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The major slope failures of significance were found to be associated with primarily excavated (cut) slopes in stiff-fissured clays and clay shales, although failures in embankment (fill) slopes constructed of high plasticity clays were also encountered in several areas. A number of remedial measures have been employed for repair of these failures, including regrading, stabilization with lime and cement additives, and various forms of restraint or buttressing structures. While excessive amounts of ground and surface water were usually present at most sites where slope failures have occurred, drainage of water has not been extensively employed as a remedial or preventative measure.

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A SURVEY OF EARTH SLOPE FAILURES AND REMEDIAL MEASURES IN TEXAS

by

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Research Report Number 161-1

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

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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 first report on the findings of Research Project 3-8-71-161, "Stability of Earth Slopes." Included herein are the results and summary of a survey of earth slope failures along Texas highways and of the remedial methods employed for repair. The areas of Texas where slope problems appear significant are identified and available information on the selection, design and performance of remedial measures is reviewed.

The authors wish to acknowledge the valuable assistance, service, and information provided by the many personnel of the District Offices of the Texas Highway Department. The assistance and advice of Mr. Chester McDowell of the Center for Highway Research, Messrs. Jim Brown and Bob Guinn of the Texas Highway Department and Mr. Tony Ball of the Federal Highway Administration are also appreciated.

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ABSTRACT

The results of a survey undertaken to identify regions in Texas where there has been a high incident rate of slope failures and to identify some of the factors responsible for these slides are presented. In conjunction with this study a review of present slope design procedures and the remedial measures employed by the Texas Highway Department for repair and maintenance of earth slopes is contained in this report.

The major slope failures of significance were found to be associated with primarily excavated (cut) slopes in stiff-fissured clays and clay shales, although failures in embankment (fill) slopes constructed of high plasticity clays were also encountered in several areas. A number of remedial measures have been employed for repair of these failures, including regrading, stabilization with lime and cement additives, and various forms of restraint or buttressing structures. While excessive amounts of ground and surface water were usually present at most sites where slope failures have occurred, drainage of water has not been extensively employed as a remedial or preventative measure.

KEY WORDS: soil mechanics, slope stability.

SUMMARY

The results of this survey of earth slope failures in Texas indicate that failures have developed most extensively in cut slopes in medium to highly plastic, overconsolidated, stiff-fissured clays in the Dallas, Fort Worth, Waco, Austin and San Antonio regions of Texas. In these regions, failures tended to be shallow, semi-circular slides which, in many instances, developed several years after the construction of the slopes involved. Moderate to excessive amounts of ground water and surface water were observed in all but a few of these slides suggesting that swelling and positive pore water pressures were the principal causes of failure.

The review of the current earth slope design procedures and remedial measures employed by the Texas Highway Department indicates that at the present time highway slopes are generally designed empirically and repaired when they fail. One or more of the following remedial measures are typically used:

1. Stabilization of the slide material by the addition of lime or cement.
2. Substitution of sands and gravels for slide material, generally in the vicinity of the toe of the slide.
3. Construction of restraint structures, usually piling and cast-in-place drilled shafts, with or without retaining walls or similar structures attached.
4. Control of ground water with interceptor trenches and surface water with ditches, curbing, crack filling, and slope planting.
5. Construction of flatter slope grades.
6. Construction of concrete rip-rap on the faces of unstable slopes.

While all of these remedial measures have been effective to a degree, there are many instances where the effectiveness of a single one is uncertain due to the combined use of several measures for repair of a single slide. Specific attention is given in this report to the restraint structure referred to as a "slide suppressor wall" and a suggested design procedure is presented for evaluating this measure.

IMPLEMENTATION STATEMENT

The results of this research indicate that the instability of earth slopes along highways in Texas represents a problem which may involve significant costs for maintenance. The major slope problem area which should be recognized encompasses many cut (excavated) slopes in stiff clays and clay shales in the San Antonio, Austin, Waco, Dallas and Forth Worth regions of Texas. One solution to the stability problems in these areas is the adoption of flatter slopes for design; however, until these can be economically justified and because a number of existing slopes are anticipated to present future stability problems, continued maintenance of earth slopes in Texas should be planned for.

The results of this research should serve as a useful guide to the highway engineer for judging the effectiveness of alternate remedial maintenance measures, based on past experience in Texas. One of these measures, the "slide suppressor wall," may also be evaluated quantitatively using the design chart and procedure presented in this report. A chart for back-calculating shear strength parameter values from observed slope failures is employed in this procedure. This chart may be used for determination of shear strength parameter values for use in the design of other remedial measures as well.

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NOMENCLATURE

Symbol	Definition
c	cohesion
c_o	cohesion at the crest of the slope
d	maximum depth of slide measured normal to slope face
E	earth pressure force in horizontal direction
F	factor of safety
H	slope height
H'	modified slope height
L	distance from toe to crest of slope measured horizontally
L'	modified distance from toe of slide to crest of slope measured horizontally
N_{cf}	stability number
N_p	earth pressure coefficient
N_p'	modified earth pressure coefficient
r_u	ratio of pore water pressure to overburden pressure
V	earth pressure force in vertical direction
X	X-coordinate of critical toe circle
Y	Y-coordinate of critical toe circle
z	resultant earth pressure force
α	inclination of trial-wedge failure plane
β	inclination of slope
γ	unit weight of soil
γc	equivalent fluid density
$\lambda c\phi$	dimensionless parameter relating γ, H, ϕ, and c
λcz	dimensionless parameter relating γ, H, c and ψ_z
ϕ	angle of internal friction
ψ_z	rate of increase in shear strength with depth

CHAPTER I

INTRODUCTION

Slope failures, in addition to creating hazards, delays, and inconveniences for the highway user, increase the cost of highway maintenance. The often high cost of repairing a slope sometimes results from the indirect use of high factors of safety for the remedial measure in an effort to prevent the recurrence of failure. In other instances, where less conservative measures have been used, failure has occurred repeatedly. By identifying some of the factors contributing to slope failures and evaluating currently used remedial measures, helpful guidelines may be established for avoiding future slope failures and designing effective, economical remedial measures.

Presented in this report is a survey of earth slope failures which was conducted to determine geologic regions where there have been high incident rates of slope failures and to identify some of the factors which may have contributed to these failures. To accomplish these objectives, engineers and geologists from the Texas Highway Department were interviewed, landslides and repaired slopes were inspected, recorded information on soil and geologic conditions was collected, and design drawings and reports pertaining to slope failures were obtained. From discussions with highway personnel associated with slope failures and from on-site inspections, some of the characteristics of the failures and the influence of these characteristics on the economy and success of various remedial measures were determined for various geologic formations. Although strength data were not obtained, Atterburg Limit tests were performed on soil samples taken from several sites, and these results are summarized in Appendix I.

In conjunction with this collection of information on slope failures, a review was made of current earth slope design procedures and remedial measures employed by the Texas Highway Department. Slope failures have commonly been repaired by regrading the slope; however, in situations where a regraded slope has continued to fail or where the slope failure has endangered a highway or bridge structure, more extensive measures have been used. Twenty-six recent slope failures where measures more extensive than just regrading were employed were investigated to determine the types of remedial measures used and their effectiveness.

The design of a remedial measure is often dependent on the shear strength of the soil. While shear strength data may be obtained from laboratory or in-situ tests, the expense and uncertainty associated with obtaining representative soil samples and performing laboratory or in-situ tests may in many instances offset the benefits derived from such tests. An alternative approach is to back-calculate the shear strength parameter values from the observed failure surface. This approach is used in this report and a chart is developed for estimating shear strength parameter values from an observed failure surface.

By employing values of back-calculated shear strengths, a series of analyses were performed to evaluate one type of remedial measure used by the Texas Highway Department. The design of this measure, a retaining structure founded on drilled piers, generally requires some knowledge or estimate of the lateral forces the structure must withstand. By performing analyses using the back calculated shear strength parameter values and a procedure described herein the lateral forces may be determined. The forces obtained in this manner are compared with the values determined by conventional procedures, which are commonly used for this purpose. The new procedure developed in this report is believed to be somewhat more rational than the existing procedures and a series of simple design charts has been developed to facilitate its use.

The information presented in this report is divided into five major parts: (1) Earth Slope Failures, (2) Remedial Measures, (3) Estimating Soil Strength Parameter Values, (4) Earth Pressure Forces for Restraint Structures, and (5) Summary and Conclusions.

CHAPTER II

EARTH SLOPE FAILURES

Introduction

Earth slope failures in Texas have developed primarily in medium to highly plastic, overconsolidated, stiff-fissured clays in which moderate to excessive amounts of ground water and surface water were present. The successful design of slopes in these materials depends to a large extent on a reliable prediction of the shear strength of the soil and the hydrologic conditions within the slope. However, predictions of the appropriate shear strength values for overconsolidated, stiff-fissured clays is complicated by many factors (Duncan and Dunlop, 1969; Skempton and Hutchinson, 1969). These include:

(1) the reduction in shear strength due to swelling,

(2) the reduction in shear strength when the strains in a heavily overconsolidated clay reach the failure strain,

(3) the anisotropic shear strength variations resulting from geologic processes,

(4) the reduction in shear strength due to fissures, fractures and slickensides, and

(5) the errors associated with sampling the soil and performing laboratory tests.

In order to establish the influence of these factors on the shear strength and stability of a particular slope, a number of extensive and sophisticated test procedures and analytical methods are commonly required.

The evaluation of the stability of slopes in the stiff clay formations commonly encountered in Texas is further complicated by what appear to be randomly developed, seasonal ground water conditions. To adequately determine the ground water regime, extensive borings would be required, to insure that isolated and randomly developed concentrations of water are discovered. Furthermore, in many instances it may be necessary to monitor such borings for extended periods of time because of the seasonal nature of some of the ground water sources and the relatively low permeability of many of the geologic formations involved. In addition, the excavation of a slope may change the seepage pattern in a manner not anticipated. For the above reasons, the determination of the shear strength and hydrologic conditions for many highway slopes in Texas would be expensive and difficult. An often more economical alternate, and an approach generally used by the Texas Highway Department, is to base the slope design on experience and to repair slides if and when they occur.

The height and inclination of a number of embankment and cut slopes constructed by the Texas Highway Department are dictated by right-of-way restrictions, highway geometry, maintenance requirements[1], and experience. Computer assisted slope stability analyses (Houston Urban Office and District 12) or stability charts (District 9) have occasionally been used to aid in the design of a slope; however, right-of-way restrictions and maintenance requirements generally govern the final slope designs. Typically, the heights of these slopes range from 20 to 40 feet, with slope inclinations ranging from 2 (horizontal): 1 (vertical) to 3:1, with slopes steeper than 2:1 generally rip-rapped with concrete slabs.

The type of soil used in the construction of embankments is generally not controlled. However, in District 12 of the Texas Highway Department, soils with liquid limits greater than 65 percent are not used for fill materials except in limited cases, and in District 2, if the plasticity index of the fill material exceeds 40 percent, the last four or five feet of the embankment are lime stabilized.

Characteristics of Slope Failures

While the present design practice of the Texas Highway Department appears in many instances to be satisfactory, there have been a number of slope failures. These have developed as small slumps, shallow semi-circular slides, and large rotational slides, with some of the slides extending several hundred feet parallel to the slope crest and involving several thousand cubic yards of material. In general, when these slides develop, the head of the slide mass drops, leaving a 4 to 12 foot scarp, while the toe of the slide bulges and flows down or off the face of the slope, as illustrated by the

[1]Maintenance requirements may govern the slope design in some instances because grass mowers can not operate efficiently on slope grades much steeper than 3:1.

typical cross section in Fig. 2.1. Although there have been several instances of failures during construction of both cut and embankment slopes, failures have generally occurred three to eight years after construction. These long-term failures may have resulted from the gradual reduction in shear strength due to swelling and to the development of positive pore water pressures. Furthermore, in stiff-fissured clays the excavation of a slope may cause some fissures to open; these open fissures, in addition to being surfaces of weakness, provide natural paths for water migration and thereby may lead to the softening of the stiff-fissured clay mass and a reduction in shear strength (Terzaghi and Peck, 1967). The majority of the slides have occurred in five Texas Highway Department Districts: 2 (Fort Worth), 9 (Waco), 14 (Austin), 15 (San Antonio), and 18 (Dallas) (see Fig. 2.2). Some slope failures have occurred in other Districts but with apparently much lower frequency than in the previously mentioned Districts. The five Districts cited encompass some of the most unfavorable geologic and hydrologic conditions for the stability of earth slopes in Texas.

Geologic Conditions

Slopes excavated in geologic formations consisting of medium to highly plastic, overconsolidated, stiff-fissured clays have proven to be the most susceptible to failure. The overconsolidated clays in the Taylor and Navarro geologic groups have caused the greatest problems in San Antonio (District 15). These geologic groups, and in addition the Del Rio clay formation, have also caused problems in Austin (District 14). The plasticity indexes of the clays sampled from slides in these geologic units have ranged from 40 to 60 percent. Slopes in the Taylor and Navarro clays typically have failed along shallow semi-circular rupture surfaces which appeared to develop in a more highly weathered zone extending approximately five to eight feet in depth below the slope, as illustrated in Fig. 2.3. For example, the rupture surface of a slide near Heep's Dairy on I.H. 35 south of Austin in a 20-foot high, 2 1/2:1 slope was shallow, with the maximum depth of the slide (measured normal to the slope face) being about six feet.

In Waco (District 9) the principal geologic formations causing instability are the South Bosque Shale, Walnut, Del Rio, and Eagle Ford; the plasticity indexes of these formations range from 38 to 60 (Font and Williamson, 1970). The stiff-fissured clays and marls in some of these formations alternate

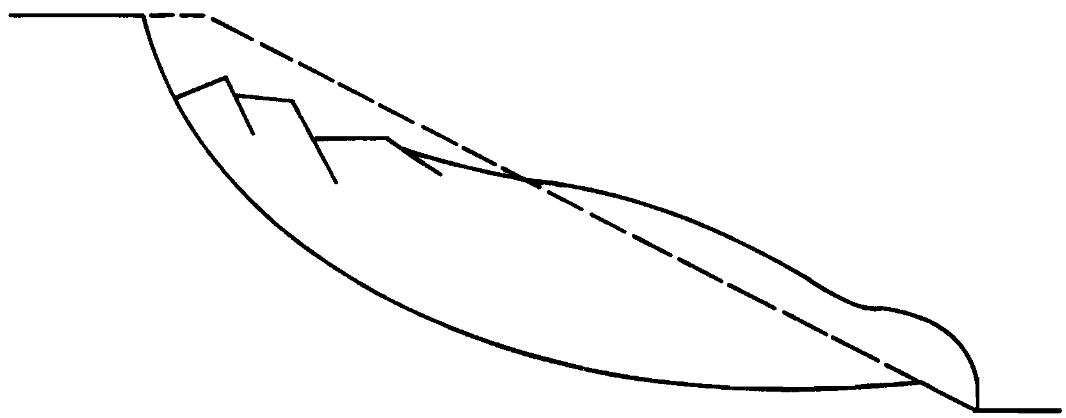


Fig. 2.1. Typical failure which occurs in Texas highway cut and embankment slopes.

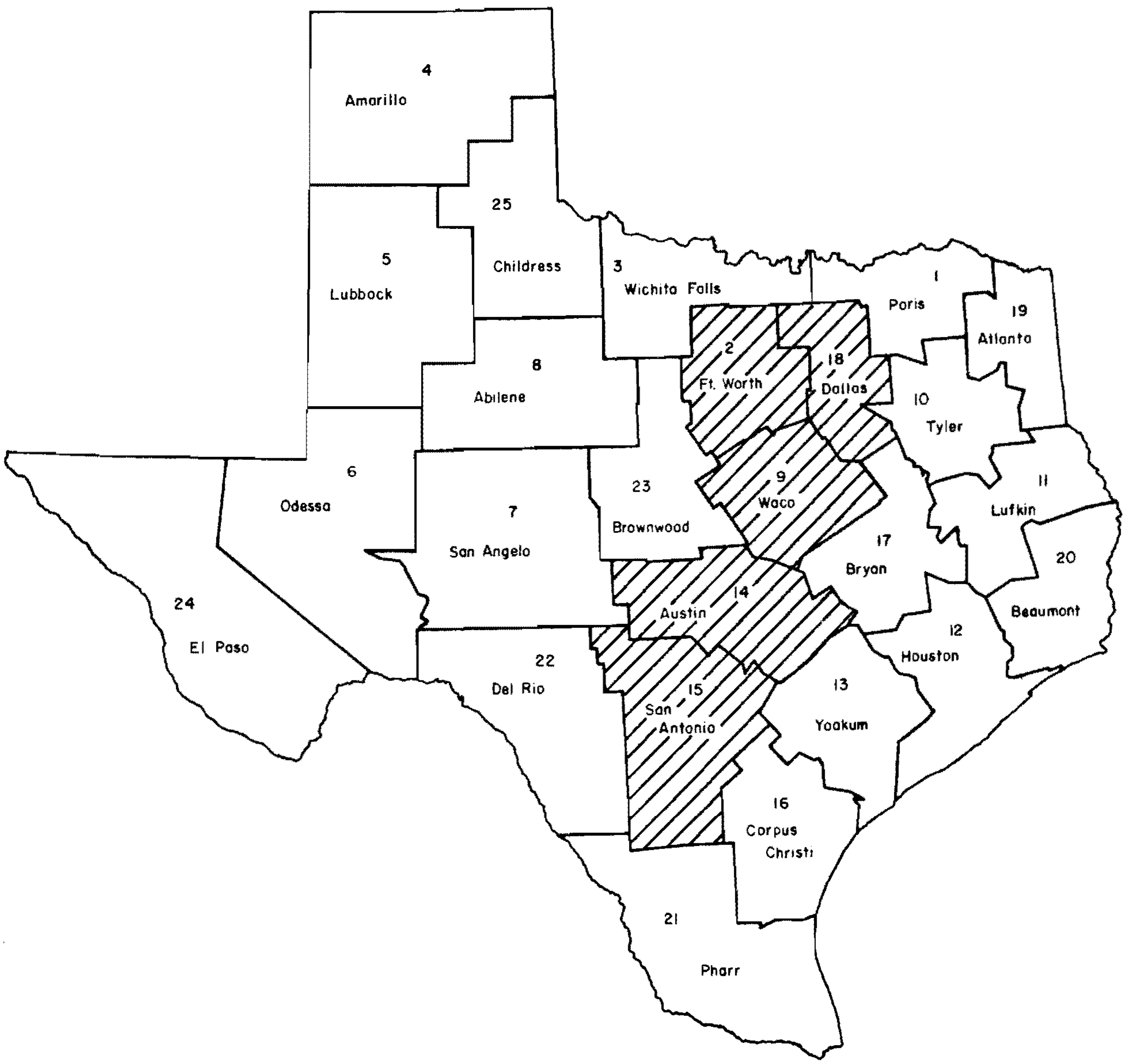


Fig. 2.2. Texas Highway Department Districts. The majority of the slope failures have occurred in the districts which are cross-hatched.

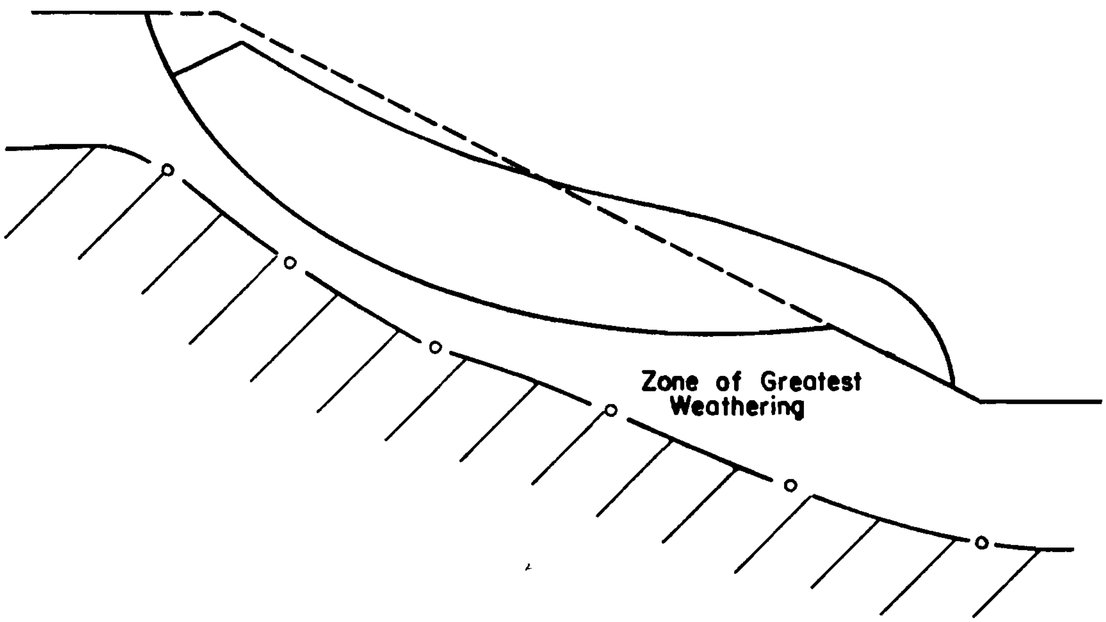


Fig. 2.3. A typical slope failure in a highly weathered zone of an overconsolidated stiff-fissured clay slope.

with thin layers of either sandstone or limestone. These thin rock layers tend to control the groundwater conditions and the mode of failure in these layered slopes. Randomly fractured and jointed rock has tended to cause erratic seepage patterns while continuous intact rock has led to the formation of perched water tables. In addition, failure surfaces in these materials tend to be non-circular, as the surface of sliding tends to develop near the base of clay layers adjacent to more resistant rock layers. A typical cross section of such a slide, which occurred north of Evant on U.S. 281, is illustrated in Fig. 2.4.

In the Fort Worth and Dallas Districts, the Mineral Wells, Eagle Ford, Taylor marl, Denton, and Kiamichi formations are some of the primary geologic formations in which landslides have occurred. Many of the clays involved in these failures are moderately to highly plastic and some, such as the Eagle Ford, are highly expansive (Texas Highway Department, 1966). Plasticity indexes ranging from 35 to 50 have been measured in a number of slides in District 2. Typically, in many of these formations thin layers of limestone or shell conglomerates alternate with thicker clay and marl beds. Perched water tables may develop above thin rock layers or, in some cases, water may seep laterally through the layers causing seeps to emerge on the face of the slope. Failures in these formations are generally similar to those shown in Figures 2.3 and 2.4.

In addition to formations of stiff-fissured clays with some interbedded limestone layers, unfavorable sequences of clay and sand layers have also contributed to landslides in several Highway Department Districts. For example, at the I.H. 45 and State Highway 7 interchange near Centerville, a layer of Wheches clay, ten to fifteen feet thick, is underlain by the Queen City sand and overlain by Sparta sand, a water bearing fine grained sand, as illustrated in Fig. 2.5. The plasticity index of the Wheches clay in this area ranges from 28 to 35 percent. During the excavation for the construction of this section of I.H. 45, the saturated Wheches clay was exposed in 4:1 and flatter slopes, resulting in the development of several shallow slides. These slides were attributed to the low shear strength of the Wheches clay resulting from seepage in the overlying Sparta sand.

Slope failures have occurred in slopes which were originally formed adjacent to limestone cliffs by falling rock debris (talus). These failures have generally been initiated in saturated materials when the toe of the

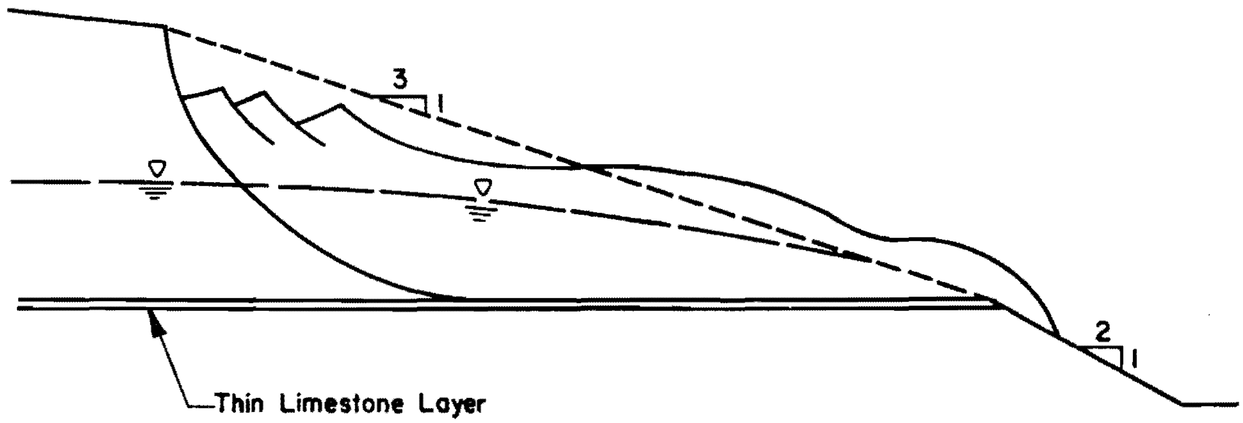


Fig. 2.4. Cross section of a slide which occurred on U.S. 281 north of Evant.

