RESEARCH REPORT 1435-2F

SHEAR STRENGTH CORRELATIONS AND REMEDIAL MEASURE GUIDELINES FOR LONG-TERM STABILITY OF SLOPES CONSTRUCTED OF HIGHLY PLASTIC CLAY SOILS

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OCTOBER 1997

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Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-98/1435-2F		

4. Title and Subtitle	5. Report Date
SHEAR STRENGTH CORRELATIONS AND REMEDIAL MEASURE GUIDELINES FOR LONG-TERM STABILITY OF SLOPES CONSTRUCTED OF HIGHLY PLASTIC CLAY SOILS	October 1997
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
A. A. Saleh and Stephen G. Wright	1435-2F
9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, TX 78705-2650	11. Contract or Grant No. 0-1435
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Texas Department of Transportation Research and Technology Transfer Section/Construction Division P.O. Box 5080 Austin, TX 78763-5080	Research Report (9/96 — 8/97)
	14. Sponsoring Agency Code

15. Supplementary Notes

Project conducted in cooperation with the Federal Highway Administration.

16. Abstract

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The Texas Department of Transportation has experienced many small slides in earth embankments that are constructed of highly plastic clays. This report addresses several issues related to (1) the design of new slopes and (2) the evaluation of remedial measures to repair slides in slopes constructed of highly plastic clays. Simple means were examined for estimating soil shear strength based on correlations with results of relatively simple index property tests. Specific procedures and recommendations are presented for estimating soil shear strength for new slopes, as well as for slopes that have experienced previous sliding. Various remedial measures for slopes have been examined and guidelines for preliminary selection of remedial measures are provided. Also, a number of maintenance activities that may be detrimental to slope stability are identified and suggestions are given for how such actions can be avoided. Finally, an evaluation of geosynthetic reinforcement for slope repair was conducted. Finite element computations were performed using a range of soil types and embankment geometry to investigate the forces mobilized in horizontal, geogrid reinforcement in embankments during construction. It was found that, for high PI soils and embankment geometries like those encountered along Texas highways, the mobilized forces are very small. Based on previous work by others, it is anticipated that the development of additional significant reinforcement forces after construction is not likely. Thus, it appears that conventional procedures for estimating reinforcement forces for design do not seem applicable and ought not be used for embankments like those in Texas.

17. Key Words	18. Distribution Statement
Geotechnical engineering, slope stability, slope maintenance, landslides	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of report)	20. Security Classif. (of this page)	21. No. of pages	22. Price
Unclassified	Unclassified	154	

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by

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Research Report 1435-2F

Research Project 0-1435
Development of Simplified Procedures for Design of Slope Reinforcement for Slide Repair

Conducted for the

TEXAS DEPARTMENT OF TRANSPORTATION

in cooperation with the

U.S. DEPARTMENT OF TRANSPORTATION
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
Bureau of Engineering Research
THE UNIVERSITY OF TEXAS AT AUSTIN

October 1997

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ACKNOWLEDGMENTS

The authors acknowledge the expert assistance provided by the research project director, C. Fu, of TxDOT's Materials and Tests Division, and C. Utley of the Lubbock District, who served as project coordinator. The authors also thank the other TxDOT members of the research team, which included J. Cravens of the FHWA, S. Ekstrom of the Paris District, L. Kelley of the Dallas District, M. McFarland of the Design Division, and S. Yin of the Houston District.

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CHAPTER 1. INTRODUCTION

The Texas Department of Transportation (TxDOT) has experienced many small slides in earth embankments. These slides are typically shallow (less than 10 feet in depth) and occur a number of years after construction (from 10 to more than 30 years). Many of these embankments are less than 30 feet high and are constructed of highly plastic clays (liquid limits 50 or greater). Although two solutions are to adopt different soils for design or to construct embankments with much flatter side slopes, many embankments of highly plastic clays now exist with slopes that are too steep. And it is likely that additional embankments will be built of highly plastic clays in the future. Accordingly, there exists a need for improved slope design when constructed in highly plastic clays, and for evaluating remedial measures to repair slides in such slopes when they fail. This report and a companion report address these needs.

Much of the responsibility for slides in earth slopes rests with the highway maintenance engineer. The maintenance engineer requires relatively simple means for identifying potential remedial measures and for determining the required parameters for design. This report attempts to address some of these needs. Chapters 2 and 3 address the issue of determining appropriate soil shear strength parameters for the design of both new slopes and remedial measures. Chapter 2 explores available correlations for estimating the shear strength from simple index properties. Chapter 3 presents recommended correlations for Texas highway slopes and provides step-by-step procedures for their use.

Chapter 4 presents an overview of the essential characteristics of slope failures and discusses their relevance to the selection and design of remedial measures. Most of Chapter 4 is devoted to presenting various remedial alternatives in a concise form that can serve as a starting point for selecting remedial measures. Various remedial measures are described along with their applicability and limitations to particular types of slope problems. Important factors that must be considered for each are also presented, along with appropriate comments on each remedial measure. Finally, Chapter 4 presents a number of maintenance activities that may be detrimental to slope stability and makes recommendations on how such actions can be avoided.

A companion report presents the results of detailed analyses that were performed to develop design charts for determining requirements for slope stabilization using additives such as lime and cement to "strengthen" the soil and geogrid layers to "reinforce" the soil. The companion report presents the design charts and illustrative examples for their use.

The design of slopes reinforced with geogrid or similar reinforcing materials requires that the force developed in the reinforcement be determined. Chapter 5 of this report presents the results of an investigation that was conducted as part of this project to establish what levels of force might be developed in the reinforcement. Particular attention was paid to slopes that are stable during construction, even without reinforcement. Most of the existing design procedures for reinforced slopes assume that the slope is unstable during construction without reinforcement, while typical slopes like those constructed in Texas using high plasticity index (PI) soils are stable during construction with no reinforcement. This presents a situation very different from that considered by most design procedures. The studies presented in Chapter 5 seek to address this issue. Final recommendations for implementation and for additional studies are presented in Chapter 6.

CHAPTER 2. CORRELATIONS FOR ESTIMATING SHEAR STRENGTH PARAMETERS OF EMBANKMENT SOILS

2.1 INTRODUCTION

The design of new slopes and of remedial measures for failed slopes requires knowledge of the shear strength of the soil. Because the shear strength is generally not known in advance, it must therefore either be measured in situ or in the laboratory (or estimated based on other information). Measurement of the shear strength and full characterization of the soil at a specific site usually involves significant cost. For repair of failed slopes, "design" is often done by maintenance personnel with neither the budget nor the resources to conduct significant field exploration and sampling or to perform the requisite laboratory (or field) strength tests. A need exists for simple means to estimate the shear strength of the soil based on correlations with results of relatively simple index property tests, which maintenance personnel can obtain. Correlations for estimating shear strengths are also useful for making preliminary estimates for design of new slopes. In some cases, soils to be used for a new slope are not even yet known and estimates of shear strength are the only alternative. This chapter addresses such correlations, with particular emphasis on, and attention to, slopes and soils in Texas.

