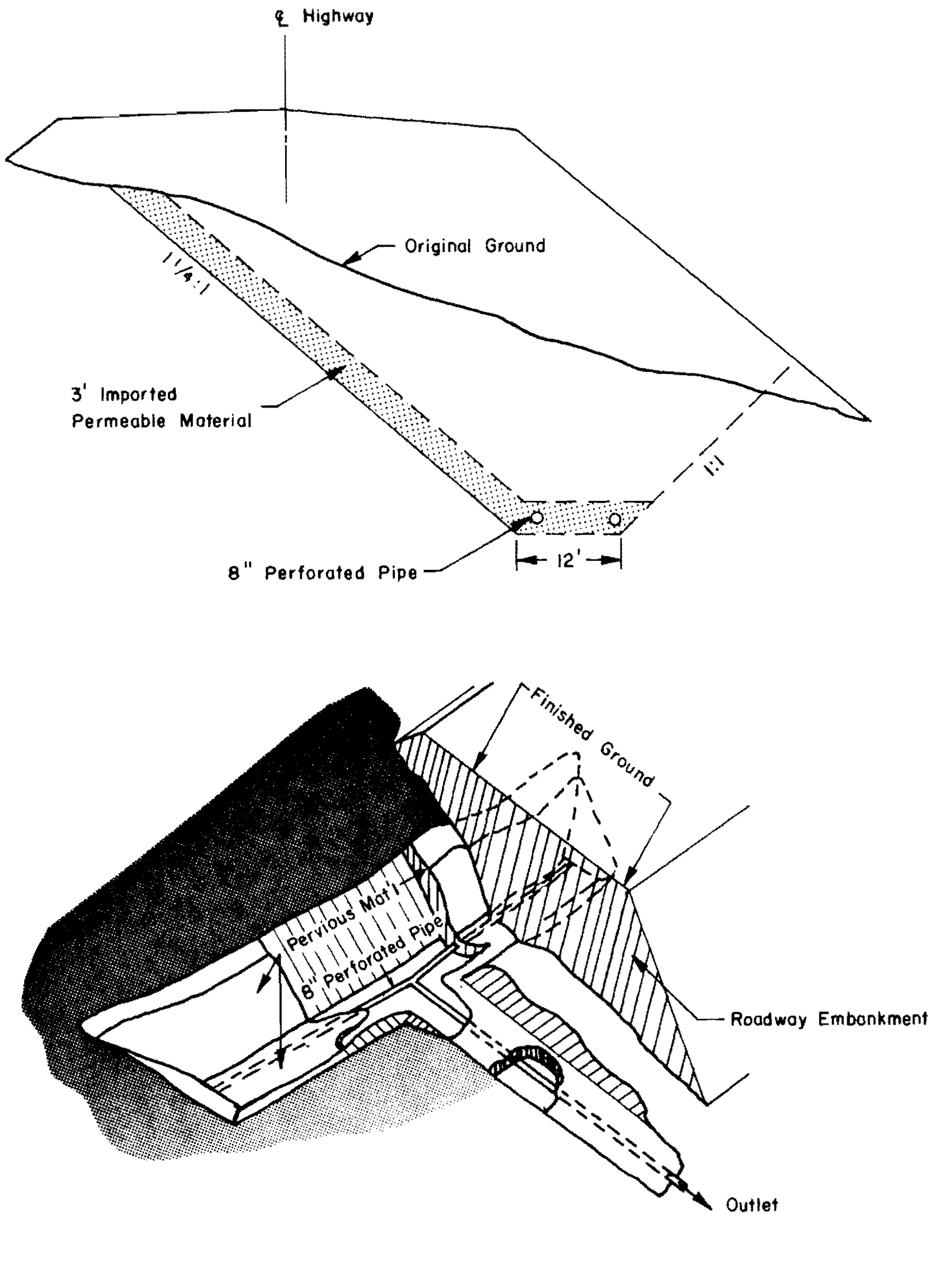


Fig 3.11. Longitudinal stabilization trench (Smith, 1964).

blanketed with 3 feet of permeable material with perforated pipes placed in the bottom to facilitate removal of subsurface water. Outlets, which in essence were transverse stabilization trenches, were provided to remove water from the lower end of the trench.

Because a larger volume of pervious, compacted material is placed in critical areas, longitudinal trenches are more effective than transverse trenches in embankment stabilization. However, cases exist where it has been impossible to construct longitudinal trenches. Cedergren (1962) reports that this was the case for the Towle Slide which occurred during the winter of 1957-58 during the relocation of U.S. Highway 40 in California. The upper 30 to 40 feet of the slide mass was a saturated, lightweight volcanic ash and shale. The most effective remedial measure appeared to be the construction of a longitudinal stabilization trench along the centerline of the proposed highway; however, it was evident that the upper side of the trench would not be stable and that the railroad would be endangered during construction. It was therefore decided to construct a series of transverse trenches extending from the existing road toward the railroad, as illustrated in Fig 3.12. Numerous small slides occurred during construction, but no serious problems were encountered. To the last reported date (1965), this corrective measure successfully stabilized the slide area.

Drainage Tunnels. Where the depth of unstable, saturated soil is too great for economical stripping or construction of drainage trenches, drainage tunnels have sometimes been used to eliminate the problem of excessive subsurface water. Tunneling has been used to successfully correct slides on the west coast (Hill, 1934; Root, 1938; Roads and Streets, 1947). Only one case was discovered where tunnels were used to correct a landslide in other areas (Whitney, 1936). The use of drainage tunnels was fairly common at one time, but at present this method is used rather infrequently, due to high costs (Root, 1958).

Hill (1934) reports the use of a unique drainage system to control a landslide on the Pacific Palisades near Santa Monica, California. The landslide threatened expensive property on top of the 180-foot bluff. A network of interconnected tunnels and drilled holes were driven into the clay stratum near the base of the bluff, as shown in Fig 3.13. Air, heated by a natural gas furnace, was blown through the network to dry out the clay and increase

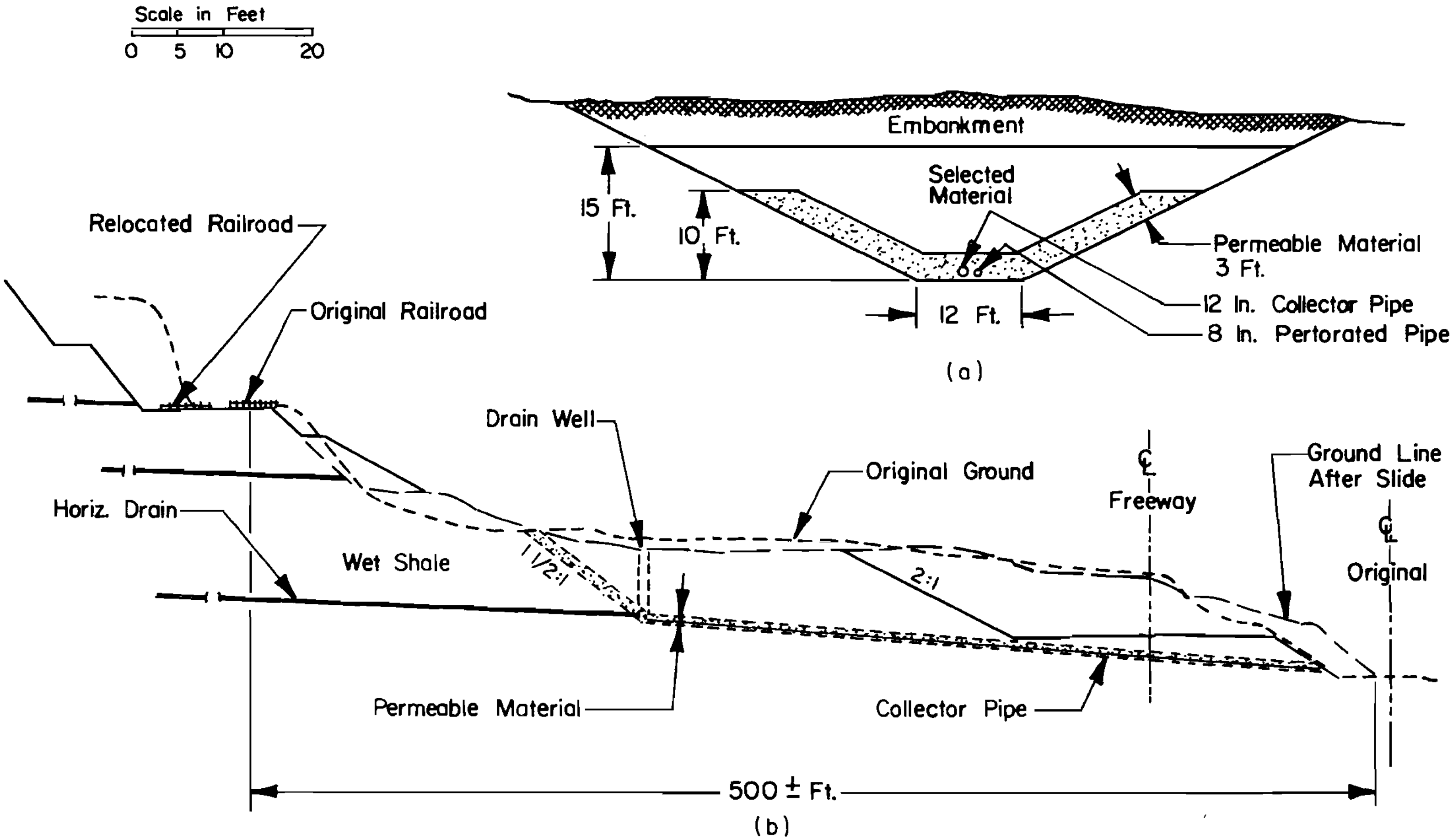


Fig 3.12. Transverse stabilization trenches - Towle slide (Cedergren, 1962).

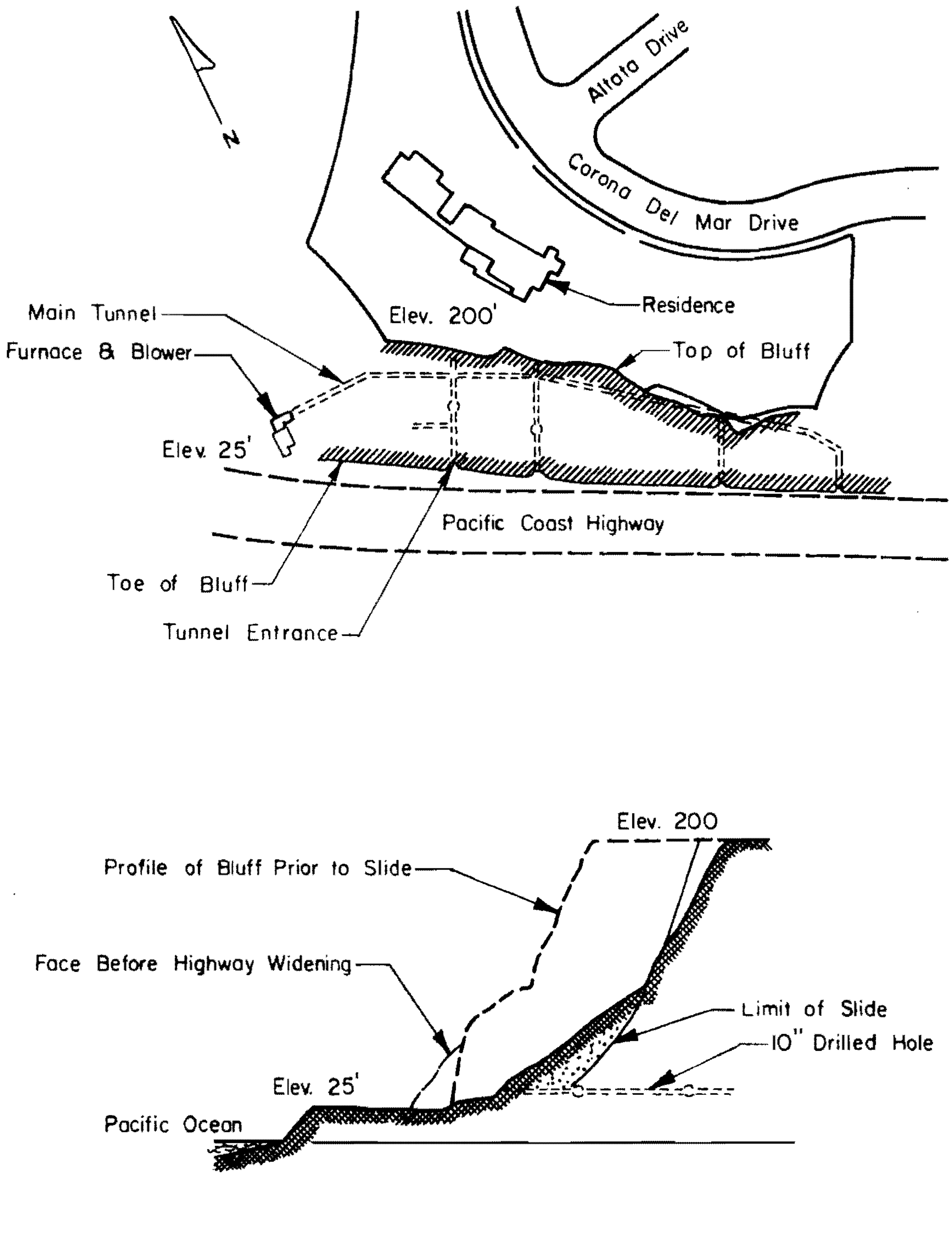


Fig 3.13. Drainage system - Pacific Palisades (Hill, 1934).

its strength. Excellent results were achieved using this procedure and it was also noted that the capitalized cost of operation of the blower was considerably less than the cost of any permanent construction that could be built to control future movements of the bluff.

Roads and Streets (1947) outlines Oregon's procedures for the use of drainage tunnels, and Whitney (1936) reports the use of drainage tunnels to control sliding of a Lake Michigan bluff. In the latter case, removal of water by the use of tunneling and supplemental drains successfully stabilized a landslide, while previous to the installation of the drainage tunnels, an anchored retaining wall failed to have any effect on the hillside stability. This technique is useful where the endangered structures are extremely valuable. In all cases reported in the literature this method was applied to large landslides to remove water from a thin seam of clay on which sliding had occurred.

Drainage Trenches. Drainage trenches or interceptor drains have been used to stabilize highway cut slopes. The purpose of these drains is normally to intercept and remove any flow of subsurface water before it can reach the slide area. However, these trench-type drains are generally limited by construction consideration to those places where water can be intercepted at depths of less than 10 to 15 feet. Usually trenches are excavated to the required depth with power equipment, a French type drain or perforated pipe is placed at the bottom, and the trench is filled with pervious material to within a few feet of the ground surface. On some large slides, depths in excess of 15 feet can be reached; however, this technique is expensive and is used infrequently (Herlinger and Stafford, 1952).

The bottom of an interceptor drain should be founded on unyielding material, and if the trench is located within the slide mass the base should be below the elevation of the slip surface. If movement destroys the base of the drain, its effect is lost and stability will not be increased.

CHAPTER 4. RESTRAINT STRUCTURES AS A REMEDIAL MEASURE FOR LANDSLIDES

One means of controlling the stability of a cut or embankment section is to increase the resistance against movement along potential failure surfaces. Buttressing, cribs, retaining walls, piling, rock bolts, and tie rodding offer means of increasing this resistance. Baker and Marshall (1958) indicate that the most advantageous use of restraint structures is as a preventive measure against large strains and time weakening effects which may cause substantial reductions in soil strength, particularly in overconsolidated, stiff-fissured clays. Restraint structures have also been used as a corrective measure with a substantial degree of success.

The increased resistance to sliding produced by restraint structures is a function of the structure's ability to resist

(1) internal shear and structural failure,
(2) overturning, and
(3) sliding and shear at or below the base of the structure.

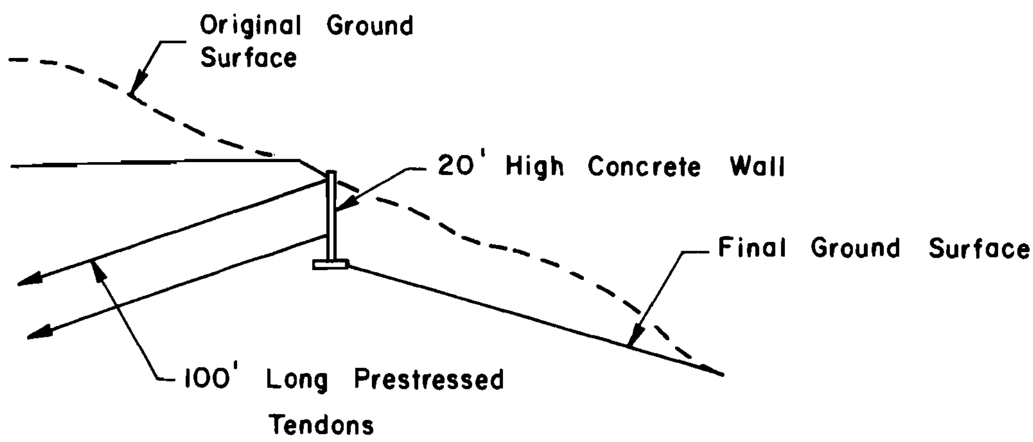
In designing retaining structures these possible modes of failure must be considered. If groundwater is present within or adjacent to a slope where restraint structures are contemplated, the design must also take into consideration the drainage problem (Root, 1958; Baker and Marshall, 1958; Smith, 1964). In addition, in most overconsolidated soils a decrease in soil strength with respect to time should be considered to achieve a permanent solution to the problem (Cassell, 1948; Skempton, 1949). Failures studied in the literature exhibited one or more of the above mentioned modes of failure.

The use of retaining structures is one of the earliest methods used for controlling landslides. The results of these attempts, particularly the earliest ones, are somewhat discouraging. Ladd (1928) reports ten failures of various retaining structures. Of four failures attributed to inadequate drainage, one was the result of rapid drawdown and three were caused by excessive seepage with the subsequent build-up of hydrostatic pressure causing overturning of the wall. Insufficient foundation depth accounted for two of the retaining wall failures, and in these cases the failure plane passed beneath the wall. Where short, small diameter concrete piles were used to increase the shear resistance, the mode of failure was direct shear of the concrete piling at the failure surface. Two failures were caused by overturning of the wall. Subsequent investigations of these ten failures led the Bureau of Public Roads to recommend drainage as the primary method of highway landslide correction in West Virginia, Ohio, and Pennsylvania (Ladd, 1928).

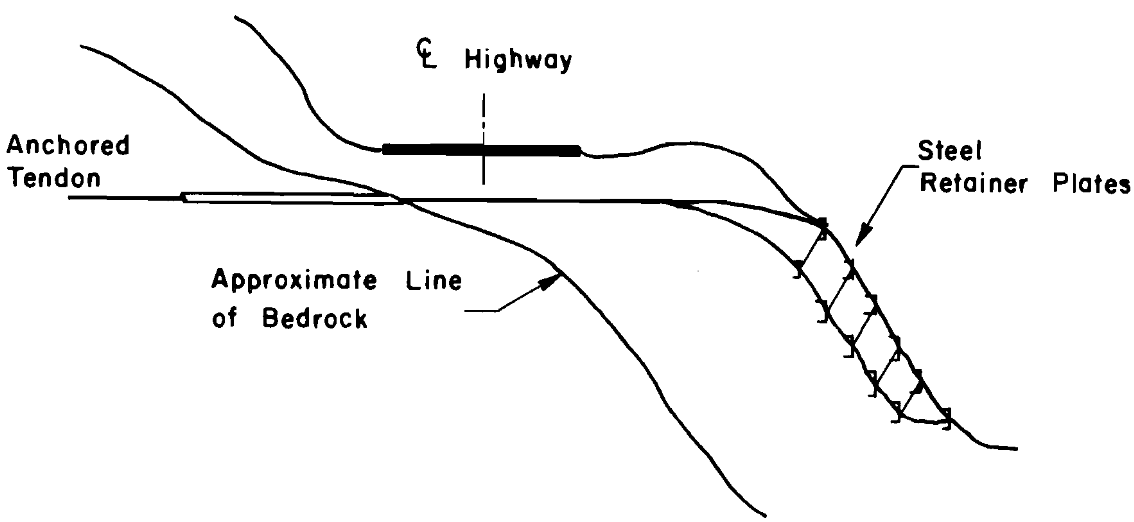
As knowledge of soil mechanics has increased, various types of restraint structures have been successfully applied to correct landslide problems. Root (1958) mentions timber bulkheads, timber and metal cribbing, concrete retaining walls, rubble and masonry retaining walls, concrete and timber piling, and toe buttressing as successful prevention and correction techniques. Reti (1964) and Gould (1970) report a successful correction of a hilltop stability problem using the anchored retaining wall illustrated in Fig 4.1a. Cutler (1932) and Allen (1937) report the use of the unique anchored baffle system illustrated in Fig 4.1b for restraint of shallow movement. However, in the case histories reported in the literature this method was unsuccessful when the soil beneath the retainer plates became saturated. For the Minneapolis Freeway slide Shannon and Wilson (1968) reported that the slit-trench buttress and retaining wall system shown in Fig 4.2 was found to be the most economical and permanent solution.

While some of the largest highway related landslides reported in the literature have been successfully stabilized using restraint structures, not all uses of retaining structures, even in recent times, have shown such success. Many failures have occurred involving slope retaining structures. Smith and Forsyth (1971) reported the failure of Potero Hill where an anchored retaining wall was used to stabilize a nearly vertical cut overlying a railroad tunnel. The grouted anchors for this wall were embedded a distance of 40 feet into the residual soil beneath the top of the hill. The cut slope supported in this manner remained stable for two years at which time movements were noted. The failure surface had passed beyond the prestressed anchors and the hillside was creeping as a large sliding block.