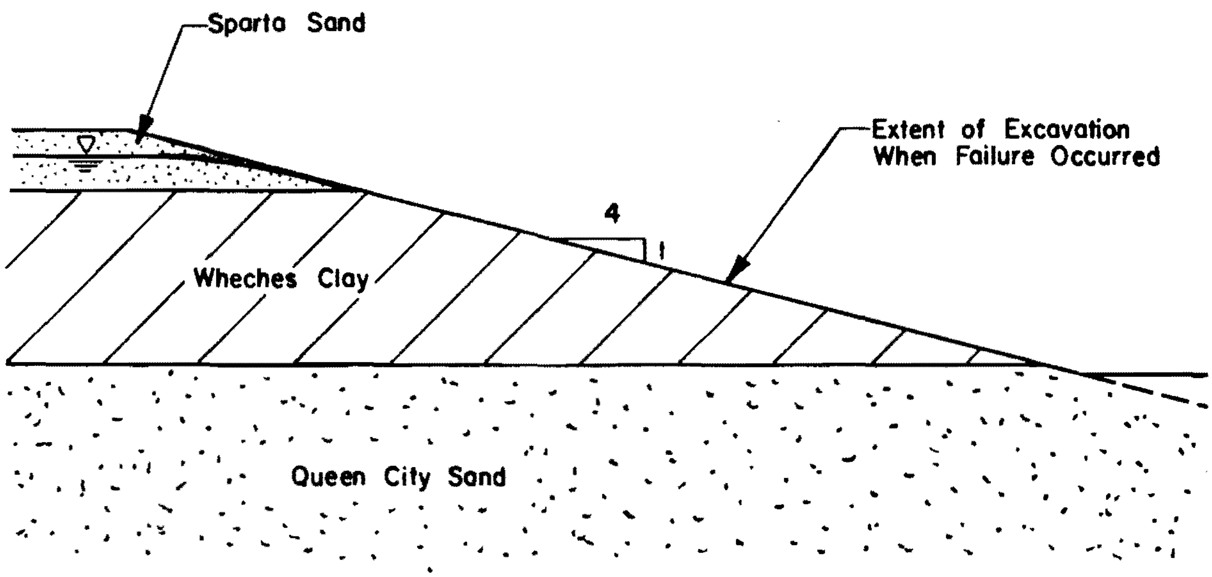


Fig. 2.5. Cross section of slope which failed along I.H. 45 near Centerville.

natural slope was removed by excavation. Several such failures have occurred in Waco (District 9), and in these the sliding surfaces appeared to develop along the contact of the talus and the underlying material.

Embankments constructed of medium to highly plastic, overconsolidated clays have also been susceptible to failures. Failures were generally observed to involve only the embankment material as in the case of the slide at the intersection of I.H. 410 and Marbach Road in northwest San Antonio. In this instance, the rupture surface passed through the crest and toe of the slope and the toe of the slide flowed a few feet beyond the toe of the slope. Only one embankment failure involving the foundation material was observed. In this instance the foundation failed beneath a 46-foot high embankment which was being built east of Dallas to raise I.H. 30 above the elevation of the then to be built Lake Hubbard. The embankment was designed with 3:1 side slopes; however, in order not to interrupt the flow of traffic on the existing I.H. 30, the south slope was built temporarily on a 2:1 grade. During construction, two failures involving approximately one tho sand feet of the south slope of the new embankment developed. The failure surfaces passed through the weak foundation soils and beyond the toe of the slope.

Hydrologic Conditions

Water originating as groundwater or surface runoff appears to be one of the principal factors contributing to the instability of clay slopes in Texas. In many of the geologic formations previously described the presence of randomly distributed limestone and sandstone strata, varying from continuous and intact to highly jointed, may result in a number of different patterns of seepage. Thus, the prediction of water conditions is considerably more complex and uncertain in these formations than in homogeneous soil deposits. In addition, the presence of fissures, as they commonly exist in stiff clay formations in Texas, further complicates the seepage patterns and influences the randomness of groundwater conditions.

Seepage within relatively pervious zones in a slope such as sand and gravel seams, porous and fractured limestones, sandstones and shell conglomerates, as shown in Fig. 2.6a, may often be a major factor in initiating slope failures. Water from a sandy clay seam is believed to have initiated two slides on a 30-foot, 2 1/2:1 slope along I.H. 35 at Moore Street in San Antonio. The first slide occurred in a section of the slope west of the

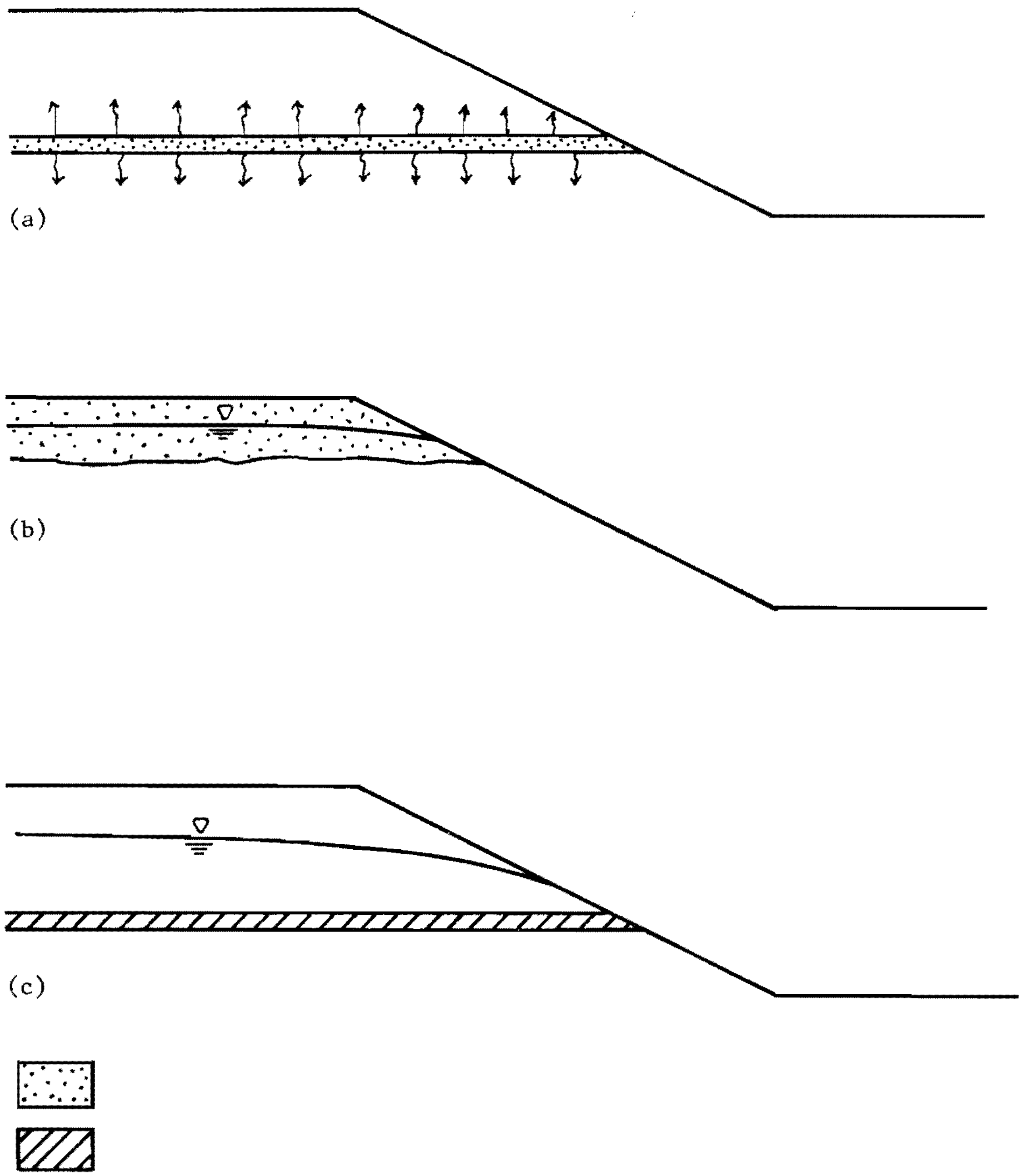


Fig. 2.6. Various sequences of pervious and impervious strata in clay slopes.

Moore Street bridge about three years after the slope had been excavated. Prior to the failure, seeps had formed on the face of the slope, and after the failure, the slide debris appeared to be saturated. Borings made near the crest of the slope disclosed a water bearing seam approximately eleven feet below the crest of the slope. Several years after this slide had been repaired a second slide developed in the concrete rip-rapped slope beneath the Moore Street bridge. In an effort to arrest the slide, holes were drilled near the toe of the rip-rap. Initially, water flowed freely from these holes, and later, when the rip-rap was removed to correct the slide, the slide material was observed to be saturated. Both of these slides are believed to have been initiated by water seeping through sandy clay seams in the predominately overconsolidated clay.

Water perched in sand, gravel, and pervious rock layers by underlying clays, as illustrated in Fig. 2.6b, may also contribute to slope failures in the clay by reducing its shear strength. This, for example, was the case in the previously discussed Centerville slides. While these slides occurred during construction, a number of years may pass before water from an overlying strata will initiate a landslide. For instance, water seeping from a fractured limestone layer into a 40-foot, 2:1 slope on I.H. 820 near Grove Oak Drive in Fort Worth appears to have contributed to the failure of this side hill cut, several years after the slope had been excavated.

Water may also be perched within the clay strata itself by an underlying rock layer of lower permeability, as depicted in Fig. 2.6c. A slide occurring on U.S. 281 north of Evant was believed to have been initiated by water perched in such a manner above a thin limestone layer. The slope in which the slide occurred had been cut in 1959; the first twelve feet of the slope were cut on a 2:1 grade and the remaining 28 feet on a 3:1 grade, as previously shown in Fig. 2.4. The slide profile consisted of alternating units of thin limestone layers and thicker clay layers. In the late 1960's, a slide occurred along a thin continuous limestone layer lying about ten feet above the toe of the slope. The toe of the slide was saturated, and borings disclosed high concentrations of water above the limestone layer where the failure occurred.

In general, slides have developed during or after periods of heavy rainfall, which may replenish or increase the flow in an aquifer such as a sand or gravel seam. The natural highly plastic clay slope along Boggy Creek near the U.S. 183 highway bridge in southeast Austin was steepened to a 2:1 grade to straighten the creek. A section of this 30-foot-high slope failed shortly after a period of heavy rainfall, approximately two months after the cut had been made. In the failed section there was a buried channel where water infiltrating from above the slope is believed to have initiated the failure.

During periods of dry weather, shrinkage cracks may develop to considerable depths in highly plastic clays, such as the Eagle Ford or Taylor Marl formations, thus providing a natural path for surface water infiltration. Surface water which entered shrinkage cracks is believed to have contributed to several embankment failures on I.H. 35E in Dallas. The material in these slopes consisted of a highly plastic silty clay with a plasticity index ranging from 30 to 50. During a period of several months, two separate slides developed in saturated material at this location following periods of heavy rainfall. These slides involved a total of approximately 500 feet of the 2 1/2:1 to 3:1 embankment slopes. A three to four-foot scarp developed along the edge of the highway shoulder and the toe of these slides passed through the lower third point of the slope face. Shrinkage cracks were observed along the shoulder of the highway several months after the slides had occurred, suggesting that surface water which infiltrated the slope through similar cracks may have contributed to the failures.

<u>Summary</u>

The majority of the earth slope failures have developed in medium to highly plastic, overconsolidated, stiff-fissured clays located in the Dallas, Fort Worth, Waco, Austin, and San Antonio region of Texas. Slope design is generally based on experience, due to the difficulties and expense of adequately defining the shear strength of the clays and the groundwater conditions within the slope. Generally, shallow semi-circular failures have occurred in embankment and cut slopes and may have developed as a result of the gradual reduction in shear strength due to swelling and the development of positive pore water pressures caused by seepage from perched water tables, aquifers, and fissures.

CHAPTER III

REMEDIAL MEASURES

Introduction

A number of different remedial measures have been utilized by the Texas Highway Department to correct earth slope failures. These measures include:

(1) stabilization with additives,

(2) soil substitution,

(3) restraint structures,

(4) control of water,

(5) slope alteration, and

(6) concrete rip-rap.

While slopes are often repaired by pushing the slide material back onto the slope (regrading), this approach is more of an effort to retain the original slope grade than to correct or offset those conditions which initiated failure. Although slopes have remained stable after being regraded, the improved stability probably results from a change in the conditions which initiated failure, such as the equilibration of pore water pressures or the drainage of a pocket of water, rather than from any improvement in the shear strength of the soil achieved by pushing the slide debris back onto the slope. Therefore, for these reasons, regrading has not been considered as a remedial measure in this report. In order to evaluate the six remedial measures listed above, twenty-six recently corrected slope failures, summarized in Table 3.1, were reviewed with attention focused on the application and effectiveness of these measures.

Stabilization with Additives

The Texas Highway Department has utilized both lime and cement additives for stabilization and repair of earth slope failures. In general the use of lime has been restricted to highly plastic clays while cement has been utilized with both clays and granular materials. The addition of lime to the soil has been accomplished by direct mixing with the soil (shallow treatment)

Table 3.1 Summary of Recent Slope Failures Which Have Received Extensive Remedial Treatment

SLOPE LOCATION	T.H.D. DISTRICT	SLOPE HEIGHT	SLOPE RATIO	CONDITIONS AT SITE		AGE OF SLOPE AT FAILURE	REMEDIAL MEASURES	REMARKS
				GEOLOGIC	HYDROLOGIC			
U.S. 80 West of Lake Arlington Tarrant County	2	25' C*	2:1	Del Rio clay	Dry	?	Soil mixed with lime and compacted on slope at a cost of $130,000	Slope failed one year after being lime stabilized. The slope will be bridged over.
U.S. 180 near Brad Palo Pinto County	2	20' C	1 1/2:1	Highly plastic clay	Excess water	Failed during construction	Retaining wall built out of I-beams and timber planks; drainage trench placed	Slide extended laterally 1025 ft. sucessfully stabilized at a cost of $12,500
S.H. 174 Johnson County	2	15' C	2:1	Silty clay	Excess water	~ 15 years	Retaining wall built out of I-beams and timber planks	Successfully stabilized at a cost of ~ $4,300
U.S. 377 south-east Tarrant County	2	40' C	3:1	Natural slope	Failed after heavy rain	?	Retaining wall built out of steel I-beams and steel guard rails	Water from utility ditch may have initiated failure
U.S. 67 near Alvarado	2	25' C	2:1		Excess water	8-9 years	Flattened slope to 4:1 and mixed lime with recompacted soil	Slope successfully stabilized
U.S. 180 near Brad Palo Pinto County	2	40' C	2:1	Side hill fill	Excess water	5-6 years	Constructed 12-ft.- deep trench drain	Slope successfully stabilized
U.S. 281 near Evant Hamilton County	9	0-12' 12-40' C	2:1 3:1 cut	Walnut formation	Excess water	1 year	Drilled holes filled with hydrated lime; limestone broken up with dynamite; seep tapped with pipe	Failure developed along clay and limestone contact; successfully stabilized
I.H. 35 and U.S. 81 McLennan County	9	18' C	2:1	Taylor Marl	Seeps at base of rip-rap	1 year	Flattened slope to 3:1 and constructed low retaining wall	Slope successfully stabilized
U.S. 59 near Greenbriar Street Harris County	12	18' F	2.6:1	Highly plastic clay	Excess water	?	Treated timber piles used to arrest slope movement. Toe of slide replaced with sand	Cost of placement of timber piling and sand ~ $100,000 Slides involved 1400 ft. of slope

*C: Cut, excavated, or natural slope
F: Fill or embankment slope

(Continued)

Table 3.1 (Continued)

SLOPE LOCATION	T.H.D. DISTRICT	SLOPE HEIGHT	SLOPE RATIO	CONDITIONS AT SITE		AGE OF SLOPE AT FAILURE	REMEDIAL MEASURES	REMARKS
				GEOLOGIC	HYDROLOGIC			
S.H. 225 and T.N. & O. Overpass Harris County	12	25' C	2-3:1	Highly plastic fill	Excess water	6-7 years	Slide arrested with timber piles	Slide developed under rip-rap
I.H. 35 near Heep's Dairy in South Austin Travis County	14	20' C	2 1/2:1	Taylor Marl	Excess water	?	Fourteen-foot-deep drainage trench placed above and parallel to crest; timber piling used on later slope movement	After drainage trench was built, the slope began to bulge This movement was corrected with piles
U.S. 183 at Boggy Creek Travis County	14	30' C	2:1	Taylor Marl	Failed after heavy rain	2 months	Two rows of cast-in-place drilled shafts on 6 ft. centers	First slide in this section of slope
U.S. 183 under Boggy Creek bridge Travis County (second slide)	14	30' C	2:1	Taylor Marl		10 months	Two staggered rows of piling on 6 and 12-foot centers; slope steepened to 1.4:1 down slope of timber piles.	Lime slurry placed in drilled holes and concrete rip-rap was used in an unsucessful attempt to prevent failure
U.S. 183 at Boggy Creek Travis County	14	30' C	1.4:1	Taylor Marl	Failed after heavy rain	4 years	Lime mixed with slide material; slope regraded to former grade	Two small slides occurred; one on each side of bridge
U.S. 87 northeast San Antonio Bexar County	15	30' C	1 1/2:1	Taylor Marl	Excess water	~ 6 years	Lime placed in drilled holes and mixed with slide material; concrete ditch built along crest	Sucessfully stabilized at a cost of ~ $12,000
I.H. 35 and Moore Street Bexar County	15	30' C	2 1/2:1	Taylor Marl P.I. ~ 54	Excess water	3-4 years	Lime placed in drilled holes and mixed with slide material; a 14-ft. drainage trench built at crest	Slide which occurred west of Moore Street bridge was repaired at a cost of ~ $24,000
I.H. 35 and Moore Street Bexar County	15	23' C	2 1/2:1	Taylor Marl	Excess water	7-8 years	Slide suppressor wall placed 1/3 the distance up the slope	Slide developed under rip-rap of Moore Street bridge
I.H. 10 and New Braunfels Avenue Bexar County	15	25' C	3:1	Taylor Marl	Excess water	3 years	Slide suppressor wall placed 1/3 the distance up the slope	Slide developed in 400 ft. of the south slope of I.H. 10; repaired at a cost of ~ $33,500

(Continued)

Table 3.1 (Continued)

SLOPE LOCATION	T.H.D. DISTRICT	SLOPE HEIGHT	SLOPE RATIO	CONDITIONS AT SITE		AGE OF SLOPE AT FAILURE	REMEDIAL MEASURES	REMARKS
				GEOLOGIC	HYDROLOGIC			
I.H. 10 and New Braunfels Avenue Bexar County	15	23' C	3:1	Taylor Marl	Excess water	~ 3 years	Thirteen, 20-ft. long drilled shafts on 8-ft. centers placed 1/3 the distance up the slope	Slide in north slope of I.H. 10 was successfully stabilized at a cost of ~ $14,900.
I.H. 10 & Roland Avenue Bexar County	15	18' C	2 1/2:1	Highly plastic clay	Excess water	~ 3 years	Slide suppressor wall placed one-third of the distance up the slope	Failure occurred beneath rip-rap
I.H. 37 and South Cross Boulevard Bexar County	15	17' C	2:1	Highly plastic clay	Excess water	?	Slide suppressor wall placed one-third of the distance up the slope	Failure occurred beneath rip-rap
I.H. 37 and New Braunfels Avenue Bexar County	15	17' C	2:1	Highly plastic clay	Excess water	?	Slide suppressor wall placed one-third of the distance up the slope	Failure occurred beneath rip-rap
Loop 410W and Marbach Road Bexar County	15	22' F	1 1/2:1	Houston clay	Excess water	1-2 years	Slide material mixed with lime	Successfully stabilized
I.H. 45 and S.H. 7 near Centerville Leon County	17	55' C	4:1	Sparta sand Wheeches clay Queen City sand	Saturated clay	During construction	System of interceptor and lateral drains	Slope was successfully stabilized at a cost of ~ $27,000.
South I.H. 35E north of Bachman Rd Dallas County	18	35' F	2:1	P.I. 30-50	Excess water	2-3 years	Slope rebuilt with same material	Slope failures repaired by mixing lime with the slide debris have occurred again in this material.
I.H. 30 at Lake Hubbard Dallas County	18	46' F	2:1	Taylor Marl	Dry	During construction	Slope flattened to 3:1	Three failures occurred in this embankment involving 1,000 ft. of slope

or by placing the lime in drilled holes (deep treatment), while cement stabilization has been restricted to direct mixing techniques (shallow treatment).

Lime Stabilization (Shallow Treatment). The stabilization of slides by the addition and mixing of lime with the soil is usually restricted to relatively shallow depths of up to approximately 4 feet. However, in one instance lime stabilization by direct mixing was carried out to depths as great as nine feet. The addition and mixing of lime with the soil are either done in-place on the face of the slope or the slide debris is removed from the slope, mixed with lime and then placed back on the slope. The stabilized material is then recompacted with tractors or conventional compaction equipment; however, the compaction may not be controlled.

For large slides, generally involving over 400 cubic yards of material, a stage construction procedure has sometimes been used for stabilization of the slide material. In these instances the slide debris is excavated by benching the slope, mixed with lime, replaced on the slope and recompacted to the final slope grade. Adjacent strips are then excavated and the procedure repeated until the entire slide area has been stabilized.

The level of lime treatment used is generally in the range of 3 to 4 percent lime by weight and no laboratory studies are made to select optimum lime content. This level of treatment is typical of the lime contents used for modification of pavement subbase materials.

While lime stabilization appears in several instances to have been a successful corrective measure, failures have occurred in slopes which were previously stabilized with shallow lime treatment. The reason for the recurring slides is uncertain, and generally insufficient information is available to decisively ascertain the courses of the recurring slides. However, several factors probably contribute to the inadequacy of the remedial measures:

(1) poor mixing of the lime with the soil,

(2) poor compaction of the lime stabilized material - no compaction control,

(3) insufficient depth of treatment and failure to stabilize the zone of movement, particularly for deep slides, and

(4) improper consideration of the causes of failure.

One of the largest shallow lime stabilization projects was carried out by Fort Worth (District 2 of the Texas Highway Department) to correct a slide along U.S. 80 west of Arlington. This slide occurred as a deep rotational failure along a 1,200 foot section of a 35-foot high, 2:1 cut slope in the Del Rio formation, a highly plastic, stiff-fissured clay shale. The slide debris was removed in 12-foot wide strips ranging in depth from five to nine feet, mixed with lime, and recompacted to the original slope grade; however, the lime stabilized surface did not intercept the failure surface of the slide. Approximately three years after the repairs were made in 1969, the slope failed again, with the stabilized strips of soil being displaced as more or less continuous blocks in the saturated slide material. Six months prior to this failure, several deep, wide cracks were observed along the face of the slope, apparently at the interface between the adjacent stabilized strips. These cracks, which may have developed as a result of renewed movement along the initial failure surface, provided natural paths for surface water to saturate and weaken the slope mass. There had been a number of other slides in this section of U.S. 80 before this more recent one and in order to prevent future problems, this slide zone will be bridged over and the slope beneath the bridge will be flattened.

In the majority of the slope failures which have occurred the presence of excess surface water and the groundwater conditions appear to have played a dominant role in the initiation of the slides. In these instances stabilization and sealing the slope face with lime may impede internal drainage and result in the development of excess hydrostatic water pressures in the soil with a corresponding reduction in strength and stability.

One of the limitations placed on the use of lime stabilization is the lack of sufficient working space to properly mix the soil with lime and recompact the material on the slope. In a number of cases, particularly in the San Antonio area, where slope failures have occurred beneath overcrossing bridges, the overhead clearance has been severely restricted. In other instances the slope was either too steep to mix the lime properly on the face of the slope or the available work area above and below the slope was too small to mix lime with slide material removed from the slope. One approach to using lime stabilization in these situations is to remove the slide material to another location for mixing and then return the stabilized material to the slope. However, the additional requirement of transporting the slide material makes this procedure more expensive than mixing lime with the slide debris at the site of the failure. The experience and policy of Houston (District 12) has been that if the material must be removed from the site for mixing with stabilizing agents, a generally more economical alternative is to replace the slide debris with a more suitable borrow material.

Lime Stabilization (Deep Treatment). Deep lime stabilization as referred to in this report is accomplished by placing lime in dry or slurry form into drilled holes. Typically the drilled holes are 8 to 12 inches in diameter and are placed in staggered rows on 5 to 10-foot centers, the exact diameter and spacing of the holes being determined by the economics of construction and the judgement of the engineers involved. The drilling of these holes is commonly done from 12 to 14-foot-wide benches excavated to provide a level working area. Either one or two rows of augered holes are used for each bench, depending on the capabilities of the drilling equipment and the engineer's judgement. The depth of the holes used for the deep lime treatment has varied from 5 to 25 feet below the drilling surface and generally an attempt is made to penetrate several feet beyond the known or estimated failure surface.

From the available case histories it is difficult to assess the extent to which deep lime treatment has contributed to the stability of a failed slope because the lime was used in conjunction with other corrective measures. However, it appears that the use of deep lime stabilization has increased the stability of affected slopes. For instance, a lime slurry was used to correct a progressive slide problem on a 1:1 slope along Business U.S. 87 in northwest San Antonio. The lime was placed in deep holes augered into a 25-foot high fissured calcareous clay slope. In addition to the lime, a shallow concrete lined ditch was built along the crest of the slope to minimize the runoff over the face of the slope. These two corrective measures successfully corrected what had previously been an annual problem.

A hydrated lime slurry was also used to stabilize the previously discussed slide near Evant in Hamilton County on U.S. 281. The soil profile of the 40-foot-high slope consisted of thin limestone layers alternating with thicker clay layers. In the late 1960's, a slide occurred along a thin continuous layer of limestone as shown in Fig. 2.4. A perched water table which contributed to the failure had developed above this limestone layer. Ten benches, wide enough to allow a truck mounted auger to drill 12-inch-diameter holes down to the zone of slippage, were cut into the slope parallel to the roadway. A total of 122 holes were augered and filled with approximately 50 tons of lime slurry. Only enough water was used to allow mixing and pumping of the lime. All the holes were checked and refilled with slurry for a period of several weeks. The limestone layer where the failure had developed was fractured with dynamite to lower the perched water table, and in one area where water continued to concentrate, a horizontal pipe was installed to remove the seepage. The project was left for six months, during which time the seeps were observed and the slope surveyed for indications of additional movement. After this period the slope appeared stable so grading and finishing were completed and rye grass was planted over the entire slope. The slope has remained stable for the past eleven years.