2.2 SCOPE OF CORRELATION

A number of correlations between shear strength and other soil properties have been reported in the literature. These correlations have been developed using shear strengths determined through extensive laboratory strength tests, field tests, or by back analyses from actual slope failures. For this study only correlations between shear strength and simple index property tests, such as Atterberg limits and soil grain size, are examined. Atterberg limits, owing to their simplicity and relatively low cost, are performed relatively routinely and have been found to be useful for correlations with shear strength. Furthermore, there is often some general knowledge from past work of the plasticity characteristics and grain size of many of the local clayey soils encountered in Texas.

2.3 MATERIAL INDEX PROPERTIES FOR TEXAS SLOPES

Over the last 23 years, a number of Center for Transportation Research (CTR) projects for TxDOT have identified and examined approximately sixty highway embankment failures (Stauffer and Wright, 1984; Gourlay and Wright, 1984; Green and Wright, 1986; Rogers and Wright, 1986; and Kayyal and Wright, 1991). Data have been collected from most of the failures examined in several districts of TxDOT (Figure 2.1). Extensive laboratory tests have also been performed to measure the index properties of the soils involved in the failures. Some additional testing has been done to measure shear strength properties.

The majority of the failures studied have occurred in compacted fill (embankment) slopes. It is believed that many soils used to construct the embankments in TxDOT's Houston District have originated from the Beaumont Clay Formation, a Pleistocene deposit that extends along the Gulf Coast. The geologic origin of many of the soils from other embankment slope failures is not known. However, it is known that almost all of the slope failures involve high plasticity clays.

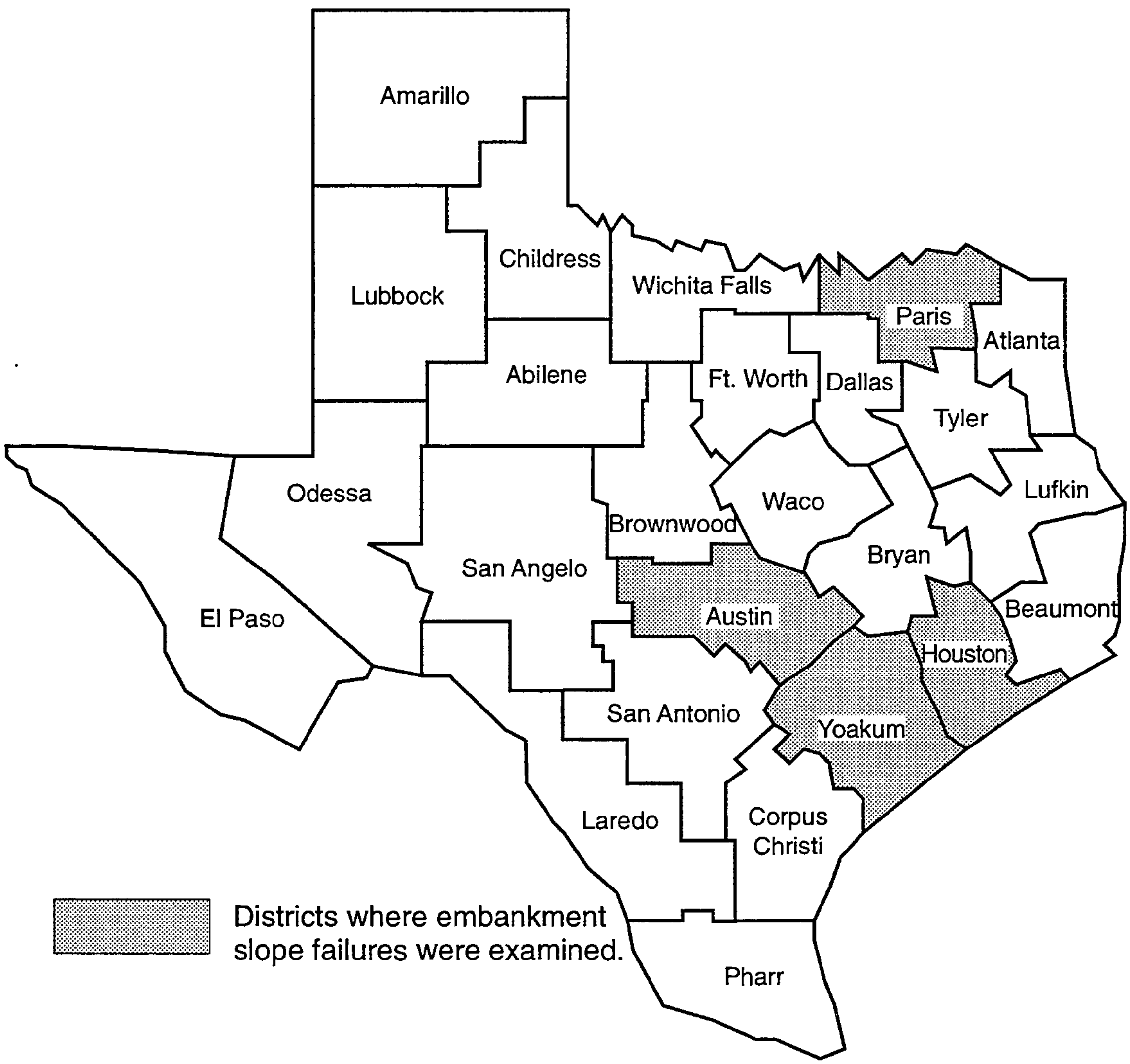


Figure 2.1 Texas state map showing districts where embankment slope failures were examined (from Stauffer and Wright, 1984)

Stauffer and Wright (1984) determined Atterberg limits for various samples obtained from embankment slopes that failed in four different districts in Texas. Figure 2.2 summarizes their data. The liquid limits range from approximately 42 to 97 percent, and plasticity indices range from 30 to 71 percent. Most of the soils are classified as highly plastic clays (CH) under the Unified Classification System (UCS). Data obtained independently by Vijayvergiya and Sullivan (1973) for soils from the Beaumont Formation are summarized separately in Figure 2.3. Vijayvergiya and Sullivan's data for the Beaumont clay compare well with the results shown in Figure 2.2.

The percent by weight finer than 2 microns for the soils examined by Stauffer and Wright (1984) ranged from 37 to 86, with the majority exceeding 50 percent. The activity, defined as the ratio of plasticity index to the percent clay size fraction by weight, ranged from 0.5 to 1.1, with a mean value of 0.79.

In previous CTR studies, shear tests were performed on some of the soils obtained from embankment slope failures. Data from the shear tests are presented and used later in this chapter to supplement correlations found in the literature.