Cantilever type retaining walls were used to support cut slopes along the Seattle Freeway. However, this method proved unsuccessful and the Washington State Highway Department developed an elaborate alternate design employing

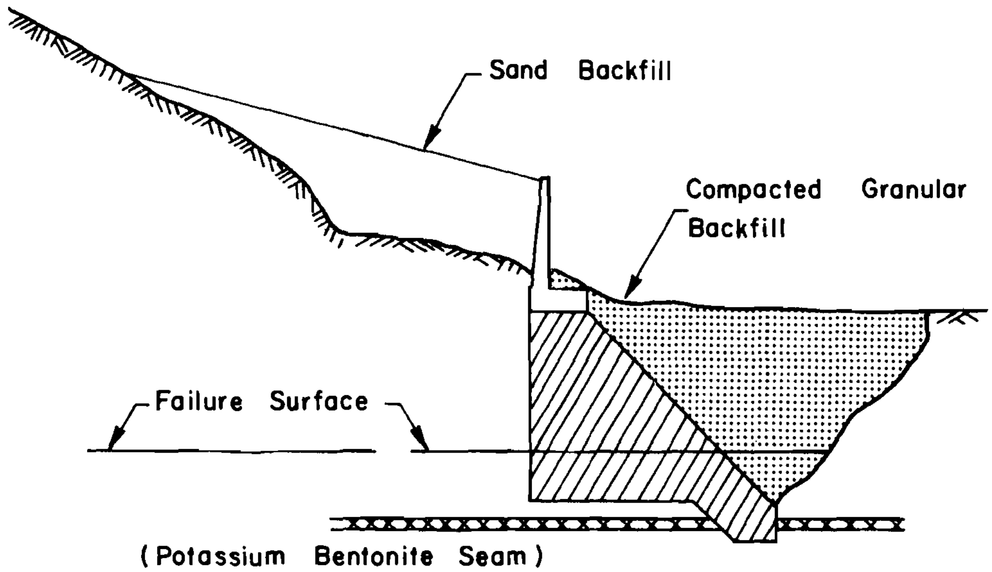


(a) Tied-Back Retaining Wall (Reti, 1964)

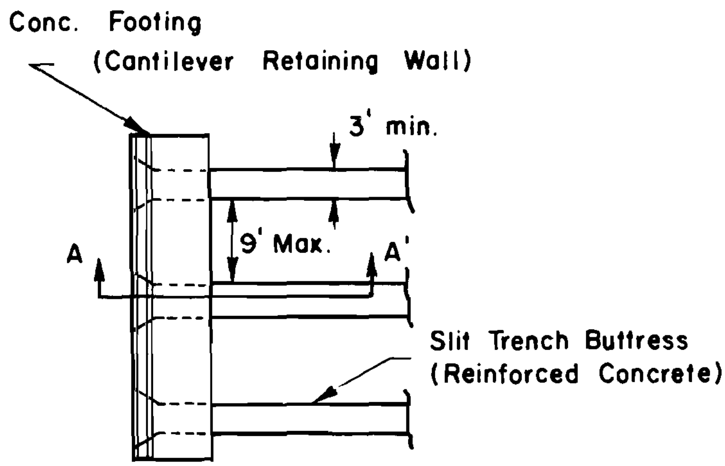


(b) Baffle Restraint System (Cutler, 1932 ; Allen 1937)

Fig 4.1. Examples of anchored and tied-back retaining systems.



a) Cross Section



b) Plan

Fig 4.2. Slit trench buttress - cantilever retaining wall used for Minneapolis Freeway slide (Shannon and Wilson, 1968).

massive cylinder pile retaining walls, illustrated in Fig 4.3 (Shannon and Wilson, 1963). These walls were successful in correcting the problem.

A long-term decrease in shear strength has accounted for several failures of retaining structures in overconsolidated weathered clays. Hypotheses as to the cause of the long-term strength decrease are well-documented in the literature. Skempton (1949) cites examples of six retaining wall failures in the overconsolidated London clay. These failures were in cut slopes and were attributed to the time dependent weakening of the soil. In all cases cited, soil strengths at failure averaged less than 50 percent of the original unconfined compressive strength. The average time to failure was 15 years. Terzaghi (1936) gave four examples of slides in stiff-fissured clays and quoted the average shear strength at the time of failure. These strengths were far less than the original undrained strength of the clay. Cassell (1953) gives further evidence of this phenomenon.

Terzaghi (1936) outlined the mechanics for softening in stiff-fissured clays. Skempton (1949) presented Terzaghi's discussion as follows:

> In a stiff-fissured clay the fissures are normally closed, but when a cut is made there is opportunity for lateral expansion toward the slope. This allows some of the fissures to open and, owing to the high strength of the clay itself, the fissures can remain open at considerable depths. Water will then start percolating through the open fissures and the clay exposed on the faces of the fissures will start softening by absorbing water. This softening will, in turn, lead to slight movements and consequently more fissures will be opened. The progressive nature of the process may eventually lead to a landslide.

When restraining structures are used in overconsolidated, stiff-fissured clays the time dependent decrease in strength should be considered. Two methods have been used to design earth retaining structures in such materials. The first consists of mobilizing the full strength of the soil by preventing lateral expansion of the cut and therefore not allowing the fissures to open (Andrews et al, 1966). The second method involves designing the structure on the basis of a limiting or residual shear strength value for the soil (Skempton, 1949).

A number of examples found in the literature for typical restraint structures used as slide preventive and corrective measures are illustrated in Figs 4.4 through 4.6. While a relatively large number of case histories involving such restraint structures was found in the literature, few of these

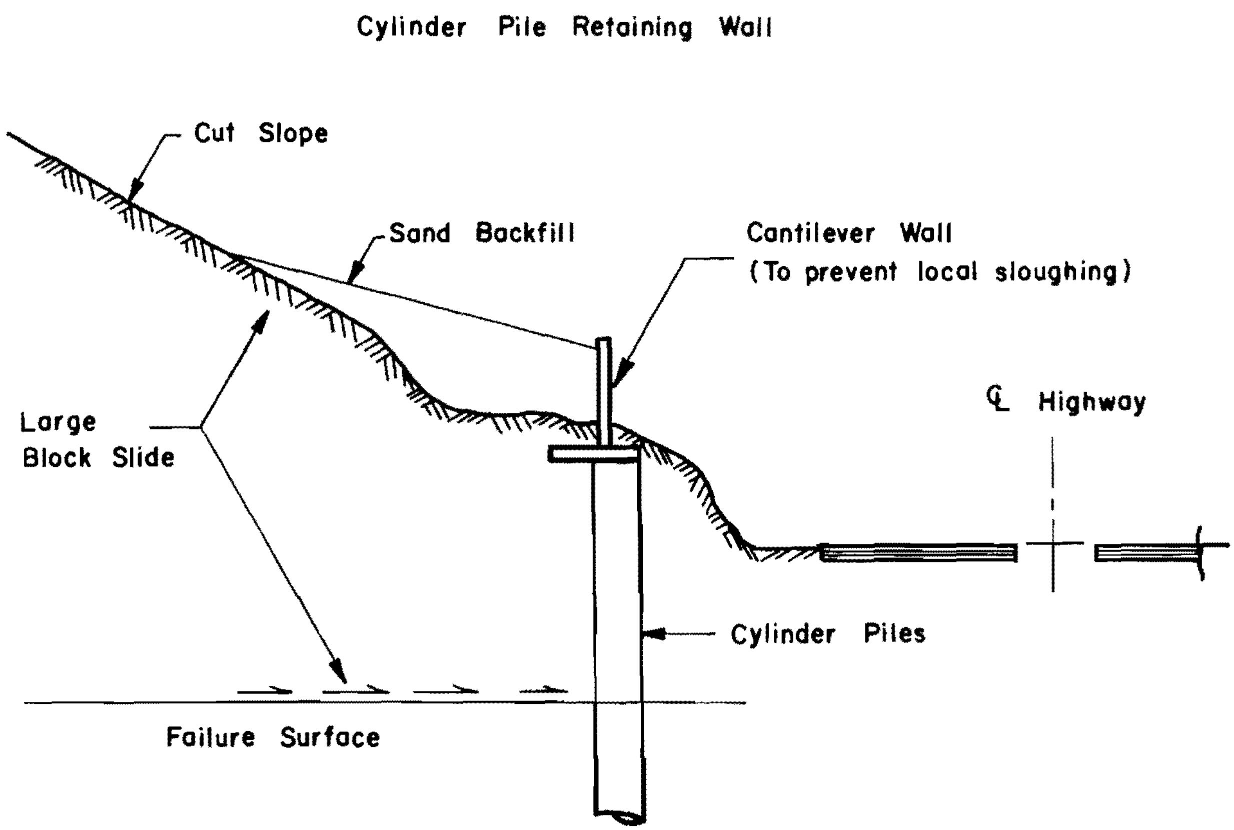
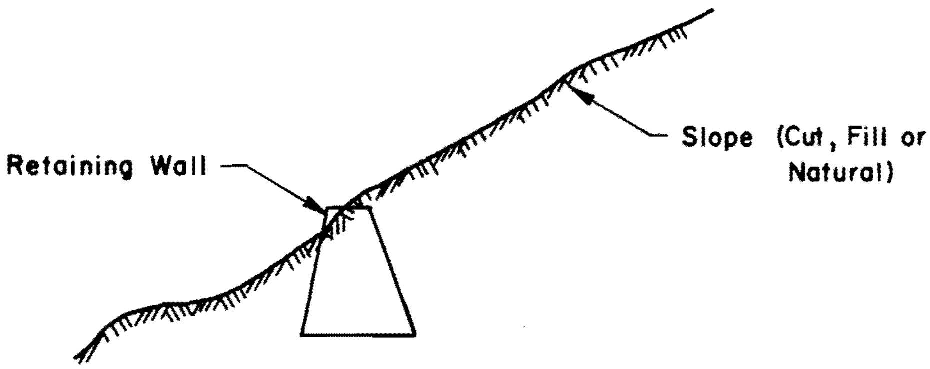
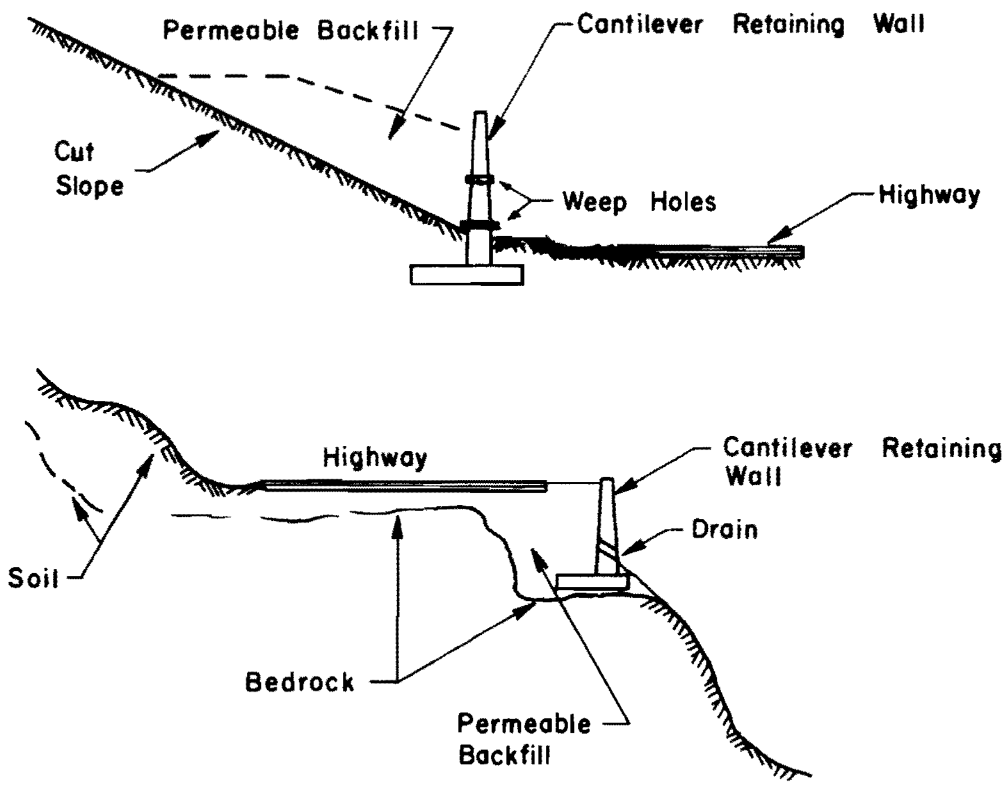


Fig 4.3. Cylinder pile retaining wall system.



a) Concrete Gravity Retaining Walls – Applicable to both cut and fill sections.



b) Cantilever Retaining Walls – Commonly used to control movements of small soil masses or sidehill fill sections.

Fig 4.4. Example uses of retaining walls for slope stabilization.

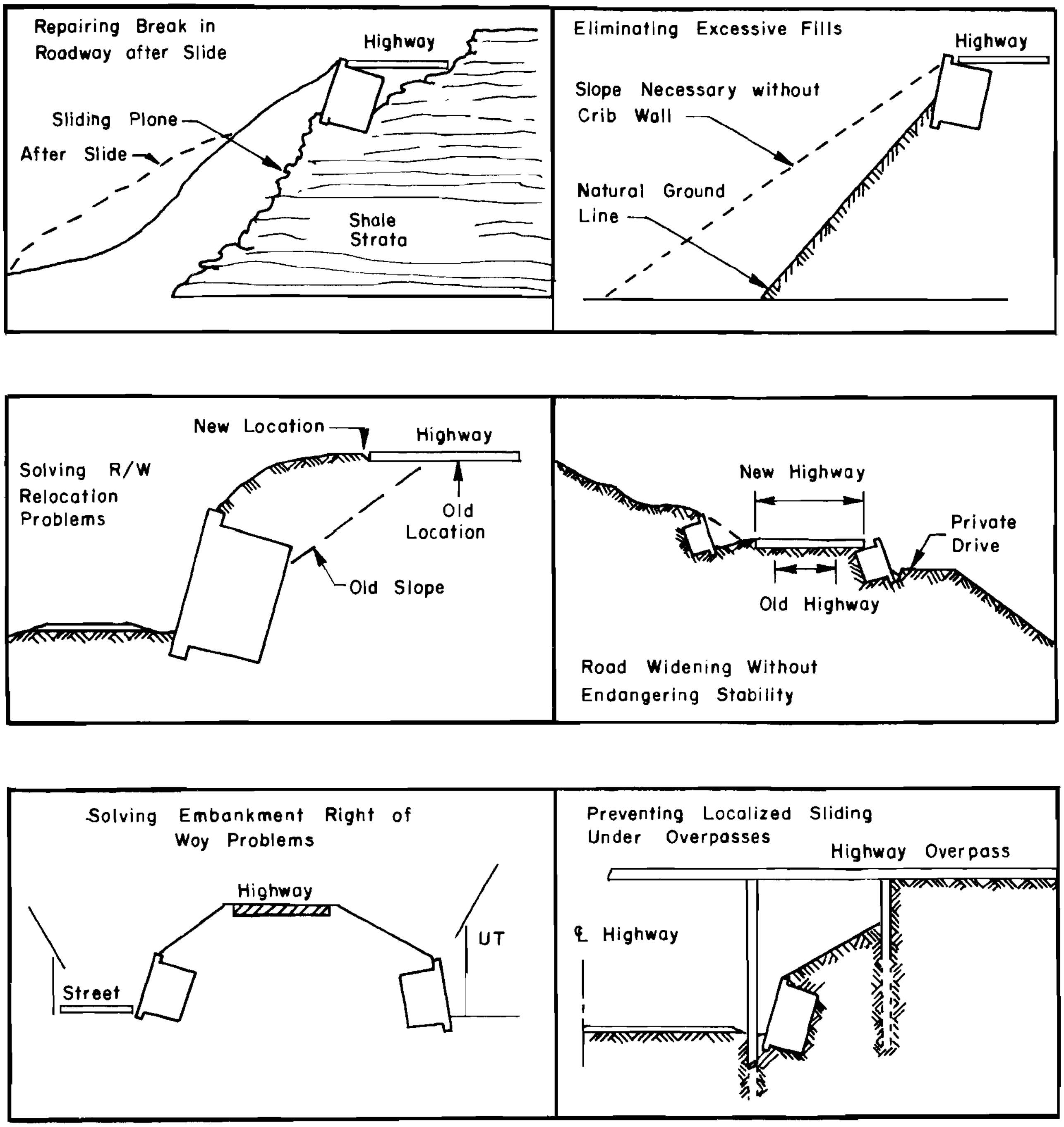


Fig 4.5. Examples of crib buttressing related to highway landslide prevention and correction.

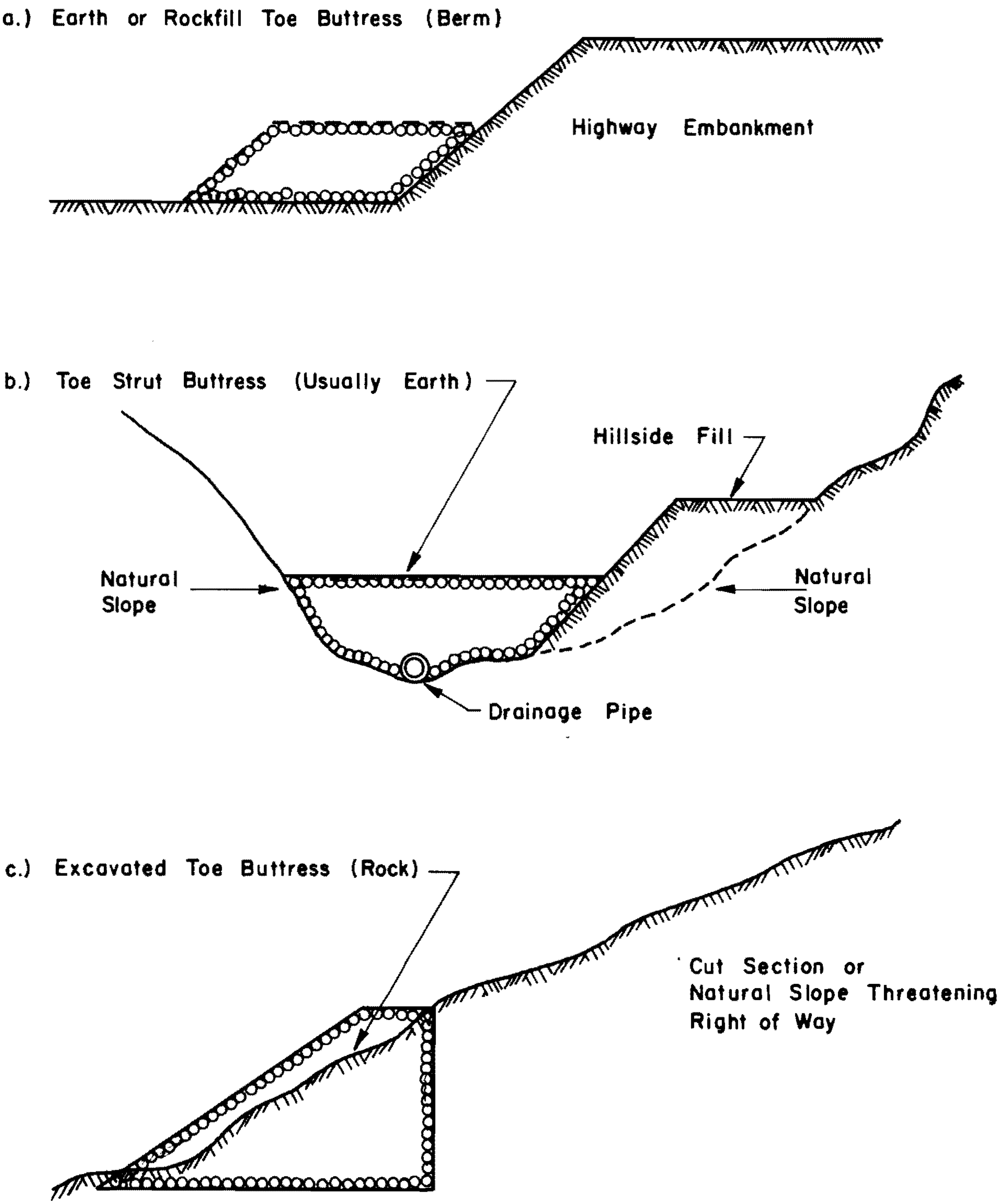


Fig 4.6. Example uses of free-draining toe buttress fills.

contain sufficient data for extensive discussion. A brief summary of the data available in the literature is presented in Table 4.1.

TABLE 4.1. SUMMARY OF CASE HISTORIES WITH RESTRAINT STRUCTURES EMPLOYED FOR REMEDIAL MEASURES

Slope Designation	Site Conditions			
Height and Inclination	Soil	Hydrologic	Comments	Remedial Measures
Green Co., Penn.; extensive sloping hillside; H(Fill) $\approx$ 20' cut/fill; 3:1; Ladd (1928).	Weathered shale to depth $\approx$ 7'.	Seepage toward the face of the slope.	Extensive slipouts extending $\approx$ to C_L of road.	Gravity-cantilever type of retaining wall keyed into solid shale foundation material.
Federal Aid Project 143-A; cut/fill section; 1-1/2:1 slope on an extensive hillside; H(Fill) $\approx$ 30'; West Virginia; Ladd (1928).	Weathered shale with shale and coal seams.	Excessive seepage from uphill shale and coal seams.	During grading operation it was found that the natural material was not stable as a sidehill fill.	A massive gravity retaining wall (reinforced concrete) keyed into solid shale was used to hold sidehill fill section. Author felt well casing piling filled with concrete would have been a better and more economical solution.
West Virginia; Federal Aid Project 19; 1:1 slope; 50' high (natural); Ladd (1928).	Weathered shale and gravel over solid bedrock.	River 50' below highway grade seepage toward river.	Flood raised river level to highway grade - failure attributed to rapid drawdown. Failure due to overturning of wall.	Reinforced concrete gravity type retaining wall keyed into solid bedrock. Random backfill of weathered shale, cobbles, and gravel weep holes provided on 7' centers.