There appear to be at least two limitations on the reliability of deep lime treatment for stabilization of an active slide. The first of these involves the uncertainty regarding the extent to which the lime migrates into the soil surrounding the drilled holes. In dense, non-fissured clays the extent of lime migration into the soil has been found to be generally less than 1 inch (McDowell, 1970). In fissured clays the degree of lime migration appears to be somewhat greater; however, the extent of migration is uncertain and probably highly variable. In addition, both the rate of lime migration and the rate of strength increase in the soil are probably relatively slow, and consequently the immediate effects of deep lime treatment are probably minimal.

A second limitation of deep lime treatment involves the potentially adverse effects of lime addition in slurry form under either gravity or pressure heads. The addition of a water-lime slurry may increase the presence of free water in the slope and result in increased hydrostatic (pore water) pressures, thus decreasing the shear strength of the soil and the stability of the slope. Also, if the lime is injected under pressure, a serious danger exists in the possibility that cracks may be developed in the injected formation and a significant reduction in stability may occur.

Cement Stabilization. The application of cement for stabilization of earth slope failures has received only limited use by the Texas Highway Department. The less extensive use of cement as compared to lime probably results in part from the fact that most slope failures have occurred in predominantly clay soils of medium to high plasticity. In these instances lime would appear to be comparably as effective as cement at approximately half the expense. However, the effects of cement stabilization may be realized much more rapidly due to the more rapid rates of strength gain with time.

In surveying the remedial methods used by the Texas Highway Department only one instance was found where cement was used to directly stabilize a fine-grained soil. This occurred on the IH 35-Moore Street slide previously described. In stabilizing this slide, cement was used to stabilize the highly plastic clay slope at the toe; however, lime was used on the remainder of the slope.

In all other instances where cement has been utilized, the cement has been mixed with cohesionless materials which have been substituted for the original slide debris. Typically about five percent cement, by weight, has been used for stabilizing these materials. In one instance Houston (District 12) has utilized oyster shells as a replacement material, stabilized with approximately seven percent cement.

Cement stabilization appears to have been restricted entirely to direct mixing techniques, and there are no known instances where deep cement treatment or grouting has been employed. The principal restrictions on the use of cement as compared to lime are the limited advantages of cement for treatment of medium to highly plastic clays, and the higher cost of cement. In general cement may also be expected to share most of the limitations and restrictions previously discussed with respect to lime.

Soil Substitution

Soil substitution involves the replacement of the slide material, generally in the vicinity of the toe of the slope, with a more suitable material, usually either a clay of low plasticity or cohesionless sands and gravels. The substituted material may serve to increase the stability of the slope in three ways:

(1) by providing a higher strength and greater shear resistance against sliding,

(2) by providing a more pervious material which will allow water to drain more freely from the slope and prevent the development of excessive hydrostatic (pore water) pressures, and

(3) by providing an increase in the weight of the soil at the toe of the slope which provides an increased passive resistance to sliding.

Generally cohesionless sands and gravels are the most suitable for this purpose as they possess a higher strength, greater permeability and somewhat higher unit weight than clayey soils. A persistent landslide problem on U.S. 59 between Shepherd Drive and Greenbriar in Houston was corrected by substituting sand for the original fill material. Timber piling was driven near the top of both the north and south fill slopes to retain the embankment while the soil below the piling was replaced with compacted sand. These slopes which had continually failed after periods of heavy rainfall have remained stable since being repaired during the early months of 1969.

Some replacement materials may be stabilized with cement to improve their resisting capacity; however, the use of fine-grained cement stabilized materials may result in undesirably low permeabilities and impedance of free drainage. The permeabilities of cement stabilized sands and gravels appear to be adequate. Indexes of the rate of flow, or drainage factors (Texas Highway Department, 1962), determined for these stabilized materials (District 12, Houston) ranged from 1200 to 1500 cc/hr.

Well compacted and well graded gravels can be used to increase the unit weight of the soil at the toe of the slope by thirty or forty percent as these materials may have densities of up to 140 lb/cu. ft. However, the overall increase in stability due to this added unit weight is probably minor as compared to the increase in stability derived from the added strength and drainage which is provided by gravel or sand substitution.

Restraint Structures

Piling and cast-in-place concrete drilled shafts have been employed relatively extensively by the Texas Highway Department for stabilization of earth slides. Where piling has been used, timber has been the most common structural material, rather than steel or reinforced concrete. In several instances a retaining wall or similar structure has been affixed to the pile heads to provide further lateral resistance to sliding and to preclude soil movement around and between the piles or drilled shafts.

Drilled Shafts. Drilled shafts, which are widely used by the Texas Highway Department for the support of bridge structures, have also been used as a slide retention measure with apparent success. Drilled shafts were used as a corrective measure along the north slope and south slope of the depressed section of I.H. 10 and New Braunfels Street in San Antonio. These slides occurred in approximately 25-foot high 3:1 slopes excavated in a highly plastic, stiff-fissured clay shale. The four hundred foot long slide which occurred along the south slope of this location was corrected by using 24-inch-diameter shafts, 20 feet in length. The drilled shafts were spaced on 8-foot centers along a single line approximately one-third of the way up the slope. In addition to these shafts a 1-foot by 4-foot vertical wall was cast in place against the upper 4 feet of shaft and an attempt was made to position the wall such that the slide plane intercepted the center of the wall. The wall was then backfilled with gravel and the slope regraded to its original shape. This particular configuration of drilled shafts and the adjacent concrete wall below grade has been locally referred to as a "slide suppressor wall."

The second and smaller slide along the north slope of the same location was stabilized with approximately 13 drilled shafts, similar in size and location of placement to those used on the south slope. However, a concrete wall was not used in conjunction with the shafts at this second slide. The remedial measures used for these slides were constructed in 1968 and the slopes had remained stable to the most recent (1971) survey.

Drilled shafts were also used to correct a slide which developed beneath a highway bridge on U.S. 183 at Boggy Creek in Austin (District 14). The first slide at this location occurred in 1965 along a portion of a 30-foot-high cut slope in highly plastic weathered shale along the bank of Boggy Creek. Fifty, 18-inch-diameter drilled shafts were placed on 6-foot centers in two staggered rows approximately 6 feet apart. The 32-foot-long shafts were located approximately one-third of the way up the original slope. Reinforcement for each shaft consisted of six evenly spaced No. 6 bars and a No. 2 spiral extending the full shaft length. In addition, three No. 9 bars were placed in the lower 20 feet of the shaft, along the upslope side, to provide resistance to bending. In conjunction with the placement of these drilled shafts, the slope was regraded and flattened to 3:1 above the elevation of the line of shafts. However, below this elevation, the slope was steepened to approximately 1.4:1 as illustrated in Fig. 3.1. While the drilled shafts have apparently been successful in halting further slide movements from above, a new slide occurred in the steepened portion of the slope below the drilled shafts during the winter of 1969-1970. This slide, which exposed the upper 6 to 8 feet of the shafts, was repaired with regrading and lime stabilization.

Steel Piling. In several locations the Texas Highway Department has utilized steel I-beams or H-piles for stabilizing slide movements. The steel piling is either driven in place or installed in a pre-bored hole and backfilled with concrete. In several instances a timber plank wall has also been fitted between the flanges of adjacent piles to provide additional slope

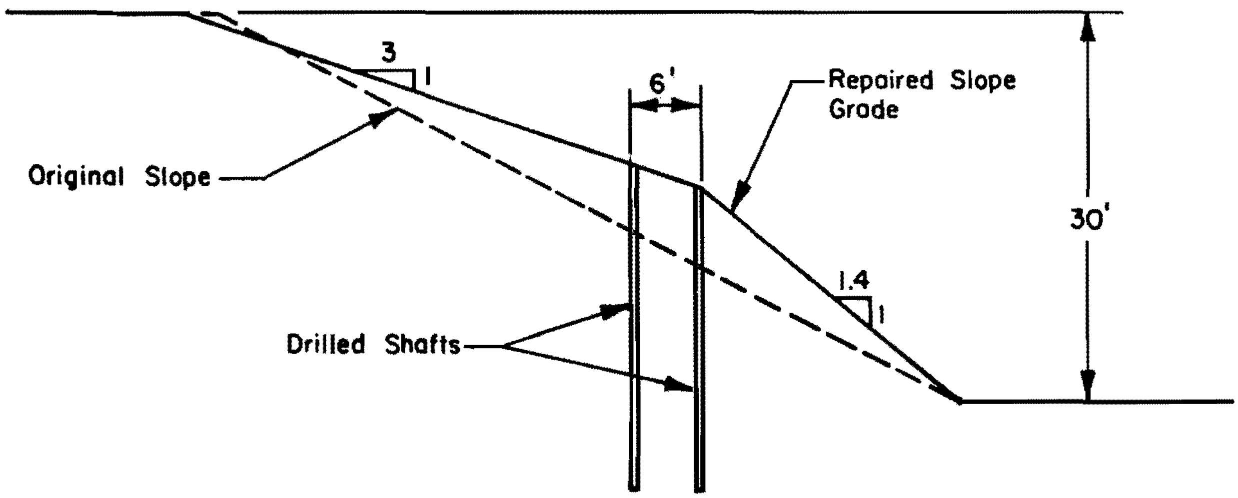


Fig. 3.1. Typical cross section of Boggy Creek near U.S. 183 in southeast Austin.

restraint, and when such a wall is used, placement of the steel beams in prebored holes is generally preferred because of the improved alignment for construction of the wall.

Steel I-beams were used to correct a slide extending approximately 1000 feet along a section of U.S. 180 near Brad in Palo Pinto County. The I-beams were driven on 8-foot centers to a depth of approximately 15 feet below the toe of the slope and treated timber planks (old railroad ties) were placed to form a 7-foot-high wall. A 3-foot-wide drainage trench, including an underdrain outlet pipe, was placed behind the wall and backfilled with filter material.

While the above wall was left exposed at the front, a similar retaining structure was included entirely within the original slope grade as a corrective measure in the stabilization of a slide along U.S. 174 near Palo Alto in Johnson County. In this instance, the steel piles were placed in pre-bored holes a distance of approximately one-third of the way down the slope from the crest. The piles extended to a depth of approximately 26 feet below the slope surface and an 8-foot-high timber retaining wall was affixed to the upper portion of the piles. A 4-foot-wide drainage trench 8 feet deep was placed behind the wall and backfilled with standard filter material including a pipe underdrain.

Another slide in Fort Worth (District 2) occurred in a natural slope located on the lower side of a highway and resulted in pavement damage extending to approximately the roadway centerline at the top of the slide scarp. This slide was believed to have been initiated by entrance of water through a 3 foot deep utilities trench constructed along the shoulder several years after completion of the highway. In this instance the slide was successfully stabilized with a restraint structure consisting of old steel guard rails attached to vertical steel I-beams. The steel beams were placed along the road shoulder in pre-bored holes and backfilled with concrete. The top of the steel wall was located at approximately the elevation of the highway and some portions of the wall were left exposed on the downslope side.

<u>Timber Piling</u>. In the correction of slides, timber piling has been used both as a primary corrective measure and as a temporary measure prior to major remedial work. In a large slide, occuring beneath the overpass structure for U.S. 183 at Boggy Creek in southeast Austin, timber piling was used as a primary corrective measure. The slide at this site developed in a 2:1 slope excavated in a highly plastic, expansive clay shale and involved about 200 feet of the 25-foot-high, concrete rip-rapped slope. The slide was corrected by

driving two rows of timber piling, one row being driven on 6-foot centers approximately at and parallel to the crest of the slope while the second row, located about 14 feet up slope from the first row, was driven on 12-foot centers. All piles were driven to refusal, approximately 20 to 30 feet deep. In conjunction with this remedial work the slope was regraded.

Timber piles were also used to correct a slide on I.H. 35 near Superior Dairies in Austin. This slide developed on a slope which had failed during a period of heavy rains approximately two years earlier and had been stabilized by installing an interceptor trench parallel to and above the crest of the slope. The new slide developed in the upper half of the 2 1/2:1, 20-foot-high slope. Timber piles were installed on 5-foot centers in two staggered rows. In this and other instances where timber piles have been used by District 14 for slide correction, there have been no problems with soil flowing around and between piles. Such piles have normally been spaced on 5 or 6-foot centers.

In at least one instance timber piles have been installed in a slide to temporarily halt the slide movement and to prevent more extensive development of an initially small slide. Such action was taken to arrest progressive movements of a slide which developed along a 520-foot section of the embankment slope on U.S. 59 between Shepherd Drive and Greenbriar in Houston. The slide developed in the lower half of the slope, and in order to prevent the slide from progressing up slope and threatening the main highway, 12-inch-diameter timber piles, 30 feet in length, were driven on approximately 2 1/2 foot centers along the upper scarp of the slide (at mid-height of the slope). The tip of the piles in this instance extended approximately 16 feet below the elevation of the toe of the slope. The purpose of these piles was to stabilize the portion of the slope above the slide for a sufficient period of time to permit the slide debris to be removed and replaced with a more select, stable material.

In several cases where timber piling has been used to stabilize slide movements, additional timber piles have been placed horizontally behind and against the vertical piles in order to provide increased lateral restraint and to reduce soil movement between the piles.

One of the difficulties which may be encountered when using timber piles is the limitation on the depth to which the piling may be driven without structural damage and "brooming" of the pile. This problem may be particularly significant in areas where slides are occurring in slopes excavated in very stiff clays and shales. However, Austin (District 14) has successfully and economically overcome this problem by driving the piles in pre-bored holes.

A second potential problem arising from the use of timber piling as a remedial method is rotting and decay of the pile material. In practically all cases where slope failures have been observed, the presence of water was clearly noted. Consequently in most of these cases timber piling will be subjected to water and probably alternate periods of partial wetting and drying, a condition which may lead to relatively rapid decay in the pile. Because of the strong likelihood of decay and loss of structural integrity, the timber piling should preferably be treated and it should be recognized that the long-term benefits of timber piling may be uncertain.

CONTROL OF WATER

The presence of uncontrolled water appears to have been one of the primary causes of nearly all earth slope failures in Texas. The introduction of water into the slope, either as groundwater or surface runoff, increases the hydrostatic (pore water) pressure in the slope and reduces the available shearing resistance of the soil. In order to restrict the water from entering the slope initially and to remove any water which does enter the slope, the Texas Highway Department has employed several methods of control.

Interceptor Trenches. One of the principal techniques for controlling subsurface groundwater uses interceptor drainage trenches, which are constructed by excavating a trench near and parallel to the crest of the slope. The depth of the trench may vary, depending on the soil profile, depth of the slide, location of groundwater and the problems and costs associated with bracing and dewatering the drainage trench. The trench is generally backfilled with a standard filter material used by the Texas Highway Department, and where large quantities of flow are anticipated, drainage tile or perforated, corrugated metal pipe underdrains are installed at the bottom of the trench. In addition, transverse drains are usually installed to remove the water collected by the interceptor drains and to discharge the water at the toe or side of the slope. These transverse drains may be similar in construction to the interceptor trenches or may consist only of a tile or metal outlet pipe.

A combined scheme of interceptor and transverse drains was used to correct a series of slides which developed during the excavation of the slopes along a section of I.H. 45 one mile from Centerville (District 17). These slides developed in an approximately 15-foot-thick clay stratum, bounded above and below by layers of sand. The clay layer had apparently become saturated by a perched water table in the upper sand stratum, and the removal of support from the clay during the excavation of the 4:1 cut slopes resulted in sliding. A series of lateral interceptor drains, averaging 6 feet in depth and 2 1/2 feet in width, were constructed in several rows paralleling the crest of the slope. The bottom of the trenches were backfilled with approximately 2 1/2 feet of standard filter material, the remainder of the trench being filled with impervious material. Where the anticipated quantity of flow was relatively large, a 6-inch-diameter perforated, corrugated, galvanized metal pipe was also placed in the bottom of the trench. Drainage of the water from the interceptor drains was accomplished by construction of several transverse drains with 6-inch-diameter metal pipe outlets to drain the water out near the toe of the slope. A total of 4500 feet of filter underdrain and 2200 feet of drainage pipe were used to correct this slide at an in-place construction cost of approximately $27,000. The use of interceptor drains in this case has proven successful. Water is still being discharged from some of the transverse outlet drains and no further stability problems have been experienced since the drains were placed in 1966.

Initial slides at Moore Street and I.H. 35 in San Antonio and on I.H. 35 near Superior Dairies in Austin were also corrected by using interceptor drainage trenches. Both of these slides occurred in highly plastic, expansive clay slopes and were corrected by excavating an interceptor trench approximately 15 feet deep near the crest of the slope. At both sites a gravel underdrain was used and only the Austin site required shoring of the trench. Additional sliding occurred at both of these sites after construction of the drainage system; however, at the San Antonio Moore Street location the new slide developed in an area adjacent to the old slide and did not involve the corrected area. At the Austin site a small slump slide occurred at the old slide location, but the new slide was much smaller in magnitude and was successfully stabilized with timber piling.

<u>Other Methods of Groundwater Control</u>. Horizontal drains extending into the face of the slope have occassionally been used to control water from isolated seeps. Generally a small diameter perforated corrugated metal pipe is used for this purpose. Horizontal drains were used in conjunction with other measures to correct a slide at Moore Street and I.H. 35 in San Antonio

and one on U.S. 281 near Evant. Initially water drained freely from drains at both sites. However, the drains have not been checked on a periodic basis, and thus, it is not known if the drains are still removing groundwater.

In several areas of the State (Districts 2, 9, and 18) perched water tables, which have developed above thin limestone strata in clay shale formations, have contributed to several slides. In at least one instance the stability of these slopes has been improved by fracturing the limestone strata with dynamite, thus improving the drainage characteristics of the formation.

Where slopes are rip-rapped with concrete, sand drainage blankets have occasionally been used to collect groundwater seeping from the slope face and to drain surface water which may enter the rip-rap from above the slope face or through construction joints. These drains are usually constructed with a minimum thickness of 6 inches and weep holes are provided in the rip-rap to allow free drainage of water from the sand blanket. Frequently, the weep holes placed at the bottom, middle, and top of the rip-rap are the only measures taken to drain the water from behind the rip-rap. While such drainage beneath the rip-rap is probably necessary, its presence does not necessarily preclude the possibility of failure of the slope due to excessive groundwater, although the groundwater is freely drained at the slope face.

<u>Control of Surface Water</u>. Control of surface water is an important and necessary phase in achieving a permanent and effective solution to slope stability problems. Surface water is controlled by ditches, curbing, crack filling, and slope planting. Concrete lined ditches placed above unstable slope areas should be constructed to properly intercept surface runoff and drain the water away from the unstable ground and the face of the slope. Where a highway or frontage road is located above a slope, the use of curbing along the roadway has provided this drainage control. However, the construction of paved ditches for the single purpose of intercepting water above the slope appears to have received only limited application by the Texas Highway Department.

In surveying the slope problem areas in Texas two specific slide locations were noted to have received inadequate consideration for surface drainage above the slope. At the Boggy Creek slide location in Austin, water was observed at one time to be ponded in areas of the ground of the slope. The existing paved drainage channels in the vicinity of the crest of the slope allowed water to pond at low points and seepage was occurring through some of the joints in the concrete ditch lining. While drainage was provided at the

Boggy Creek location, the drainage appeared inadequate for complete and proper control of the surface water at this site.

At a second site, located on S.H. 337 in Palo Pinto County, the complete absence of surface drainage appears to have been a contributing factor in the loss of stability. The slide at this location occurred along a portion of a 70-foot-high cut slope in a shale-sandstone formation. In the area of the slide the presence of excessive moisture could be clearly noted and a moderate amount of surface erosion had occurred on the face of the recently completed slope. Inspection of the area above the slope clearly showed the evidence of large amounts of surface runoff between the crest of the slope and the right-of-way line. Immediately above the slide area, at the crest of the slope, surface runoff has become channelized in an erosion gully varying from 6 to 10 inches in depth. While subsurface groundwater probably played a significant role in initiating the slide at this location, the infiltration of uncontrolled surface runoff undoubtedly aggrevated the slide problem.

In addition to paved drainage ditches, surface water may be controlled and its entrance into the slope may be reduced by filling and sealing cracks which form near the crest of the slope due to shrinkage and slope movements. If such cracks are allowed to remain open, they provide a natural path for entrance of runoff into the slope and the subsequent development of high pore water pressures. The Texas Highway Department has employed clay, RC-2 asphalt, and lime for filling open cracks; however, the solution is only temporary since new cracks generally develop within a short period of time. In the San Antonio area shrinkage cracks formed in one instance to a depth of approximately 20 to 25 feet. These formed in an embankment of approximately the same height (20-25 feet) constructed of a highly plastic clay borrow material. Attempts were made to seal the cracks by pumping an estimated 1000 gallons of RC-2 liquid asphalt into the openings. Later inspection (after the occurrence of a slide in the same embankment) revealed that below a depth of several feet the asphalt had remained in a liquid state and had failed to cure.

Sealing of the slope surface with a membrane to prevent cracking due to shrinkage has met with only limited success. In a number of instances cracking has been observed to extend into the paved highway surface, the existence of the paving having little influence on the prevention of cracking. San Antonio (District 15) has attempted to cover and seal the crest of the slope with a flexible asphalt membrane; however, these attempts have generally been unsuccessful because cracks soon develop in the membrane itself.

Grass and other vegetation planted on the slope for erosion control may also aid in the removal of some surface water and prevent deep infiltration of water into the slope through the face during wet periods of the year. However, during dry periods the slope vegetation may aid in the formation of shrinkage cracks, thus aggrevating the problem of surface water infiltration, especially during the first fall or winter rains.

Slope Alteration

The stability of several Texas highway slopes has been improved by flattening the slope grade by either removing or adding material. The flattening of a slope by excavation tends to reduce the forces which drive the slide mass, while the addition of compacted soil, principally in the toe region, tends to increase the forces which resist movement. The grade to which a slope is flattened has been determined primarily on the basis of experience and right-of-way restrictions. Typically, where there have been no right-of-way restrictions the slope has been flattened to the next higher integer ratio, i.e., from a 2:1 to a 3:1 slope grade. The slope failures in a natural slope at the intersection of I.H. 35 and U.S. 81 in Waco (District 9) and the previously discussed slides on the I.H. 30 embankment which crossed Lake Hubbard in northeast Dallas were successfully corrected by flattening their respective slope grades. The 18-foot-high, highly plastic natural slope and the 46-foot-high embankment slope were both flattened from a 2:1 to a 3:1 grade.

The use of slope flattening as a corrective measure may be restricted by the cost of additional right-of-way, limited sources of borrow material, or long haul distances to remove or add material. In addition, when soil is added to the toe region of a cut slope, care must be exercised to avoid sealing off groundwater flow. Disruption of this flow may lead to a build up of hydrostatic head which in time may cause the slope to fail again.

Concrete Rip-Rap

The presence of concrete rip-rap[1] on the slope face appears to have contributed to some slope failures while in other instances similar rip-rap has apparently prevented failures. This apparent anomaly appears to result in

[1] Concrete rip-rap as referred to in this report is a continuous slab of concrete generally six to eight inches thick constructed on the face of the slope.

part from the presence of surface and subsurface water and the effect which concrete rip-rap has on the control of this water. The general characteristic of failures associated with rip-rap and the role of rip-rap in preventing slope failures are discussed below.

Failure Modes. Instability and failure of concrete rip-rap have generally occurred in two manners. In some instances the rip-rap has moved downslope as a gradual creep movement. This mode of failure commonly produces a bulge near the toe of the rip-rap, sometimes extending into the paved shoulder of the road. The second mode of failure may be classified as complete failure of the rip-rap due to a shallow or deep soil failure in the underlying slope.

Rip-rap distress resulting from gradual creep movements has generally been confined to relatively steep slopes, usually 2:1 or steeper. Problems of this type have been found in Houston (District 12) and to a limited extent in other areas of the state. In one instance, in Fort Worth (District 2), rip-rap movements apparently resulted in large lateral loads being transmitted to a bridge support column, producing hairline tension cracks on a portion of the downslope side of the column. This problem was remedied by removing and replacing a small portion of the rip-rap around the column to allow for adjustments of the rip-rap without transferring load to the column.

The problem of downslope creep of the rip-rap has generally been reduced by the use of either flatter slopes (District 2) or toe walls varying from 9 to 10 inches in thickness and 2 to 3 feet in depth. However, in some instances these procedures have not performed satisfactorily and additional measures have been taken. District 15 has in recent instances adopted the use of an intermediate "key" wall located midway up the slope to provide additional anchorage for the rip-rap. These have been used for cut slopes which are 2:1 or steeper with a slant height of more than 45 feet. The intermediate wall is approximately 18 inches deep and 10 inches wide and the toe walls are either approximately 18 or 36 inches deep and 9 inches in width. The deeper toe wall is used with a 5-inch-thick rip-rap section, while the smaller wall is used with 4-inch rip-rap. Rip-rap and slope movements, which occurred at the intersection of U.S. 81 and I.H. 35 in District 9, were successfully corrected with a 3-foot high cantilevered wall, above ground, at the toe of the slope. The rip-rap was replaced on the flattened slope, extending back from the top of the wall.

The second category of slope rip-rap failures has generally been a result of slides initiating in the slope itself. However, in many of these instances it appears that the rip-rap may have indirectly contributed to the failure by impeding the free drainage of groundwater and permitting surface runoff to infiltrate and become entrapped in the slope. The impeded drainage of the groundwater, observed in sandy clay seams in the stiff clay slope which failed at Moore Street and I.H. 35 in San Antonio, is believed to have been partially responsible for the failure. In several other instances in this area similar influences of rip-rap are believed to have contributed to slides.