2.4 DESIGN SHEAR STRENGTH PARAMETERS

All of the embankment slope failures examined by the authors occurred in fine-grained, predominately clayey soil. Accordingly, attention is restricted to fine-grained soils. In determining the shear strength of fine-grained soils, a distinction must be made between the short-term, "undrained" shear strength and the long-term, "drained" shear strength.

2.4.1 Short-Term, "Undrained" Shear Strength

The short-term, undrained shear strength is applicable to the conditions where the time between construction of the slope and failure is short enough that the soil does not have ample time to "drain." Failures where the undrained shear strength is applicable occur during—or very shortly after—construction. The undrained shear strength is expressed by the Mohr-Coulomb equation as

$$s = c + \tau \tan \phi \tag{2.1}$$

where s is the undrained shear strength, σ is the total normal stress on the failure surface, and c and ϕ are the "cohesion" and "friction angle," respectively. In general, the undrained shear strength is a function of the total confining pressure (σ). For saturated soils ϕ is zero, and the undrained shear strength is expressed by a cohesion value only, i. e.,

$$s = c \tag{2.2}$$

The undrained shear strength can be measured in unconfined compression tests or "unconsolidated-undrained" triaxial compression tests. Undrained shear strengths are sometimes estimated using such dynamic penetration tests as the Texas Cone Penetrometer and the Standard Penetration Test. However, dynamic penetration tests do not provide reliable estimates of undrained shear strength. The Texas Triaxial test (Texas SDHPT, 1962) has also been used to measure undrained shear strength. This test, however, should also be avoided; O'Malley and Wright (1987) showed that the test could introduce significant errors.

Except for cases in which the foundation is significantly weaker than an overlying embankment and the slope fails during or very shortly after construction, most highway slope failures examined in Texas occur some time—typically 10 to 30 years—after construction was completed. In such cases the undrained shear strength is not applicable; the long-term, drained shear strength is the strength of interest. Accordingly, the balance of this chapter is devoted to the drained shear strength.

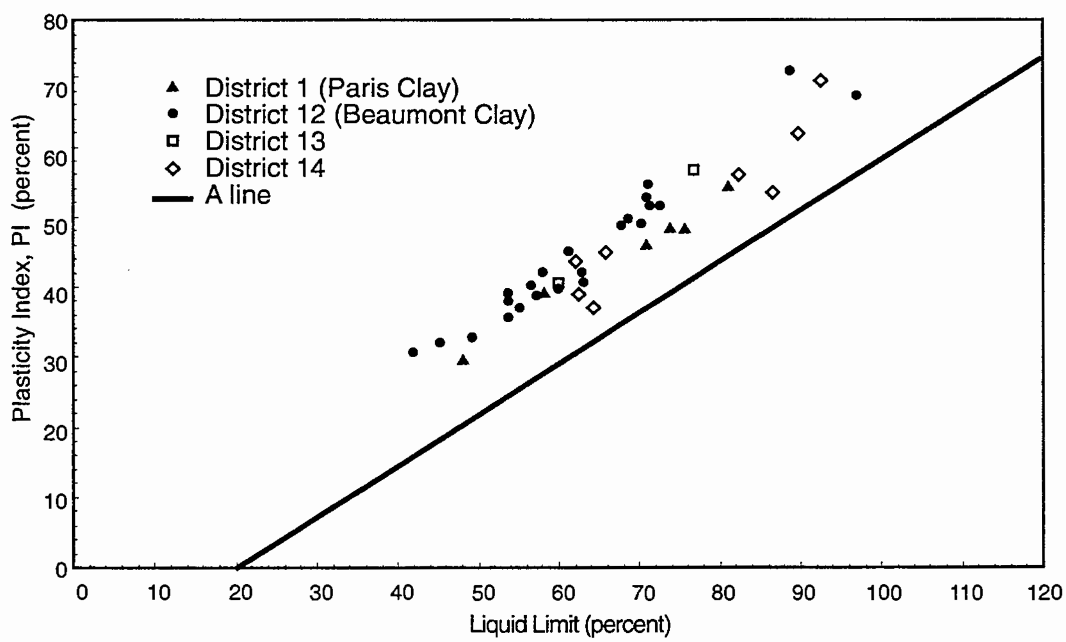


Figure 2.2 Plasticity chart with Atterberg limits for soils from embankments that have experienced slope failures

2.4.2 Long-Term, "Drained" Shear Strength

The applicable shear strength for long-term stability of slopes is the *drained* shear strength. Although the term *drained* is used here and throughout geotechnical engineering, the term is somewhat of a misnomer: *Drained* simply means that water has had the opportunity to move out of the soil, *or into the soil*, e.g., by the soil expanding over time. It does not mean that all water has been removed from the soil. In fact, the soil water content may increase when the soil "drains," if the soil swells.

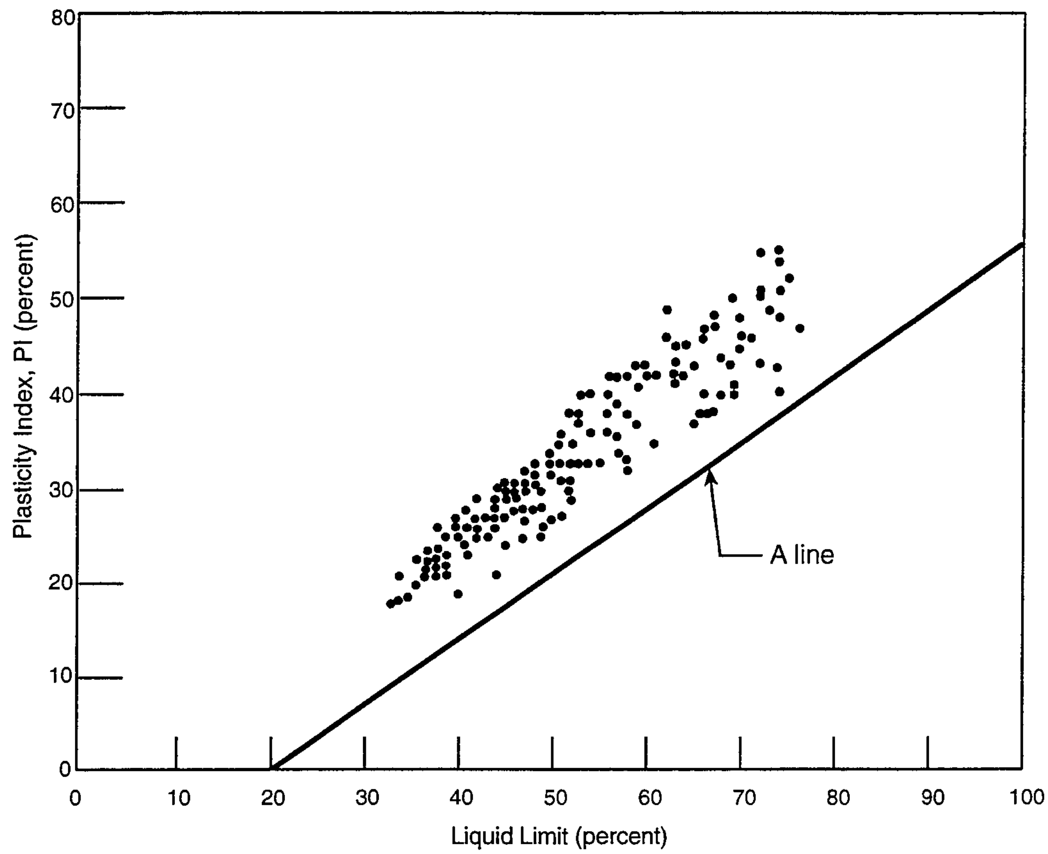


Figure 2.3 Plasticity chart with Atterberg limits for Beaumont clay (Vijayvergiya and Sullivan, 1973)