(continued)

TABLE 4.1. Continued

Slope Designation	Site Conditions			
Height and Inclination	Soil	Hydrologic	Comments	Remedial Measures
Putnam Co., West Virginia; sidehill fill; 1:1; 50'; Ladd (1928).	Random fill of decomposed shale.	Excessive rain. Short term saturated fill.	Rain caused the fill to become saturated.	Reinforced concrete retaining wall used as a prevention; failure caused by inadequate backfill and lack of weep holes.
Cabell Co., West Virginia; 1-1/2:1; 65' high fill across a ravine 450' long; Ladd (1928).	Foundation consisted of an old stream bed of saturated silt and clay. Fill was decomposed shale.	Foundation material was saturated and unconsolidated under weight of fill.	Initial slide was a small slipout extending to ℄ of highway. First correction was to recompact the fill material in the slide area. One year later a deep slide was evident. Rock foundation found ≈ 12' below ground level. Could not be corrected until drainage scheme was employed.	Concrete pile correction: 8" diameter holes were bored 8' into bedrock, reinforced with 4-1/2" diameter rods. 120 total posts, 2 rows 3' c-c. One year later these failed. Second correction: 25' oak pile driven 1/3 up the slope; slope regraded. One year later signs of failure were evident.
California Caisson's Cut; Editors, Civil Engineering (1958).	Homogenous clay slope.	No data.	Movement slowed from 1-1/2"/day to less than 0.3"/day.	20' concrete caissons each reinforced with 2 tons of 11" bar 10' above and 10' below failure plane.

(continued)

TABLE 4.1. Continued

Slope Designation Height and Inclination	Site Conditions Soil	 Hydrologic	Comments	Remedial Measures
Hilltop slide, California; 30'-40'; 1:1 cut/fill; Reti (1964), Gould (1970).	15'-20' of residual soil overlying shale and sandstone of the Miocene sediments; τ-min-500-600-psf.	GWT 17' below surface. Seepage along shale/fill interface.	Dames and Moore Project. Tiebacks prestressed to 150k design load look. 1 KSF design pressure on wall = $2 \times K_A$ derived pressure.	20' high × 200' long × 8" thick gunite retaining wall. Two tie rods at each pilister each 100' long; 60' embedment length; 13 psi bond.
Bagnel Dam Approach; 1-1/2:1; 25' fill; Cutler (1932).	No data.	No data.	Retaining wall considered but too expensive; extent of slide 200' long, 25' high, 10-12' deep.	Suspender system used to hold surface of fill in place (see typical plan and section under type of restraint structures).
Fill 1-1/2:1; 30' high; Indiana; Allen (1937).	Fill was not compacted.	No data.	Same method as Bagnel Dam. (Patented technique by Willcox.)	Suspender system anchored by tieback bored and grated into rock.

(continued)

TABLE 4.1. Continued

Slope Designation	Site Conditions			
Height and Inclination	Soil	Hydrologic	Comments	Remedial Measures
Ohio Route; 20'-30'; 1-1/2:1 fill; Krauser (1950).	No data.	No data.	Roadway relocated on or near solid rock ledge. Wasted rock backfill used as much as possible. Above this random fill placed. See typical plan and cross section under types of restraint structures.	A. With a shallow rock ledge concrete gravity retaining wall used. B. With a deep ledge low 89 pile set in bored holes in rock 1/3 length in rock concrete placed in bored holes. Cribbing placed behind piling.
Hillside fill; 1-1/2:1; 40'; Kane (1935).	Interbedded blocky limestone separated by clay layer.	One clay layer was funneling water into the top of the fill.	Slide caused by excess water.	First attempt was to place French drains at all locations of expected settlement and reconstruction of the fill. The slide continued and as a result dry rubble retaining wall was placed at the toe. Good results.

(continued)

TABLE 4.1. Continued

Slope Designation	Site Conditions		Comments	Remedial Measures
Height and Inclination	Soil	Hydrologic		
Wetteren Railway Cut; 30:35'; 1-1/4:1 (England); Marivoet (1948).	Loam underlain by medium blue clay underlain by stiff clayey fine sand.	GWT at surface of slope.	Combined correction cross section and stability analysis.	Required for stability: lower GWT and buttress toe; 2:1 slope would yield F.S. 20% greater; counterfort drains 30% average in F.S.
Toddington 45' Cut; 2.5:1 slope (England); Cassel (1948).	Basically stiff blue hias clay. Top 10-12' badly deteriorated.	Groundwater movement toward the face ≈ 8-12' below the surface of the cut.	Slip 8-12' deep; stiff fissured clay 40 years old at time of failure. Residual strengths required a 7:1 slope for stability.	Heavy rock toewalls have stopped further movement from 1949 to 1953.
Hook Norton 58' Cut; 2:1 slopes (England); Cassel (1948).	Stiff fissured hias clay. LL = 63.2% PL = 33% PI = 30% LI is (-)	Several springs evident on face of cut. Groundwater movement toward face of cut.	σd = 105 lb/ft^3 σ in situ = 123-130 lb/ft^3 σ in failure plane 95 lb/ft^3 70 years old at failure.	Gravity type concrete reinforced retaining walls and heavy counterfort drains.

(continued)

TABLE 4.1. Continued

Slope Designation	Site Conditions			
Height and Inclination	Soil	Hydrologic	Comments	Remedial Measures
Hullavington 40' Cut; 2:1 slopes (England); Cassel (1948).	Stiff fissured hias clay. LL = 57% PL = 23% PI = 33% LI = (-) to 0.14 W%Failure zone = 29% Above failure = 25% Below failure = 17%	GWT 3-4' below surface of cut; flow toward.	44 years old at failure. σ = 125 lb/ft^3 above f-zone σ = 113 lb/ft^3 in f-zone σ = 132 lb/ft^3 below f-zone 7.5:1 slope required for stability 210' = depth of slip.	No remedial measures reported. In adjacent sections where deep counterfort drains have been installed no slides have <u>ever</u> been reported.
Typical slides in weathered London clay; H $\approx$ 40'; slope = 2.5:1 (England); Cassel (1948).	Weathered London clay. LL = 70% PL = 26% Qu = 5000 lb/ft^2 MC < 30% Strength required for F.S. = 1 = 560 lb/ft^2	GWT fluctuated between 2' of the surface and 8' of the face of the cut slope. Seepage toward the cut.	All slides occurred in the weathered top layer of the London clay. Average depth of sliding was $\approx$ 3-5'. Slopes were $\approx$ 45 years old at time of failure. All slips occured in upper 1/2 of the slope.	Low, relatively thin, toe walls have been used to stop the problem in these plastic fissured clays. Gravel backfill has been used in addition to transverse gravel filled trenches extending 4-6' below the surface of the slope at 10' intervals; weep holes provided in the walls.

(continued)

TABLE 4.1. Continued

Slope Designation	Site Conditions			
Height and Inclination	Soil	Hydrologic	Comments	Remedial Measures
Watford Bypass; H = 15'; slope = 2.5:1 (England); Skempton (1948).	Brown weathered London clay.	No GWT complications although clay was weathered by rainfall and sun.	Several slides occurred between 5 and 10 years after construction in 1927. F.S. = 1 $\phi = 0$ analysis $C = 220\ lb/ft^2$ Average strength in adjacent soil = $1000\ lb/ft^2$	Maintenance operations repeatedly patched the slide.
Retaining Wall Failures; London clay; H = 20-55'; backfill slope 3:1 to horizontal (6 failures); Skempton (1948).	All slides occurred in brown (weathered) London clay. Grey London clay at greater depths.	GWT parallel slope at 5-8' depths.	Time to failure shown graphically. All were cut slopes; strengths at failure versus original unconfined compressive strengths shown graphically.	Retaining wall failures.
Wood Green Station Retaining Wall Failure; H = 21' at 3:1; H = 16' vertical; Henkel (1957).	Brown weathered London clay; S = 1500 to $3000\ lb/ft^2$; LL = 78 PL = 30 W% = 38% C' = $250\ lb/ft^2$ (wedge method of analysis)	GWT 5' below surface followed slope of the cut to weep holes in retaining wall. Seepage toward face of cut.	Wall constructed in 1893, failed 1948; 55 years old at failure. Effective stress method of analysis assuming ϕ' fully mobilized and = 20^o value of C' determined for stability of C' 0 with geologic time.	Retaining wall failure. Generally correction has been to install counterfort drains without toe wall for support. Approximate 25' c-c 4' wide (each counterfort). See method of correction.

(continued)

TABLE 4.1. Continued

Slope Designation Height and Inclination	Site Conditions Soil	 Hydrologic	Comments	Remedial Measures
Northolt Cut; 2.5:1; H = 33'; Henkel (1957).	Brown weathered London clay; LL = 78% PL = 28% W% ≈ 30% Q-Test at W = 30% 2100 lb/ft^2	GWT parallels slope 35' below surface. Pore pressure reduced rapidly with installation of counterfort drains.	Failure along weathered/unweathered clay interface. ≈ circular failure surface. Along failure surface very soft clay W = 44%; vane shear = 270 lb/ft^2. Analysis by Bishop's method.	Failure occurred completely above a low toe wall. Counterfort drains (as above) used to stabilize the slide.
Natural slope; 4:1 to 5:1; 100-130'; Peynircioglu (1969).	Very stiff green and blue clay with interbedded sand and gravel lenses.	GW flow ≈ parallel to and toward the hillside. Artesian conditions developed in the sand and gravel lenses.	Slides occurred from 1963 to present - of progressive nature. Extensive laterally loaded pile tests run to determine design of piles for correction of landslides.	(1) A pile founded retaining wall at the toe of the slope. (2) A drainage trench through the center of the slide (toe to crest) with vertical extensions. (3) A pile founded retaining wall at the crest of present movements to halt future progressive slides.

(continued)

TABLE 4.1. Continued

Slope Designation	Site Conditions		Comments	Remedial Measures
Height and Inclination	Soil	Hydrologic		
Seattle Freeway; H $\approx$ 100'; 4:1 original slope cut section; Andrews et al (1967); Engineering Report by Shannon and Wilson (1963).	Lacustrine deposited layers of silts and clays overlain by variable thicknesses of sand and gravel. Silts and clays were highly over-consolidated and were highly expansive upon relief of lateral pressures.	Very erratic GW conditions. In general water content appeared to decrease with depth. Perched water tables were present in zones between the silt and clay layers.	During the construction of the Seattle Freeway small cuts (20-30') were made at the toe of the original 4:1 slopes. Original design called for cantilever retaining walls to be used to support these cuts. However, as the sections were cut the release of the lateral pressures caused movements of the entire hillside. It was concluded that the design must either be to prevent failure (replace K_o pressure) or design for post failure strengths.	Remedial measure was the cylinder pile retaining wall. (See previous section on types of restraint structures.) These were designed to prevent failure; even though some lateral strains were evident it was concluded that if lateral strains are kept below some limiting value, the soil will retain a relatively high shear strength and at the same time earth pressures will be relatively modest.

(continued)

TABLE 4.1. Continued

Slope Designation Height and Inclination	Site Conditions Soil	 Hydrologic	Comments	Remedial Measures
Minneapolis Freeway; H $\approx$ 75'; originally cut at 2:1; 1200' long; Schwantes and Adolfson (1968); Engineering Report by Shannon and Wilson (1968); Gedney (1970).	Hard shale overlain by alternate layers of sandy till, sand and gravel. Failure occurred along a bentonitic seam below the shale formation.	Although the GW level was generally high, no complications resulted. Retaining walls were designed so that no build-up of hydrostatic pressures resulted.	Slope indicators were used to obtain the exact location of the failure plane. Failure occurred during excavation of cut slope. Cause was the release of confining pressure which reduced the effective strength of the bentonitic seam. Failure was analyzed as a sliding wedge along the bentonitic seam.	A sand berm temporarily halted movements until design of various types of retaining structures could be completed. Several buttressing techniques were contemplated. Split trench buttressing was chosen as the final solution. (See preceding section on types of restraint structures.)

CHAPTER 5. ELIMINATION AND AVOIDANCE OF LANDSLIDES

Elimination and Avoidance

Early in 1959, the California Division of Highways in conjunction with the city of Los Angeles retained the consulting firm of Morgan, Proctor, Mueser, and Rutledge to investigate the landslide troubled Palisades on the Pacific Coast Highway north of Los Angeles. At the time of the initial investigation the Highway Department felt that an extensive drainage system incorporated with berms at twelve major troubled spots would be required to remedy the situation. It was believed that this method was the only economically feasible plan for correction of the problem. In January of 1961, following an extensive engineering investigation, the final report was issued and it indicated that the costs for the proposed remedial measures were completely out of proportion to the property values in the area. The alternative solution consisted of relocation of the highway seaward. This alternative involved construction of a rock causeway or bridge, and was considered much less expensive than the extensive regrading and drainage suggested earlier. It was also noted in the engineers' report that this solution guaranteed the safety of future traffic using the highway.

Avoidance as a remedial alternate consists of relocation of the highway in order to avoid the consequences of the slide. This may consist of bridging the unstable area, or relocation of the highway away from the vicinity of the slide. On the other hand, elimination as a remedial measure consists of removing unstable material which may prove detrimental to the structure in question. Avoidance has been most economically applied as a preventive measure during stages of highway planning. At this time, field surveys and photogrammetry studies may be used to identify potentially unstable areas. Alternative routes may be chosen, and costs for additional right-of-way may be explored. Although the primary application of avoidance is as a prevention rather than a correction, several instances exist where avoidance, combined with drainage or excavation, has provided an economical solution to an existing problem (Root, 1958; Baker and Marshall, 1958; Ladd, 1928; and Engineering News Record, 1961).

Elimination involving complete removal of the slide has proven feasible for small slides and is most economical when firm foundation material is available at relatively shallow depths. Several instances appear in the literature in which the existing slide was stripped of all unstable material, backfilled with drain gravel, and covered with the original material (Ladd, 1928; Root, 1958; Cedergren, 1962; Cedergren and Smith, 1962). When ground water or seepage has been identified as the cause of small slides, excellent success has been reported using this method (Smith, 1964).

The importance of avoidance as a means of controlling highway related landslides was realized by the Bureau of Public Roads as early as 1927. Ladd (1928) has indicated that the West Virginia Road Commission concerned themselves with the problem of relocation at obvious points of danger during the preliminary stages in the design of a proposed highway. The final decision for relocation was based upon the present cost of additional grading or earthwork versus the probability that part of the road might be lost in the future and the related cost. In many cases confronting West Virginia engineers, briding or relocation of the highway would have been less expensive than the combined cost of maintenance and temporary remedial measures undertaken (Ladd, 1928).

The major advantage offered by avoidance or elimination methods is that, with respect to a given slide, future stability may be insured. No other remedial measure offers as permanent a solution to the problem. In some cases relocation offers improved highway alignment. Sidehill fill sections have often been stabilized with a resulting improved alignment (Kane, 1935). In effect, this method consists of placing the relocated highway on a broad bench cut into the head of the unstable slope. If the load removed is sufficient to produce stability in the entire mass, the solution will prove effective and permanent. A sidehill fill near East Liverpool, Ohio, was successfully treated in this manner (Baker and Marshall, 1958).

Bridging consists of spanning the unstable mass by means of a highway bridge or sidehill viaduct. This method is most applicable to relatively narrow slides on steep slopes (Root, 1958; Baker and Marshall, 1958). If the slope is flatter than 2:1 or the length to be spanned is greater than 200 feet, it is doubtful that this method will be economically feasible and other methods of correction will usually be used (Baker, 1953; Root, 1958). In view of the above applications and restrictions the use of bridging has been mostly confined to mountainous terrain, and since it involves no novel approaches the coverage in the literature is quite scarce.

Several disadvantages of elimination or avoidance methods lie in the physical difficulties which may be encountered during extensive excavation, location change of the highway, or construction of a bridge or sidehill viaduct. In many cases these alternatives will be far more costly than less permanent methods of correction. However, care must be taken not to base economic calculations on initial cost alone; future maintenance expenditures must also be considered. At times relocation which is satisfactory in terms of stability will result in an unacceptable alignment. A summary of the relative advantages and limitations of these methods is presented in Table 5.1.

TABLE 5.1. SUMMARY OF AVOIDANCE AND ELIMINATION

Method	Best Applications	Cost	Limitations	Remarks
Complete removal of slide material (elimination)	Small slides with shallow soil profiles. Area above slide should be stable.	Excavation right-of-way changes or damages of adjacent property.	May be very costly for large slides. Area above slide may be undermined by excavation.	A stability analysis may be necessary to determine stability above the eliminated slide.
Relocation of highway (avoidance)	Applicable to every type movement, but may be prohibitive due to cost. Best when the roadway has been undermined and stable soil or bedrock is available immediately uphill.	Excavation. Pavement and subbase. Right-of-way.	Large initial investment. Movement usually not controlled.	A detailed cost estimate is required for further economic comparisons.
Bridging (avoidance)	Steep hillsides with a relatively narrow slide. Firm foundation material available only at deep depths.	Bridge.	Very high cost.	Bridging structure must withstand possible movements in future.

CHAPTER 6. EXCAVATION METHODS FOR LANDSLIDE CORRECTION AND PREVENTION

Introduction

Slope excavation methods are generally utilized to increase the stability of a slope by reducing the driving forces; in this manner a more favorable balance of resisting and driving forces is achieved. In several instances an optimum balance of these forces has best been achieved by the use of excavation methods in conjunction with one or more of the methods used for increasing the resisting forces in the slope, as outlined in previous chapters. In this chapter the excavation methods most commonly employed, removal of soil at the head of the slope, benching, and slope flattening, are described and examples of the use of these techniques are presented.

Correction of landslides using slope excavation results in a permanent solution to the problem if a proper investigation and analysis precedes the use of these methods (Baker and Marshall, 1958). Economic comparison between these and other methods of correction often shows that slope excavation (either flattening or benching) is the least expensive alternative for providing a permanent solution to the problem. However, economics alone do not provide sufficient data on which to base an engineering design. For example, a study of the landslide which occurred during the construction of the Minneapolis freeway indicated that slope flattening was the most desirable method of correction on the basis of construction costs. However, aesthetic considerations and availability of right-of-way precluded the use of this measure. As a result, slit trench buttressing and cantilever retaining walls were used to preserve the original hillside above the cut (Shannon and Wilson, 1968). Peck and Ireland (1953) indicate that excavation techniques are generally most economical when used to correct deep slides involving from 20,000 to 2,000,000 cubic yards of material. In many cases where extremely large slides and prime property are involved, excavation has not been considered, even though a suitable design could have been achieved (Forbes, 1947; Root, 1955a; and Reti, 1964). Of ten sildes reported in the Montana Landslide Research Study (1969), six were corrected by slope excavation. The economics of excavation as a remedial measure in terrain where land is undeveloped and right-of-way is readily available appear favorable.

Case Histories and Excavation Schemes

Several excavation procedures have been used for the prevention or correction of landslides in slopes composed primarily of soil type materials. Removal of the head or upper portion of the slope has been used to reduce the driving forces and thereby increase the stability of the slope. Baker and Marshall (1958) state that this method is most applicable to deep slides with circular failure surfaces. Further, as a preventive measure they recommend that one to two times the quantity of soil removed at the toe in a sidehill cut should be removed at the head. When used as a correction it has been recommended that from 15 to 25 percent of the moving mass should be removed from the head of the slope. These criteria seem of little general applicability, and use of these recommendations has not been apparent in the literature. More specifically, it appears that soil type, slope geometry, and proper stability analyses should be used to determine the amount of excavation required to produce the desired increase in stability.

At times a lowering of the grade line of a highway may produce results similar to those achieved by removal of the head. This method consists of locating the highway on a broad bench cut into unstable material rather than constructing a fill on the surface, which may increase the driving forces. Root (1958) recommends that the grade reduction be no less than 10 percent of the height of the slope.

Only two case histories were found in the literature relating to removal of the head of the slope. Both were summarized by the Committee on Landslide Investigation in 1958 (Highway Research Board, 1958). In one of these, the stabilization of the Cameo slide reported by Peck and Ireland (1953) and illustrated in Fig 6.1, stability analyses disclosed that removal of the head of the slide (shaded area B in figure) would provide a factor of safety of 1.3, while removal of a similar volume of material near the toe of the slope (area A) would produce a factor of safety of only 1.01, based on an initial factor of safety of unity. Removal of area B resulted in a permanent solution to the problem.