The entrance of water into the slope and the impedance of free drainage contributes to instability by permitting the soil (commonly stiff, highly plastic clays in cut slopes) to swell and lose strength due to the development of increased pore water pressures. Entrance of surface water into the rip-rap may occur through construction joints and openings which form at the top of the rip-rap. In several instances expansion of the soil beneath the rip-rap, due to either subsurface moisture or small amounts of surface water infiltration has contributed to the formation of cracks and partings in the rip-rap. Where the rip-rap is used for slope protection beneath highway overpasses, the top of the rip-rap has been observed to pull several inches away from the supporting concrete foundation cap for the bridge, thus providing a natural conduit for surface water to infiltrate the slope.

Efforts to remove surface and subsurface water entrapped behind relatively impervious concrete rip-rap have included the use of weep holes and the occasional use of sand blanket drains. Weep holes have generally been placed near the base of the rip-rap; however, weep holes have also been placed above intermediate key walls, and near the top of some rip-rapped slopes to drain granular material backfilled behind bridge abutments. Rip-rapped slope failures involving a soil failure have been corrected in a number of instances with a slide suppressor wall (for additional details, see the discussion on Drilled Shafts).

Rip-Rap as a Preventive Measure. Whether rip-rap increases or reduces the stability of a slope is at the present time uncertain, and probably, depending on the circumstances, rip-rap may either increase or decrease the stability. In San Antonio (District 15) there have been a number of slides in concrete rip-rapped slopes beneath highway overpasses both before and after failures in adjacent slopes with no rip-rap, suggesting that the rip-rap had not improved the stability of these slopes. Although these rip-rapped slopes were generally steeper (2-2 1/2:1) than the adjacent slopes (3-3 1/2:1), the relatively lower stability of the rip-rapped slopes as compared to the flatter adjacent slopes may have been largely offset or improved by the inclusion of drilled shafts in the crest of these rip-rapped slopes, the drilled shafts being installed to support the overpass bridge structure. While it is difficult to ascertain the degree to which the drilled shafts have improved the stability of the slope, the presence of considerable sub-surface moisture in the slide debris suggests that high pore water pressures may have contributed more to the instability of the rip-rapped slope than did the steepness of these particular slopes.

While the experience in San Antonio may suggest that rip-rap contributed to slope failures, an apparently contradictory experience exists in the urban area of Fort Worth in District 12. No failures of concrete rip-rapped slopes have been reported with the exception of the single problem previously described with respect to creep movement in the rip-rap. In many instances slopes which were not rip-rapped have failed while adjacent slopes, having the same inclination, but with concrete rip-rap anchored with a 3-foot-deep toe wall, have experienced no problems. One apparent reason for the effectiveness of rip-rap in these cases is the absence of excessive subsurface moisture and the prevention of surface water infiltration by the rip-rap.

On the basis of the limited amount of evidence available, it appears that, where surface runoff is the primary source of moisture, concrete rip-rap may reduce the amount of infiltration and reduce expansion and shrinkage in the soil, provided that cracks and open joints do not exist in the concrete. Thus, the rip-rap may aid in maintaining the stability of the slope. However, if subsurface moisture and seepage are present, rip-rap may provide little improvement to the stability of the slope and in some instances may even adversely effect the stability. Consequently the effectiveness of concrete rip-rap probably depends to a large extent on the sources of moisture and the integrity of the concrete. Because of the difficulty in predicting and determining either of these in advance, the reliability of rip-rap for improving and maintaining the stability of earth slopes appears uncertain at the present time.

CHAPTER IV

ESTIMATING SOIL STRENGTH PARAMETER VALUES

The design of a reliable and economical remedial measure depends to a large extent on the accuracy with which the shear strength of the slope can be determined. The shear strength may be estimated from experience, measured or back calculated from the slope failure. While experience is a valuable guide in the design of a corrective measure it may not be sufficient to execute a reliable as well as an economical design. Experience may be supplemented with strength data obtained from laboratory or in-situ soil tests. The evaluation of the strength test data will be influenced by, among other factors, the extent to which the soil samples represent the in-situ conditions and by the extent the test procedures reproduce the stress conditions in the slope. The cost of overcoming the uncertainty in obtaining and subjecting representative samples to laboratory and in-situ tests, which accurately model the in-situ soil conditions, would represent a high percentage of the total cost for a corrective measure on a typical Texas Highway slope. Furthermore, an extensive sampling and testing program may require more time than can be allowed to design and install a corrective measure.

In many instances, including those where cost or time prohibits extensive sampling and testing, the shear strength can be estimated by performing a stability analysis using the actual failure surface to determine the strength parameters which would be required to give a factor of safety of unity. This approach has been used to develop a chart from which an average cohesion value and angle of internal friction can be estimated from a slope which has failed.

Back Calculation of c and ϕ

The values of cohesion (c) and angle of internal friction (ϕ), corresponding to a factor of safety of unity, were determined for a range of slope conditions. For convenience the most critical failure surface was in all instances assumed to be a circular arc passing through the toe of the slope, a valid assumption for most homogeneous slopes. The procedure used for the analyses in this chapter is a procedure of slices based on the assumptions presented by Spencer (1967). This procedure has been shown to be an extremely accurate procedure based on static equilibrium and a computer program employing this method was readily available for performing the necessary analyses for back calculation of c and ϕ values (Wright, 1969, 1971).

In determining c and ϕ values, which correspond to a factor of safety of unity, it is convenient to use the dimensionless parameter $\lambda_{c\phi}$, defined as

$$\lambda_{c\phi} = \frac{\gamma \cdot H \cdot \tan \phi}{c} \tag{4.1}$$

where H is the slope height and γ is the total unit weight of soil. For a given slope angle (β) and value of $\lambda_{c\phi}$ a unique stability number, N_{cf}, exists which defines the factor of safety (F) in the form

$$F = N_{cf} \cdot \frac{c}{\gamma H} \tag{4.2}$$

The value of N_{cf} is dependent only on the values of $\lambda_{c\phi}$ and β regardless of the values of c, ϕ, γ, and H. Similarly, for a given combination of $\lambda_{c\phi}$ and β values, a unique set of values for ϕ and the ratio $\frac{c}{\gamma H}$ can be shown to exist for a factor of safety of unity.

The values of ϕ and $\frac{c}{\gamma H}$ corresponding to F = 1.0 were determined in the following manner for values of $\lambda_{c\phi}$ ranging from 0 to 100 and slope ratios (cotangent β) ranging from 1/1 to 4/1. First, for each combination of values of $\lambda_{c\phi}$ and β, the value of N_{cf} was determined. Next, the value of $\frac{c}{\gamma H}$ was calculated for a factor of safety of unity using Eq. 4.2 in the form

$$\frac{c}{\gamma H} = \frac{1.0}{N_{cf}} \tag{4.3}$$

Finally, the value of ϕ corresponding to the particular $\lambda_{c\phi}$ value being used was calculated from Eq. 4.1 in the form

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

The values of $\frac{c}{\gamma H}$ and $\tan \phi$ obtained could be plotted in terms of $\lambda_{c\phi}$ and β, and if $\lambda_{c\phi}$ and β were known for an existing slope which had failed, such a plot could be used to determine the cohesion value and angle of internal friction for the slope in question. However, $\lambda_{c\phi}$ cannot be determined without prior knowledge of the shear strength itself.

Proceeding, it can be noted that in addition to finding a unique value of N_{cf}, a geometrically similar and unique failure surface exists for a given set of $\lambda_{c\phi}$ and β values. For circular failure surfaces passing through the toe of the slope, it is convenient to describe the location of the critical failure surface in terms of the ratios obtained by dividing the x and y coordinates of the center of the circle (measured from the toe of the slope) by the slope height. The resulting dimensionless coordinate numbers, $\frac{Xc}{H}$ and $\frac{Yc}{H}$, respectively, will then define the center of a unique critical circle for a particular combination of slope angle (β) and $\lambda_{c\phi}$. The values of the dimensionless center coordinate numbers were determined for values of $\lambda_{c\phi}$ ranging from 0 to 100 and slope ratios (cotangent β) ranging from 1 to 4. The centers for the critical toe circles corresponding to these ranges of $\lambda_{c\phi}$ values and slope ratios are tabulated in Appendix II. These dimensionless center coordinate numbers are plotted in Fig. 4.1. A negative value of the ratio $\frac{Xc}{H}$ in this figure indicates that the center of the circle is located in a direction toward the slope from the toe, while positive values indicate that the center of the critical circle lies beyond the toe of the slope.

If the position of the failure surface can be located for an actual slide and an equivalent circular arc can be defined to approximate the observed surface, the value of $\lambda_{c\phi}$ can be estimated from Fig. 4.1. This procedure would require that the x and y coordinates for the center of the estimated critical failure surface be divided by the slope height to obtain the dimensionless coordinate numbers $\frac{Xc}{H}$ and $\frac{Yc}{H}$. The value of $\lambda_{c\phi}$ could then be obtained by interpolation once the center point for the estimated critical circle was located on Fig. 4.1. It may be noted that in this procedure the estimated center coordinate from an observed failure may not lie on the curve corresponding to the actual slope angle. This discrepancy is likely to occur due to the inaccuracies in the estimated position of a circular failure surface, the assumption of a circular failure surface and the assumption that the shear strength can be characterized in terms of a single value of c and ϕ. However,

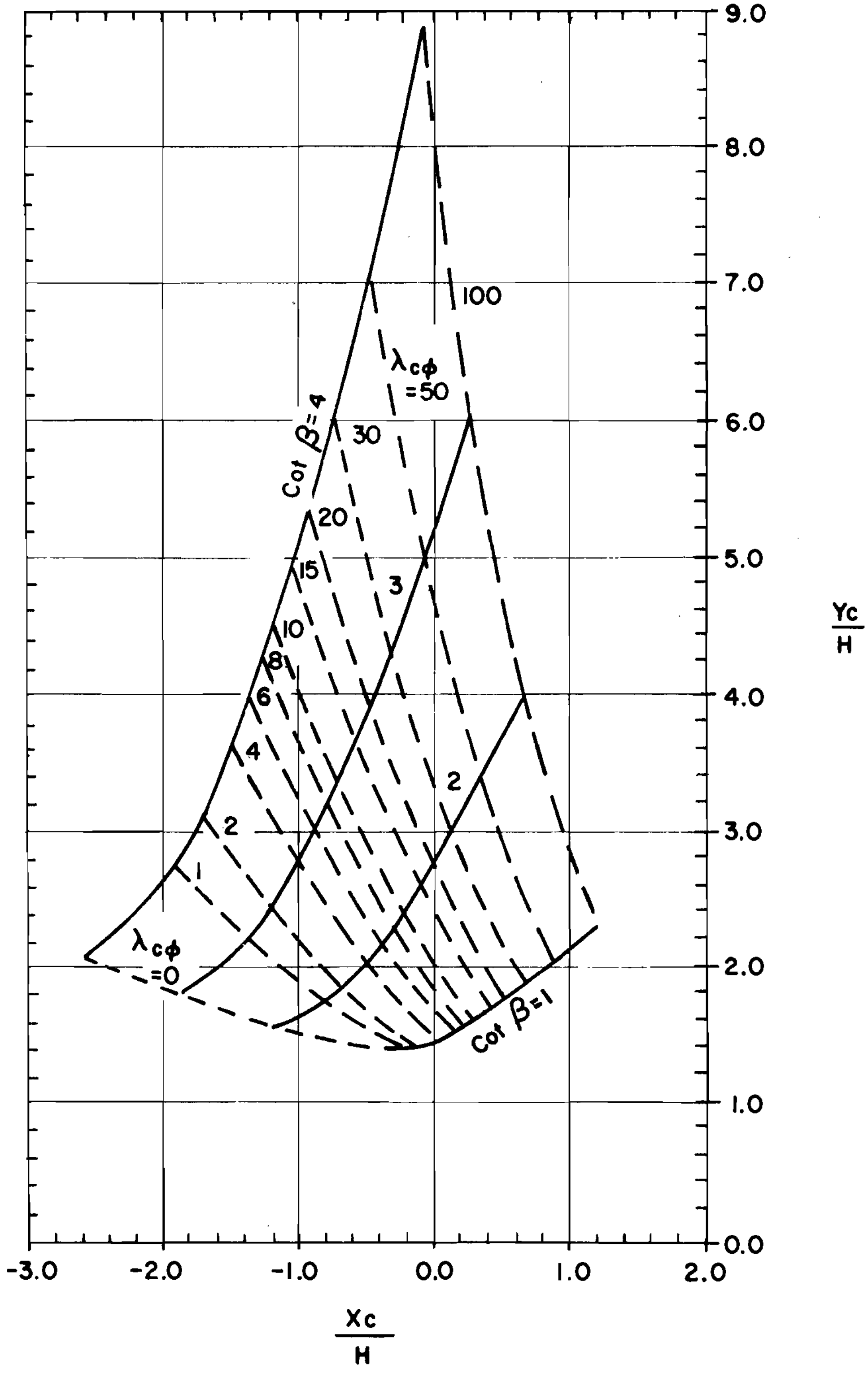


Fig. 4.1. Critical centers for toe circles with origin of coordinates at toe of slope.

errors in determining the value of $\lambda_{c\phi}$ for a particular slope failure are probably relatively insignificant inasmuch as the value of $\lambda_{c\phi}$ is only used to determine the relative balance between the value of $\frac{c}{\gamma H}$ and ϕ. Regardless of the relative balance between the values of $\frac{c}{\gamma H}$ and ϕ any set of values back-calculated from an actual slope failure will yield a factor of safety of unity, the objective of estimating $\lambda_{c\phi}$ being only to obtain a set of values which corresponds more realistically with the observed failure surface.

The above procedure for determining $\lambda_{c\phi}$ was judged to be somewhat inconvenient because of the difficulty involved in estimating and fitting a circular shear surface to an observed failure. For this reason a somewhat more convenient procedure was developed, based also upon the existence of a geometrically unique circular failure surface for a given combination of $\lambda_{c\phi}$ value and slope inclination. However, rather than characterize this surface in terms of the x and y coordinates of its center, the relative maximum depth of the critical surface was used. The depth was represented as the ratio (d/H) obtained by dividing the depth (d) of the critical circle, measured normal to the face of the slope, by the slope height. For a given slope inclination and $\lambda_{c\phi}$ value a unique value of this depth ratio (d/H) exists. Similarly, if the slope inclination and the depth ratio for the critical circle are known, the value of $\lambda_{c\phi}$ can be found. Thus, for a particular slope and depth ratio it follows that a unique set of values for c and ϕ can be found corresponding to a factor of safety of unity.

The values of $\frac{c}{\gamma H}$ and $\tan\phi$, corresponding to a factor of safety of unity, are shown in Fig. 4.2 for the range of slope ratios and $\lambda_{c\phi}$ values previously described. The solid line curves in this figure were obtained by plotting, for each slope inclination, the locus of points corresponding to the $\frac{c}{\gamma H}$ and $\tan\phi$ combinations which were obtained using different $\lambda_{c\phi}$ values. For each slope the (d/H) ratios for the corresponding critical circles were also computed and intermediate plots of the d/H ratio versus $\tan\phi$ were developed. From these plots the value of $\tan\phi$ was determined for selected values of the depth ratio and this information was used in developing the lines of constant depth ratio (broken lines) shown superimposed on Fig. 4.2.

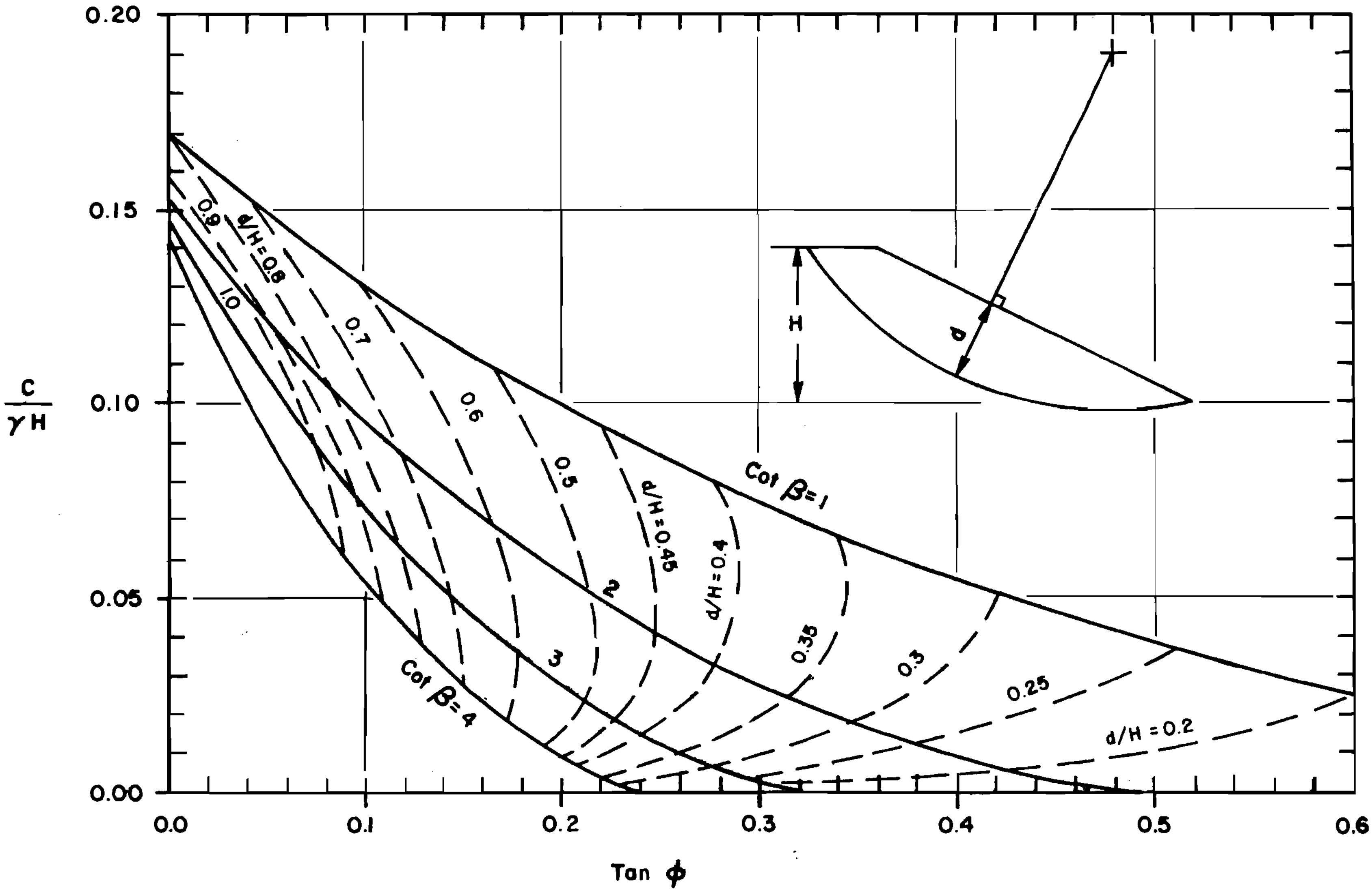


Fig. 4.2. Values of $\frac{c}{\gamma H}$ and Tan ϕ corresponding to a factor of safety of unity.

Application of Charts

The chart illustrated in Fig. 4.2 may be used to determine a value of the soil cohesion and angle of internal friction from a known slope failure. The use of the chart for this purpose can be best illustrated by considering the following example. Suppose that a failure has occurred in a 2:1, 25-foot high-slope and that the estimated depth of the slide (measured normal to the slope face) is approximately 7 feet. Further, let us assume that the unit weight of soil is approximately 110 pounds per cubic foot. The depth ratio for this slope failure can be computed as

$$\frac{d}{H} = \frac{7}{25} = 0.28$$

Referring to Fig. 4.2 and the solid curve corresponding to a 2:1 slope the following values can be obtained by interpolation between d/H ratios of 0.25 and 0.30:

$$\frac{c}{\gamma H} \cong 0.015$$

$$\tan \phi \cong 0.36$$

Thus,

$$c = 0.015 \times 110 \times 25 \cong 41 \text{ psf}$$

and

$$\phi \cong 20^{\circ}$$

The critical circular failure surface corresponding to these values can also be obtained by using Fig. 4.1. For this example, the value of $\lambda_{c\phi}$ is determined from the above information as follows:

$$\lambda_{c\phi} = \frac{\tan \phi}{c/\gamma H} = \frac{0.36}{0.015} = 24$$

Referring to Fig. 4.1 the dimensionless critical center coordinate numbers for a 2:1 slope and $\lambda_{c\phi}$ value of 24 are found to be

$$\frac{X_c}{H} \cong 0.1$$

$$\frac{Y_c}{H} \cong 2.8$$

Thus, the x and y coordinates for the center of the critical circle are

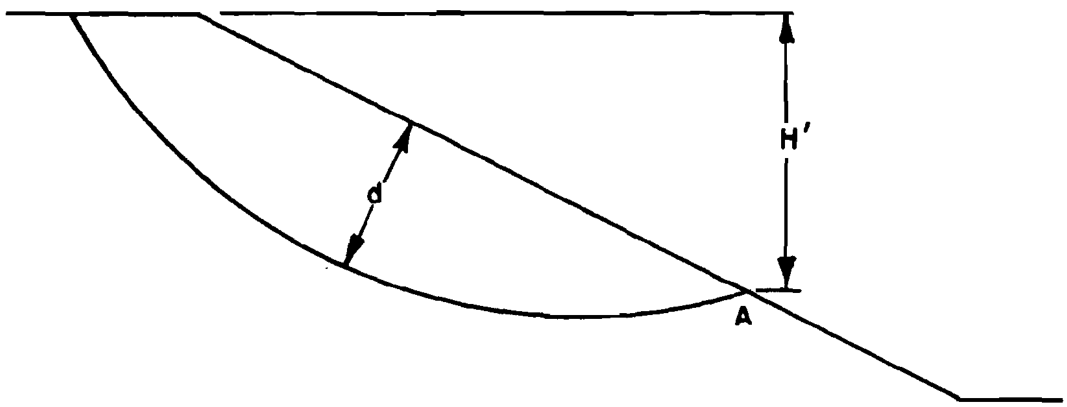
$$x = 0.1 \times 25 = 2.5 \text{ ft.}$$

$$y = 2.8 \times 25 = 70 \text{ ft.}$$

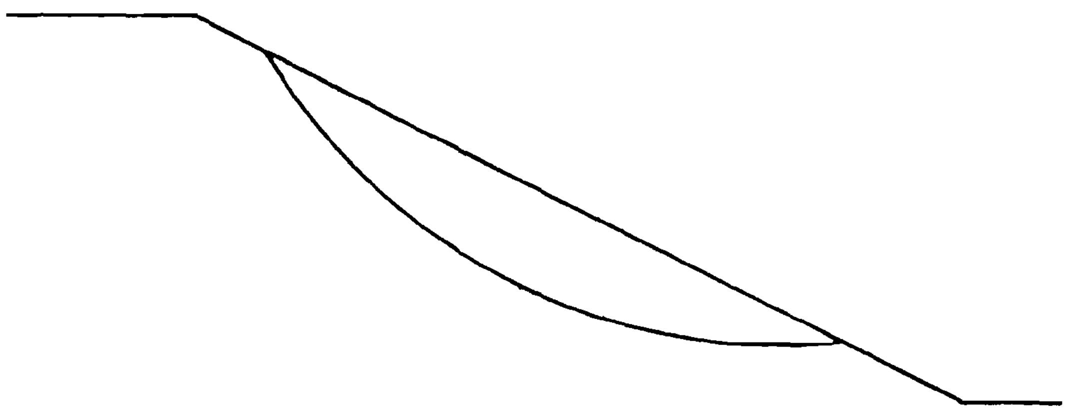
The center of the critical circle is then located at a point 70 feet above and 2.5 feet beyond the toe of the slope.

In most applications it may be useful to plot the critical circle on a cross section of the slope in order to judge the reasonableness of the assumed depth of slide (d). In many instances several depths may be assumed to establish the sensitivity of the back-calculated shear strength parameters to variations in the slide depth over an estimated range. However, it should be noted that all sets of strength values so obtained correspond to a factor of safety of unity, and, thus, significant errors are not necessarily introduced by poor estimates in the depth of slide.

In a number of instances the observed failure surface for slopes in Texas has been found not to pass through the toe of the slope, but rather through some point located above the toe, on the face of the slope. In many of these instances the charts illustrated in Figs. 4.1 and 4.2 may also be used, provided that the slide scarp intersects the crest of the slope in the manner illustrated in Fig. 4.3a. For such a case the slope height should be taken as the distance (H') shown in this figure. When the chart in Fig. 4.1 is used to locate the critical circle the coordinates obtained from this chart will then correspond to measured distances from the toe of the failure surface (Point A in Fig. 4.3) rather than the toe of the slope. In those cases where the failure surface does not intersect the crest of the slope, as illustrated in Fig. 4.3b, the charts presented in this chapter may not be entirely valid, as their application to such cases remains to be established.



(a)



(b)

Fig. 4.3. (a) Case for which charts illustrated in Figs. 4.1 and 4.2 may be used.
(b) Case for which charts are not applicable.

Summary and Conclusions

In this chapter a means for estimating soil strength parameters by back-calculation from an observed failure is presented. This procedure is probably in many instances preferable to obtaining strength values from conventional laboratory tests for several reasons:

1. The back-calculated values probably reflect to a more significant degree the influence of cracks, joints and inhomogeneities in the soil profile.
2. The errors introduced by influences of sample size, anistropy and disturbance in laboratory samples as well as the errors in laboratory tests themselves are in part eliminated.
3. The cost and time involved in an extensive sampling and testing program are virtually eliminated by the back-calculation procedure.
4. Errors which inevitably exist in the methods employed for performing stability analyses may be partially compensated for when the same analysis procedures used for re-design of a slope are also employed to back-calculate the strength values used in subsequent analyses.