Drained shear strengths are expressed in terms of the Mohr-Coulomb equation and effective stresses as

$$S = \bar{c} + (\sigma - u) \tan \bar{\phi} \qquad (2.3)$$

where $\bar{c}$ and $\bar{\phi}$ are the shear strength parameters expressed in terms of effective stresses. The term, σ - u, represents the *effective stress,* which is the total stress on the failed surface, less the pore water pressure, u. The effective stress reflects not only the total loads, but also the effect of any water present in the soil: The higher the water pressures *(u),* the lower the shear strength of the soil.

2.5 DRAINED SHEAR STRENGTH PARAMETERS

Several different sets of drained shear strength parameters ($\bar{c}$ and $\bar{\phi}$) can be defined, depending on the stress history and on the strains that the soil is subjected to. "Peak," "fully softened," and "residual" strength values may each be defined. "Peak" shear strengths refer to the strength of the soil at the point of maximum resistance and with no prior failure. "Fully softened" is a term that is applied to the strengths that many highly plastic soils exhibit after a number of years of exposure to climate and environmental changes. "Residual" shear strengths are the shear strengths that the soil exhibits as an ultimate, minimum resistance after shear to very large strains. Residual shear strengths are applicable to soils and slopes, which have experienced prior sliding. Each of these shear strengths (peak, fully softened, and residual) is treated separately below.

2.5.1 Peak Shear Strengths

Peak shear strengths are applicable to soils that have not experienced prolonged exposure to environmental changes and have not experienced prior failure. They represent the maximum drained resistance that the soil can provide.

2.5.2 Fully Softened Strengths

Fully softened strengths are the strengths that are believed to eventually develop in clays after long periods of exposure to environmental conditions (shrink-swell, wetting-drying, etc.). Fully softened strengths are believed to be the governing strength for first-time slides in both excavated and fill slopes in highly plastic clays. The term *fully softened* is generally applied to field conditions, rather than to a condition that is routinely reproduced in the laboratory. However, Green and Wright (1986), Rogers and Wright (1986), and Kayyal and Wright (1991) successfully reproduced this condition in the laboratory by subjecting specimens to repeated cycles of wetting and drying before measuring the strength properties.

It has been suggested that the fully softened strength is equivalent to the peak strength of normally consolidated clays (Skempton, 1969, 1970, and 1977). Normally consolidated clays are clays that have not been subjected to pressures higher than those that exist at the time the soil is sheared; specimens of normally consolidated clay can be created in the laboratory by mixing soil with water into a slurry and then reconsolidating the soil into specimens. Kayyal showed that the strength of fully softened specimens after wetting and drying agreed favorably with the strength measured on specimens reconsolidated from a slurry.

2.5.3 Residual Shear Strengths

Residual shear strengths are measured either by repeatedly shearing soil back-and-forth in a direct shear device or by using torsional shear devices that permit essentially unlimited rotation and shear of the specimen. The residual shear strength is applicable to potential sliding surfaces where sliding has previously occurred. Although it would be conservative to use residual shear strengths for all cases, residual shear strengths may be significantly less than even fully softened strengths; thus, the use of residual shear strengths might be overly conservative in many cases.

2.5.4 Peak vs. Fully Softened Strength—Special Case

Peak strengths should generally be used with caution because in many cases they do not apply to long-term stability in highly plastic clays. Fully softened strengths are more applicable than peak strengths for many highly plastic clays. For low plasticity soils with low shrink-swell tendencies, peak strengths may be applicable. There is also indirect evidence that when highly plastic clays are covered with impervious cover, specifically concrete riprap, the cover greatly reduces moisture fluctuations and, thus, reduces the "softening" effect. Slopes with concrete riprap have been observed to remain stable, while adjacent unprotected slopes have failed, even when the unprotected slopes were flatter than the protected ones. The slope protection in these cases is not believed to actually prevent moisture from reaching the soil, but rather is believed to prevent the changes (fluctuation) in moisture that are detrimental and lead to softening of the soil. In such cases, peak, rather than fully softened, shear strengths may be applicable, but this has not been confirmed.

2.5.5 Secant vs. Tangent Friction Angle

Equation 2.3 expresses a linear relationship between shear strength and effective stress (Figure 2.4a). In some cases, the shear strength does not vary linearly with effective stress; rather, the Mohr-Coulomb envelope is curved (Figure 2.4b). In such cases, the shear strength is often expressed using a secant envelope with the slope of the envelope representing a secant friction angle (Figure 2.5a). In this case, the secant friction angle is a function of the effective stress; different friction angles will be used depending on the range of stresses involved. Because the secant friction angle is different from the tangent friction angle (Figure 2.5b), it is important to distinguish between the two (secant vs. tangent). Throughout this report we will use the secant friction angle (Figure 2.5a) when the Mohr-Coulomb envelope is curved.

2.6 EXISTING CORRELATIONS

The correlation of effective stress shear strength parameters ($\overline{c}$ and $\overline{\phi}$) with index properties depends on the specific type of drained shear strength properties being considered: Different correlations exist for peak, fully softened, and residual shear strengths. Accordingly, each of these strengths is considered separately.

Most of the strength correlations reported in the literature are based on data for undisturbed specimens or for specimens prepared by consolidating soil from a high water content slurry in the laboratory. Very few correlations seem to be based on data for compacted clays. However, extensive laboratory work has been performed on compacted soil samples obtained from various Texas highway embankments that experienced slope failures (Stauffer and Wright [1984], Gourlay and Wright [1984], Green and Wright [1986], Rogers and Wright [1986], and Kayyal and Wright [1991]). These data have been used to verify the correlations reported in the literature for other soils.