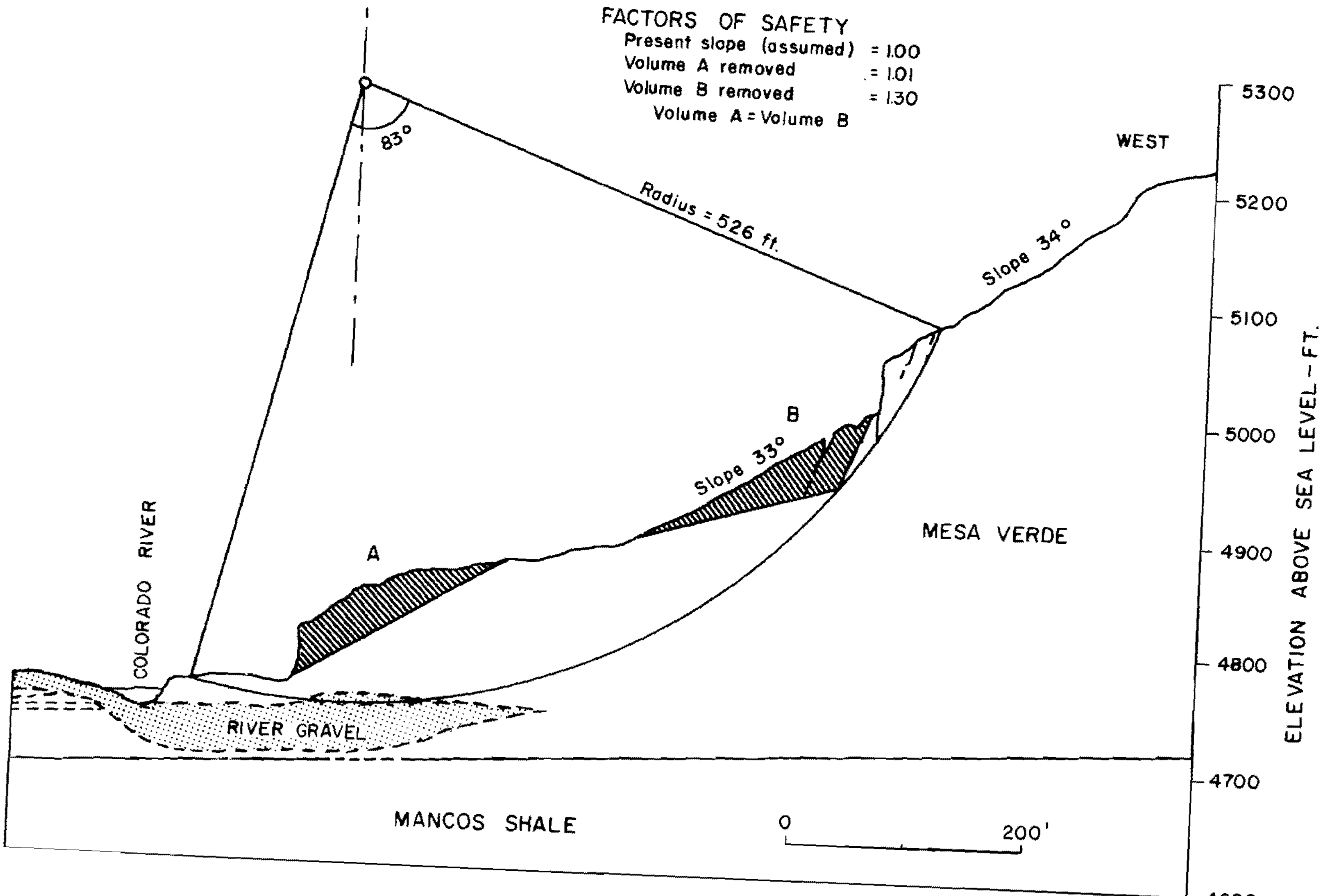


Fig 6.1. Comparative evaluation of excavation alternatives - Cameo slide (Peck and Ireland, 1953).

Benching of slopes has also been used to provide stable slope conditions. Stuart (1916) reported the use of benched slopes to stabilize a 196-foot cut in the Catskill Mountains. Hennes (1959) indicates that the chief advantages of this method as compared to slope flattening lie in the reduced right-of-way requirements and the reduced volume of excavation required to produce a given increase in the factor of safety. As in slope reduction, benching unloads the head of the slope, and a properly designed bench should reduce the height of a slope to such an extent that an increased factor of safety will be achieved.

Both the Dyerville Cut and the Carquinez "Big Cut" reported by Cedergren and Smith (1962) offer examples in which a benched cut design was incorporated into the overall stabilization procedures. These slides incorporated benched slopes and horizontal drains at approximately 100-foot intervals on the side slopes of the cuts (Figs 6.2 and 6.3).

Slope flattening, another excavation technique, has been used as both a preventive and a corrective measure for improving stability. This method has been most effective when undercutting of the slope has produced shallow slides that extend only a short distance beyond the top of the slope (Root, 1958).

An excellent example of the use of slope flattening and regrading as a remedial measure is provided by the 320-foot Mulholland Cut for the San Diego Freeway illustrated in Fig 6.4 (Cedergren, 1962). During construction it was noted that the 1:1 benched slopes, originally designed for the predominately sandstone and interbedded shale cut, were unstable. Subsequently the cut was redesigned by employing uniform 3:1 side slopes, raising the highway grade line 60 feet, and constructing earth fill buttresses against the bottom 70 feet of the final cut. The redesigned slope proved to be the most economical alternative and has remained stable since completion.

While slope excavation and regrading procedures may sometimes be restricted in their application as remedial measures by the availability of right-of-way, these techniques have been successfully employed in a number of cases to correct unstable conditions in earth slopes. Although there probably are numerous other instances in which these techniques have been utilized, no record of their use has been reported in the literature, thus accounting for the limited amount of well-documented information on the analysis, design, and performance of excavation methods for slope stabilization. However, the selection and evaluation of alternate methods of slope excavation can be aided

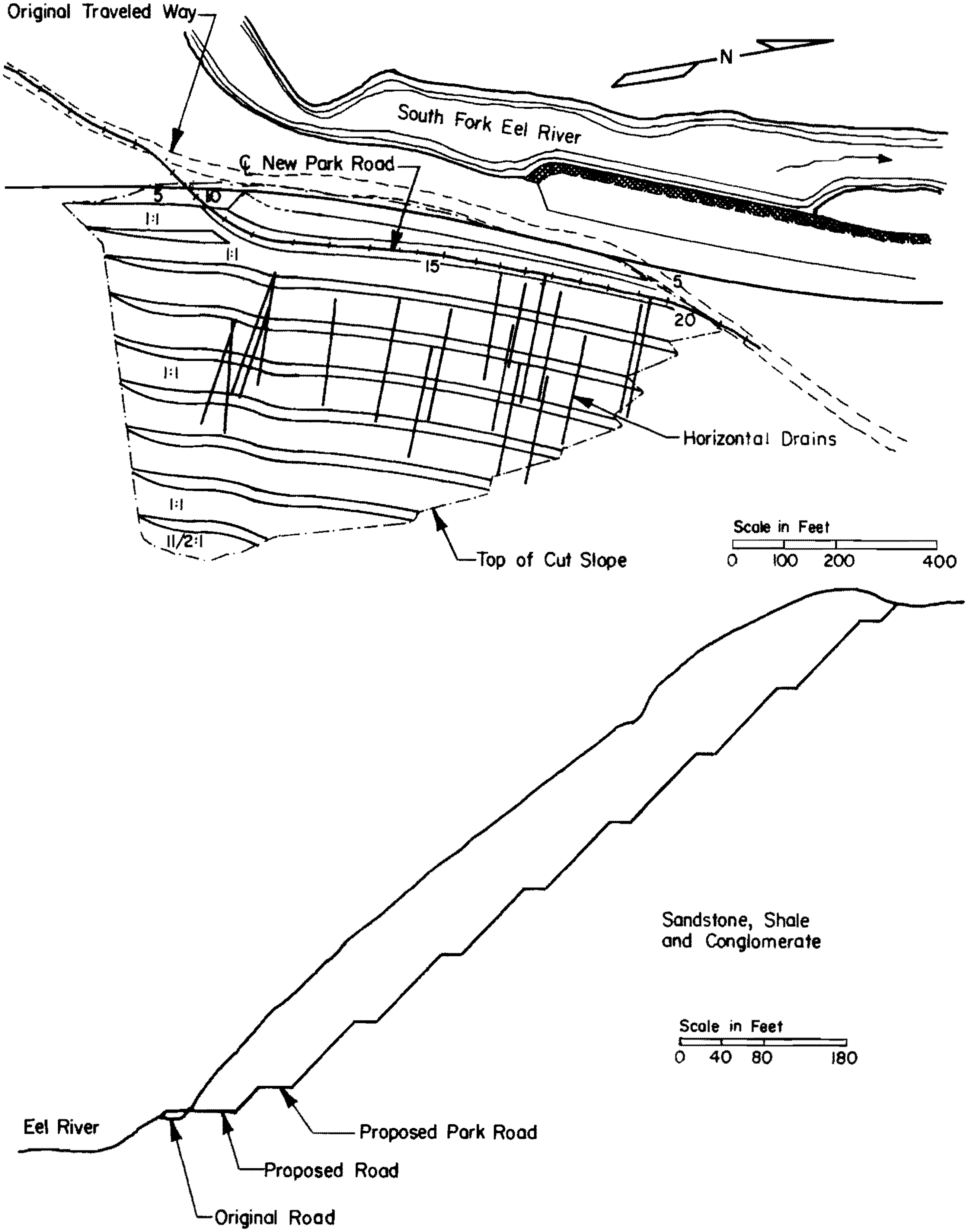


Fig 6.2. Combined benching and drainage for slope stabilization - Dyerville cut (Cedergren, 1962).

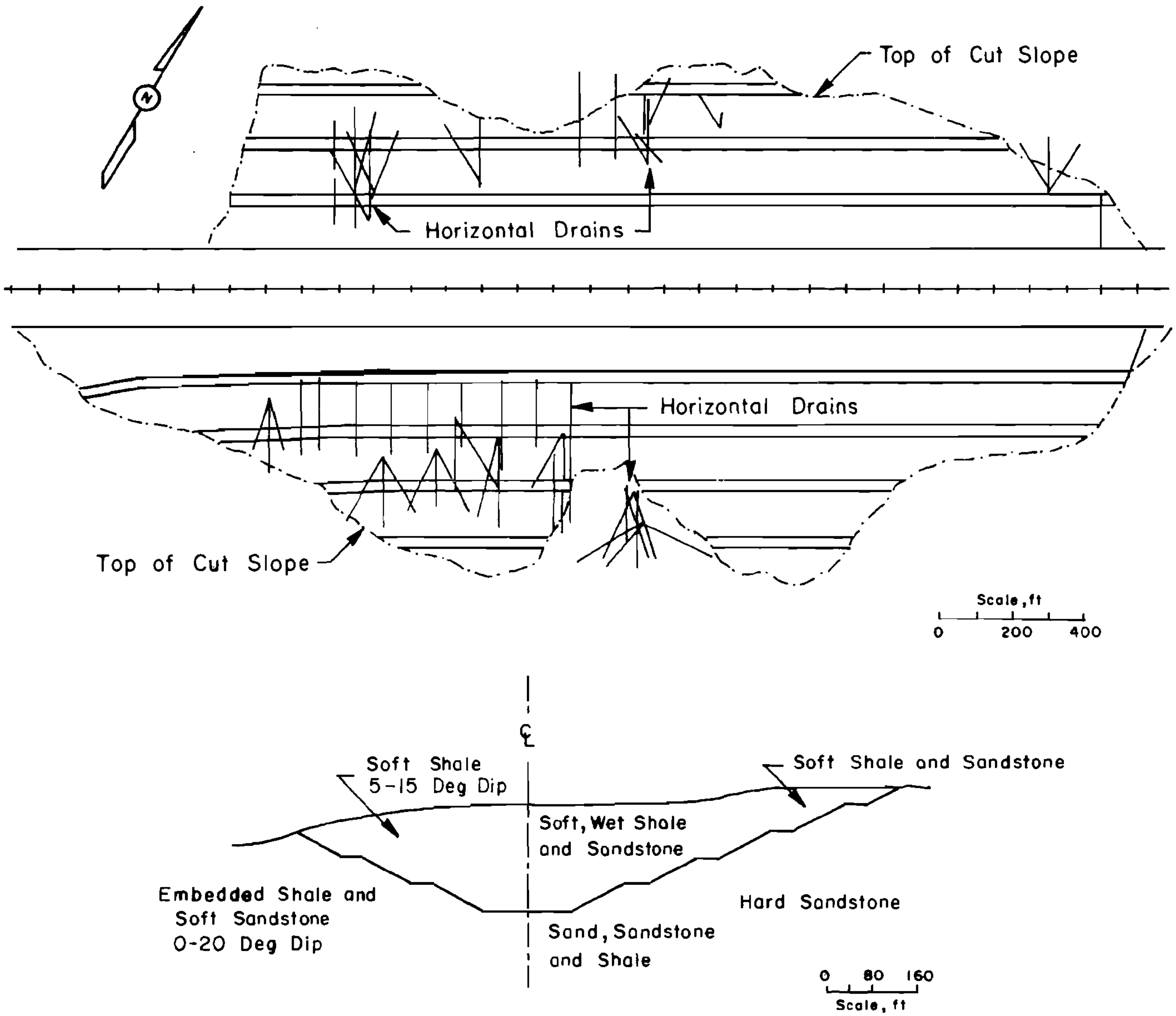


Fig 6.3. Combined benching and drainage for slope stabilization - Carquinez "Big Cut" (Cedergren, 1962).

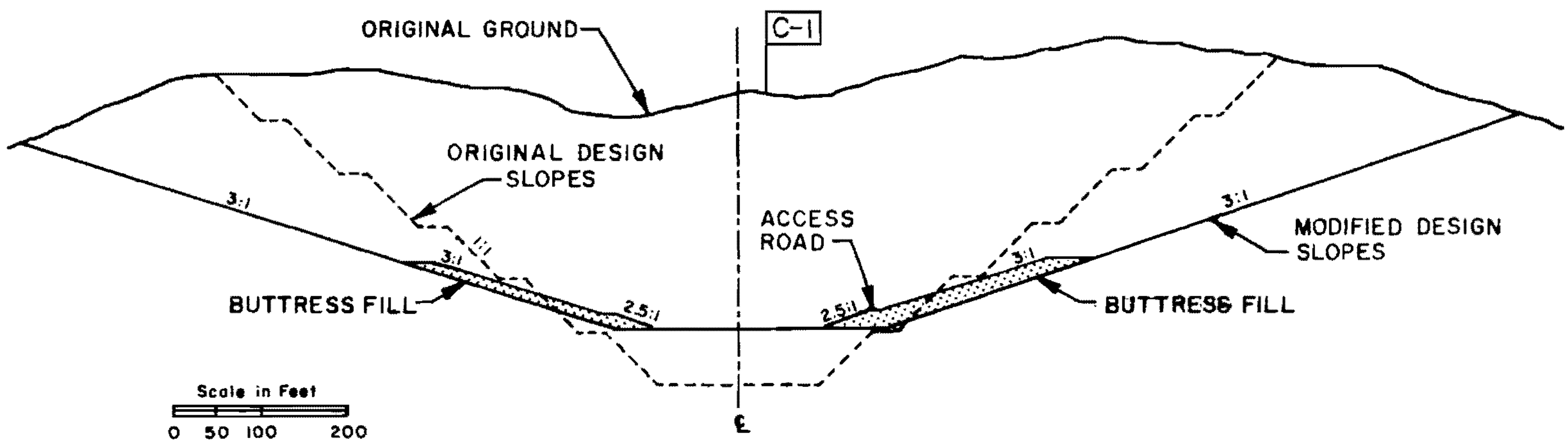


Fig 6.4. Slope flattening and grade change - Mulholland cut, San Diego Freeway (Cedergren, 1962).

considerably by the use of appropriate analytical techniques. These techniques for evaluating slope excavation alternatives are described in Chapter 8.

CHAPTER 7. OTHER METHODS OF SLOPE STABILIZATION

Introduction

In addition to the more frequent application of drainage, retaining structures, slope excavation, and highway relocation, several new or innovative methods of control have been successfully employed to arrest or prevent slides. These methods include treatment with additives, thermal stabilization, electro-osmosis, slope vegetation, reinforced earth, and freezing of the soil. While it has been shown in the literature that these methods do present a feasible means for slope stabilization, their use has been limited because of high costs and limitations imposed on the methods due to soil conditions. However, in some instances they have represented the most economical means of stabilization. In this chapter these methods are reviewed and the relative advantages and disadvantages of each are briefly discussed.

Electro-Osmosis

Several large-scale applications of electro-osmotic stabilization procedures have been reported in the literature (Casagrande, 1953; Casagrande et al, 1961). However, these deal with the stabilization prior to movement of open cuts or embankments. Of these references, only the paper by Casagrande et al (1961) deals with the stabilization of a slide.

The construction of the Trans-Canada Highway near Marathen, Ontario, required excavation of an 80-foot by 40-foot open cut, 15 feet deep, for installation of a portion of the pile foundation for a bridge. During the excavation of this cut a slide developed in an adjacent 2.5:1 slope in saturated silt, threatening a portion of the bridge which had already been completed (Casagrande et al, 1961). Several methods of stabilizing the slide were considered, including freezing, chemical additives, caissons, slope flattening, relocation, and electro-osmosis; but after evaluating these, electro-osmosis was selected as the best and most economical overall solution to the problem. Three months after the installation of the electro-osmotic dewatering system, the groundwater level had been lowered by between 33 and 45 feet, the average water content of the slope had decreased by approximately 4 percent, and the average unit weight of the soil was increased by an average of 6 pounds per cubic foot. While the slope was steepened to 1:1 and was subjected to vibrations of nearby pile driving operations, the electro-osmosis was successful in stabilizing the slope.

Electro-osmosis produces an increase in the shear strengths of fine-grained soils by inducing the migration of water from the soil pore spaces under the application of an electrical current between electrodes driven into the soil. By removal of water accumulated at the cathode, the moisture content of the soil can be appreciably reduced, with a corresponding increase in shear strength. However, it should be noted that for long-term stabilization of landslides, the electric current required over a long period of time may render this method prohibitive (Root, 1958).

Stabilizing Additives and Chemical Treatment

While chemical treatment may be effectively used to increase the shear resistance of unstable materials, a review of the literature indicates that chemical stabilization is very infrequently applied to landslide stabilization. Smith and Peck (1955) report the American railroad use of pressure grouting with a cement slurry as a preventive measure against slope failures. In these cases the cement slurry was injected into water pockets and open cracks in embankment sections and success was reported in stabilizing areas of predominately fine-grained, plastic soils which had previously been plagued with slide problems. Two factors were found to have increased the stability of the sections: (1) The void spaces were filled, and therefore water was kept out of the embankment sections; and (2) the slurry provided added strength to the soil mass as the cement cured. It was also believed that hydration of the cement may have reduced the water content of the fill, further contributing to the increased shear resistance of the embankment. It was reported that a volumetric grout acceptance of 2 to 3 percent of the total fill volume was successful in halting slides. Root (1958) reports that cement grout injections have been used successfully for cementing coarse sands and gravel but that the method is considered less effective with fine-grained soils.

Lime treatment has been used to increase the shear resistance of soil masses. Generally, three types of lime treatment techniques have been used to solve various stability problems: (1) lime-soil compaction, (2) high-pressure in-situ lime slurry injections, and (3) hydrated lime poured into cracks and fissures. However, these methods have been predominately used for subgrade stabilization and not for landslide control or correction (Lundy, 1968).

Hardy (1967) reports a landslide corrected by the injection of a quicklime slurry. Preceding the attempt to stabilize the area with lime injections, drainage systems, concrete piles, and timber bulkheads had all failed to halt the slide. In 1963, a 250 by 75-foot area was stabilized by pouring 20 tons of quicklime into 6-inch-diameter holes on 5-foot centers. Within one year of this application the lime had migrated one foot from the holes, the apparent cohesion had increased from 0.6 psi to 1.4 psi, and the angle of internal friction had increased from 17 to 21 degrees. The factor of safety for the slide increased from 0.9 to 2.6, and the slide movements in the stabilized area were arrested while areas adjacent to the treated section have continued to slide. Hardy (1970) reported that the treatment has remained effective six years after the original application.

Thermal Treatment

Thermal treatment has proven an effective means of stabilizing slides in embankment and cut sections, particularly in highly plastic clays where the method has been successfully used to increase the shear strength and permeability of the soil. However, in addition to application to highly plastic clays, thermal treatment has been used to decrease the sensitivity of loess to repeated soaking. Most of the results of experimental work and field applications of thermal stabilization have been published by Russian engineers. Hill (1934) reported the only data which were found on thermal treatment in the United States.

Hill (1934) reported the successful stabilization of a landslide along the Pacific Palisades near Santa Monica, California (see Chapter 3). The system employed to correct the situation was an extensive series of interconnected drainage tunnels through which air, heated by a natural gas furnace, dried the material in the vicinity of the failure plane. The increase in shear strength of the soil provided by this measure was sufficient to stabilize the slope against further movements. Since that time other references have appeared in the literature suggesting thermal treatment as a means of improving the stability of earth masses (Belles et al, 1958; Litvinov et al, 1961).

Laboratory investigations on thermal treatment of soils indicate that the plasticity index of high-plasticity clays is significantly reduced at treatment temperatures higher than 400° C (Belles et al, 1958). This decrease in plasticity results in a comparable reduction in creep deformations. Litvinov et al (1961) report an increase in total shear strength of approximately 100 percent when loess soils were subjected to in-situ thermal treatment in the temperature range of 300 to 500° C. Rao and Wodhaven (1953) report an increase in permeability following thermal treatment. Other authors report a substantial decrease in compressibility of soils composed of the clay minerals (Salas et al, 1955).

Litvinov et al (1961) report that the burning of liquid or gaseous fuels in airtight holes has proven the most economical and most effective method of in-situ thermal stabilization. This method has undergone considerable development in Russia and has been used for correcting problems arising from landslides. Litvinov et al also indicate that when using this method it is possible to employ thermo-chemical stabilization, a process whereby special chemical additives are introduced both during and after thermal treatment.