While the concept of back calculating shear strengths from actual slope failures is not new, in the past back-calculation of shear strengths was restricted to determining an average shear strength expressed as either a value of cohesion with ϕ assumed equal to zero or an angle of internal friction with c assumed equal to zero (Harty, 1953; Gould, 1960; Skempton, 1964; Hutchinson, 1969). In addition, plots of c versus ϕ similar to Fig. 4.2 have been utilized to provide a convenient means to determine if shear strength parameters evaluated from laboratory tests, performed on soil samples from an actual slope failure, correspond to a factor of safety of unity (Crawford and Eden, 1967; Peterson et al., 1960; Singh, 1970). However, in instances where plots of c versus ϕ have been used in the past, the knowledge of the actual failure surface has received only limited consideration. The procedure presented in this chapter provides a means to evaluate from an actual failure a balanced combination of c and ϕ corresponding to a factor of safety of unity.

The values of the strength parameters (c and ϕ) which are determined using the chart illustrated in Fig. 4.2, are based on total stresses and no direct consideration of pore water pressures has been included. However, the strength values so determined should indirectly reflect in part the influence of pore water pressures on the soil strength. While a procedure for back calculating effective stress shear strength parameters could also be developed, the procedure would be considerably more complex and would require additional parameters for characterization of the groundwater conditions. The development of such a procedure is beyond the scope of this report and was not considered in the present study.

CHAPTER V

EARTH PRESSURE FORCES FOR RESTRAINT STRUCTURE

The correction of a number of slope failures in Texas has been accomplished by the use of concrete retaining walls located entirely within the original slope grade, as illustrated in Fig. 5.1. These structural restraint systems, locally referred to as slide suppressor walls, are generally founded on drilled shafts located approximately one-third the distance up the slope with the wall being placed so as to intersect the known or estimated failure surface. Once the slide suppressor wall has been placed, a drainage trench is excavated immediately behind the wall and backfilled with filter material. The slope is then regraded to its original grade.

The earth pressure force (E) for which the slide suppressor wall is designed is commonly determined by using either an equivalent-fluid procedure or a trial-wedge procedure. In the equivalent fluid procedure, which has been used by the Texas Highway Department, the backfill is assumed to act as a fluid. In this procedure there is no requirement that the shear strength of the soil be known other than as a possible guide in the selection of an equivalent fluid density. However, there is no rational procedure to determine the fluid density, and thus, the value selected for a given situation is based on experience and handbook recommendations. The fluid density selected in this manner may lead to either overly conservative or unconservative results.

The trial-wedge procedure, a force equilibrium method based on plane failure surfaces, is a more rational approach for estimating the maximum earth pressure (Bowles, 1968). However, unlike the equivalent-fluid procedure, the soil strength parameters must be determined either by performing laboratory tests or by back calculating the soil strengths from the slope failure. When the position of the failure surface can be located for an actual slide and an equivalent circular arc can be defined to approximate the observed surface, the total stress strength parameters (c and ϕ) may be estimated by the procedure outlined in Chapter III. The equivalent circular arc can be characterized by the depth ratio (d/H), obtained by dividing the depth (d) of the circular arc,

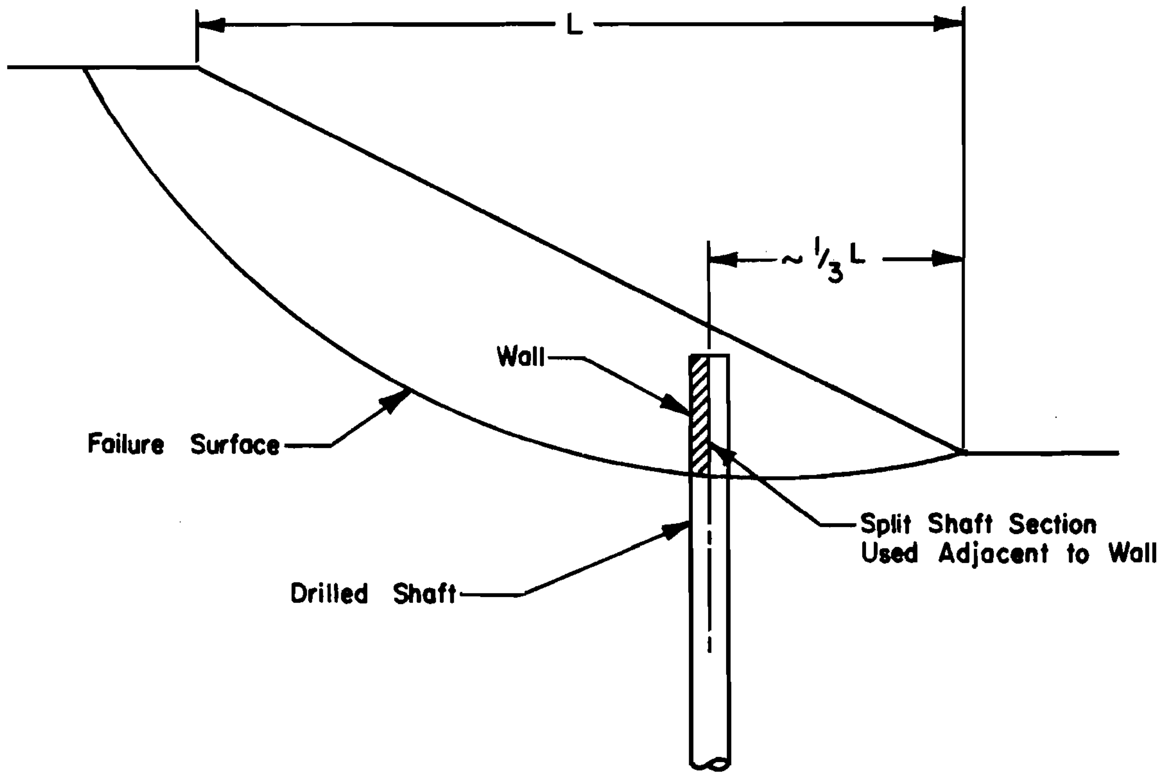


Fig. 5.1. Slope stabilized with slide suppressor wall.

measured normal to the face of the slope, by the slope height (H). The depth ratio and the inclination of the failed slope are sufficient to determine the unique values of c and ϕ, corresponding to a factor of safety of unity. These values may then be used to perform a trial-wedge analysis for estimating the earth pressure force acting on the proposed slide suppressor wall. However, while the back-calculated values of c and ϕ might be used with the trial-wedge procedure, a more rational procedure for using these values to calculate earth pressure forces would appear to be one based on the assumption of circular shear surfaces like those originally used to back-calculate the values of c and ϕ.

Procedure for Calculation of Earth Pressure Forces

The earth pressure force imposed by a material sliding along a circular failure surface can be calculated using a method-of-slices-procedure based on the assumptions made by Spencer (1967). In this procedure the side forces acting on the vertical boundaries between slices are calculated using the three basic conditions of static equilibrium. The interslice side force calculated by this procedure can be assumed to be equal to the earth pressure forces which would act on slide suppressor walls located at various positions along the failure surface. For example, if the portion of the failure mass encompassing the area ABC, shown in Fig. 5.2a, were removed, a force equal to the side force (Z_o) would have to be resisted by a slide suppressor wall. The magnitude of the earth pressure force exerted on the wall will depend on the location of the wall because the side forces vary along the failure surface as illustrated in Fig. 5.2b.

The inclined side force (Z_o) can be resolved into a horizontal component (E_o) and a vertical component (V_o). The horizontal force (E_o) will impose bending and shearing stresses in a wall. The vertical force (V_o) will tend to reduce the bending stresses in the wall; however, the reduction in the bending stresses will normally be small because the concrete walls attached to drilled shafts are only about one foot thick. For this reason the vertical component of the side force was neglected and only the horizontal force (E_o) was considered.

For a given value of $\lambda_{c\phi}$ and slope inclination β there exists a unique failure surface (d/H ratio) for which the magnitudes of the horizontal side forces depend only on their relative position along the shear surface, the

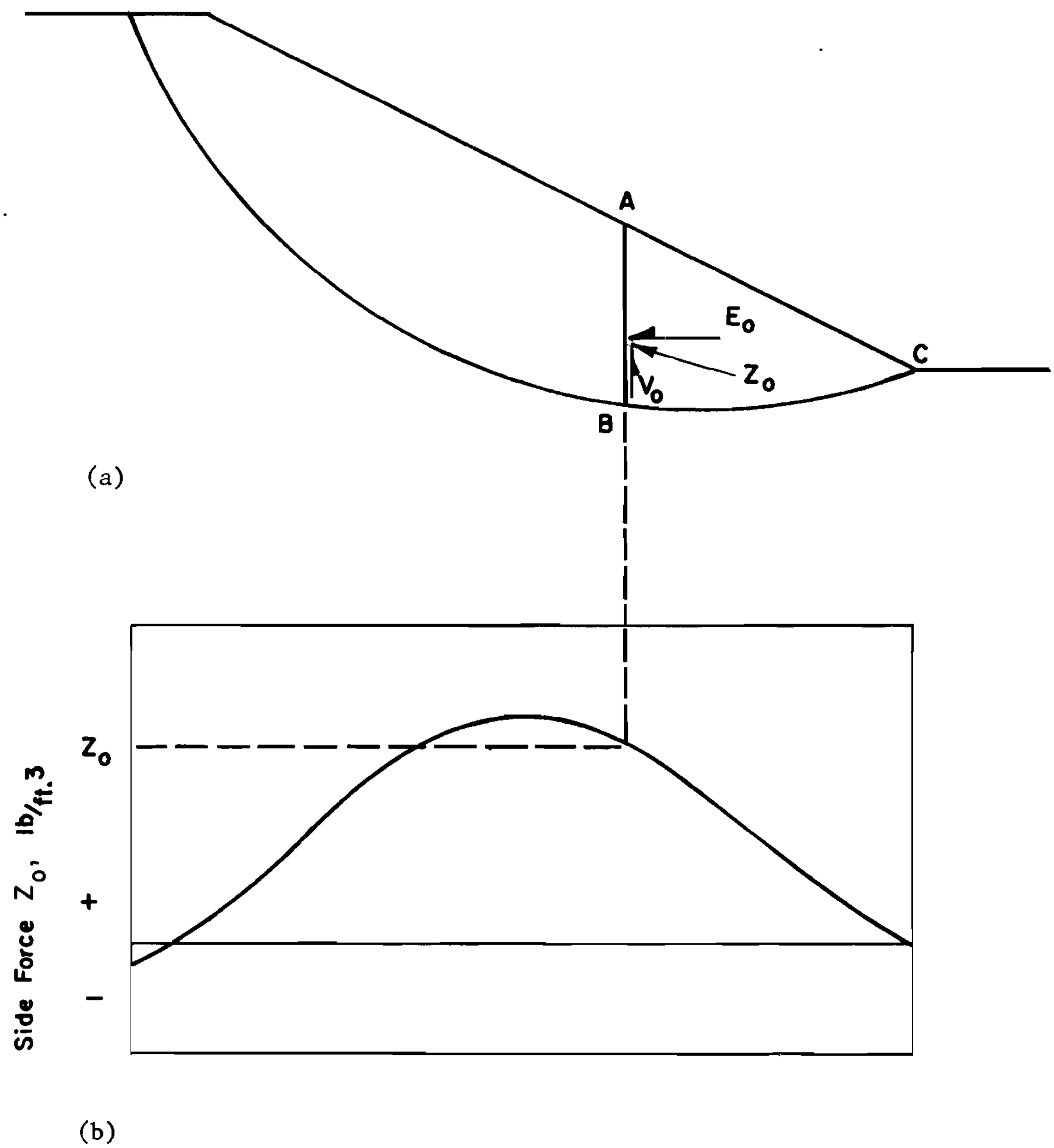


Fig. 5.2. Variation of interslice side forces along the length of the slope. (a) Location of slices. (b) Variation of interslice forces.

unit weight of the soil γ, and the height of the slope. This can best be demonstrated by an example, as follows. The shear surface illustrated in Fig. 5.3 corresponds to a value of $\lambda_{c\phi}$ equal to ten and a slope ratio (cotangent β) of 2:1. The horizontal earth pressure forces acting on planes AB, DE, and FG were calculated for various combinations of c, ϕ, γ, and H corresponding to $\lambda_{c\phi}$ values of ten. These forces (E) and the combinations of parameters used for their calculation are summarized in Table 5.1. By dividing each of these forces by the product of the unit weight of soil and the slope height squared (γH^2) a set of unique dimensionless earth pressure coefficients (N_p) were derived; these values are summarized in the last three columns of Table 5.1.

The coefficients, obtained in the above manner, depend only on the relative position of the earth pressure force along the shear surface (L/4, L/3, and L/2). For every $\lambda_{c\phi}$ value (or d/H ratio) and slope inclination a similar set of earth pressure coefficients (N_p) can be derived, each set corresponding to various positions of the earth pressure force along the failure surface. Tables or charts of these dimensionless coefficients developed for a wide range of d/H ratios, slope inclinations, and slide suppressor wall locations would provide a convenient means for estimating from an observed failure surface an earth pressure force which would act on a slide suppressor wall. In the remainder of this chapter earth pressure coefficient charts will be developed and presented; in addition, the use of earth pressure coefficients to calculate earth pressure forces will be compared with other conventional procedures used for this purpose.

<u>Non-Dimensional Coefficients</u>

Non-dimensional earth pressure coefficients (N_p) were determined for critical circles corresponding to a range of slope conditions. For convenience, the most critical failure surface was in all instances assumed to be a circular arc passing through the toe of the slope. An available computer program employing Spencer's method-of-slices procedure was utilized to perform the analyses (Wright, 1971). For each critical failure surface analyzed, earth pressure coefficients were determined for a wall one-fourth, one-third, and one-half the distance up the slope, as illustrated in Fig. 5.3. Values of N_p were calculated from critical circles corresponding to values of $\lambda_{c\phi}$ ranging from 0 to 100 and slope ratios of 2:1 and 3:1. Depth ratios (d/H) were found to provide a convenient means for characterizing the critical failure circles

Table 5.1 Earth Pressure Forces (E) and N_p values for $\lambda_{c\phi} = 10$ and 2:1 slope.

c, lb./ft.^{2}	ϕ, Degrees	γ, lb./ft.^{3}	H, ft.	F.S.[1]	E, lb./ft. L/4	E, lb./ft. L/3	E, lb./ft. L/2	N_p L/4	N_p L/3	N_p L/2
1000.0	45.0	100	100	3.40	362,000	486,200	631,800	.3620	.4862	.6318
182.0	20.0	100	50	1.24	90,500	121,550	157,950	.3620	.4862	.6318
29.4	16.4	50	20	1.00	7,240	9,724	12,636	.3620	.4862	.6318

[1]Note values of N_p are independent of the Factor of Safety

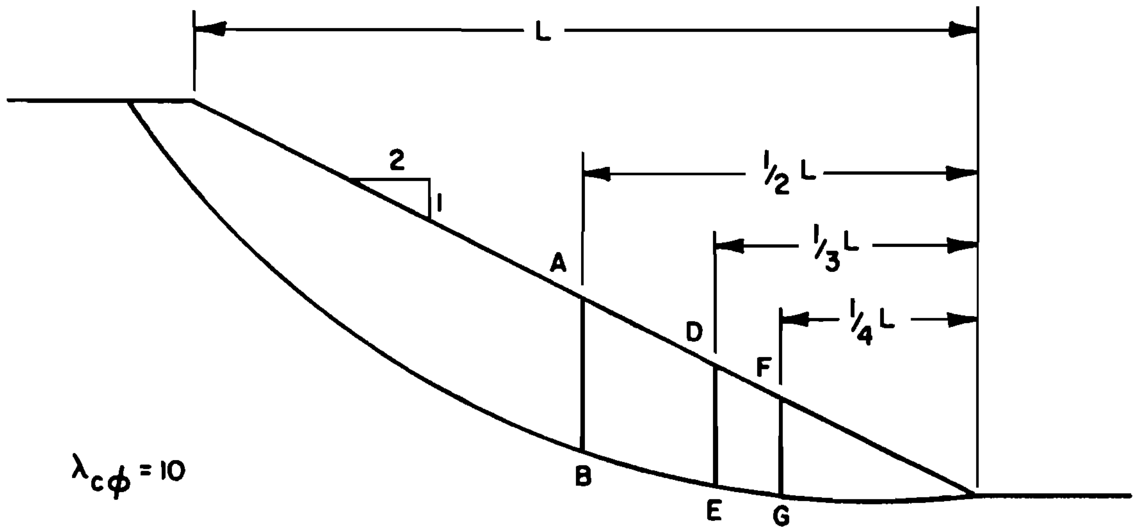


Fig. 5.3. Failure surface for $\lambda_{c\phi}$ = 10 and 2:1 slope.

for each slope inclination and the corresponding earth pressure coefficients. The N_p values evaluated for the three relative locations of the earth pressure force are tabulated in Appendix III.

Influence of Soil Profile Characterizations

The analyses performed to evaluate the non-dimensional earth pressure coefficients were total stress analyses in which the shear strength parameters were assumed to be defined by values of c and ϕ. However, for a given critical circle, other total stress shear strength characterizations may give values of earth pressure coefficients which differ from N_p values based on total stress values of c and ϕ. To determine how the earth pressure coefficients may be influenced by different characterizations of the soil strength profile, the earth pressure coefficients obtained using a c and ϕ soil profile characterization were compared with earth pressure coefficients determined for the following two cases:

Case I: The shear strength of the slope was assumed to have an initial value (c_o) at the crest of the slope and to increase linearly with depth, as depicted in Fig. 5.4a.

Case II: The shear strength of the slope was assumed to have the same value (c_o) at any point along the slope surface with the shear strength increasing linearly below the slope surface. See Fig. 5.4b.

The depth ratios (d/H) calculated from the failure surfaces corresponding to Case I and Case II can be demonstrated to be uniquely related to the slope inclination and the dimensionless parameter, λ_{cz}, expressed as

$$\lambda_{cz} = \frac{c_o}{H\psi_z}$$

where c_o is the initial value of strength, ψ_z is the rate of increase in strength with depth, and H is the height of the slope. However, as previously noted, the values of c_o and ψ_z have somewhat different interpretations for the two cases under consideration, and, thus, for a given λ_{cz} and slope inclination these cases will have different d/H ratios. For Case I, earth pressure coefficients were determined for critical circles corresponding to λ_{cz} values of 0.005, 0.02, 0.05, and 1.5, and for Case II, earth pressure coefficients were determined for λ_{cz} values of 0.01, 0.1, and 1.0. Slope ratios of 2:1 and 3:1 were used for both cases. For Case I, the critical circles for values of λ_{cz}

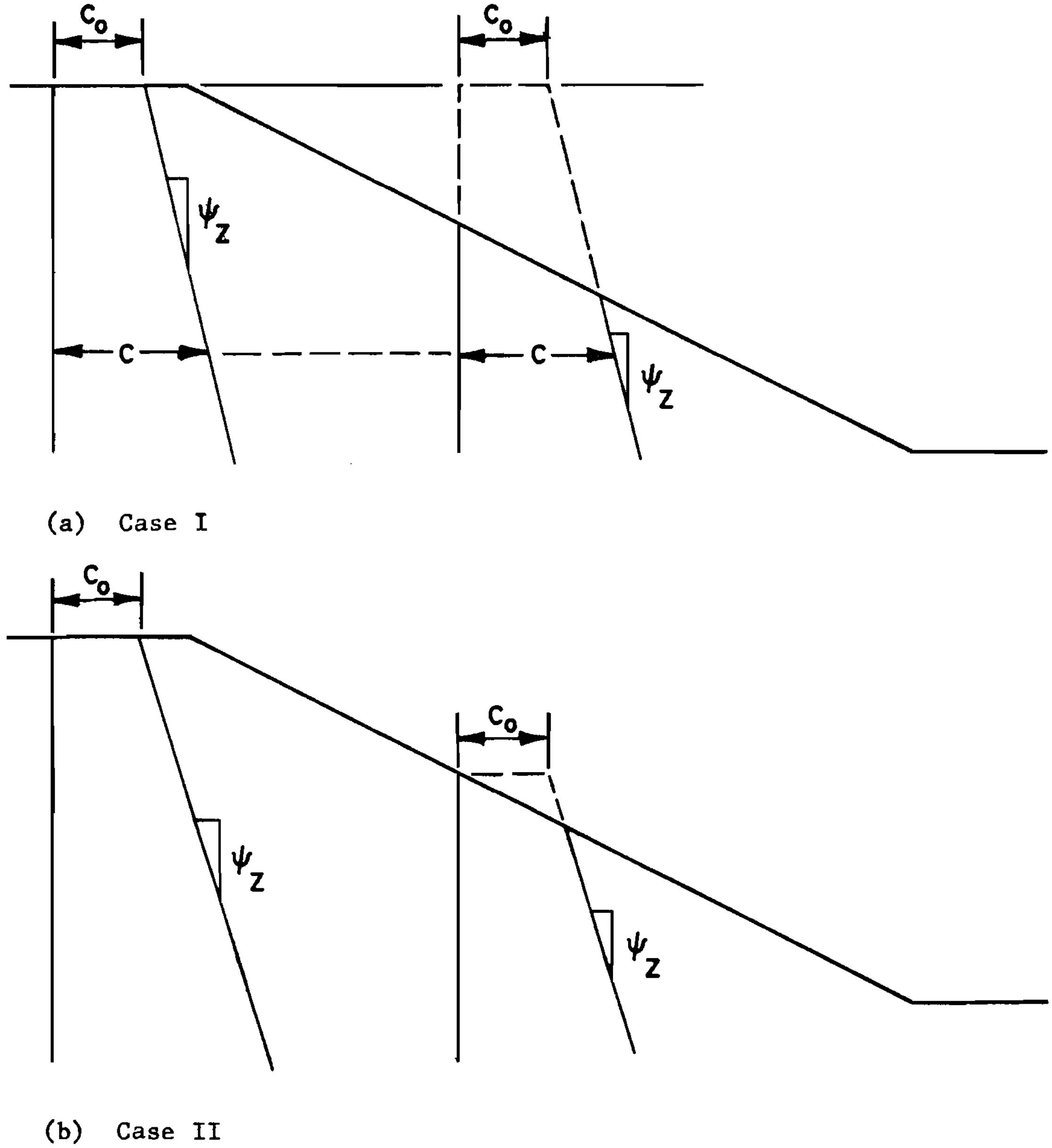


Fig. 5.4. Total stress shear strength characterizations of the soil strength profile.

less than 0.05 were found to be essentially identical and therefore, the range of values for the earth pressure coefficients was limited on one end to values corresponding to a value of λ_{cz} equal to 0.05. The earth pressure coefficients and depth ratios corresponding to the critical circles analyzed for Case I and Case II are plotted in Figs. 5.5 and 5.6. It can be noted from these figures that the earth pressure coefficients corresponding to the strength variation assumed for Case I are greater than the values of N_p based on a c-ϕ soil characterization, while the earth pressure coefficients corresponding to Case II are less than the coefficients determined for the c-ϕ soil.

Total stress analyses performed using the shear strength variations assumed for Case I and Case II resulted in earth pressure coefficients which differed from N_p values evaluated using a c and ϕ soil characterization. The use of a c and ϕ strength characterization in an effective stress analysis also resulted in earth pressure coefficients which differed from those determined from a total stress analysis using a c-ϕ soil characterization. Effective stress analyses were performed to determine the magnitude of the differences between the earth pressure coefficients determined from effective total stress approaches. The pore water pressure distribution used in the effective stress analyses was characterized in the following two ways:

Case III: The pore water pressure was assumed to be equal to some constant fraction (r_u) of the vertical overburden pressure at all points in the soil profile.

Case IV: The pore water pressure was expressed in terms of a piezometric surface, inclined toward the toe of the slope as shown in Fig. 5.7.

For Case III it may be shown that for a given value of $\lambda_{c\phi}$, slope inclination, and r_u a unique critical circle and corresponding earth pressure coefficients exist. Earth pressure coefficients were evaluated for 2:1 and 3:1 slopes, $\lambda_{c\phi}$ values of 4, 20, and 100, and r_u values of 0.4 and 0.6. The value of r_u equal to 0.6 represents an extremely high pore water pressure distribution. These earth pressure coefficients for Case III plotted in Figs. 5.8 and 5.9 were found to be larger than the values determined from the total stress, c-ϕ analyses. The maximum difference between the effective and total stress analyses occurred for high values of $\lambda_{c\phi}$ and r_u: for r_u = 0.6 and $\lambda_{c\phi}$ = 100 the difference was about 18 percent.