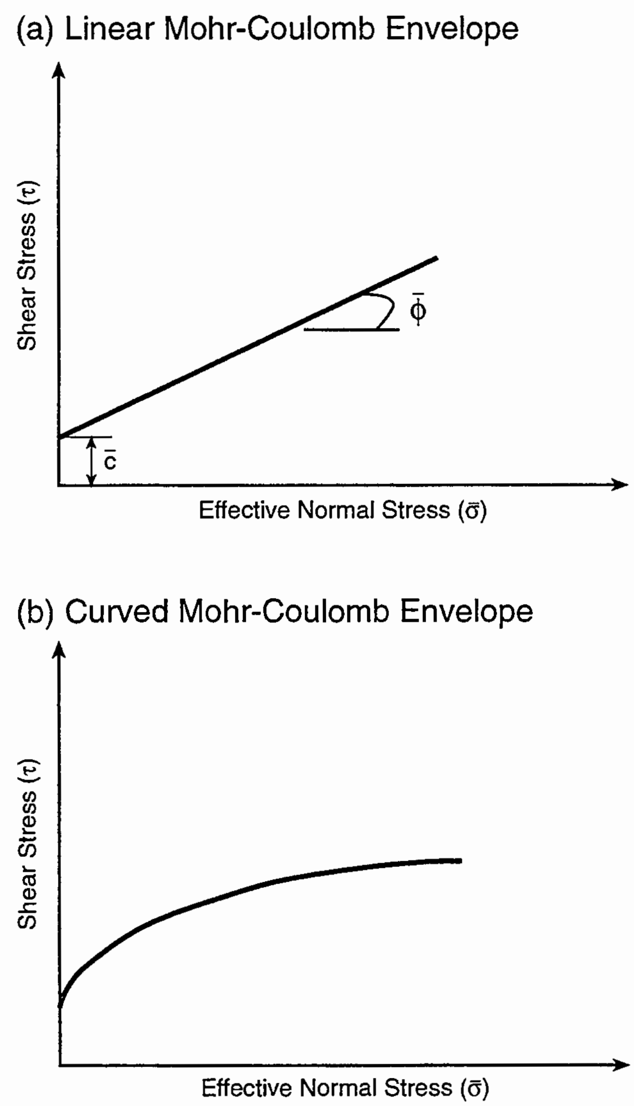


Figure 2.4 Linear and curved Mohr-Coulomb failure envelopes

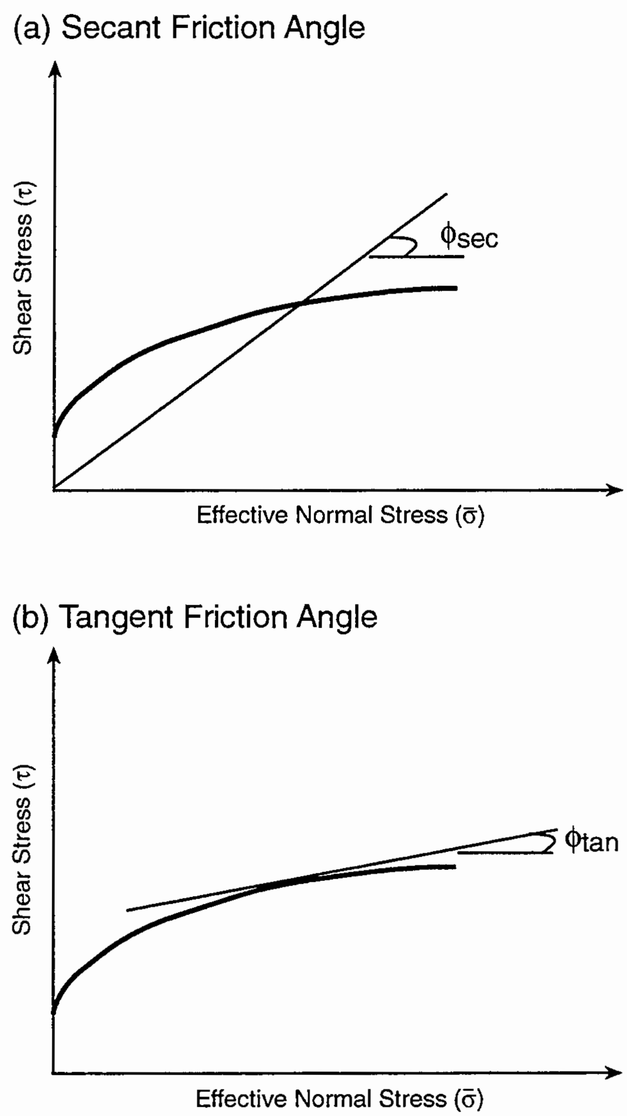


Figure 2.5 Illustration of secant and tangent friction angles

2.6.1 Correlations for Peak Effective Stress Shear Strength Parameters

Most of the correlations found in the literature are for peak shear strength parameters. Furthermore, most of the correlations for the peak shear strength parameters are based on data pertaining to either normally consolidated clays or remolded clays.

Numerous correlations have been developed between peak effective stress friction angle, $\overline{\phi}$, and simple index parameters. Ladd et al. (1977) present the correlation between $\overline{\phi}$ and plasticity index shown in Figure 2.6. The correlation in Figure 2.6 is based on normally consolidated clays tested in triaxial compression. Mitchell (1976) shows a similar correlation where $\overline{\phi}$ is shown to decrease with increasing plasticity index and increasing mineral activity (Figure 2.7). Tavenas and Leroueil (1987) and many others have corroborated the trends shown by Mitchell and by Ladd. A number of other correlations between the peak effective friction angle and plasticity index has been summarized by Kanji; these are shown in Figure 2.8.

Very little work has been done to correlate effective stress cohesion ($\overline{c}$) with index properties. This may be owing partly to the fact that cohesion is often small and may be related to the stress history of the soil. Atterberg limits involve complete remolding of the soil (thus destroying stress history effects); consequently, they may not correlate with cohesion.

The range in effective stress friction angle presented in the literature is considerable, even among soils having similar Atterberg limits. The broad range has prompted many engineers to question the value of such correlations. However, in a recent paper, Stark and Eid (1995) have shown that "scatter" in the data may be misconceived owing to the effects of clay size fraction and effective normal stresses. Stark and Eid (1995) conducted an extensive laboratory test program and developed the relationship shown in Figure 2.9 between peak effective stress friction angle and soil index properties for normal stresses of 50, 100, and 400 kPa. All the specimens tested by Stark and Eid (1995) were normally consolidated. Their correlation differs from previous correlations in that it shows a significant nonlinearity for the effective stress shear strength envelope for normally consolidated soils having clay size fractions greater than 25 percent. In addition, their correlation shows that the nonlinearity is most noticeable at effective normal stresses less than approximately 200 kPa (i.e., in the range of effective stresses applicable to TxDOT embankments).

2.6.2 Fully Softened Strength Correlations

As discussed earlier, the fully softened strength of soils is considered to be equivalent to the peak strength of normally consolidated soil. Because all the strengths reported by Stark and Eid (1995) were peak values for normally consolidated soil, they considered the strengths to be the same as fully softened strengths and presented them as such. Thus, the correlations involving normally consolidated soil specimens described in the previous section may be used to estimate the fully softened shear strength parameters.