Belles et al (1958) indicate that thermal treatment has resulted in permanently stabilized landslides in both cut and embankment sections. A railroad embankment constructed of highly plastic clay and located in Russia was subjected to thermal treatment. In this instance horizontal borings beneath the track and vertical borings on each side of the embankment were heated to approximately 500° C for 36 hours. The permanent results were a drying of the embankment beneath the track and the release of water accumulated in the subbase through a permeable zone created around the borings by heating. It was reported that this treatment prevented heaving of the track and that other embankment sections in the vicinity of the treated area have continued to heave. In the same manner as that reported above, a large landslide near the Black Sea was also corrected by thermal treatment.

Slope Vegetation

Toms (1948) has suggested that vegetation may be useful as a slide deterrent by providing a protective cover which prevents or slows the process of physical weathering. In this manner, the original strength of the soil may be preserved. In addition, evidence exists that vegetation decreases the natural water content of the soil to a substantial depth, ranging from approximately 8 feet for a dense cover of grass to greater than 15 feet for large bushes and trees (Felt, 1953).

Toms (1948) also states that the use of vegetation as a prevention of landslides is most effective for sandy soils, where the overall stability is primarily a function of stability near the surface. It was indicated that by using a bituminous surface treatment in conjunction with slope planting, an immediate increase in surface stability was achieved and the growth of the slope vegetation was both protected and stimulated.

Moran (1948) concluded that planting grass on clay slopes had little effect on the overall stability and indicated that for clay slopes, planting grass might even loosen the soil and cause softening with depth more rapidly. Moran also states that slope planting is most effective for sandy soils and reports the stabilization of two sand dunes with grass.

Reinforced Earth

"Reinforced earth" is a relatively new process which has been useful in preventing slides associated with highway construction and is gaining acceptance for use in the construction of relatively high earth-retaining structures, particularly on poor foundation materials. The reinforced earth method employs thin, galvanized steel strips placed at selected intervals within a compacted earth fill. The ends of each of these strips are restrained at the faces of the earth fill by attachment to semi-cylindrical galvanized steel anchor plates, the objective being to provide resistance to lateral movements in the fill through the tensile resistance and lateral confinement afforded by the anchored steel strips. This method has been in common use since 1965 in France, Africa, and Canada, with the French-Italian Highway successfully employing ten reinforced earth structures as slide prevention measures. No failures have been reported.

Slope Freezing

Freezing of soil to prevent sliding during construction is a method which has been used on at least one large project. Gordon (1937) describes the use of freezing to stabilize a slide which halted the construction of Grand Coulee Dam. In this example, salt brine was cooled to 25° F by large refrigeration units and the cooled brine was pumped through pipes at the toe of the slope. In this manner the landslide was "frozen" and construction could proceed.

Although successful, the freezing process is slow, relatively costly, and limited to a temporary treatment for landslide control. Freezing is more commonly used to maintain open cuts during construction.

CHAPTER 8. STABILITY CHARTS FOR SLOPE FLATTENING AND BENCHING IN SLOPES OF HOMOGENEOUS SOILS

Introduction

The effect of remedial excavation on the stability of a slope may be determined through appropriate stability analysis. The analyses described in the following sections were performed to investigate the influence of slope flattening and benching upon slope stability and to develop charts that may be used to determine the increased stability affected by either method.

Review of Previous Work

Hennes (1959) presented a series of stability analysis charts for use in evaluating the influence of remedial excavation on slope stability. In developing these charts, to include the effects of slope benching and flattening, Hennes relied on previous chart solutions presented by Janbu (1954); however, after reviewing this work, two sources of potentially large inaccuracies were recognized. First, the analyses presented by Janbu were based on the Ordinary Method of Slices analysis procedure, a procedure which is now recognized as being relatively inaccurate for slope stability analysis. Further, in order to utilize Janbu's charts for analysis of the influence of benching, it is necessary to employ several simplifying assumptions, the effects of which are not known.

In order to eliminate some of the uncertainty and inaccuracy involved in the use of the existing charts, a series of analyses were performed using more accurate and representative analysis procedures. These analyses are explained and stability charts are presented for slope flattening and benching in the remainder of this chapter.

Slope Flattening

In order to determine the influence of slope flattening with respect to increasing the factor of safety for a given slope, and to develop charts useful for this purpose, a series of analyses were performed utilizing a relatively accurate stability analysis procedure based on the assumption of a log spiral shear surface. This procedure has been found to yield results which are comparable to several of the other accurate analysis procedures available, including those employing the assumption of a circular shear surface, and was selected on the basis of readily available solutions which were useful in studying the present problem.

In developing simple slope stability charts, it is convenient to utilize the dimensionless parameter, $\lambda_{c\phi}$, which was introduced by Janbu (1954) and expressed in the form

$$\lambda_{c\phi} = \frac{\gamma \cdot H \cdot \tan \phi}{c} \tag{8.1}$$

where

γ = the unit weight of soil,

H = the slope height,

ϕ = the angle of internal friction of the soil,

c = the cohesion intercept on the Mohr-Coulomb strength invelope.

It may be shown that for a given slope inclination, β , and value of $\lambda_{c\phi}$, the factor of safety may be expressed in the form

$$F = N_{cf} \cdot \frac{c}{\gamma H} \tag{8.2}$$

where N_{cf} is a dimensionless stability number and depends only on the values of $\lambda_{c\phi}$ and β , regardless of the particular values of c , ϕ , γ , and H . Thus, the influence of changing the slope inclination, β , by flattening the slope can be represented in terms of changes in the value of N_{cf} , the change in the value of N_{cf} being directly proportional to the change in the factor of safety for the slope.

The influence of slope flattening was investigated for slope inclinations ranging from 1:1 to 5:1. For each of these slopes the values of N_{cf} were calculated for values of $\lambda_{c\phi}$ in the range from zero to 100, assuming that the most critical failure surface (spiral) passes through the toe of the slope. The results of these analyses were then used to calculate the ratios between the values of N'_{cf}, corresponding to the flattened slopes, and the values of N_{cf} for the original slopes, each ratio corresponding to a particular value of $\lambda_{c\phi}$. These ratios, which represent the ratios between the new factor of safety with the slope flattened and the original factor of safety, are presented in chart form in Figs 8.1 through 8.3, each chart representing a given initial slope ratio (cot β).

In using the presented charts to evaluate the influence of slope flattening, the value of $\lambda_{c\phi}$ must be known. The value of $\lambda_{c\phi}$ may be either calculated using Eq 6.1 or determined by back-calculation from actual slope failure data using procedures similar to those described by Abrams and Wright (1972). Once the value of $\lambda_{c\phi}$ is known, the appropriate stability chart for a given initial slope may be used in two ways:

(1) if the desired increase in stability (N'_{cf}/N_{cf}) is known, the new slope ratio (cot β') required to attain this increase may be found; or

(2) if the new slope ratio is known or perhaps limited by right-of-way restrictions and excavation costs, the corresponding increase in stability may be determined for the given excavation scheme.

It is interesting to note that the influence of slope flattening is most significant in those cases where the value of $\lambda_{c\phi}$ is relatively high, corresponding, as can be noted from Eq 8.1, to materials having relatively low values of cohesion compared to their frictional resistances. Consequently, slope flattening generally provides the greatest increase in stability for cohesionless materials, where the depths of sliding are usually shallow, while in cohesive soil deposits, where slide movements may be relatively deep, the influence of slope flattening is minimal. This conclusion appears to be in agreement with the literature reviewed in previous sections of this report, and it should be noted that under certain circumstances (limited right-of-way

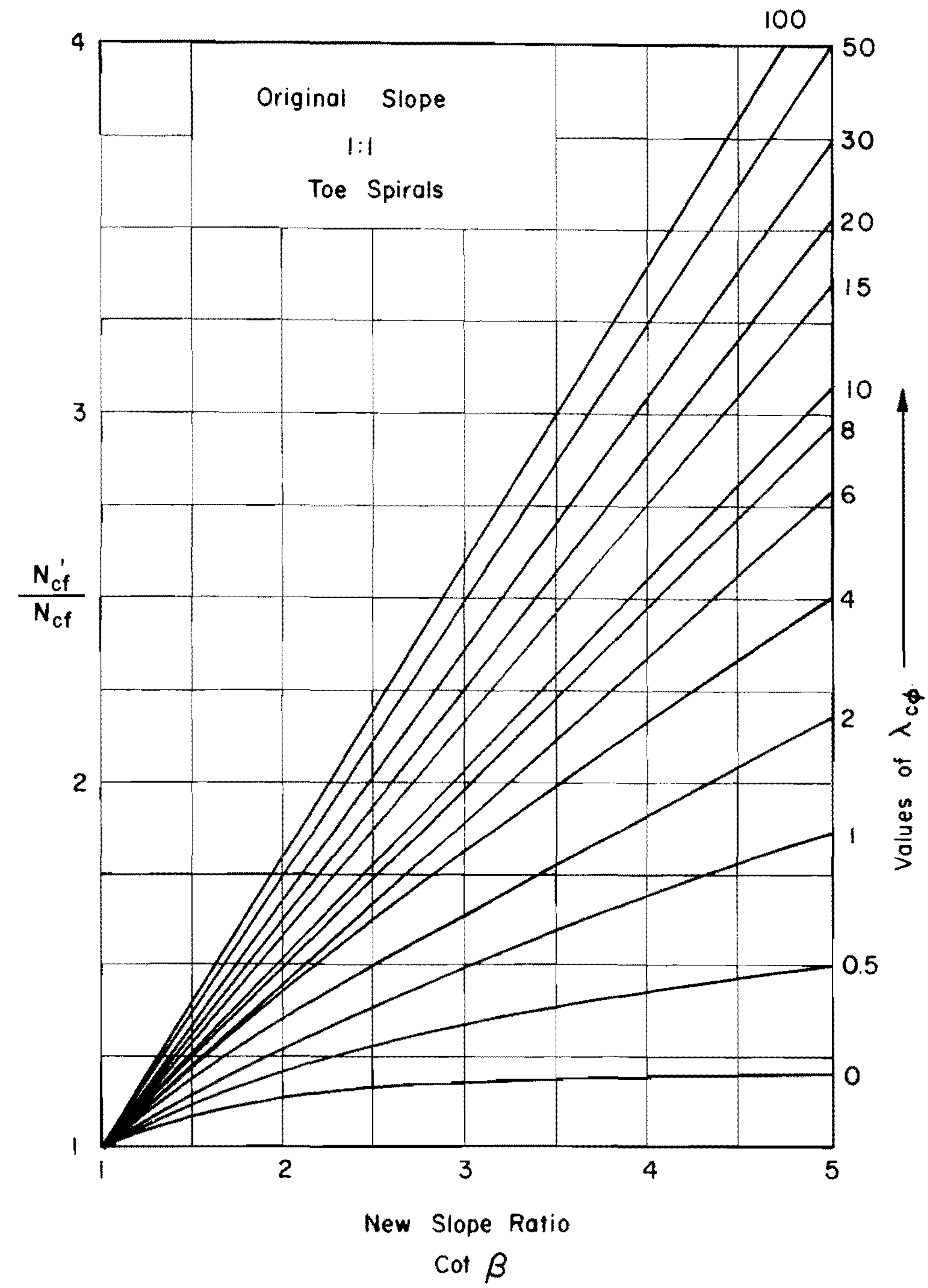


Fig 8.1. Stability chart for evaluating the effectiveness of slope flattening - 1:1 original slope.

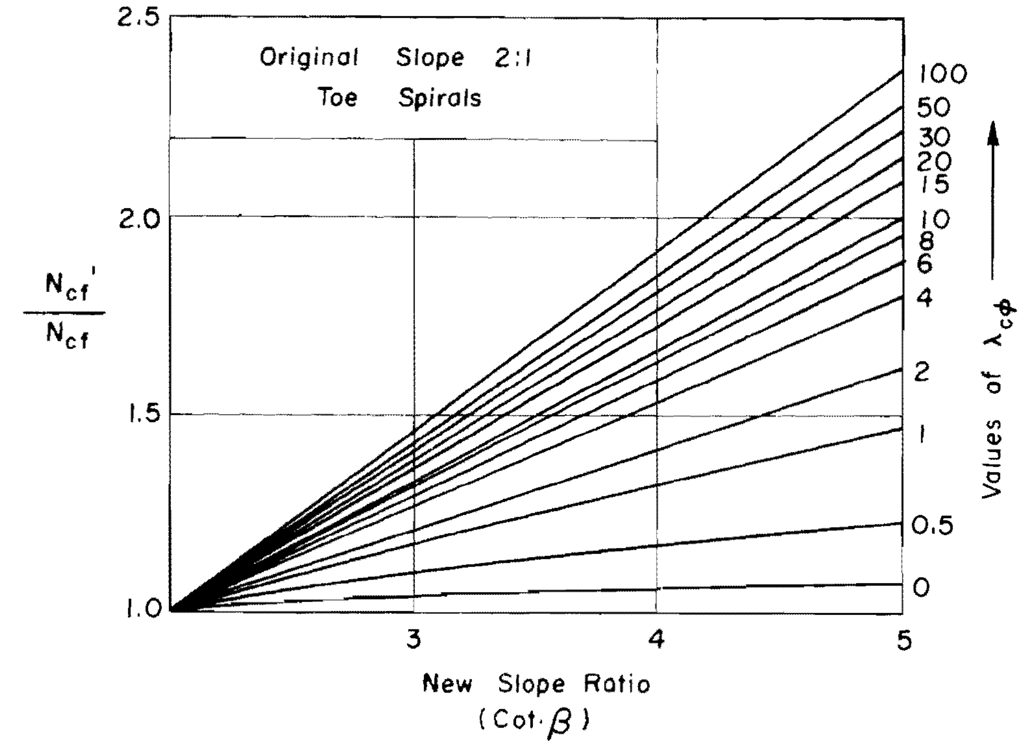


Fig 8.2. Stability chart for evaluating the effectiveness of slope flattening - 2:1 original slope.

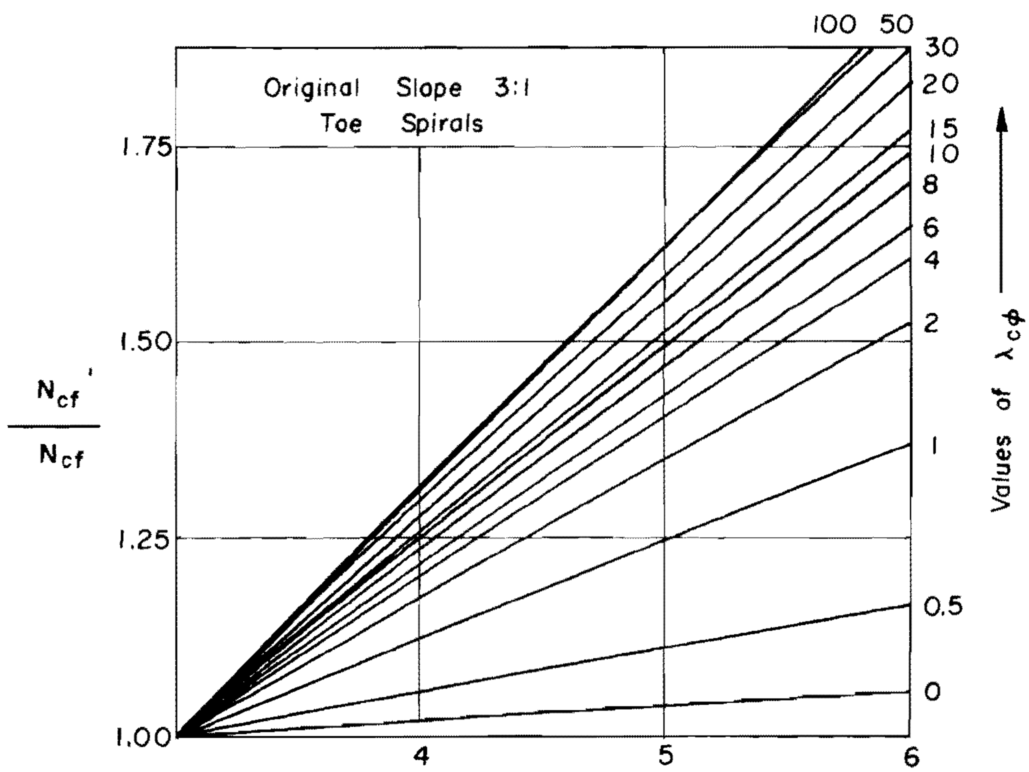


Fig 8.3. Stability chart for evaluating the effectiveness of slope flattening - 3:1 original slope.

or very low values of $\lambda_{c\phi}$) it may prove economically impossible to achieve the desired increase in stability through the use of slope flattening. In these cases, other remedial methods should be explored.

The volume of excavation and the distance that the top of the slope must be moved back to achieve the desired increase in stability may be calculated by considering the geometry involved in slope flattening. Figure 8.4 illustrates this geometry. The distance that the top of the slope must be moved back to attain the new slope ratio required for stability is expressed by Eq 8.3:

$$x = (\cot \beta_2 - \cot \beta_1)H \tag{8.3}$$

The volume of excavation required per lineal foot of slope may then be calculated by

$$\text{Vol} = 1/2\,(x)H \text{ per lineal foot} \tag{8.4}$$

The following example problem illustrates the use of the stability charts for slope flattening. This problem was used by Hennes (1959) and, by using his example problem, a direct comparison between the two procedures is readily available.

Example Problem for Slope Flattening

Given: initial slope, $\cot \beta = 1$,

$c = 600$ psf,

$\phi = 22^{\circ}$,

$\gamma = 120$ pounds per cubic foot,

$H = 60$ feet.

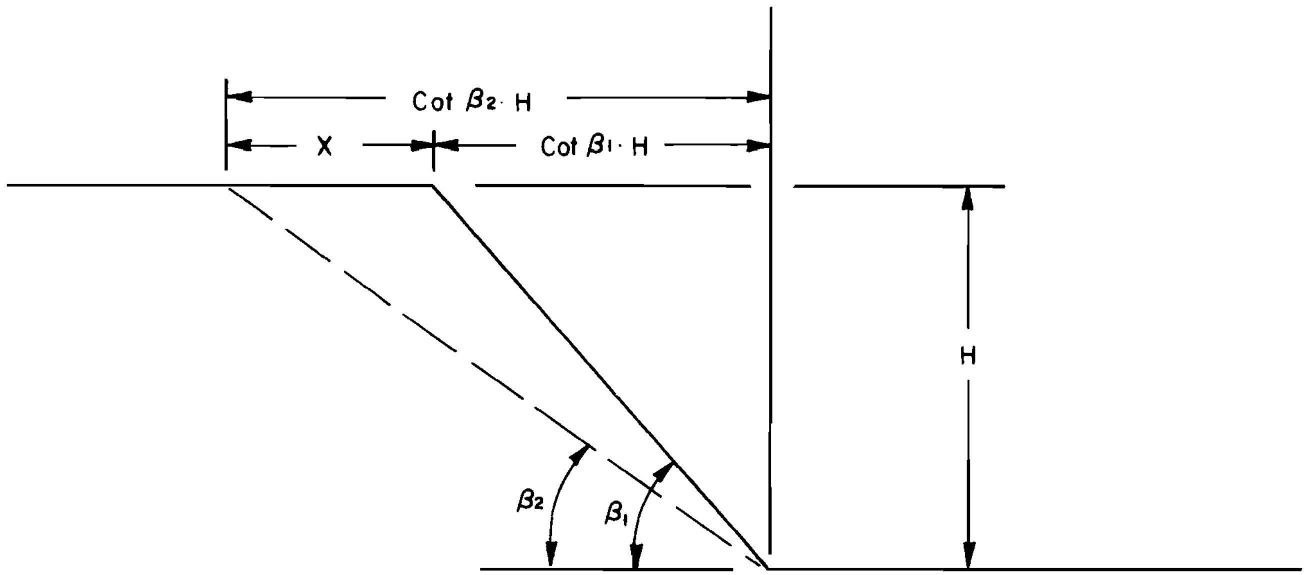


Fig 8.4. Geometry of original and flattened slope.

Problem: Determine the slope flattening required to increase the factor of safety of the original slope to 1.5.

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

$$\lambda_{c\phi} = \frac{\gamma H \tan \phi}{c} = \frac{120(60)(.404)}{600} = 4.85$$

With $\lambda_{c\phi}$ and the original slope ratio known, it is possible to determine the stability number of the original slope by using any one of several methods. For this case, the stability number (N_{cf}) is 14.6 . The factor of safety for the original slope may then be determined by Eq 8.2:

$$F = N_{cf} \frac{c}{\gamma H} = 14.6 \frac{600}{7200} = 1.21$$

The required increase in stability number may then be calculated:

$$\frac{N_{cf}}{N_{cf}} = \frac{F}{F} = \frac{1.50}{1.21} = 1.24$$

By entering Fig 8.1 with the ratio $\frac{N_{cf}}{N_{cf}}$ and the value of $\lambda_{c\phi}$, the new slope ratio may be determined. From Fig 8.1, the new slope ratio (cot β_2) equals 1.55. The distance that the top of the slope must be moved back is calculated from Eq 8.3:

$$\begin{aligned} x &= (1.55 - 1.0)(60) \\ &= 33 \text{ feet} \end{aligned}$$

Equation 8.4 is used to determine the volume of excavation required per lineal foot of slope:

$$\text{Vol} = 1/2\ (33)\ (60) = 990 \text{ cubic feet per lineal foot}$$

The data obtained using this example problem and the stability charts developed by Hennes (1959) yield cot $\beta_2 = 1.68$, a volume of excavation of 1220 cubic feet per lineal foot of slope, and 41 feet as the distance that the top of the slope must be moved back to achieve the desired increase in stability.