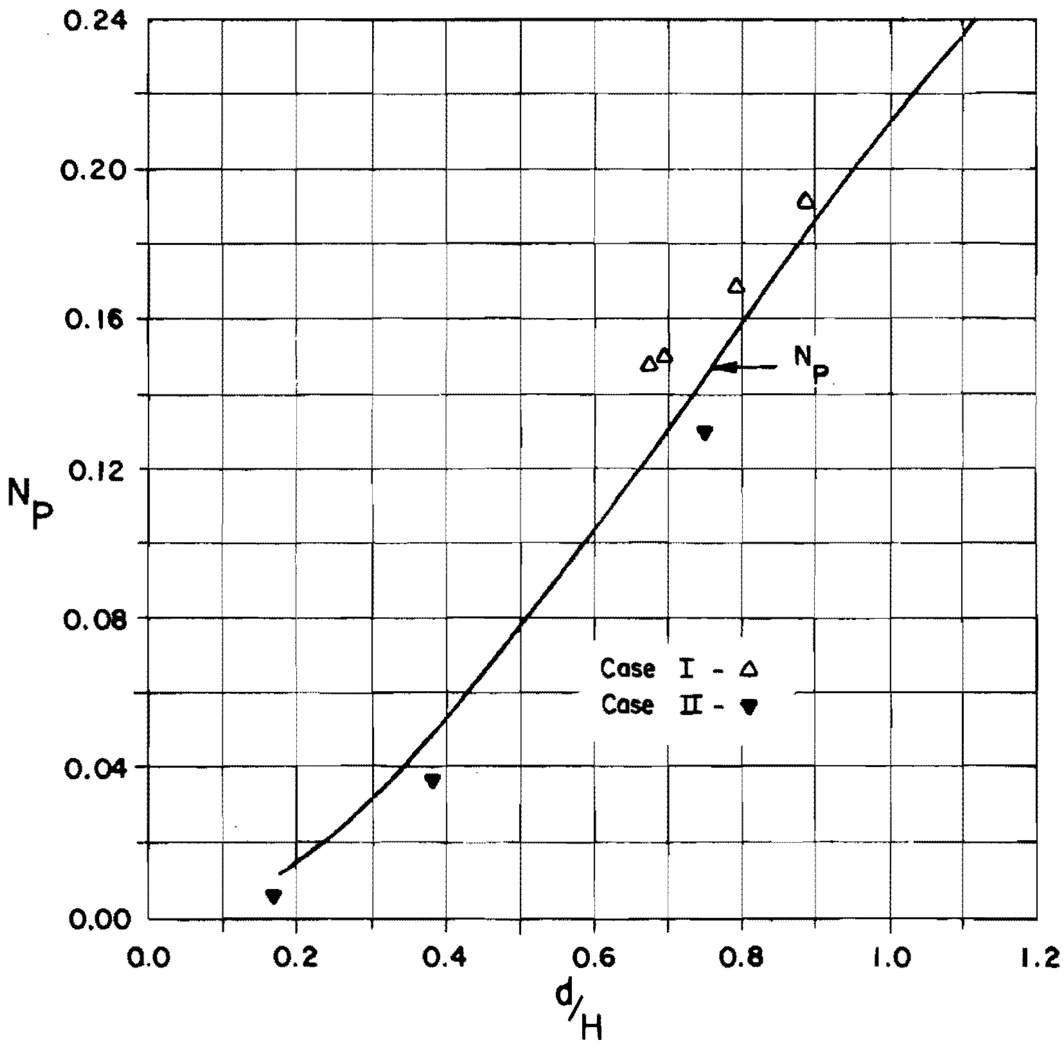


Fig. 5.5. Comparison of Case I and II earth pressure coefficients with values of N_p evaluated for a 2:1 slope at L/3.

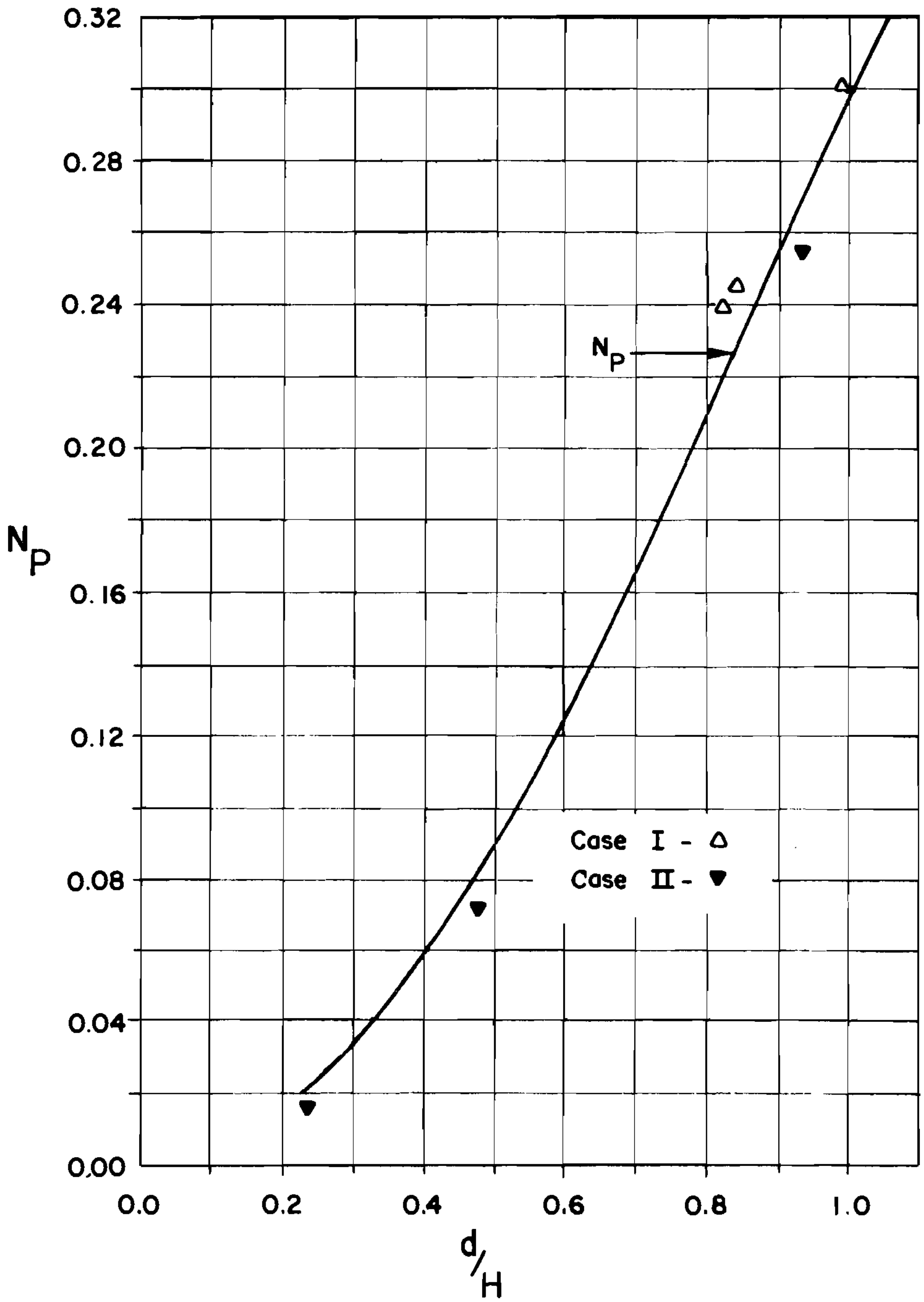


Fig. 5.6. Comparison of Case I and II earth pressure coefficients with values of N_p evaluated for a 3:1 slope at L/3.

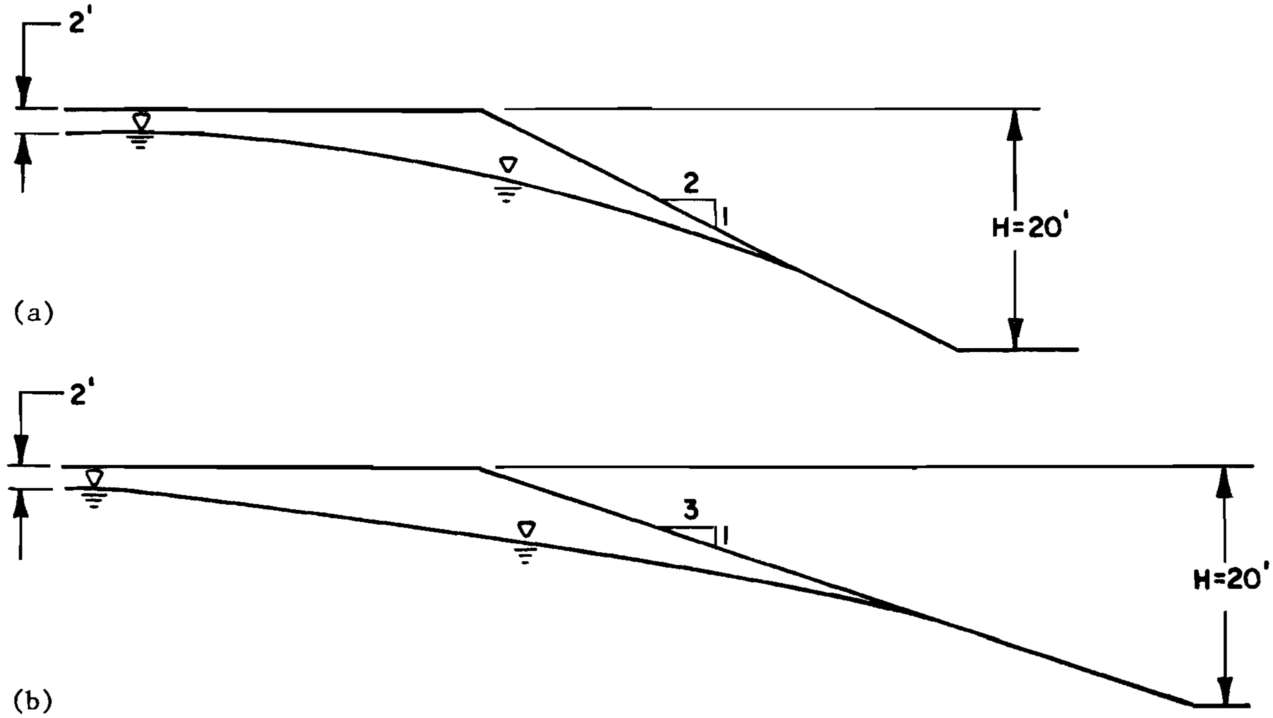


Fig. 5.7. Piezometric lines used in effective stress analyses for (a) 2:1 and (b) 3:1 slopes.

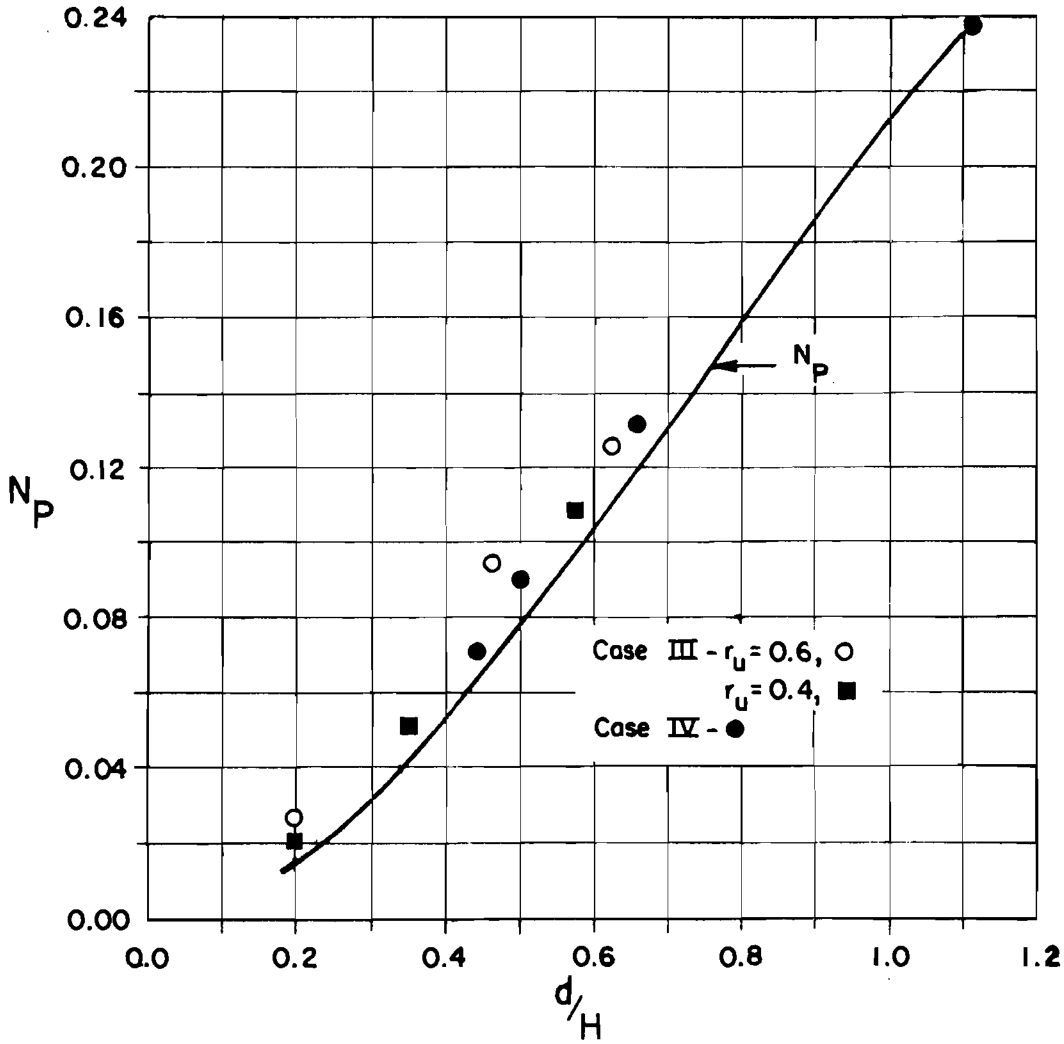


Fig. 5.8. Comparison of Case III and IV earth pressure coefficients with values of N_p evaluated for a 2:1 slope at L/3.

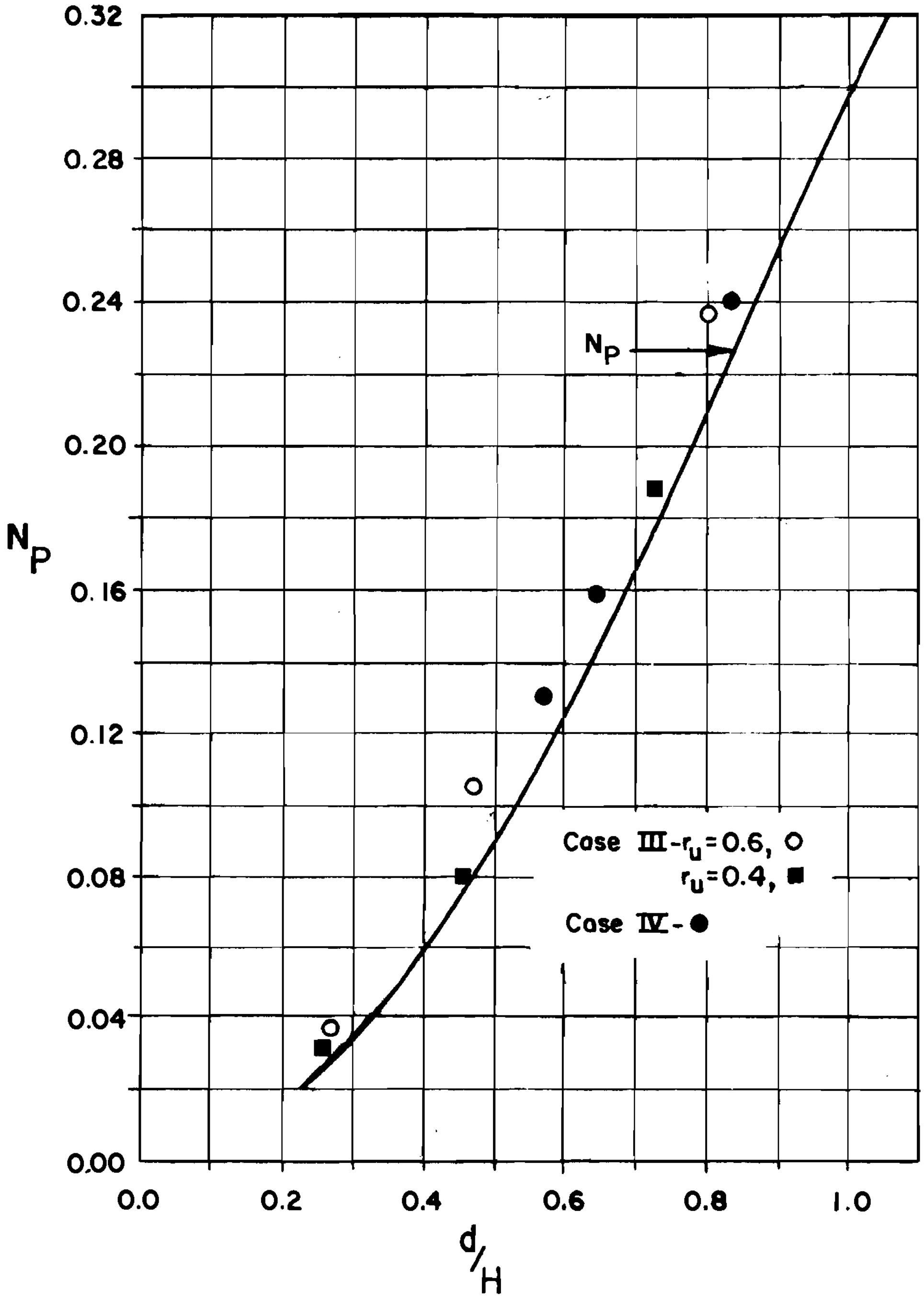


Fig. 5.9. Comparison of Case III and IV earth pressure coefficients with values of N_p evaluated for a 3:1 slope at L/3.

The piezometric lines used in the comparative analyses for Case IV were developed using the Schaffernak-Iterson procedure presented by Casagrande (1937). Earth pressure coefficients were determined for a 2:1 and 3:1, 20-foot-high slope and for the relatively high piezometric lines illustrated in Fig. 5.7. The two slope geometries were analyzed for the following soil strengths:

(1) $c = 300$ lb/ft^2; $\phi = 31°$ ($\lambda_{c\phi} = 4$)
(2) $c = 120$ lb/ft^2; $\phi = 31°$ ($\lambda_{c\phi} = 10$)
(3) $c = 80$ lb/ft^2; $\phi = 31°$ ($\lambda_{c\phi} = 15$)

For values of $\lambda_{c\phi}$ greater than 15, the upper portion of the failure surface intersected the slope face, a condition not taken into account by the total stress analyses for N_p values. Thus, this condition was excluded from consideration. The earth pressure coefficients determined from the Case IV effective stress analyses, shown in Figs. 5.8 and 5.9, were from eleven to twelve percent greater than the values determined from the total stress, c-ϕ analyses.

These effective stress analyses for Cases III and IV suggest that earth pressure coefficients determined from total stress, c-ϕ analyses may be unconservative. While the effective stress analyses give higher earth pressure coefficients, the development of charts utilizing effective stress earth pressure coefficients is hindered by the difficulties associated with characterizing the pore water pressures in the slope. Therefore, in order to develop earth pressure coefficient charts which encompass the coefficients previously determined for Cases III and IV, values of N_p determined from total stress, c-ϕ analyses were modified.

Modification of Earth Pressure Coefficients

To increase the magnitude of the N_p values, the failure surfaces corresponding to total stress-shear strength parameters c and ϕ were reanalyzed to evaluate modified earth pressure coefficients (N_p'). These modified earth pressure coefficients (N_p') were evaluated in the following manner.

First, a critical circle was determined for given values of $\lambda_{c\phi}$ and slope inclination. The slope was then assumed to be represented by a surface such as the surface ABCDF shown in Fig. 5.10. The distance CD in this figure corresponds to the depth of the failure surface one-fourth of the distance up the slope. Next, a force equilibrium procedure of slices, in which the side forces (Z) were assumed to act horizontally, was used to calculate the force

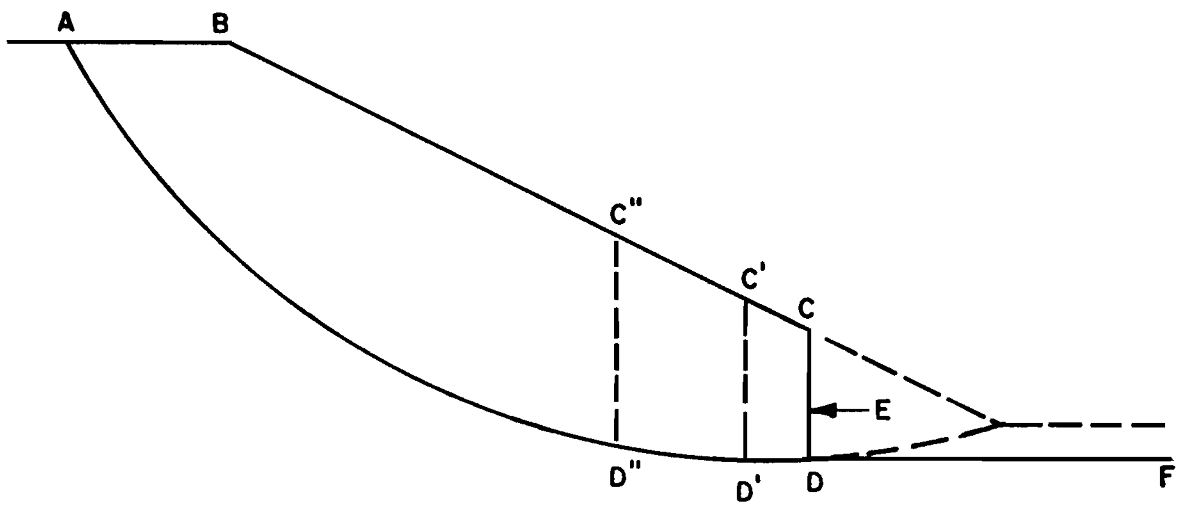


Fig. 5.10. Typical slope cross section used to evaluate values of N_p'.

(E) required to maintain the stability of the soil mass ABCD bounded by the circular surface AD. Using the same critical circle, the forces required for equilibrium of the soil masses ABC'D' and ABC"D" were also calculated. The distances C'D' and C"D" correspond to the depth of the failure surface one-third and one-half the distance up the slope, respectively.

Values of N_p' were determined for critical toe circles corresponding to values of $\lambda_{c\phi}$ ranging from 0 to 100 and slope ratios (cotangent β) of 2:1 and 3:1. Values of N_p' determined at a point one third of the distance up the slope are plotted in Figs. 5.11 and 5.12 for comparison with previously determined earth pressure coefficients. It can be seen from these figures that the N_p' values encompass all of the previously determined earth pressure coefficients with the exception of the earth pressure coefficients determined for Case III with $r_u = 0.6$. However, this high value of r_u represents an extreme and unlikely pore water pressure condition and, thus, appears to be of little practical interest. Values of N_p' are plotted in Figs. 5.13, 5.14 and 5.15 for three locations (L/4, L/3, and L/2) of the earth pressure force, the range of $\lambda_{c\phi}$ values, and slope ratios described previously.

Point of Application of Earth Pressure Forces

The earth pressure forces calculated using Spencer's procedure of analysis and total stress shear strength parameters c and ϕ were found to act near the lower quarter point of the slice boundaries for low values of $\lambda_{c\phi}$ and near the lower third point for high values of $\lambda_{c\phi}$. The positions of these forces determined at one-fourth, one-third, and one-half the distance up a 2:1 and 3:1 slope are summarized in Appendix IV. Additional analyses using Spencer's procedure indicated that the formation of a tension crack will tend to move the point of application of the earth pressure to a higher elevation. Because of the influence of the formation of a crack on the location of an earth pressure force and the uncertainty in predicting the earth pressure distribution in an actual slope, it is recommended for design that the earth pressure force be assumed to act at the mid-point of a wall.

Application of Charts

The charts presented in Figs. 5.13, 5.14, and 5.15 may be used to determine the earth pressure forces for the design of a slide suppressor wall, provided the wall is to be installed within the original slope grade and is positioned to intercept the failure mass at the known or estimated rupture

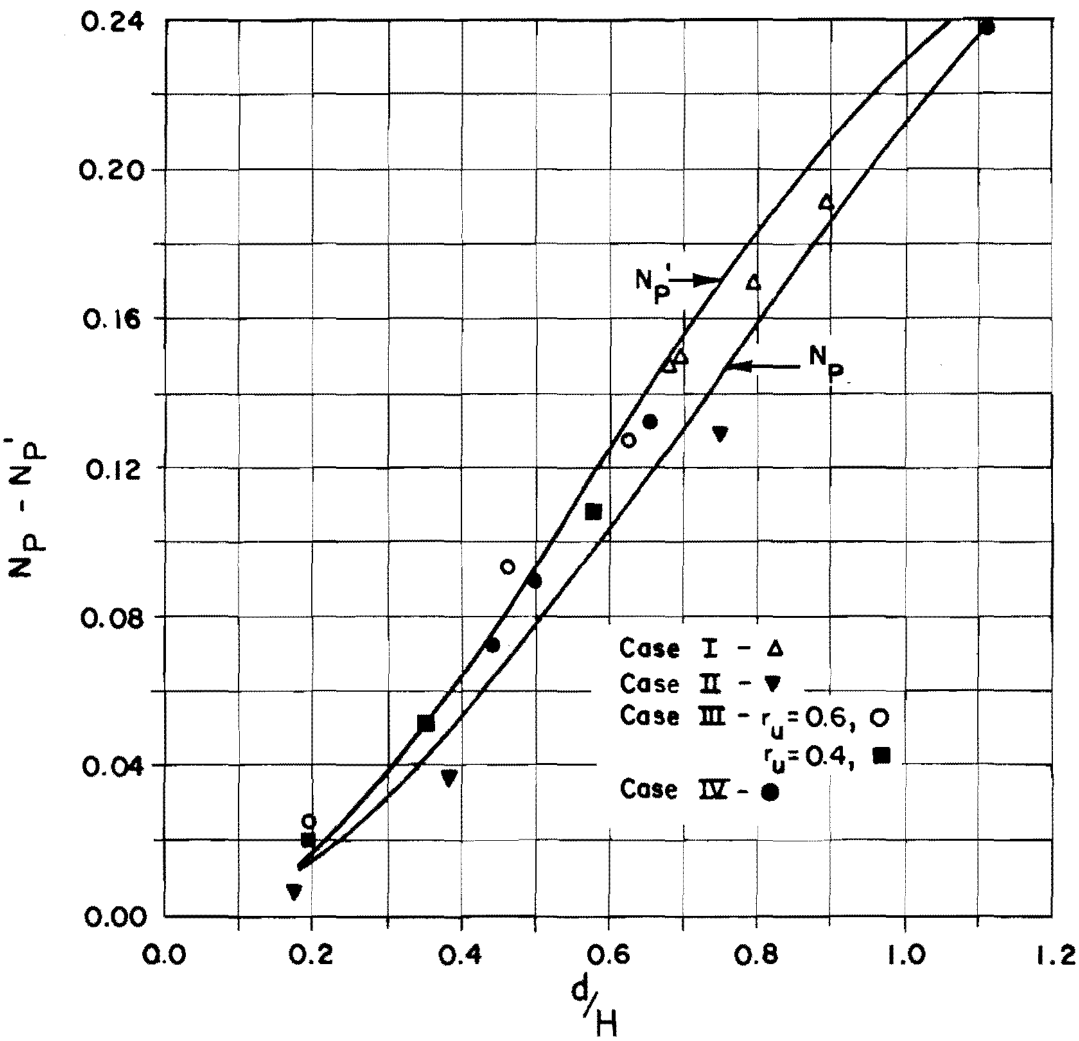


Fig. 5.11. Comparison of earth pressure coefficients determined from total stress and effective stress analyses with values of N_p' for a slide suppressor wall installed one-third of the distance up a 2:1 slope.