2.6.3 Residual Shear Strength Correlations

Residual shear strengths represent "ultimate" shear strengths that are developed at large strains. The residual shear strength parameters are normally determined using drained direct shear or torsional shear tests performed with very slow loading rates to ensure complete drainage. Tests often require many days or even weeks to perform.

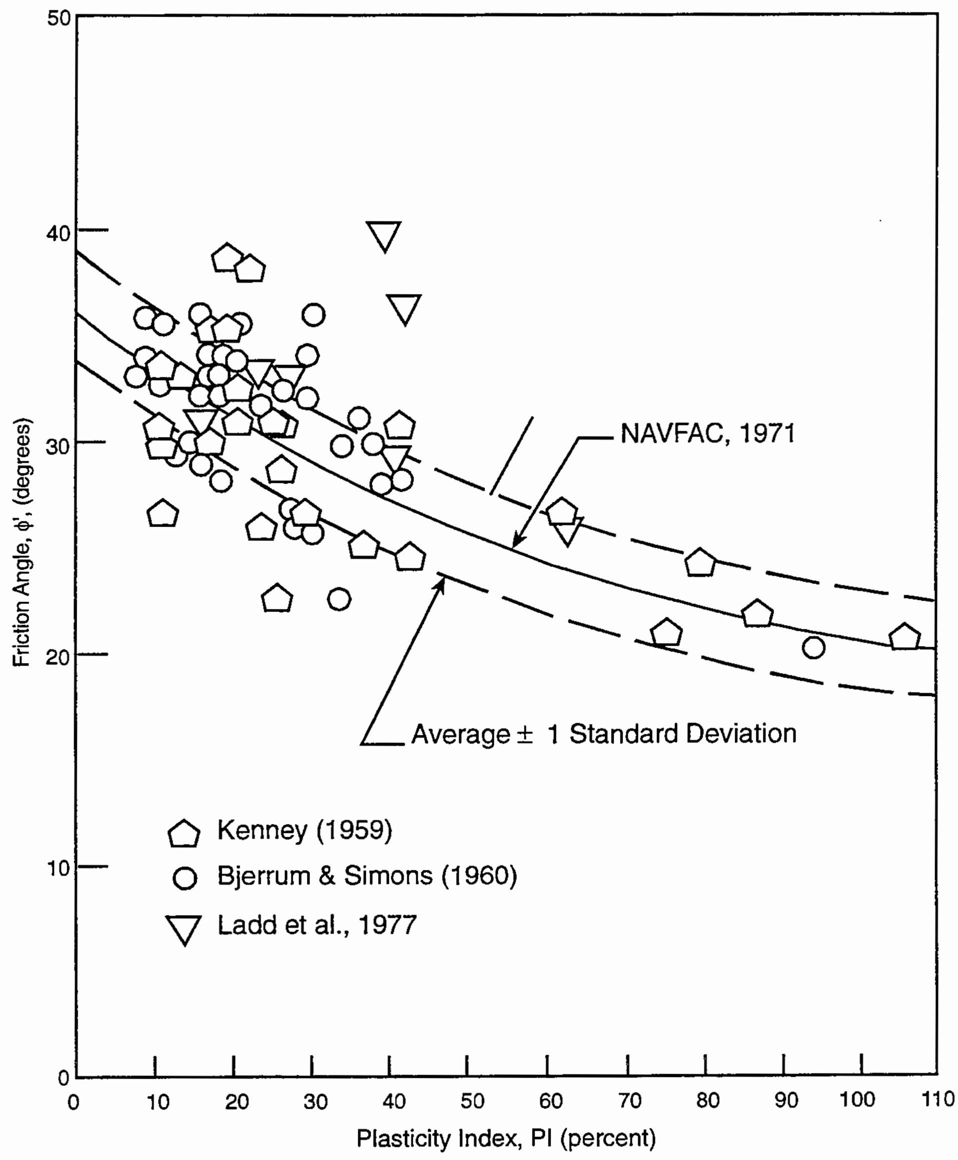


Figure 2.6 Correlation between peak friction angle and plasticity index from triaxial tests on normally consolidated clays (Ladd et al., 1977)

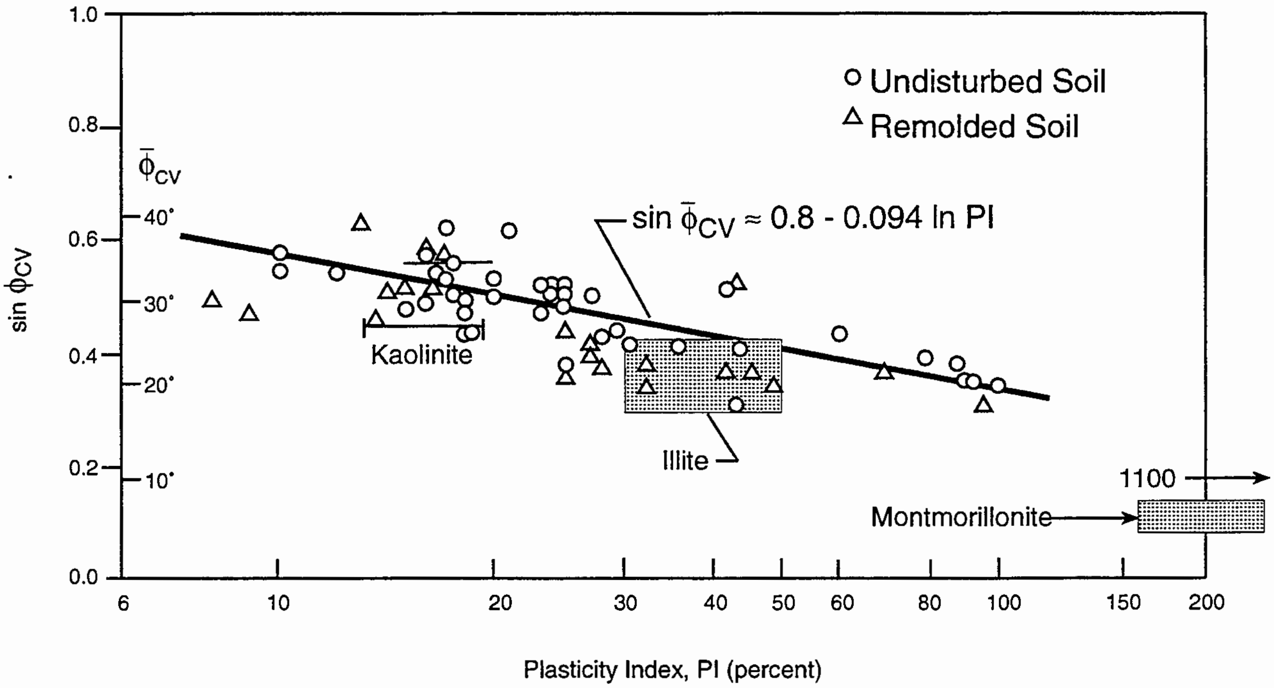


Figure 2.7 Correlation between peak friction angle and plasticity index for normally consolidated clays (Mitchell, 1976)