Calculation of the factor of safety by use of the log spiral method and the slope ratio recommended by Hennes (cot $\beta_2 = 1.68$) yields a value of $FS = 1.65$. This value represents a factor of safety 10 percent greater than that required by the problem statement. The total volume of excavation obtained using the charts developed in this chapter is approximately 20 percent less than that obtained by Hennes (1959). This difference may be attributed to the Ordinary Method of Slices analysis procedure and the simplified assumptions required for use of Jambu's charts by Hennes in the development of his charts. For all except $\phi = 0$ materials, the charts presented by Hennes (1959) will produce conservative factors of safety (actual factors of safety will be greater than those calculated).

Slope Benching

The stability charts presented in this section are used to evaluate the increase in stability of homogeneous slopes due to benches of varying widths and depths. A variety of slopes were analyzed with cot β varying from 1 to 5 and $\lambda_{c\phi}$ values ranging from 0 to 100. Upon preliminary analysis of the data, it became apparent that benching proves most effective for slopes with a $\lambda_{c\phi}$ value between 1 and 8. Values higher than $\lambda_{c\phi} = 8$ showed a gradual decrease in percent stability gain when the dimensions of the bench were held constant. Figure 8.5 illustrates the rate of increase in the stability number as a function of $\lambda_{c\phi}$ for a typical bench and slopes of 1:1 and 2:1. The slope angle of the original slope also determines to what extent benching will increase the stability. As shown in Fig 8.6, for a bench of typical dimensions and given excavation quantity, the effect of benching decreases as the slope ratio (cot β) increases. Based on the results of Figs 8.5 and 8.6 as well as similar results for additional studies it was decided to restrict development of subsequent charts to 1:1 and 2:1 slopes and $\lambda_{c\phi}$ values ranging from 1 to 8.

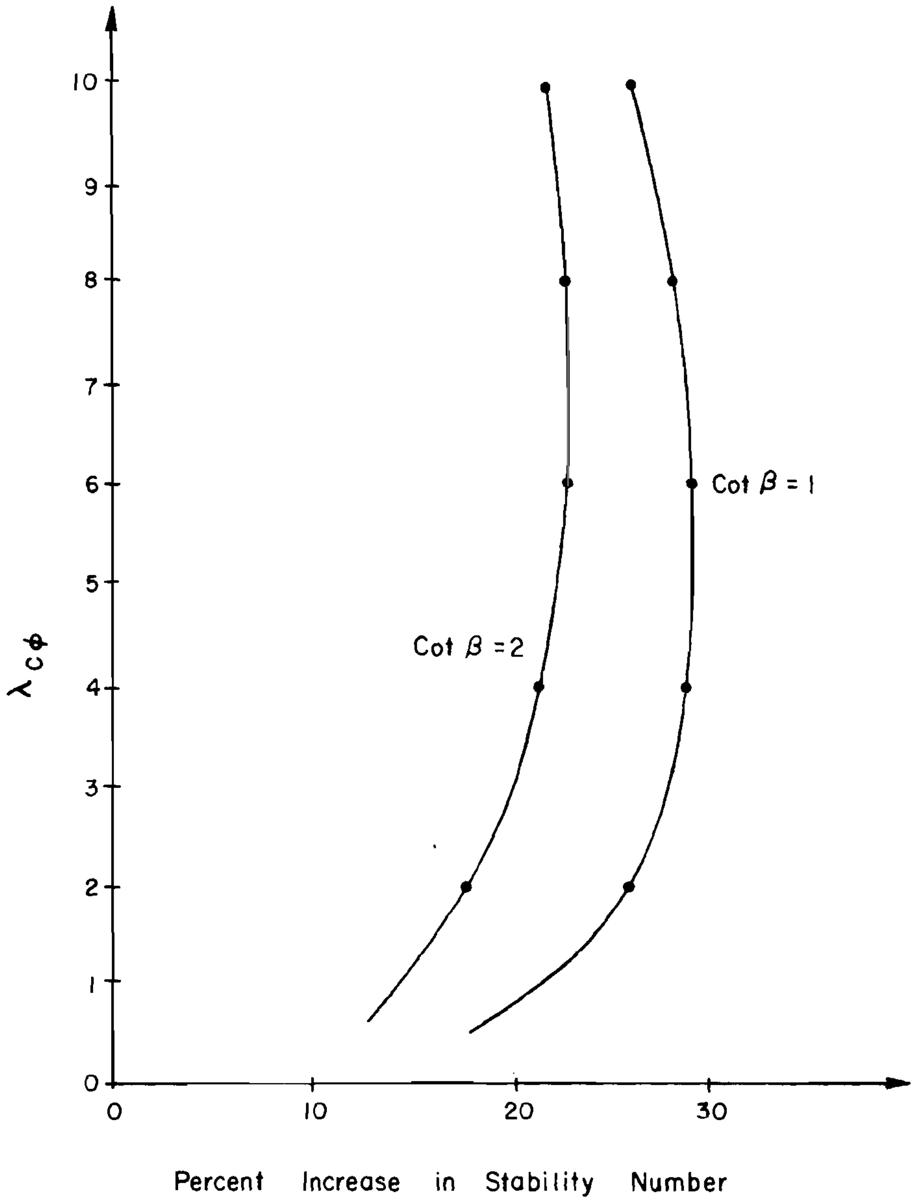


Fig 8.5. Effectiveness of typical bench as a function of $\lambda_{c\phi}$.

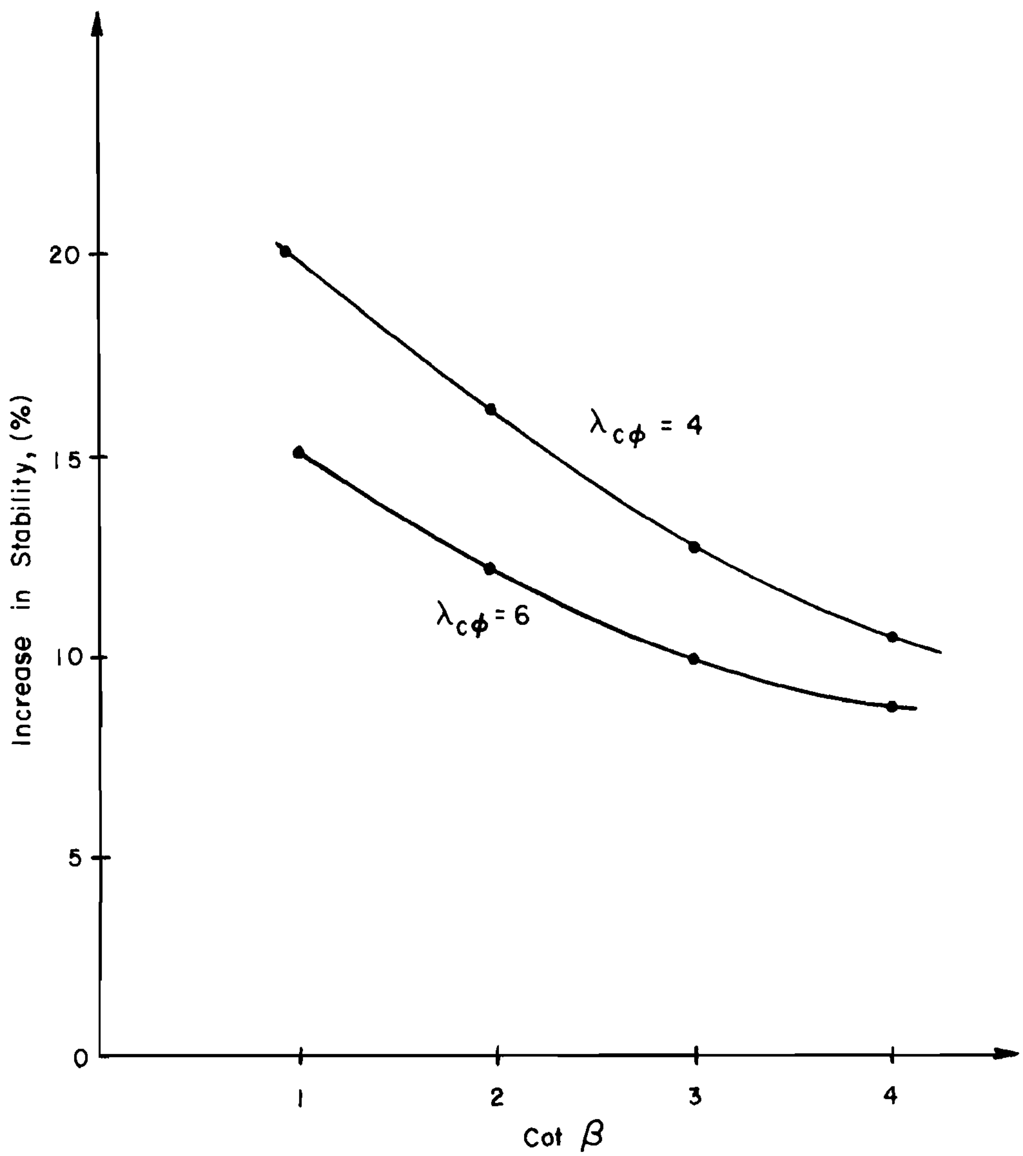


Fig 8.6. Effectiveness of a typical bench as a function of slope ratio (cot β).

To present the data in the form of stability charts, width factors (N_w) and height factors (N_h) were used to express the dimensions of the bench. These factors are defined in the following equations:

$$\frac{\text{height of bench}}{\text{slope height}} = N_h \tag{8.5}$$

$$\frac{\text{width of bench}}{\text{slope height}} = N_w \tag{8.6}$$

The area of the bench and therefore the volume of excavation per lineal foot of slope is determined by an area factor such that

$$N_a = N_w \cdot N_h \tag{8.7}$$

The volume of excavation per lineal foot of slope can therefore be computed by

$$\text{Vol} = N_a \cdot H^2 \tag{8.8}$$

Because of the complex slope geometry involved with benched slopes, it was necessary to utilize a computer program to locate and analyze the most critical failure surface. A computer program, SSTAB1, which was available at The University of Texas at Austin, was used for this purpose. Slopes were analyzed with height factors (N_h) of 0.1, 0.2, 0.3, 0.4, and 0.5 and width factors (N_w) of 0.25, 0.50, and 0.75. These factors correspond to dimensions of typical benches reported in the literature.

The method used in the development of stability charts for benched slopes required the solution of two separate problems. It was determined that for a particular height factor (N_h), $\lambda_{c\phi}$, and cot β , there existed a minimum width factor (N_w) where the factor of safety for the lower portion of the slope (Case II) is identical to the factor of safety for the entire slope (Case I). Case I and Case II critical circles are illustrated in Fig 8.7. For this minimum bench width, the height of the slope is effectively reduced, and any bench wider than the minimum value appears to be uneconomical because no further increase in stability is achieved.

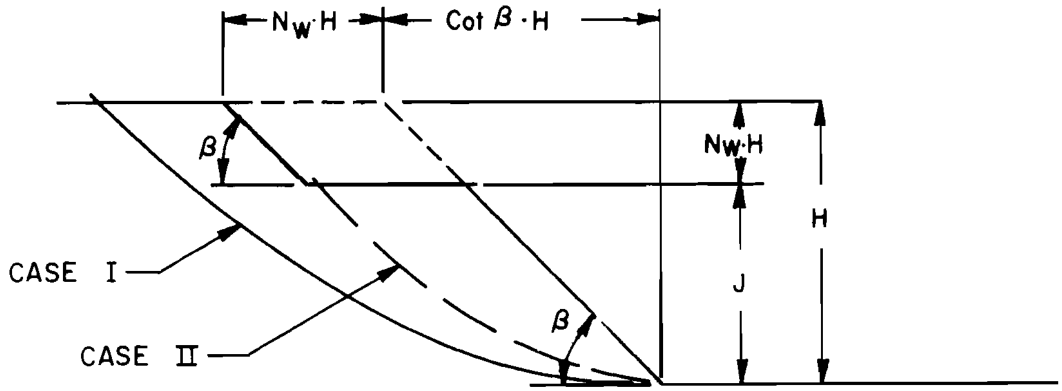


Fig 8.7. Critical circles with same factor of safety.

The first step in the solution was to obtain the stability number corresponding to Case I as a function of bench width for a particular $\lambda_{c\phi}$, $\cot \beta$, and height factor. An example of this curve is shown in Fig 8.8a for a slope where $\cot \beta = 1$, $\lambda_{c\phi} = 4$, and $N_h = 0.30$. Following this, a stability number for the portion of the slope beneath the bench was determined. By plotting the stability number for the lower portion of the slope (Case II) on the same graph with the stability number for the entire slope (Case I), the optimum bench width factor and corresponding N_{cf} value were obtained as shown in Fig 8.8b.

The procedure illustrated in Fig 8.8 was repeated for slope ratios of 1:1 and 2:1, values of $\lambda_{c\phi}$ from 0.5 to 8.0, and height factors ranging from 0.1 to 0.5. In this manner curves of stability number versus area factor, N_a, could be developed as a function of $\lambda_{c\phi}$ and N_h. The completed charts for determining the stability of benched slopes are shown in Figs 8.9 and 8.10.

The following example problem is used to illustrate the manner in which the stability charts for benched slopes are used. The slope geometry and data for the slope flattening example problem is used for this problem.

Example Problem for Benching

Given: initial slope, $\cot \beta = 1$,

$c = 600$ psf,

$\phi = 22^{\circ}$,

$\gamma = 120$ pounds per cubic foot,

$H = 60$ feet.

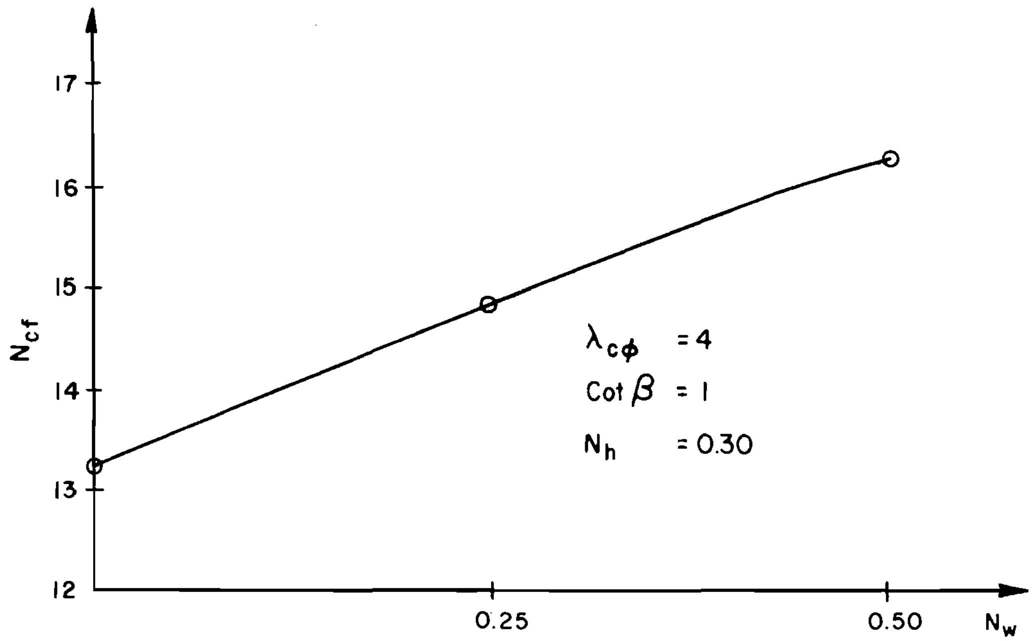


Fig 8.8a. Variation in stability number (N_{cf}) with bench width - Case I critical circle.

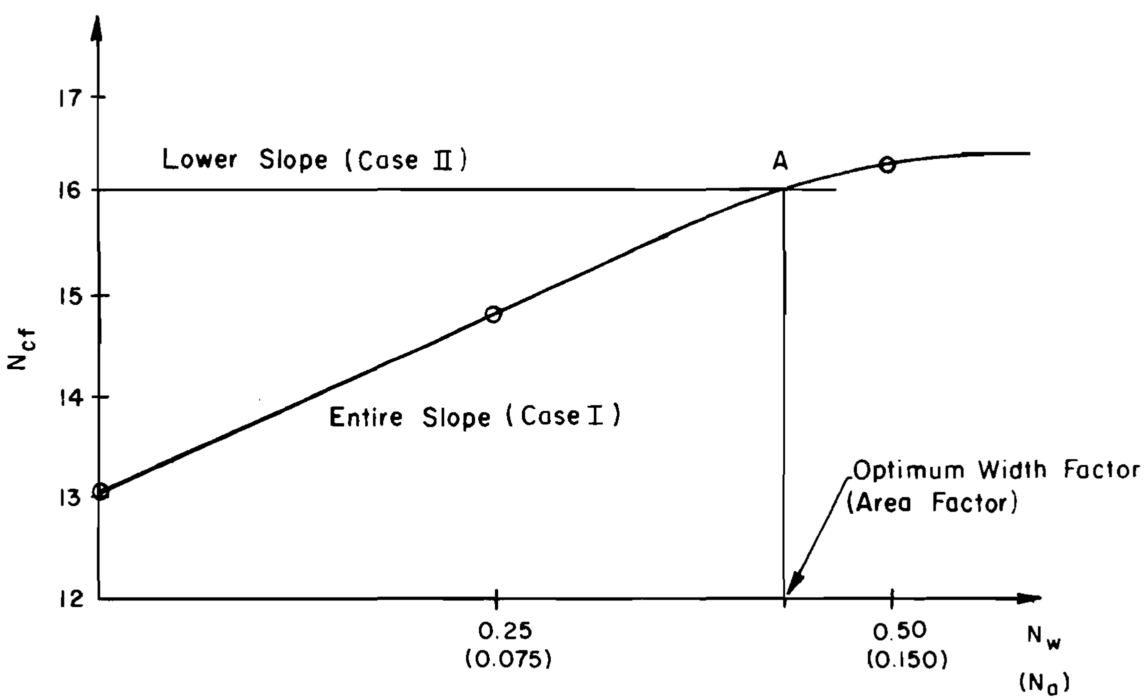


Fig 8.8b. Determination of optimum bench width factor and corresponding stability number.

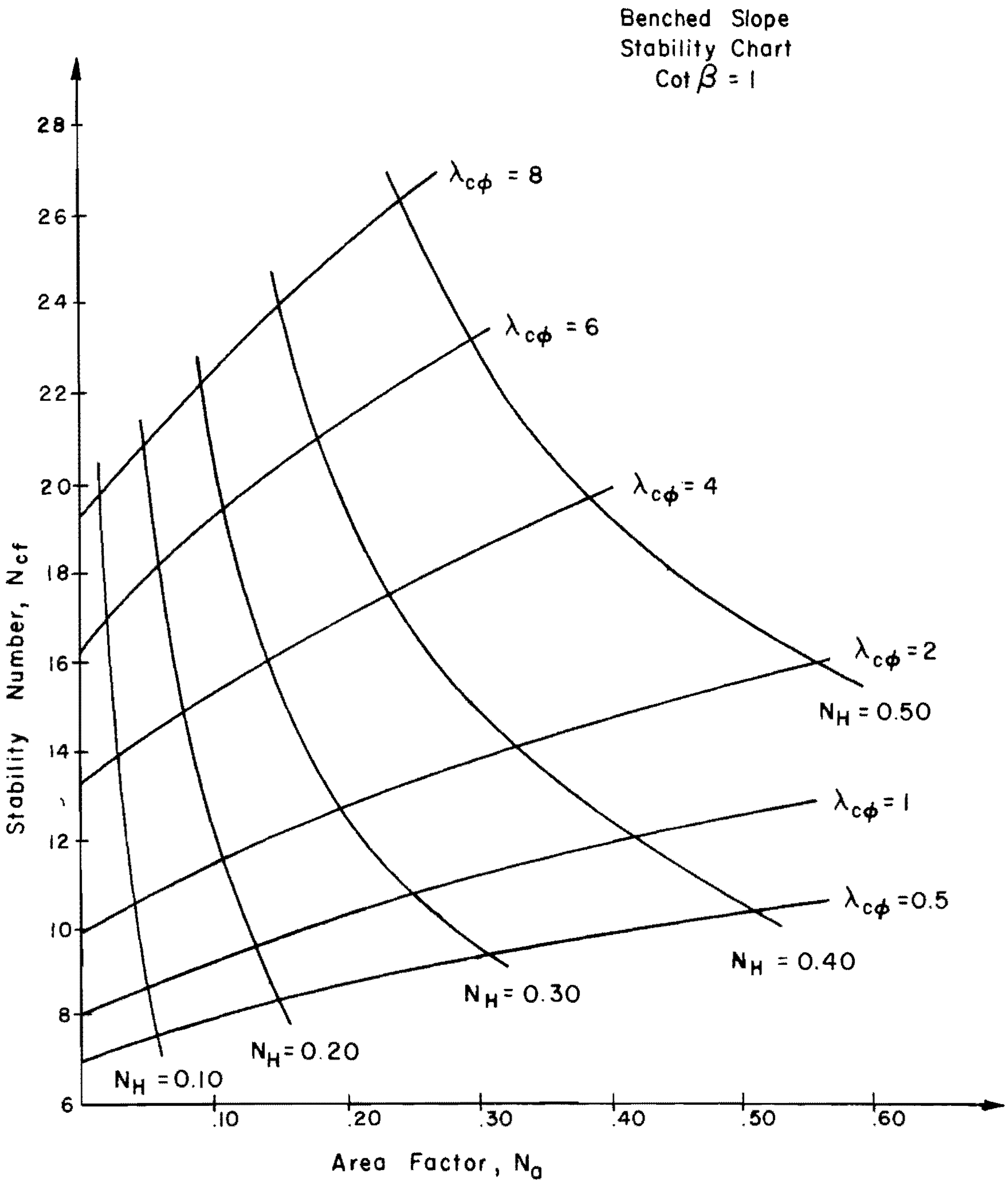


Fig 8.9. Stability chart for determining optimum bench sizes and corresponding stability numbers - 1:1 slope.