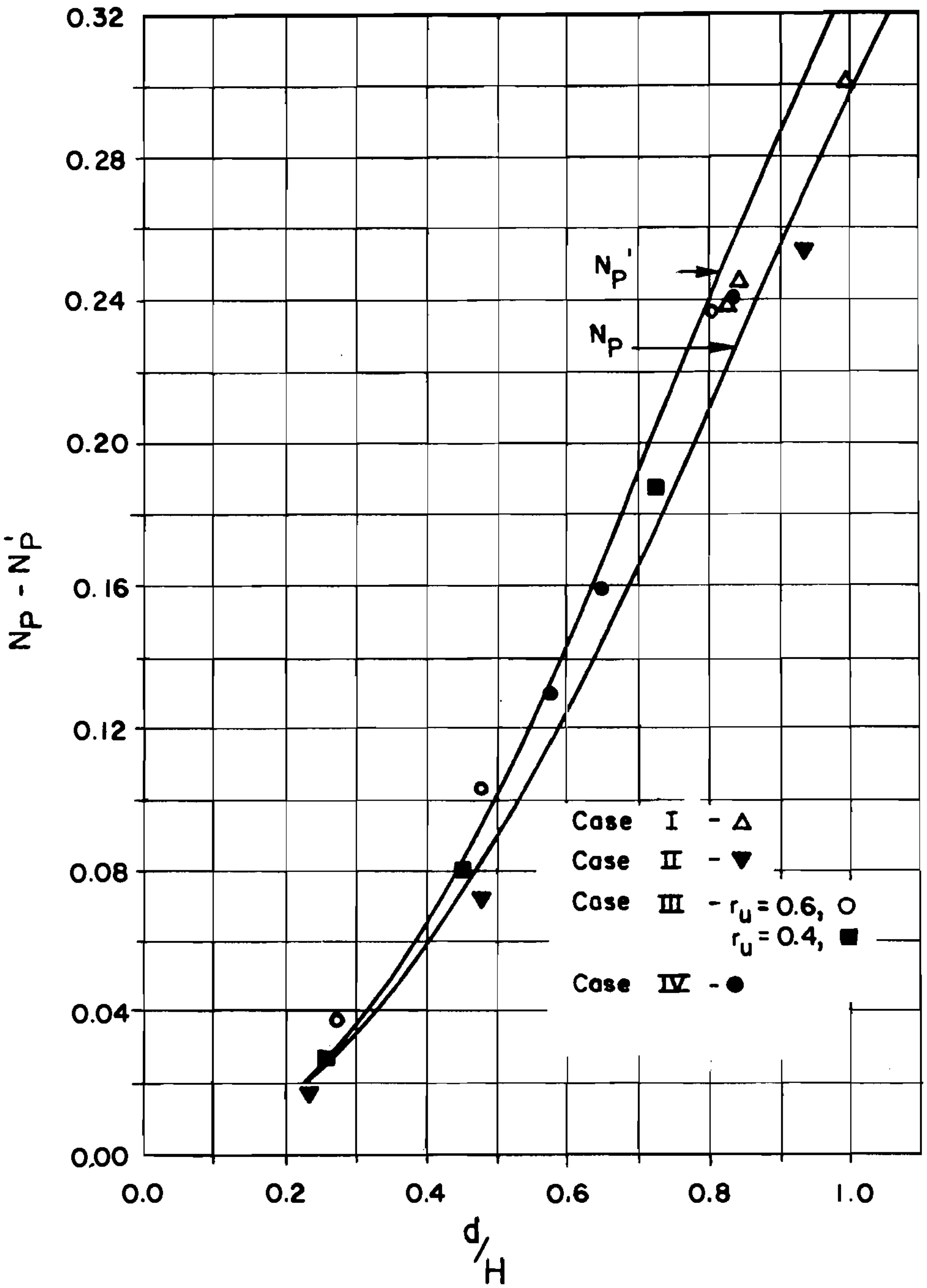
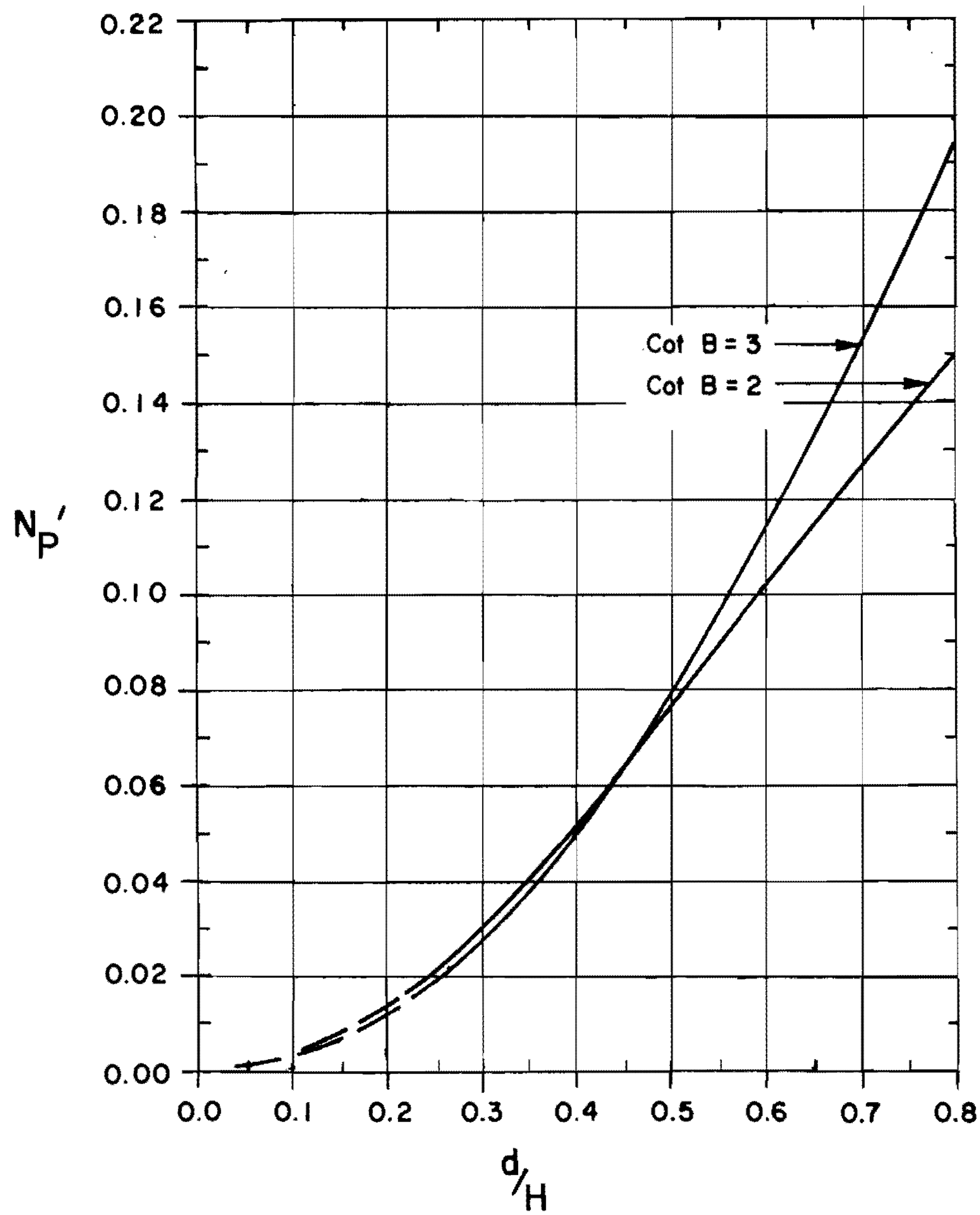
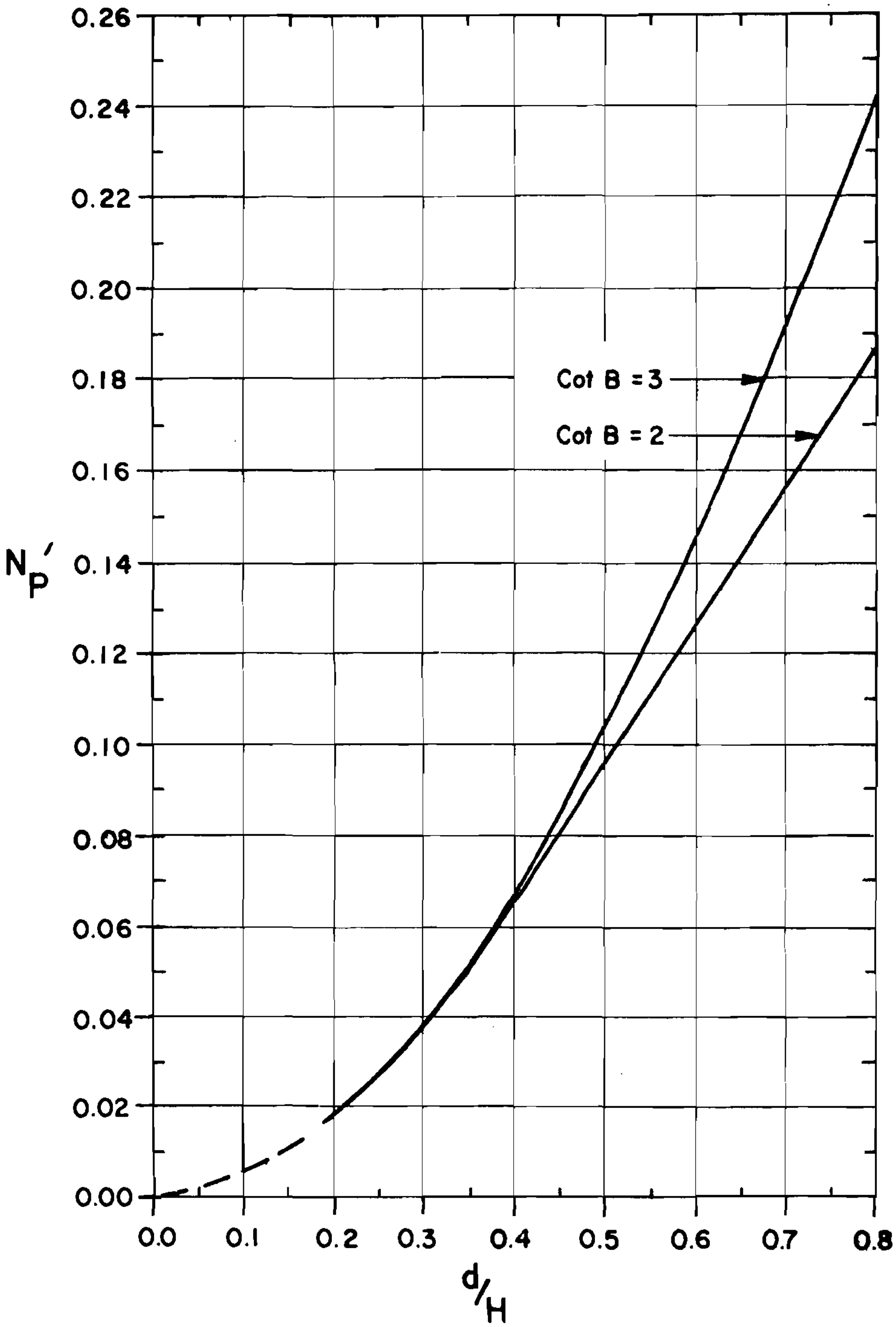


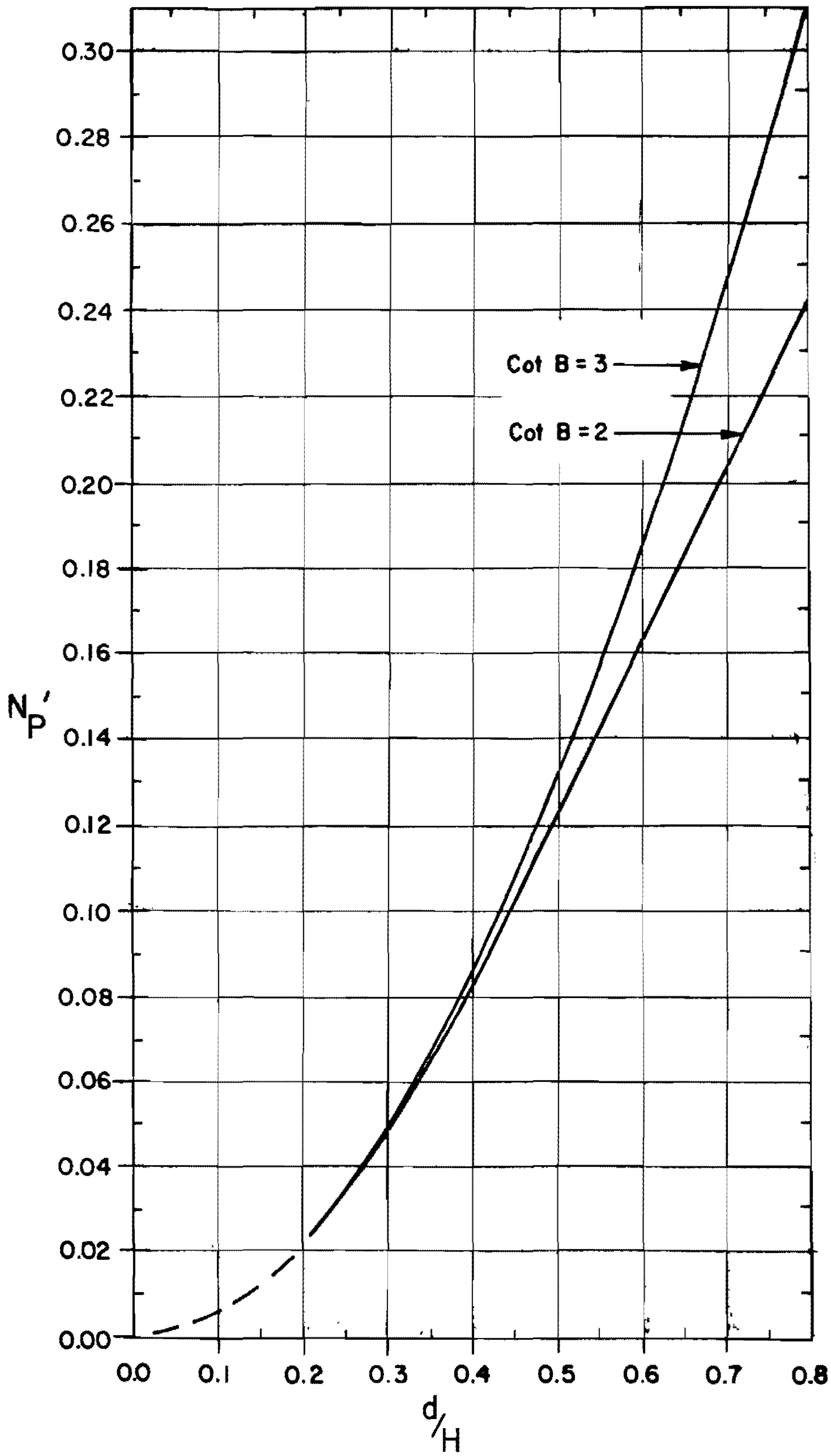
Fig. 5.12. Comparison of earth pressure coefficients determined from total stress and effective stress analyses with values of N_p' for a slide suppressor wall installed one-third the distance up a 3:1 slope.



5.13. N_p' values for slide suppressor wall placed one-fourth of the distance up a 2:1 or 3:1 slope.



5.14. N_p' values for slide suppressor wall placed one-third the distance up a 2:1 or 3:1 slope.



5.15. N_p' values for slide suppressor wall installed one-half of the distance up a 2:1 or 3:1 slope.

surface, as shown in Fig. 5.1. The use of the charts for this purpose can be illustrated by considering as an example the previously discussed slide which occurred on I.H. 35 beneath the Moore Street crossover bridge. A slide suppressor wall was used at this site to correct the failure which occurred in the 23-foot-high, 2 1/2:1 slope as shown in Fig. 5.16. The average unit weight of the soil at this site was approximately 122 pcf. In this instance an assumption of a circular failure surface appeared to be reasonable, and the maximum depth of this surface, measured normal to the slope face, was estimated to be between 6 and 7 feet.

The failure surface of this slide did not pass through the toe of the slope. However, in cases where the scarp of the failure surface intersects the crest of the slope the earth pressure coefficient charts may still be used provided the height of the slope is taken to be H' and the horizontal length of the slope is taken to be L', as shown in Fig. 5.16. The modified height (H') and length (L') for this example are 21 feet and 52.5 feet respectively.

The depth ratios corresponding to the 6-foot and 7-foot estimated depths of the failure surface are

$$\frac{d}{H} = \frac{6}{21} = 0.29$$

and

$$\frac{d}{H} = \frac{7}{21} = 0.33$$

Referring to Fig. 5.14, the following earth pressure coefficients may be obtained:

$$\frac{d}{H} = 0.29; \quad N_p' = 0.035$$

$$\frac{d}{H} = 0.33; \quad N_p' = 0.045$$

Therefore, the earth pressure forces corresponding to the two depth ratios, 0.29 and 0.33 respectively, would be

$$E = 122 \cdot (21)^2 \cdot 0.035 = 1880 \text{ lb/ft}$$

and

$$E = 122 \cdot (21)^2 \cdot 0.045 = 2420 \text{ lb/ft}$$

The units of the earth pressure force are pounds per lineal foot of wall. It is interesting to note that a small change in the depth of the failure surface results in a relatively large increase in the earth pressure force. In this example, a 17 percent increase in the depth of the slip surface resulted in a

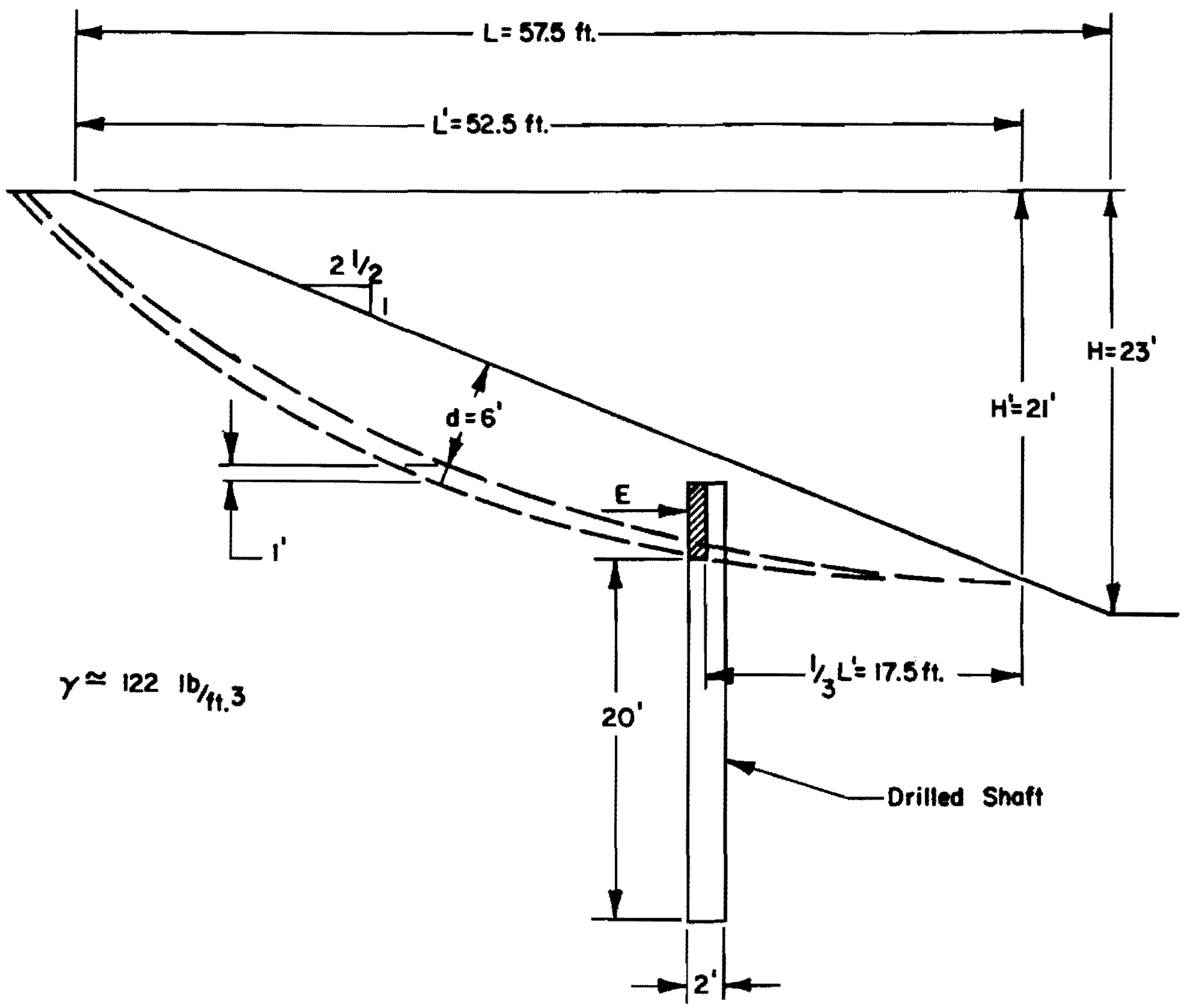


Fig. 5.16. Simplified cross section of the slope on I.H. 35 under the Moore Street Bridge.

29 percent increase in the earth pressure force.

In the example being considered, the earth pressure coefficients were obtained from Fig. 5.14 because the wall was located approximately one-third the distance up the slope. The earth pressure forces may also by determined for other locations of the wall using the appropriate chart from Fig. 5.13, 5.14, or 5.15. Earth pressure coefficients for intermediate wall locations or slope inclination between 2:1 and 3:1 may be determined by linear interpolation between the appropriate figures.

Comparison of Earth Pressure Coefficient Procedure with Conventional Procedures

The earth pressure forces for which slide suppressor walls are designed have been conventionally determined by using either an equivalent-fluid procedure or a trial-wedge procedure. While the equivalent-fluid procedure is not directly applicable to walls with sloping ground surfaces, the Texas Highway Department has extended the equivalent-fluid procedure to the case of a wall with a sloping backfill by making a series of simplifying assumptions. In this modified procedure, the equivalent-fluid pressure is first calculated on plane ABC as indicated in Fig. 5.17. The fluid pressure acting on the surface BC is then assumed to be the earth pressure acting on the wall. The Texas Highway Department has generally used approximately the minimum fluid density recommended by AASHO (1969), 36 pcf, for design of slide suppressor walls. Using the modified equivalent-fluid procedure and assuming a fluid density of 36 pcf, the earth pressure force acting on the slide suppressor wall in Fig. 5.16 would be 2290 pounds per foot. This value compares favorably with the earth pressure force calculated for the same wall using the earth pressure coefficient procedure. Additional analyses, summarized in Table 5.2, were made comparing the modified equivalent-fluid procedure with the earth pressure coefficient procedure. For heights of slide suppressor walls approximately equal to the depth of the failure surfaces, soil unit weights of 100 to 120 pcf, and a fluid density of 36 pcf, these analyses indicate that in certain instances, the equivalent-fluid procedure may lead to unconservative results.