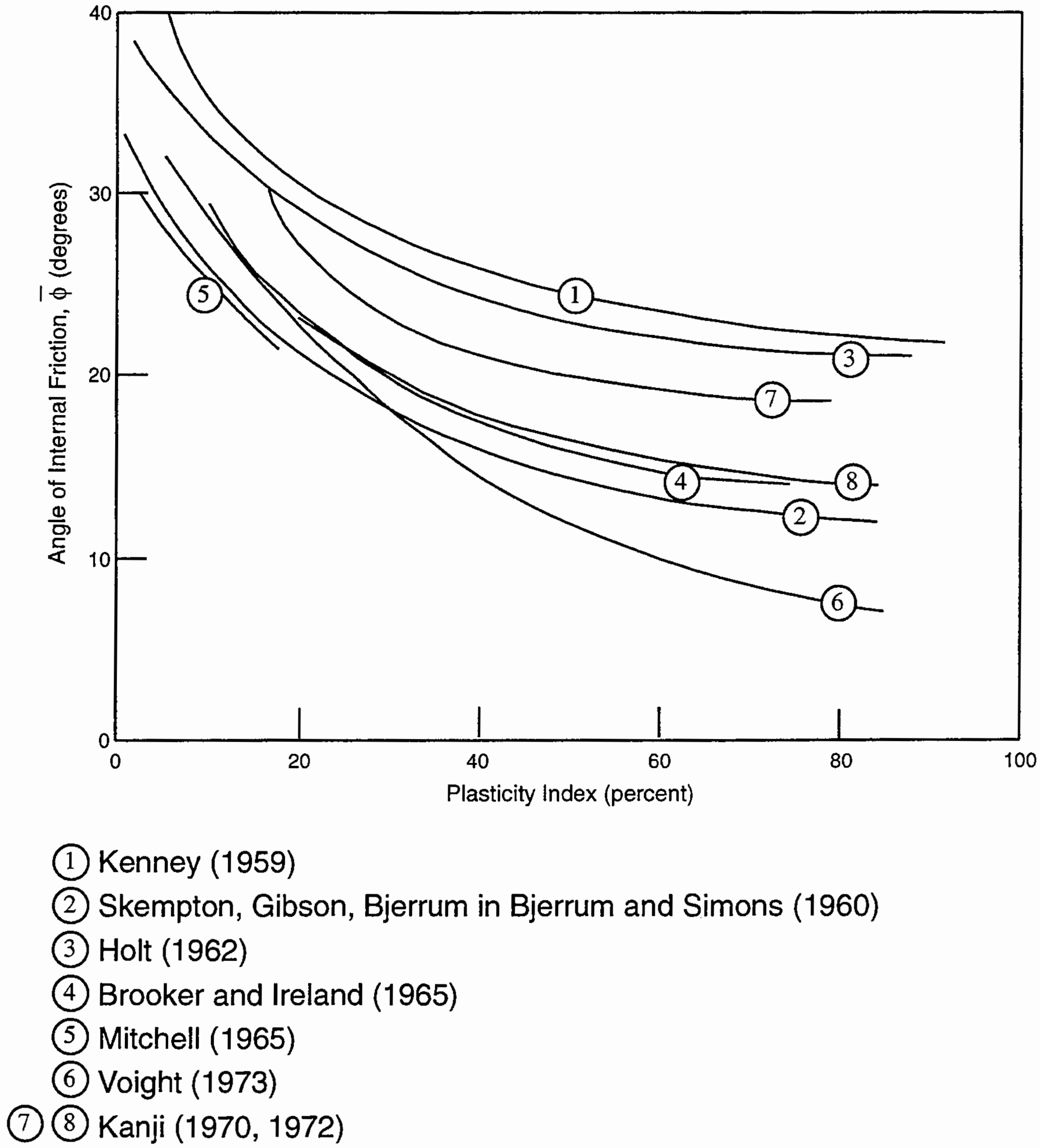


Figure 2.8 Variation in peak friction angle with plasticity index as determined by various investigators (Kanji, 1974)

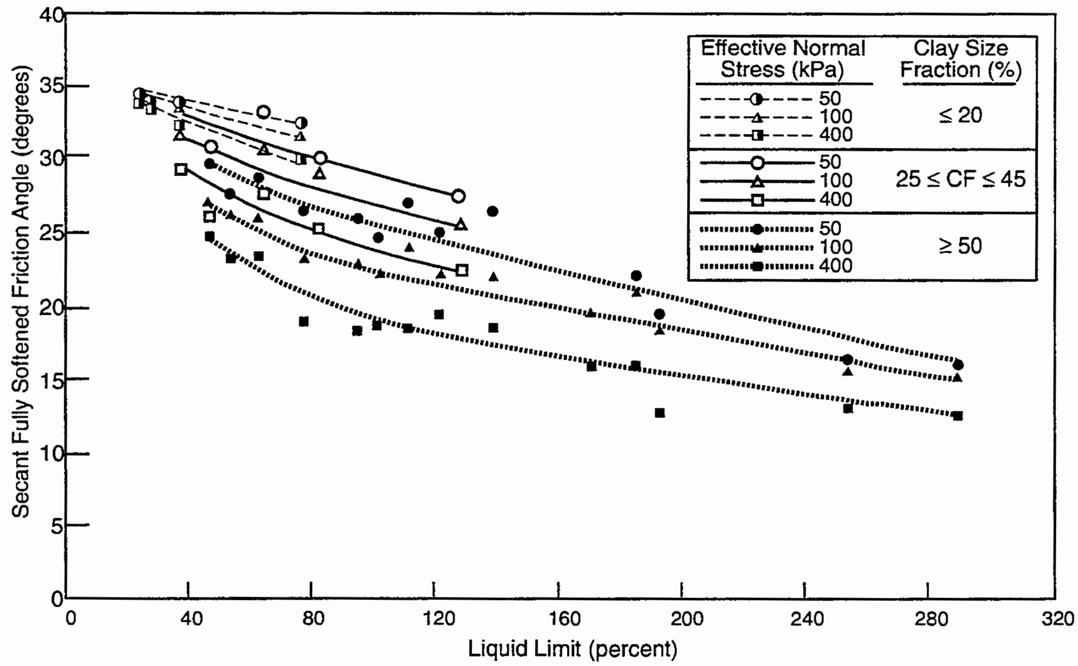


Figure 2.9 Relationship between fully softened friction angle and liquid limit (Stark and Eid, 1977)

Most studies have shown that the residual shear strength envelope exhibits no cohesion, and that clay size fraction and mineralogy are probably the most important parameters in estimating the residual friction angle, $\bar{\phi}_r$ (Lupini et al. [1981], and Skempton [1985]). It has also been found that the Mohr-Coulomb envelope for residual strengths is curved. Thus, secant values of $\bar{\phi}_r$ are stress dependent and decrease as effective stress increases.