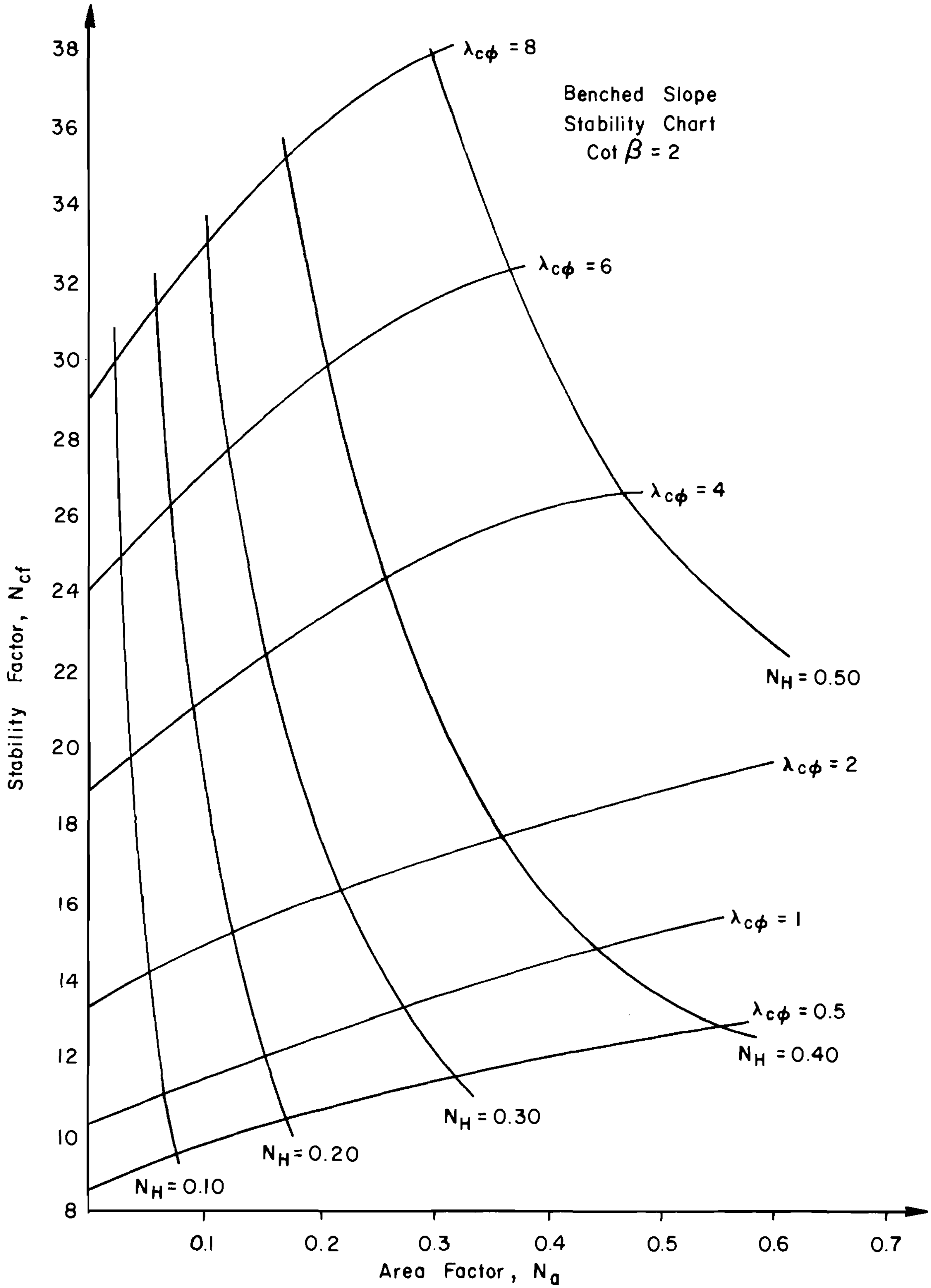


Fig 8.10. Stability chart for determining optimum bench sizes and corresponding stability numbers - 2:1 slope.

Problem: Determine the optimum bench dimensions and required volume of excavation to increase the factor of safety of this slope to 1.5.

From the slope flattening example problem

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

The stability number required to increase the factor of safety of this slope to 1.5 may be determined from Eq 8.2:

$$N_{cf} \text{ (required)} = F \frac{\gamma H}{C}$$

$$N_{cf} \text{ (required)} = 1.5 \frac{(120)(60)}{600} = 18$$

By entering Fig 8.9 with the known $\lambda_{c\phi}$ value and the stability number (N_{cf}) required to increase the factor of safety, it is possible to determine the optimum bench dimensions. By linearly interpolating between $\lambda_{c\phi} = 4$ and $\lambda_{c\phi} = 6$, the height factor (N_h) and area factor (N_a) may be read directly. From Fig 8.9

$$N_h = 0.32$$

$$N_a = 0.13$$

With these values known, the width factor N_w may be calculated as follows:

$$N_w = \frac{N_a}{N_h} = \frac{0.13}{0.32} = 0.40$$

With these factors known the optimum dimensions and volume of excavation of the bench may be calculated:

$$\text{Depth of bench} = N_h \times H = 0.32\,(60) = 19 \text{ feet}$$

Width of bench $= N_w \times H = 0.40\,(60) = 24$ feet

Volume of excavation $= N_a \times H^2 = 0.13\,(60)^2$

$= 467\ ft^3$/lineal foot of slope

The values obtained by Hennes (1959), as given in the slope flattening problem, are 20 percent higher than indicated by this analysis. For this problem and his charts

Depth of bench = 22 feet

Width of bench = 25 feet

Volume of excavation = 575 ft^3/lineal foot of slope

As a check the computer program SSTAB1 was used to calculate the factor of safety for this slope with the bench dimensions calculated using the analysis recommended in this chapter. The calculated minimum factor of safety was 1.51. This value is in excellent agreement with the factor of safety obtained using the stability charts for benched slopes developed in this section and should be, inasmuch as no approximations were introduced by expressing the results in chart form.

By comparison, the factor of safety calculated using the bench width and height recommended by Hennes (1959) resulted in a factor of safety of 1.53. Although the factor of safety is only slightly higher it should be noted that the total volume of excavation required using Hennes charts is 20 percent greater. At this point, it is interesting to note that the analysis for benched slopes and for slope reduction are not conservative by the same amount. This seems to indicate that by considering the earth removed in a benched slope as an upward force on the slope, an error is introduced in the analysis, this error being the difference in the respective factors of safety derived using Hennes' analysis, approximately 10 percent.

To further aid the engineer in understanding the effects of benching upon the stability of a slope, Figs 8.11 and 8.12 were prepared. These charts

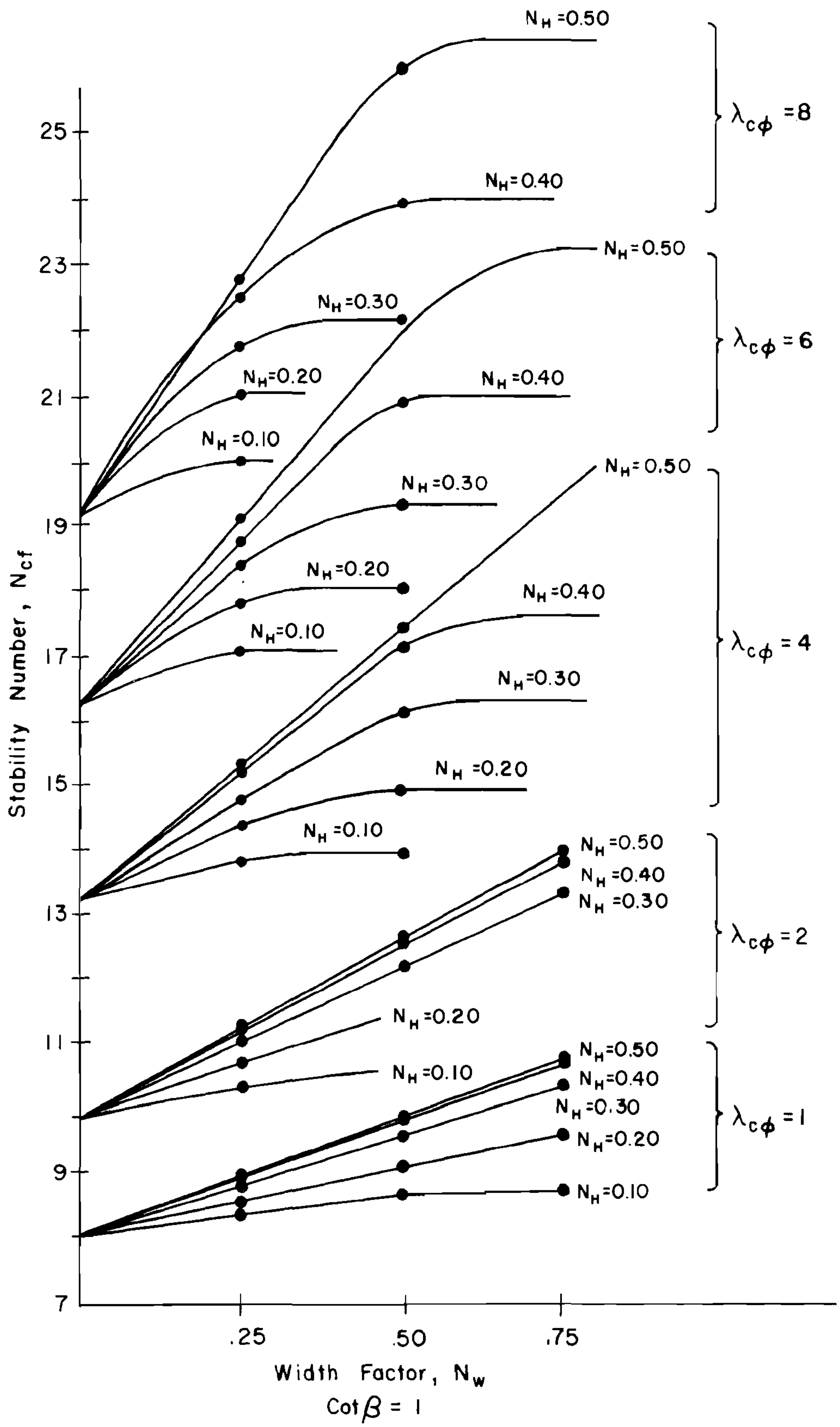


Fig 8.11. Stability numbers for selected bench dimensions 1:1 slope.

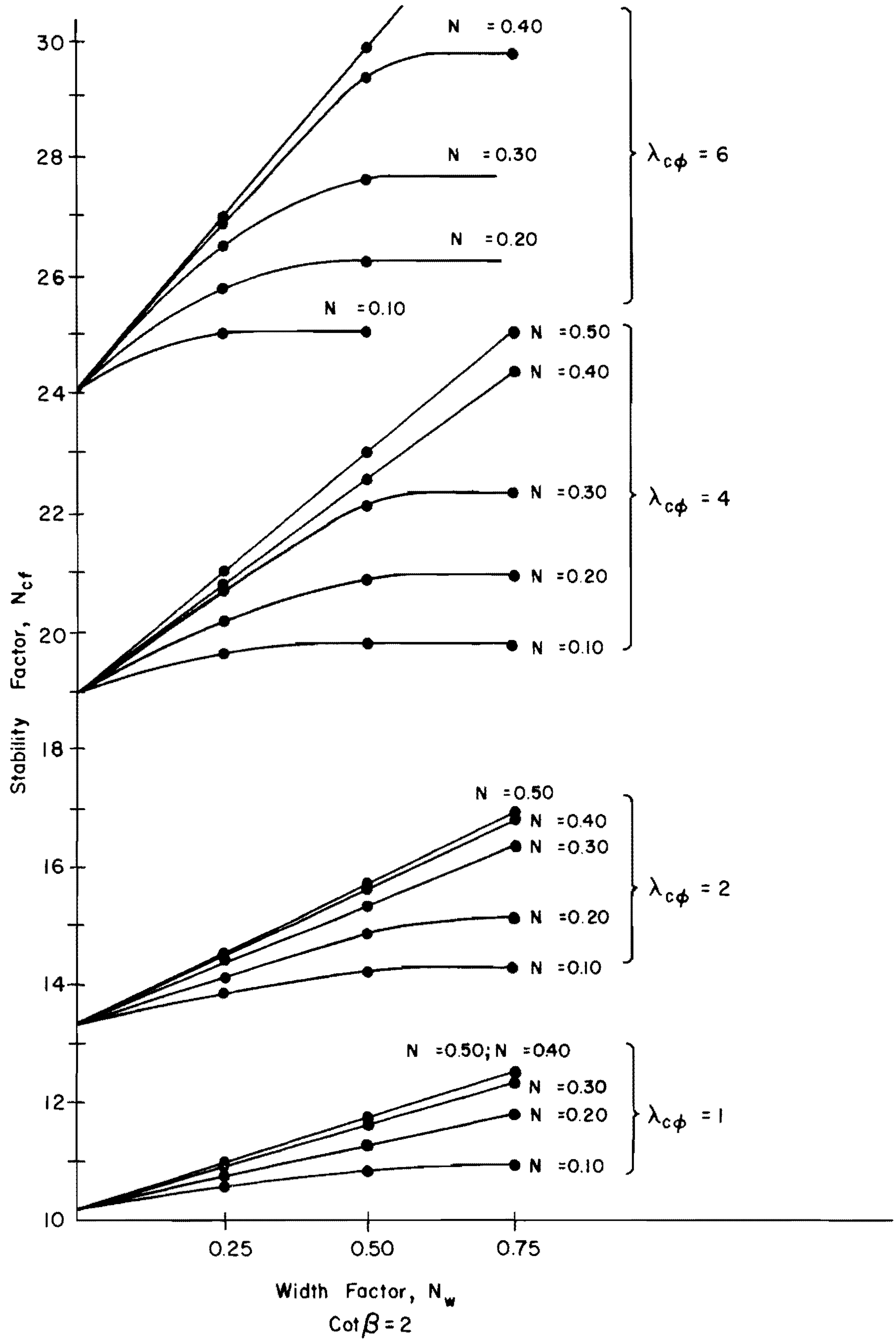


Fig 8.12. Stability numbers for selected bench dimensions - 2:1 slope.

illustrate the increase in stability produced by a given height and width of bench as a function of $\lambda_{c\phi}$. With $\lambda_{c\phi}$ and the stability number (N_{cf}) required to produce a given factor of safety known, the engineer may directly select a height and width factor that will produce these results. It should be noted that the points on these curves do not necessarily illustrate the most economical dimensions for the bench. However, if the width of the bench is restricted by right-of-way requirements, a height factor may be obtained such that the desired stability number can be obtained.

Conclusions

The stability charts presented in this chapter represent a method by which proper slope ratios and dimensions of benched slopes can be achieved. It has been shown that these charts are more accurate than those previously presented by Hennes (1959). It has been shown that slope flattening is most effective for slopes with high $\lambda_{c\phi}$ values, while benching is more effective for lower values of $\lambda_{c\phi}$. The graphs presented in this chapter should aid the engineer in determining whether benching or slope flattening presents an economical solution to his stability problem.

CHAPTER 9. SUMMARY AND CONCLUSIONS

A survey of the remedial measures employed for earth slope failures, the soil and groundwater conditions at the site, and the performance of the remedial measures has been presented. The remedial measures reported in the literature included:

(1) drainage, consisting of
   - (a) surface water control,
   - (b) horizontal drains,
   - (c) vertical drains and well systems,
   - (d) stripping of unsuitable soils and backfilling with a select free-draining material,
   - (e) transverse and longitudinal drainage trenches, and
   - (f) tunnels;

(2) restraint structures, consisting of piles, piers and retaining walls;

(3) elimination and avoidance of the slide area by excavation or relocation;

(4) benching and slope flattening by regrading; and

(5) special procedures, including
   - (a) electro-osmotic stabilization,
   - (b) addition of stabilizing additives and chemical treatment,
   - (c) thermal treatment,
   - (d) slope planting,
   - (e) use of reinforced earth, a patented process, and
   - (f) freezing.

The review of remedial measures has shown that a number of remedial measures have been used and, depending on the site conditions, all have enjoyed some degree of success. However, the success of any remedial measure is dependent upon the actual soil and groundwater conditions in the slope and the degree to which these are fully recognized in the selection and design of the remedial measure. Further, the selection of the remedial measure must be governed to a large extent by the economics associated with the consequences of the failure and of future recurrences.

From the review of previous experiences with remedial measures it appears that substantial costs are often incurred in stabilizing earth slope failures. Thus, when earth slope failures occur or if inadequate steps are initially taken to insure against failures anticipated after construction, it is important to have appropriate procedures for selection and design of remedial measures. The information presented in this report should be useful in establishing preliminary selection of remedial measures for slide stabilization. However, this study does not provide all necessary tools for final selection and design.

The final selection and design of a remedial measure currently seems to be accomplished in either one of two ways. The first approach is based on previous experience in a particular area and use of empirical guidelines. This approach generally involves using procedures which have been previously tried and found successful. Such an approach is generally restricted to areas where a considerable amount of experience is available or else a high degree of uncertainty must be assigned to the remedial measure employed. No quantitative information is available from this approach and the degree to which remedial measures may be overdesigned is not easily established. The empirical approach has in many instances been successful but in others has not. If such an approach is followed by the Texas Highway Department, the information provided herein and in the report by Abrams and Wright (1972) should aid the designer in his judgement.

The second approach to final selection and design of remedial measures generally involves a thorough site investigation, which includes soil borings, laboratory undrained triaxial or drained triaxial and direct shear tests (depending on the slope conditions at failure), and, finally, an appropriate series of stability analyses. The stability charts presented in Chapter 8 of this report should be useful in evaluating the stability of benched or flattened slopes and are intended for use with back-calculated shear strength values. However, for a large number of slope failures and remedial measures, a more thorough evaluation of the soil profile and properties, and a detailed stability analysis performed with the aid of a computer will be required to obtain a realistic and reliable solution. Without such an effort, the effectiveness of many remedial measures is at best uncertain.

REFERENCES

1. Abrams, Timothy G., and Stephen G. Wright, "A Survey of Earth Slope Failures and Remedial Measures in Texas," Research Report No. 161-1, Center for Highway Research, The University of Texas at Austin, December 1972.

2. Allen, G. H., "Old Slide Compels Radical Remedy," Engineering News Record, August 17, 1939, pp 70-71.

3. Allen, G. H., "Steel Baffles Stop Sliding Hill," Engineering News Record, July 1, 1937, pp 32-33.

4. Allen, G. H., "Arresting a Landslide on an Indiana Highway," Civil Engineering, July 1937, p 385.

5. Andrews, G. H., et al, "Cylinder Pile Retaining Walls," ASCE Structural Engineering Conference, Preprint 295, 1966.

6. Baker, R. F., "Analysis of Corrective Actions for Highway Landslides," Proceedings, Vol 79, No. 190, American Society of Civil Engineers, May 1953, pp 1-25.

7. Baker, R. F., "Determining Corrective Action for Highway Landslides," Bulletin 49, Highway Research Board, 1952, pp 1-27.

8. Baker, R. F., "Regional Concept of Landslide Occurrence," Bulletin 216, Highway Research Board, 1959, pp 17-30.

9. Baker, Robert F., and Harry E. Marshall, "Landslides and Engineering Practice," Special Report 29, Highway Research Board, 1958, pp 150-188.

10. Balter, R. B., "Soil and Foundation Investigation on the Patapso Tunnel Project," Bulletin 236, Highway Research Board, 1960, pp 17-39.

11. Belles, A. A., et al, "Thermal Treatment as a Means of Improving the Stability of an Earth Mass," Geotechnique, Vol 7, December 1958, p 158.

12. Benedict, H. W., "Redwood Freeway," California Highways and Public Works, September 1957, p 52.

13. Bertram, G. E., "An Experimental Investigation of Protective Filters," Publications of the Graduate School of Engineering, Harvard University, No. 267, January 1940.

14. Bertram, G. E., "Giant Spikes Halt Earth Movement in California," Civil Engineering, January 1958, p 96.

15. Bohman, R. A., and A. W. Root, "Special Drills Drain Landslides," Western Construction, Vol 30, No. 8, August 1955, pp 29-32.

16. Brawner, C. O., "Landslide Stabilization Using Horizontal Drains," British Columbia Professional Engineer, October 1966, pp 8-11.

17. Brawner, C. O., "The Landslide Problem in British Columbia Highway Construction," Proceedings of the 13th Canadian Snow and Soil Mechanics Conference, National Research Council of Canada, September 1959.

18. Buckingham, E. M., "Investigation and Correction of Landslides," American Society for Testing Materials, Special Technical Publication No. 322, 1962, pp 135-138.

19. Casagrande, Arthur, "Characteristics of Cohesionless Soils Affecting the Stability of Slopes and Earth Fills," Contribution to Soil Mechanics 1925-40, Boston Society of Civil Engineers, Boston, 1940, pp 257-277.