Analyses were also performed to compare the horizontal earth pressure forces predicted by the trial-wedge and earth pressure coefficient procedures. The earth pressure forces which are compared are those forces which would act on a slide suppressor wall placed one-third of the distance up the slope, as illustrated in Fig. 5.18. In order to perform the trial-wedge analyses, the

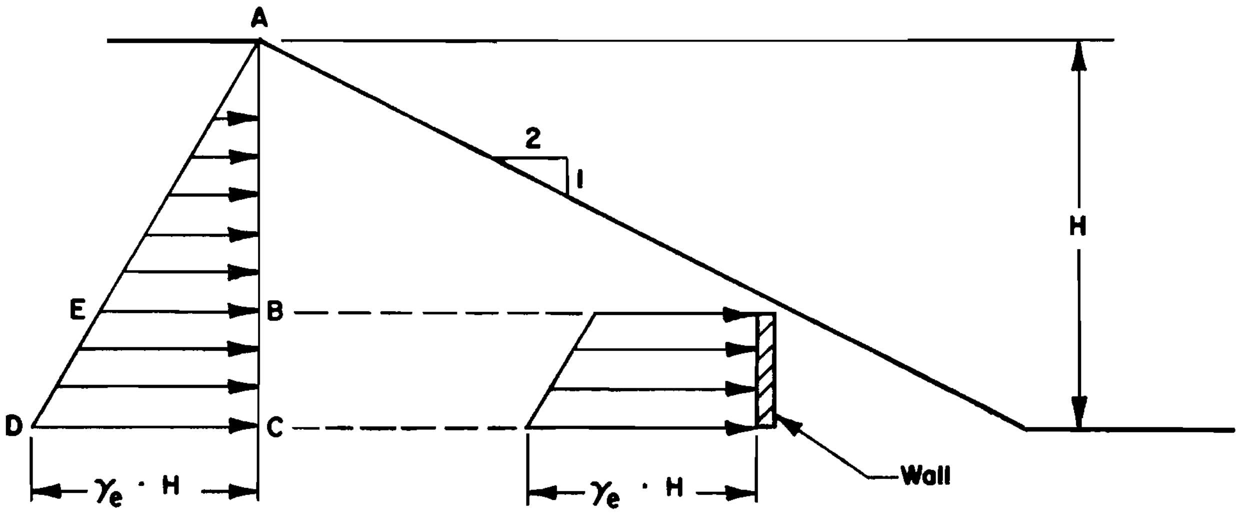


Fig. 5.17. Assumed fluid pressure distribution used in the modified equivalent-fluid procedure.

Table 5.2. Comparisons of Earth Pressure Forces Determined from Equivalent-Fluid Procedure and N_p Procedure (Equivalent fluid density equal to 36 pcf).

$\lambda_{c\phi}$	Length ABC*, ft.	Length BC**, ft.	Height of slope, ft.	γ, lb./ft.^{3}	Equivalent Fluid Force, lb./ft.	N_p' Force***, lb./ft.
			(Cot B = 2)			
4	21.1	7.7	20	100	4780	3200
4	42.2	15.4	40	100	9560	12800
4	42.2	15.4	40	120	9560	15400
10	19.5	6.2	20	100	3660	1940
10	39.0	12.4	40	100	7320	5760
10	39.0	12.4	40	120	7320	9330
40	17.5	4.2	20	100	2330	840
40	35.0	8.4	40	100	4660	3360
40	35.0	8.4	40	120	4660	4000
			(Cot B = 3)			
4	23.5	10.2	20	100	6760	5580
4	47.0	20.4	40	100	13520	20300
4	47.0	20.4	40	120	13520	26800
10	21.4	8.0	20	100	5000	3330
10	42.8	16.0	40	100	10000	13200
10	42.8	16.0	40	120	10000	15900
40	18.6	5.3	20	100	3040	1400
40	37.2	10.6	40	100	6080	5600
40	37.2	10.6	40	120	6080	6700

* Length ABC Defined in Fig. 5.17.
** Length BC Defined in Fig. 5.17.
*** N_p' values determined from Fig. 5.14.

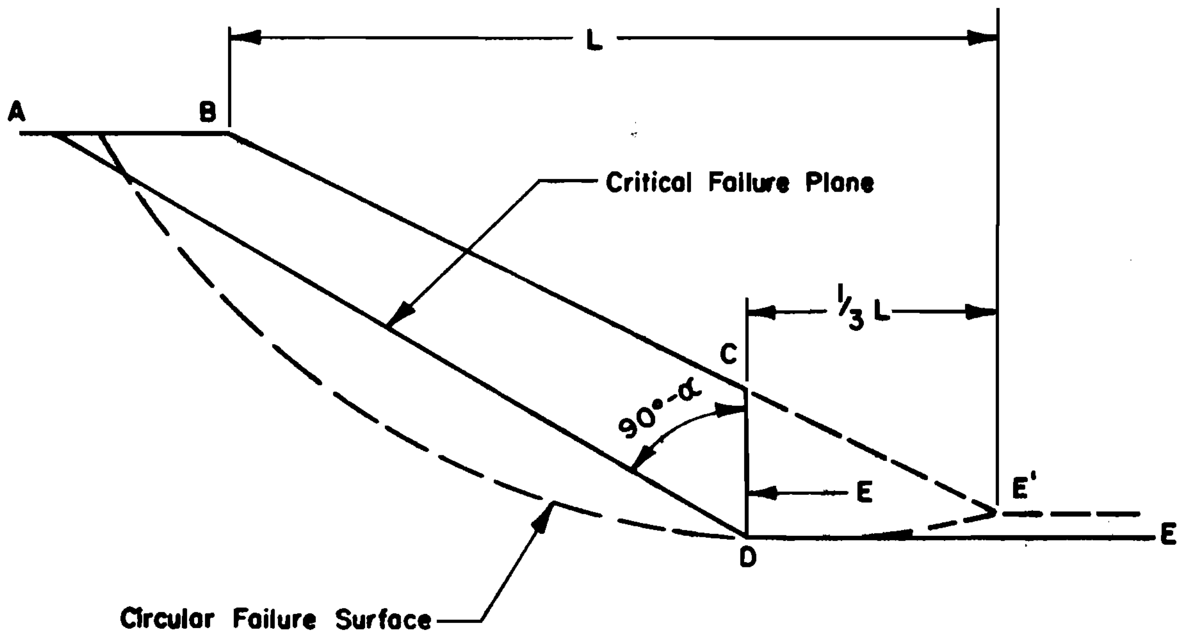


Fig. 5.18. Slope cross section showing the assumptions made in the trial-wedge analyses.

surface of the slope was assumed to be defined by the surface ABCDE shown in Fig. 5.18. The height of the wall (CD) was assumed to be equal to the depth of the critical circular failure surface corresponding to a given $\lambda_{c\phi}$ and slope inclination; the height of the wall (CD) will, therefore, change as the depth of the circular failure surface changes.

The comparative analyses were performed in the following manner. First, a critical circle was determined for a slope inclination and $\lambda_{c\phi}$ value. The horizontal earth pressure force corresponding to the location of the wall and the height of the wall (CD) were then determined for this circular failure surface. Next, a trial-wedge analysis was performed to determine the maximum earth pressure force corresponding to a plane failure surface passing through the base of the wall. Earth pressure forces were compared for values of $\lambda_{c\phi}$ ranging from 0 to 50 and slope inclinations of 2:1 and 3:1. For each trial-wedge analysis corresponding to a given value of $\lambda_{c\phi}$ and slope inclination, the total stress-shear strength parameters (c and ϕ) were selected to result in a factor of safety of unity for each circular failure surface investigated. The results of the trial-wedge analyses which were performed using a computer program are tabulated in Appendix V.

In the trial-wedge analyses the earth pressure forces were assumed to act normal to the wall and, for the analyses with circular shear surfaces, the resultant earth pressure forces (interslice side forces) were assumed to be inclined at the critical inclination required to satisfy force and moment equilibrium in Spencer's method-of-slices procedure. The earth pressure forces (E) calculated by both procedures were divided by the unit weight of the material and the square of the original slope height in order to calculate dimensionless earth pressure coefficients. These values are plotted in Fig. 5.19 against the depth ratios (d/H) corresponding to the critical circular failure surfaces for the previously described values of $\lambda_{c\phi}$ and slope inclinations. From Fig. 5.19 it can be noted that the earth pressure forces calculated from the trial-wedge analyses are from 10 to 30 percent lower than the earth pressure forces calculated employing Spencer's procedure and circular failure surfaces. The differences between these earth pressure forces are minimum values since the assumption of a horizontal earth pressure force in a trial-wedge analysis will result in a <u>maximum</u> force and the assumption of a critical side force inclination in a circular analysis will result in a <u>minimum</u> horizontal earth pressure force consistent with a reasonable line of thrust. If the earth pressure force

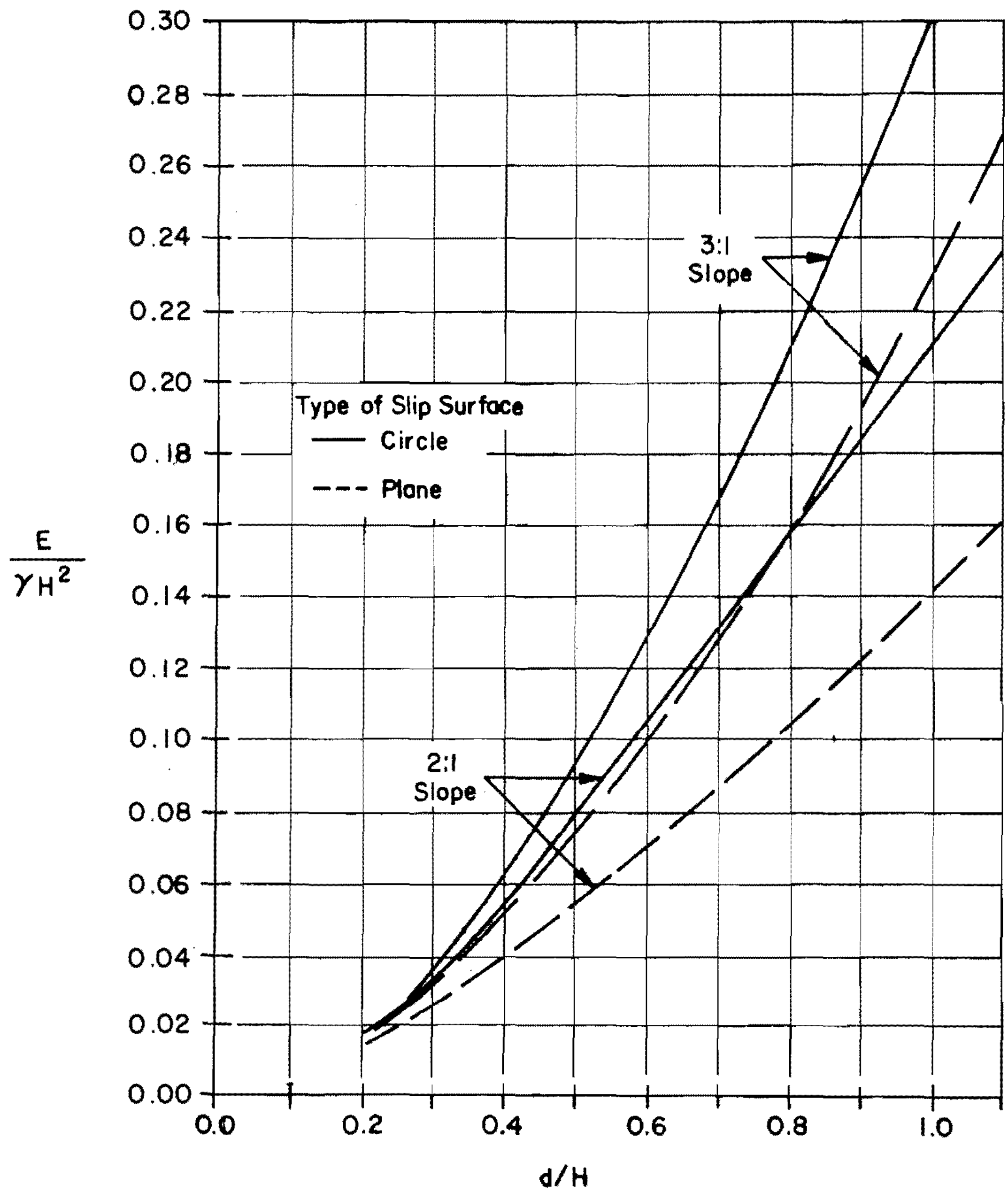


Fig. 5.19. Comparison of earth pressure forces determined from trial-wedge analyses (plane failure surfaces) and Spencer's method-of-slices procedure (circular failure surfaces).

inclinations were assumed to be identical in both of these analyses, the differences between the magnitudes of the earth pressure forces would have been greater.

Summary and Conclusions

Presented in this chapter are means for calculating from an observed failure surface, an earth pressure force which can be used for design of slide suppressor walls of the type described herein. This earth pressure coefficient procedure was specifically developed to calculate the earth pressure force which would act on a slide suppressor wall and it was assumed that failure would reoccur along the initial failure surface if such a wall were not installed to maintain the stability of the slope. This procedure is preferable to conventional procedures for obtaining earth pressure forces for several reasons:

1. The earth pressure coefficient procedure utilizes a better approximation of the actual failure surface than either the equivalent-fluid or trial-wedge procedure.
2. The method-of-slices procedure used to develop earth pressure coefficients is theoretically more correct than either the equivalent-fluid or trial-wedge procedures of analysis.
3. Application of the earth pressure coefficient procedure is relatively simple once the position of the failure surface has been estimated.

CHAPTER VI

SUMMARY AND CONCLUSIONS

The results of a survey of earth slope failures in Texas and the remedial measures utilized by the Texas Highway Department to correct these slides have been presented. Earth slope failures were observed to develop primarily in cut slopes in medium to highly plastic, overconsolidated, stiff-fissured clays in the Dallas, Fort Worth, Waco, Austin, and San Antonio regions of Central Texas. In these regions, failures tended to be shallow, semi-circular slides which, in many instances, developed several years after the construction of the slopes involved. Moderate to excessive amounts of ground water and surface water were observed in all but a few of these slides, suggesting that swelling and positive pore water pressures were the principal causes of failure.

In conjunction with the survey of slope failures, a review was made of the current earth slope design procedures and remedial measures employed by the Texas Highway Department. At the present time, highway slopes are generally designed empirically and repaired when they fail, using one or more of the following remedial measures:

1. Stabilization of the slide material by the addition of lime or cement.
2. Substitution of sands and gravels for slide material, generally in the vicinity of the toe of the slide.
3. Construction of restraint structures, usually piling and cast-in-place drilled shafts, with or without retaining walls or similar structures attached.
4. Control of ground water with interceptor trenches and surface water with ditches, curbing, crack filling, and slope planting.
5. Construction of flatter slope grades.
6. Construction of concrete rip-rap on the faces of unstable slopes.

While all of these remedial measures have been effective to a degree, there have been instances where the use of lime stabilization or concrete rip-rap has not been sufficient to prevent the reoccurrence of failure.

The design of a remedial measure, such as a restraint structure, will be influenced by the soil shear strength and the magnitude of the potential earth pressure forces. To facilitate the determination of the soil shear strength, a chart has been developed for estimating total stress shear strength parameters (c and ϕ) from an observed failure surface. In addition, a series of dimensionless earth pressure coefficient charts were developed to estimate, also from an observed failure surface, the earth pressure force acting on a restraint structure, referred to as a slide suppressor wall. These charts provide a more accurate means of estimating the earth pressure force than either the equivalent-fluid or trial-wedge procedures currently used by the Texas Highway Department, and the use of the charts results in generally higher earth pressure forces than predicted by either of these current procedures. The identification of some of the characteristics of earth slope failures, the evaluation of currently used remedial measures, and the development of design charts to estimate soil shear strengths and earth pressure forces should provide guidance for avoiding future failures and designing effective, economical remedial measures.

APPENDIX I

ATTERBURG LIMIT TESTS

APPENDIX I

ATTERBURG LIMIT TESTS

Atterburg Limit tests performed on soil samples taken from a number of slides which occurred in District 2 are summarized below. These tests indicate that the majority of the slides in this district occurred in medium to highly plastic clays.

Slide Location	W%*	Liquid Limit	Plasticity Index	Remarks
U.S. 377 near Benbrook Reservoir	26.6	79.0	55.0	Yellow clay; some caliche
U.S. 281 near Morgan Mill	29.6	64.0	46.0	Yellowish grey clay sample taken from toe of slide
U.S. 281 near Morgan Mill	79.0	33.0	4.0	White clayey silt sample taken from top of slide
U.S. 183 near Carter Airport in Ft. Worth	31.9	72.0	44.0	Yellow-reddish-brown clay - Eagle Ford formation - Sample taken from top of slide
U.S. 183 near Carter Airport in Ft. Worth	32.6	60.0	37.0	Black to dark grey clay - Eagle Ford formation - Sample taken from toe of slide
F.M. 2871 just north of T&P Railroad	17.8	54.0	36.0	Yellow-grey clay, some caliche present - Sample taken from north end of slide
F.M. 2871 just north of T&P Railroad	29.9	45.0	26.0	Yellow-grey clay, some small shells in sample taken from middle of slide
Loop 820 near Rufe Snow Drive	14.5	61.0	36.0	Reddish-brown clay - Denton-Weno-Paw Paw formation
Loop 820 near Grove Oak Drive	27.6	59.0	35.0	Yellow-grey brown clay - Woodbine formation
Loop 820 near U.S. 30	13.2	43.0	26.0	Reddish-brown, yellow clay - Woodbine formation

(Continued)

Slide Location	W%*	Liquid Limit	Plasticity Index	Remarks
F.M. 2871 just south of T&P Railroad	22.9	42.0	23.0	Yellow-grey clay - Kiamichi formation
S.H. 337 - 8 miles north of Mineral Wells	16.7	39.0	23.0	Reddish brown, yellow-grey clay - Sample taken from top of the slide
U.S. 180 - 6 miles east of Brad	24.5	29.0	11.0	Yellow-grey silty clay - Sample taken from toe of the slide

*W% - Moisture content of sample taken several weeks after slides occurred, may not be representative of moisture content at failure.

APPENDIX II

PARAMETERS USED IN ANALYSES

APPENDIX II

PARAMETERS USED IN ANALYSES

Constituent values of $\lambda_{c\phi}$ used to evaluate dimensionless coordinate numbers in Chapter IV and earth pressure coefficients in Chapter V are tabulated in section A of this appendix. In addition the stability numbers (N_{cf}) and the coordinates of critical toe circles evaluated using the $\lambda_{c\phi}$ values in section A for slope ratios ranging from 1:1 to 4:1 are tabulated in section B.

A. $\lambda_{c\phi}$ Constituent Values

$\lambda_{c\phi}$	H, ft.	γ, pcf	ϕ, deg.	c, psf
0	100	100	0.00	1000.0
1	100	100	5.71	1000.0
2	100	100	11.31	1000.0
4	100	100	21.80	1000.0
6	100	100	30.96	1000.0
8	100	100	38.66	1000.0
10	100	100	45.0	1000.0
15	100	100	45.0	666.7
20	100	100	45.0	500.0
30	100	100	45.0	333.3
50	100	100	45.0	200.0
100	100	100	45.0	100.0

B. Critical Toe Circles

(X and Y coordinates relative to toe of slope)

$\lambda_{c\phi}$	N_{cf}	X_c	Y_c
	(1:1 Slope)		
0	5.87	-45.0	140.0
1	8.03	-21.0	140.0
2	9.89	-9.0	141.0
4	13.22	6.0	147.0
6	16.46	16.0	152.0
8	19.14	23.0	155.0
10	21.91	30.0	160.0
15	28.51	43.0	169.0
20	34.90	50.0	175.0
30	46.96	67.0	187.0
50	70.16	87.0	203.0
100	125.58	119.0	231.0
	(2:1 Slope)		
0	6.55	-120.0	157.0
1	10.22	-83.0	174.0
2	13.31	-68.0	186.0
4	18.91	-51.0	204.0
6	24.11	-40.0	218.0
8	29.11	-31.0	231.0
10	33.98	-25.0	239.0
15	45.76	-12.0	259.0
20	57.21	-2.0	275.0
30	79.54	12.0	298.0
40	101.35	25.0	321.0
50	123.01	33.0	335.0
60	144.33	42.0	352.0
80	186.71	55.0	375.0
100	228.87	66.0	396.0
	(3:1 Slope)		
0	6.82	-189.0	181.0
1	11.97	-137.0	221.0
2	16.28	-119.0	245.0
4	24.12	-100.0	277.0
6	31.49	-88.0	301.0
8	38.62	-79.0	320.0

(Continued)

$\lambda_{c\phi}$	N_{cf}	X_c	Y_c
	(3:1 Slope, Continued)		
10	45.59	-72.0	336.0
15	62.58	-58.0	369.0
20	79.19	-47.0	397.0
30	111.77	-32.0	435.0
40	143.76	-17.5	475.0
50	175.62	-5.0	511.0
60	207.06	-1.0	519.0
80	269.67	20.0	581.0
100	332.10	27.0	599.0
	(4:1 Slope)		
0	6.95	-257.0	207.0
1	13.54	-189.0	275.0
2	19.03	-169.0	311.0
4	29.08	-148.0	360.0
6	38.60	-135.0	395.0
8	47.84	-125.0	424.0
10	56.90	-117.0	448.0
15	79.08	-103.0	493.0
20	100.83	-90.0	538.0
30	143.64	-71.0	606.0
50	227.82	-45.0	702.0
100	434.84	-0.5	872.0

APPENDIX III

VALUES OF N_p

APPENDIX III

VALUES OF N_p

Values of N_p determined from total stress, c-ϕ analyses and presented in Chapter V are listed below for a slide suppressor wall located one-fourth (L/4), one-third (L/3), and one-half (L/2) the distance up the slope.

$\lambda_{c\phi}$	d/H	N_p L/2	N_p L/3	N_p L/4
		(2:1 Slope)		
0	1.109	0.3402	0.2377	0.1757
1	0.743	0.1955	0.1430	0.1068
2	0.621	0.1480	0.1102	0.0824
4	0.506	0.1057	0.0799	0.0598
6	0.445	0.0848	0.0647	0.0483
8	0.403	0.0711	0.0545	0.0407
10	0.377	0.0632	0.0486	0.0362
15	0.330	0.0496	0.0383	0.0285
20	0.299	0.0414	0.0322	0.0239
30	0.263	0.0327	0.0255	0.0188
50	0.222	0.0237	0.0185	0.0136
100	0.178	0.0153	0.0121	0.0088
		(3:1 Slope)		
0	1.497	0.7632	0.5390	0.3982
1	0.937	0.3656	0.2703	0.2005
2	0.776	0.2640	0.1983	0.1470
4	0.633	0.1834	0.1397	0.1032
6	0.559	0.1455	0.1115	0.0822
8	0.510	0.1227	0.0945	0.0695
10	0.476	0.1078	0.0832	0.0611
15	0.418	0.0840	0.0652	0.0478
20	0.380	0.0699	0.0544	0.0398
30	0.336	0.0552	0.0431	0.0314
50	0.278	0.0380	0.0298	0.0217
100	0.228	0.0257	0.0202	0.0146

APPENDIX IV

POINT OF APPLICATION OF EARTH PRESSURE FORCES

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POINT OF APPLICATION OF EARTH PRESSURE FORCES

The earth pressure forces calculated using Spencer's procedure of analysis (as discussed in Chapter V) and strength parameters (c and ϕ) were found to act at the locations listed below for slices located one-fourth (L/4), one-third (L/3), and one-half (L/2) the distance up the slope. The point of application is presented as a ratio of the distance (y) (measured between the base of the slice and the point of application) to the height of the slice (h).

$\lambda_{c\phi}$	y/h		
	L/4	L/3	L/2
	(2:1 Slope)		
0	0.19	0.23	0.24
1	0.23	0.26	0.27
2	0.25	0.27	0.29
4	0.27	0.29	0.30
6	0.28	0.30	0.31
8	0.29	0.30	0.31
10	0.29	0.31	0.32
15	0.30	0.32	0.32
20	0.31	0.32	0.33
30	0.31	0.32	0.33
50	0.32	0.33	0.34
100	0.33	0.34	0.34
	(3:1 Slope)		
0	0.26	0.29	0.30
1	0.28	0.30	0.32
2	0.29	0.31	0.32
4	0.30	0.32	0.33
6	0.31	0.32	0.33
8	0.31	0.33	0.33
10	0.32	0.33	0.34
15	0.32	0.33	0.34
20	0.32	0.33	0.34
30	0.33	0.33	0.34
50	0.33	0.34	0.34
100	0.33	0.34	0.35

APPENDIX V

SUMMARY OF TRIAL-WEDGE ANALYSES

APPENDIX V

SUMMARY OF TRIAL-WEDGE ANALYSES

Summarized below are the shear-strength parameters, slope and wall heights, and earth pressure forces (E) used to develop the trial-wedge earth pressure force curves in Fig. 5.19. In all the trial-wedge analyses the unit weight of the soil was assumed to be 100 pcf.

$\lambda_{c\phi}$	c, lb./ft.^{2}	ϕ, degrees	Height of wall, ft.	Height of slope, ft.	E, lb./ft.	α^*, degrees
			(2:1 Backfill)			
0	305.4	0.00	13.3	26.7	6450	29.3
1	195.8	5.58	10.3	23.6	3750	29.5
2	150.2	8.54	9.0	22.4	2950	29.9
4	105.8	11.95	7.7	21.1	2120	30.1
6	82.9	13.97	7.0	20.4	1840	30.4
10	58.9	16.40	6.2	19.5	1430	30.6
20	35.0	19.26	5.1	18.5	990	30.7
30	25.2	20.67	4.6	17.9	800	30.7
40	19.7	21.53	4.2	17.5	670	30.6
50	16.3	22.12	4.0	17.3	600	30.5
			(3:1 Backfill)			
0	293.4	0.00	19.7	33.0	16990	27.3
1	167.1	4.77	13.4	27.3	8150	26.1
2	122.9	7.00	12.0	25.3	6020	25.7
4	82.9	9.42	10.2	23.5	4360	25.3
6	63.5	10.78	9.1	22.5	3540	24.9
10	43.9	12.37	8.0	21.4	2780	24.7
20	25.3	14.17	6.5	19.9	1830	24.0
30	17.9	15.02	5.8	19.2	1480	23.7
40	13.9	15.55	5.3	18.6	1210	23.3
50	11.4	15.89	4.9	18.2	1040	23.0

*α is the inclination of the plane failure surface from a horizontal plane as shown in Fig. 5.18.

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