20. Casagrande, Leo, "Electro-osmosis," Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Vol 1, 1948, pp 218-228.

21. Casagrande, Leo, "Electro-osmosis Stabilization of Soils," Contribution to Soil Mechanics 1941-53, Boston Society of Civil Engineers, Boston, 1953, pp 285-317.

22. Casagrande, Leo, R. W. Laughness, and M. A. J. Matich, "Electro-osmotic Stabilization of a High Slope in Loose Saturated Silt," Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1961, pp 555-561.

23. Cassel, F. L., "Slips in Fissured Clay," Second International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1948, pp 46-50.

24. Cedergren, H. R., "Slope Stabilization with Drainage," Seepage, Drainage, and Flow Nets, John Wiley and Sons, 1967, pp 299-344.

25. Cedergren, H. R., "Seepage Requirements of Filters and Pervious Bases," Journal, Vol 86, No. SM5, Proceedings of the American Society of Civil Engineers, October 1962.

26. Cedergren, H. R., and T. W. Smith, "Cut Slope Design and Landslides," American Society for Testing Materials, Special Technical Publication No. 322, 1962, pp 135-158.

27. Clark, D. D., "A Phenomenal Landslide," Proceedings, Vol 30, American Society of Civil Engineers, March, May, and August 1904.

28. Clark, D. D., "A Phenomenal Landslide - Supplement," Transactions, Vol 82, American Society of Civil Engineers, 1918, p 767.

29. Cleveland, O., "The Railway Slide at Cleveland," Engineering Record, Vol 48, November 14, 1903.

30. Civil Engineering, June 1970, p 67.

31. Collin, Alexandre, "Landslides in Clays," translation by W. R. Schriever, University of Toronto Press, 1956.

32. "Control of Stability of a Sliding Slope in a Railroad Cut Near Wetteren," Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1948, pp 38-42.

33. Cooper, Forrest, "Slide Stabilization by the Horizontal Drain Method," Proceedings, AASHO, 1956, pp 265-270.

34. "Curing Slides with Drainage Tunnels," Roads and Streets, Vol 90, No. 4, April 1947, pp 72-76.

35. Cutler, T. H., "Slipping Roadbed Held Up by Steel Suspenders," Engineering News Record, December 8, 1932, pp 684-685.

36. Defense Documentation Center, "A Report Bibliography," prepared for R. J. Schweizer, Search Control No. 056340, February 17, 1971.

37. Dennis, T. H., and R. J. Allan, "Slide Problem: Storms Do Costly Damage on State Highways Yearly," California Highways and Public Works, July 1941, pp 1-6.

38. "Designing Cuts and Fills to Avoid Costly Slides," Rural Roads, February 1963, p 14.

39. Downs, W. S., "Earth Slip Hazards and Controls in Highway Maintenance," Engineering News Record, May 15, 1930, pp 794-798.

40. Eager, William L., "Drilled Horizontal Drains for Slide Stabilization," Proceedings, 34th WASHO, 1955, pp 123-131.

41. Eager, William L., "Slide Stabilization with Drilled Horizontal Drains," Roads and Streets, Vol 99, No. 2, 1956, pp 87-88.

42. Edwards, Tom, "Slide Stabilization," Proceedings of the Eleventh Northwest Conference on Road Building, pp 111-117.

43. Eide, O., and Bjerrum, "The Slide at Bekkelaget," Geotechnique, Vol 2, No. 1, 1955, p 88.

44. "Failure of a Side Hill Highway Fill," Civil Engineering, May 1963, pp 40-41.

45. Fellenius, Bror, "The Landslide at Guntrop," Geotechnique, Vol 5, No. 1, March 1955, pp 120-125.

46. Felt, E. J., "Influence of Vegetation on Soil Moisture Content and Resulting Soil Volume Changes," Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering, Vol 1, 1953, pp 24-27.

47. Focht, J. A., and R. A. Sullivan, "Two Slides in Overconsolidated Pleistone Clays," Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico, 1969, pp 571-576.

48. Forbes, Hyde, "Landslide Investigation and Correction," Transactions, Vol 120, American Society of Civil Engineers, 1947, pp 377-442.

49. "Frozen Earth Stops Slide at Grand Coulee," Engineering News Record, November 19, 1936.

50. Fukvoka, Masarni, "Landslides in Japan," 31st International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1953, pp 234-238.

51. Gedney, David S., "Rock Mechanics in Highway Engineering," Highway Focus, Vol 1, No. 3, September 1969, pp 1-35.

52. Gordon, Grant, "Freezing Arch Across Toe of East Forebay Slide, Grand Coulee Dam," Reclamation Era, Vol 27, 1937, pp 12-16.

53. Gould, J. P., "Lateral Pressures on Rigid Permanent Structures," Specialty Conference on Lateral Stresses and Design of Earth Retaining Structures, American Society of Civil Engineers, New York, June 1970, pp 219-269.

54. Greeley, D. H., "Prevention of Slides as a Safety Factor," California Highways and Public Works, May 1940, p 13.

55. Grey and Mitchell, "Fundamental Aspects of Electro-osmosis in Soils," Journal, Vol 93, No. SM6, American Society of Civil Engineers, November 1967, pp 209-239.

56. Handy, R. L., et al, "Chemical Stabilization of an Active Landslide," Civil Engineering, August 1967, p 62.

57. Hardy, R. M., et al, "Landslides in Overconsolidated Clays," Engineering Journal, Canada, June 1962, p 81.

58. Hardy, editorial, Civil Engineering, 1967.

59. Hardy, editorial, Civil Engineering, 1970.

60. Harty, V. D., "Slide in Fort Henry Embankment, River Shannon, Ireland," Third International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1953, pp 255-258.

61. Hellesoe, G. F., "Los Gatos - Santa Cruz Highway Slipouts Cost State $64,000," California Highways and Public Works, December 1941, pp 3-7.

62. Henkel, D. J., and A. W. Skempton, "A Landslide at Jackfield, Shropshire in a Heavily Overconsolidated Clay," Geotechnique, Vol 5, No. 2, June 1955, pp 131-137.

63. Henkel, D. J., "Investigations of Two Long-Term Failures in London Clay Slopes in Wood Green and Northolt," Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1957, pp 315-320.

64. Hennes, R. G., "An Appraisal of Measures for Improvement of Slope Stability," Bulletin 216, Highway Research Board, 1959, pp 17-30.

65. Hennes, R. G., "The Control of Landslides," First International Conference on Soil Mechanics and Foundation Engineering, Vol 1, 1936, pp 133-135.

66. Herlinger, E. W., and G. Stafford, "Orinda Slide," California Highways and Public Works, January-February 1952, p 45.

67. Herrmann, H. G., and Wolfskill, "Residual Shear Strength of Weak Shales," Engineering Properties of Nuclear Craters, Report 5, December 1966.

68. Hill, R. A., "Clay Stratum Dried Out to Prevent Landslides," Civil Engineering, Vol 4, 1934, pp 403-407.

69. "Hillside Stability - The Modern Approach," Civil Engineering, June 1960, p 66.

70. Hirsch, A. D., "Stabilizing Earth Slopes Through Installation of Drains with Hydrauger Equipment," WASHO Conference, Reno, Nevada, 1955.

71. Highway Research Board, Landslides in Engineering Practice, Edwin Eckel, editor, Special Report 29, 1958, 232 pp.

72. Holm, O. S., "Stabilization of a Quick Clay Slope," Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1969, pp 625-627.

73. Holtz, W. G., "Investigation and Solution of a Landslide Problem Involving a High Transmission Tower," American Society for Testing Materials, Special Technical Publication No. 322, 1962, pp 199-210.

74. "How an Unusual Slide in a Highway Cut Handled West of Sapinero, Colorado," Roads and Streets, Vol 77, No. 4, April 1934, p 157.

75. "Huge Slide Halter by Hydrauger Holes," Contractors and Engineers, October 1952, pp 20-23.

76. Hughes, A. C., "Soil Stability Problems in Road Engineering," Proceedings of the Institute of Civil Engineers, Vol 2, Part 2, 1953, p 219.

77. "Hydraulicking Slides on Redwood Highway," California Highways and Public Works, Vol 4, No. 1, January 1927, p 11.

78. Hyrschfield, R. C., et al, "Review and Analysis of Available Information on Slopes Excavated in Weak Shales," Engineering Properties of Nuclear Craters, Report No. 3, 1965.

79. Janbu, Nilmar, "Stability Analysis of Slopes with Dimensionless Parameters," Harvard Soil Mechanics Series No. 46, Cambridge, Massachusetts, 1954.

80. Kane, Wallace B., "Stabilizing a Slipping Fill on a Hillside Road," Engineering News Record, August 8, 1935, p 184.

81. Kerr, P. F., and I. M. Drew, "Clay Mobility, Portuguese Bend, California," Project No. 8623, Scientific Report No. 1, United States Air Force Terrestrial Science Laboratory, April 1967.

82. Kerr, P. F., and I. M. Drew, "Clay Mobility in Ridge Route Landslides, Gorman, California," Project No. 8623, Scientific Report No. 14, United States Air Force Terrestrial Science Laboratory, July 1970.

83. Kerr, P. F., and I. M. Drew, "Quick Clay Movements in Anchorage, Alaska," Project No. 8623, Scientific Report No. 5, United States Air Force Terrestrial Science Laboratory, December 1965.

84. Kerr, P. F., and R. A. Stroud, "Clay Mobility in Landslides, Ventura, California," Project No. 8623, Scientific Report No. 5, United States Air Force Terrestrial Science Laboratory, May 1969.

85. Krauser, H. L., "Piles into Rock," Roads and Streets, May 1950, pp 56-58.

86. Krynine, D. P., "Landslides and Pile Action," Engineering News Record, Vol 107, No. 122, 1931.

87. Krynine, D. P., "On the Methodology of Landslide Investigation in Soviet Russia," Bulletin 236, Highway Research Board, 1959, pp 17-30.

88. Krynine, D. P., and W. R. Judd, "Landslides and Other Crustal Displacements," Principles of Engineering Geology and Geotechnics, McGraw-Hill Book Co., Inc., New York, 1957, pp 636-671.

89. Ladd, G. E., "Methods of Controlling Highway Landslides," Roads and Streets, Vol 68, 1928, pp 529-538.

90. Laird, M. A., "Soils Engineering Applied to a Highway Stability Problem," Highway Focus, Vol 2, No. 5, December 1970, pp 1-10.

91. "Landslides Threaten Residences Near San Pedro California," Engineering News Record, Vol 103, 1929, p 75.

92. Larew, G. H., "Use of Field, Laboratory and Theoretical Procedures for Analyzing Landslides," Bulletin 49, Highway Research Board, 1952, pp 28-39.

93. Litvinov, I. M., et al, "Stabilization of Soil for Constructional Purposes," Proceedings of the Fifth International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1961, pp 775-780.

94. Lundy, H. L., et al, "Evaluation of Deep In-Situ Soil Stabilization by High Pressure Lime Slurry Injections," Highway Research Record No. 235, 1968, pp 27-35.

95. Marivoet, L., "Control of the Stability of a Sliding Slope in a Railway Cut Near Wetteren," Third International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1948, pp 38-42.

96. "Methods of Overcoming Slides on Side Hill Roads," Engineering News Record, Vol 89, 1922, p 359.

97. "Montana Landslide Research Study," Special Report to the Montana Highway Department, 1969.

98. Moram, B. J., "The Use of Vegetation in Stabilizing Artificial Slopes," Proceedings of the Conference on Biology and Civil Engineering, Institute of Civil Engineers, 1948, p 113.

99. Nerrian, R. H., "Portuguese Bend Landslide in the Palos Verdes Hills, Southern California," Geological Society of America, Vol 69, 1958, p 1696.

100. "New Drainage Methods Solve Slide Problems," Western Construction, Vol 34, No. 6, pp 74-79.

101. Nordfelt, C. R., "Vertical Wells Stabilize California Hillside," The Highway Magazine, September 1956, pp 196-199.

102. Palmer, E. L., et al, "The Control of Landslides by Subsurface Drainage," Proceedings, Vol 30, Highway Research Board, 1950, pp 503-508.

103. Parrott, W. T., "Control of Landslides by Vertical Sand Drains," Bulletin 115, Highway Research Board, 1955, pp 51-52.

104. Peck, R. B., and H. O. Ireland, "Investigation of Stability Problems," Bulletin 507, American Railroad Engineers Association, 1953, pp 112-133.

105. Peynircioglu, Handi A., "Investigations of Landslides on a Natural Slope and Recommended Measures," Seventh International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1969, pp 645-651.

106. Philbrick, Shailer S., and Arthur B. Cleaves, "Landslides and Engineering Practice," Special Report 29, Highway Research Board, 1958, pp 93-111.

107. "Pipe Drains Stop an Active Slide," Engineering News Record, Vol 61, 1942, p 576.

108. "Practical Lessons in Landslide Correction," Indian Road Congress, Vol 30, No. 3, September 1967, pp 411-441.

109. "Putting Brakes on Landslides," Compressed Air Magazine, Vol 53, No. 8, August 1948, pp 191-192.

110. Rao, K. S., and S. K. Wodhaven, "The Effect of Heating the Soil on Permeability Under Prolonged Submergence of Soil in Water," Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering, Vol 1, 1953, pp 178-179.

111. "Reinforced Earth," Bureau d'Etude de la Terre Armee, France, 1968.

112. Reti, G. A., "Slope Stabilized by Anchored Retaining Walls," Civil Engineering, April 1964, pp 49-53.

113. Root, A. W., "Control of Slides by Underdrainage," American Society of Agricultural Engineers, California Department of Public Works, June 1958, 14 pp.

114. Root, A. W., "Correction of Landslides and Slipouts," Transactions, Vol 120, American Society of Civil Engineers, 1955a, pp 280-289.

115. Root, A. W., "Horizontal Drains - New California Approach in Landslide Control," California Highways and Public Works, March-April 1955b, p 26.

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

117. Rowan, W. H., et al, "Failure of a Side Hill Fill," Civil Engineering, May 1963, pp 40-41.

118. Salas, J. A., et al, "Swelling Clays," International Association of Road Congress, Vol 19, 1955, pp 10-47.

119. Schroter, G. A., and R. O. Maurseth, "Hillside Stability, the Modern Approach," Civil Engineering, June 1960, p 66.

120. Scott, E. T., "Grading Completed for Widening of Grapevine Grade from Three to Four Lanes," California Highways and Public Works, February 1941, pp 18, 19, and 26.

121. Scott, E. T., "Grapevine Grade Sliding Hill to be Stopped by Buttress Fill and Drains," California Highways and Public Works, May 1936, pp 309-314.

122. Shannon and Wilson, "Report on Foundation Studies, Seattle Freeway," Consulting Report, private correspondence, 1963.

123. Shannon and Wilson, "Slope Stability Investigation, Vicinity of Prospect Park," Consulting Report, private correspondence, 1968.

124. Simpson, F. H., et al, "Progress Report by the Committee on Roadbed Stabilization," Bulletins 479 and 486, American Railroad Engineers Association, 1949 and 1950.

125. Skempton, A. W., "The $\phi = 0$ Analysis of Stability and Its Theoretical Basis," Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Vol 6, 1948, pp 72-77.

126. Skempton, A. W., "The Rate of Softening in Stiff Fissured Clays, with Special Reference to London Clay," Second International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1949, pp 50-53.

127. "Slides Plague Scenic Coast Highway," Engineering News Record, May 11, 1961, p 21.

128. Smith, R., "Roadbed Stabilization," Bulletin 479, American Railroad Engineers Association, 1949, p 674.

129. Smith, R., and R. B. Peck, "Stabilization by Pressure Grouting on American Railroads," Geotechnique, Vol 5, 1955, pp 243-257.

130. Smith, Rockwell, "Chapter Two: Economic and Legal Aspects," in "Landslides and Engineering Practice," Special Report 29, Highway Research Board, 1958, pp 6-19.

131. Smith, T. W., "Ground Water Controls for Highways," Special Report No. 57, Highway Research Board, December 1964, pp 35-52.

132. Smith, T. W., "Major Slides of the Trans-Sierra Highway," Pacific Council, American Society of Civil Engineers Convention, April 1962.

133. Smith, T. W., and Tom Stafford, "Horizontal Drains on California Highways," Journal of the Soil Mechanics and Foundation Division, Vol 83, SM7, American Society of Civil Engineers, July 1957.

134. Smith, T. W., et al, "Pinole Slide, I-80, California," Highway Focus, Vol 2, No. 5, December 1970, pp 51-62.

135. Smith, Travis, and Raymond Forsyth, "Potrero Hill Slide and Correction," Journal of the Soil Mechanics and Foundation Division, Vol 97, SM3, March 1971, pp 541-564.

136. Spellman, H. J., "Handling Landslides on Mountain Roads," Engineering News Record, Vol 98, 1927, pp 434-438.

137. "Stability and Performance of Slopes and Embankments," American Society of Civil Engineers, Soil Mechanics and Foundation Division, Berkeley, California, August 22-26, 1969.

138. Stanton, T. E., "California Experience is Stabilizing Earth Slopes," Second International Conference on Soil Mechanics and Foundation Engineering, Vol 3, June 1948.

139. Stanton, T. E., "Stabilization of Earth Slopes Through Installation of Horizontal Drains," California Highways and Public Works, January-February 1948.

140. Stuart, F. L., "Four Engineering Innovations on the Magnolia Cut-Off," Engineering News Record, Vol 75, 1916, pp 1218-1222.

141. Symons, I. F., "The Magnitude and Cost of Minor Instability in the Side Slopes of Earthworks on Major Roads," Report LR331, Road Research Laboratory, 1970.

142. Taylor, D. W., "Stability of Earth Slopes," Contribution to Soil Mechanics, Vol 1.

143. Terzaghi, K., "Critical Height and Factor of Safety of Slopes Against Sliding," Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering, Vol 1, 1936, pp 156-161.

144. Terzaghi, K., "Mechanism of Shear Failures on Clay Slopes and the Creep of Retaining Walls," Public Roads, Vol 10, 1929.

145. Terzaghi, K., and R. B. Peck, Soil Mechanics in Engineering Practice, John Wiley and Sons, 1967.

146. Terzaghi, K., and R. B. Peck, "Stabilization of an Ore Pile by Drainage," Journal of the Soil Mechanics and Foundation Division, Vol 83, No. SM1, American Society of Civil Engineers, January 1957.

147. Terzaghi, Karl, "Stability of Slopes of Natural Clay," Proceedings of the Harvard Conference on Soil Mechanics, Vol 1, 1936, p 161.

148. Tompkin, J. M., and S. B. Britt, "Landslides, A Selected Annotated Bibliography," Bibliography 10, Highway Research Board, 1951.

149. Toms, A. H., "The Effect of Vegetation on the Stabilization of Artificial Slopes," Proceedings of the Conference of Biology and Civil Engineering, Institute of Civil Engineers, 1948, p 99.

150. Toms, A. H., "The Present Scope and Possible Future Development of Soil Mechanics in British Railway Civil Engineering Construction and Maintenance," Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Vol 4, 1948, pp 226-237.

151. Toms, A. H., "Recent Research Into the Coastal Landslides at Folkestone, Warren, Kent, England," Third International Conference on Soil Mechanics and Foundation Engineering, Vol 2, 1953, pp 288-293.

152. Toms, A. W., and D. L. Bartlett, "Application of Soil Mechanics in Design of Stabilizing Works for Embankments, Cuttings, and Track Formations," Proceedings of the Institution of Civil Engineers, Vol 21, April 1962, pp 705-747.

153. U.S. Army Corps of Engineers, Engineering and Design: Stability of Earth and Rock Fill Dams, No. EM 1110-2-1902, January 1970.

154. U.S. Army Corps of Engineers, U.S. Army Engineering and Design of Retaining Walls, No. EM 1110-2-2502, May 29, 1961.

155. U.S. Bureau of Reclamation, "Investigation and Solution of a Landslide Problem Involving a High Transmission Tower," American Society for Testing Materials, Fourth Pacific Area National Meeting, October 1962.

155. U.S. Bureau of Public Roads, Landslide Investigations: A Field Handbook for Use in Highway Location and Design, Arthur R. Cleaves, U.S. Government Printing Office, Washington, D.C., July 1961, 67 pp.

156. Vaughn, E. W., "Stabilizing Earth Fill with Rock," Engineering News Record, July 1, 1938, p 27.

157. Vidal, Henri, "Reinforced Earth Steel Retaining Walls," Civil Engineering, February 1970, pp 72-73.

158. Von Gottstein, E., "Two Examples of Underground Sliding Caused by Construction of Embankments and Static Investigations of the Effectiveness of Measures Provided to Insure Their Stability," Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering, Vol 3, 1926-27, pp 122-128.

159. Waterways Experiment Station, "Soil Stabilization - Investigations of Quicklime as a Stabilizing Material," Technical Report No. 3-455, March 1962.

160. Waterways Experiment Station, "Soil Stabilization - Investigations of a Chemically Modified Quicklime as a Stabilizing Material," Technical Report No. 3-455, Report 6, June 1963.

161. Whitney, Charles S., "Stabilizing a Lake Michigan Bluff," Civil Engineering, Vol 6, No. 5, May 1936, pp 309-314.

162. Wilson, S. D., "Observational Data on Ground Movements Related to Slope Instability," Proceedings, Vol 96, SM5, American Society of Civil Engineers, September 1970, pp 1521-1544.

163. Wilson, S. D., "The Use of Slope Measuring Devices to Determine Movements in Earth Masses," Field Testing of Soils, ASTM Special Technical Publication No. 322, 1962, pp. 187-197.

164. Zaruba, Q., and V. Mencl, "Landslides and Their Control," Czechoslovak Academy of Sciences, 1969, 208 pp.