Figure 2.10 presents a correlation between $\bar{\phi}_r$ and the plasticity index that was first developed by Voight (1973) and later modified by Bovis (1985). Skempton (1964, 1985) presented the correlations of residual friction angle, $\bar{\phi}_r$, and clay size fraction shown in Figure 2.11 for an effective normal stress equal to approximately 1 atmosphere. The correlations were developed using data for both normally consolidated and overconsolidated clays. Lamb (1985) showed how the residual friction angle changes with change in effective normal stress and plasticity index (Figure 2.12). Kulhawy and Maine (1990) recommended that $\bar{\phi}_r$ be obtained from Skempton's chart and then modified for the effect of normal stress using Lamb's correction. As noted above, the values in Skempton's chart are appropriate for an effective normal stress equal to approximately 1 atmosphere.

Stark and Eid (1994) conducted drained torsional shear tests on thirty-two different clays obtained from various parts of the world and developed a general correlation for the residual shear strength incorporating the liquid limit, clay size fraction, and effective normal stress. Their tests and stability analyses performed on selected known slope failures indicate that correlations based on either clay size fraction or Atterberg limits (liquid limit, or plasticity index) alone may not be sufficient. They found that for clays with a clay size fraction greater than 50 percent and liquid limits ranging from 60 to 220 percent, the residual shear strength envelope exhibits significant nonlinearity. Stark and Eid (1994) reported their data in terms of the secant residual friction angle, which corresponds to a linear failure envelope passing through the origin (Figure 2.5a). The secant friction angle corresponds to a particular effective normal stress. They showed that relationships exist between the secant residual friction angle and both liquid limit and clay size fraction for various effective normal stresses (Figure 2.13). Figure 2.14 from Stark and Eid's paper clearly shows the nonlinearity as expressed by the variation of the secant residual friction angle with effective normal stress.

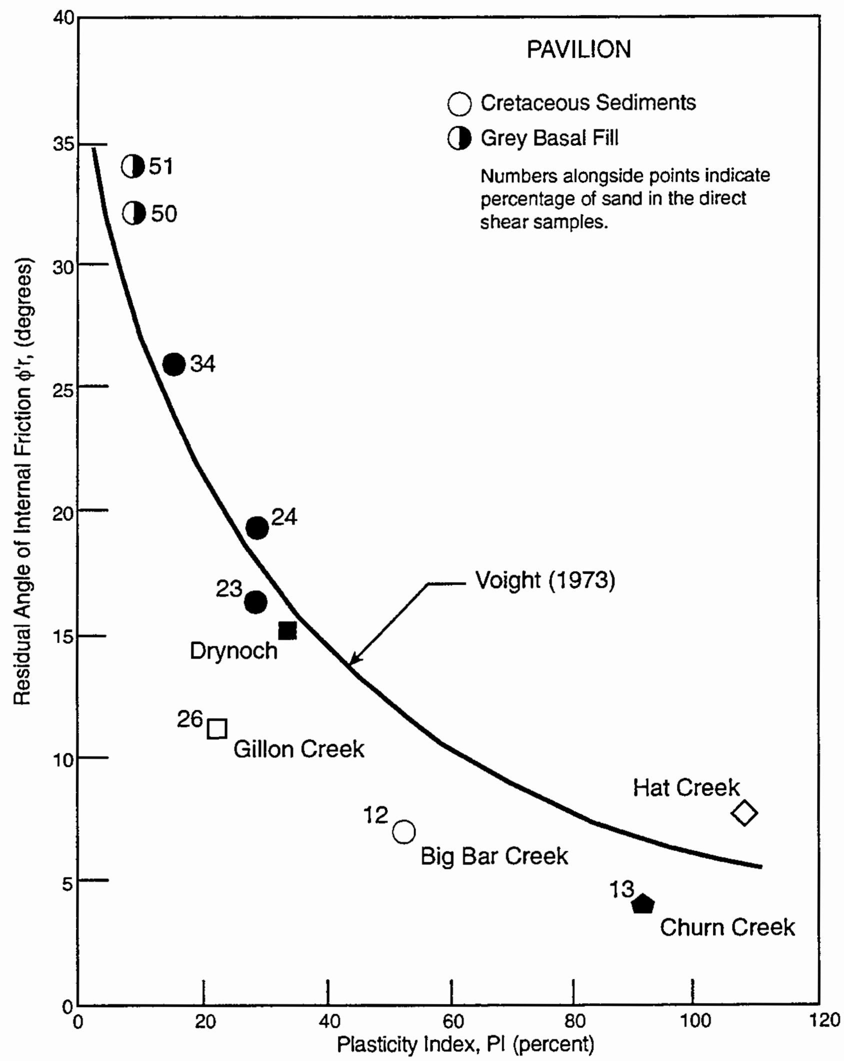
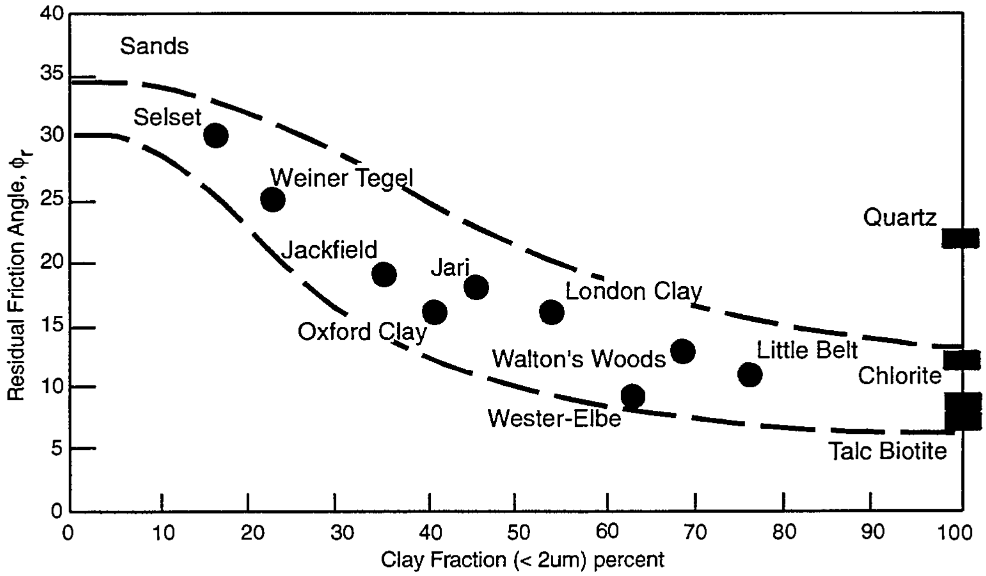
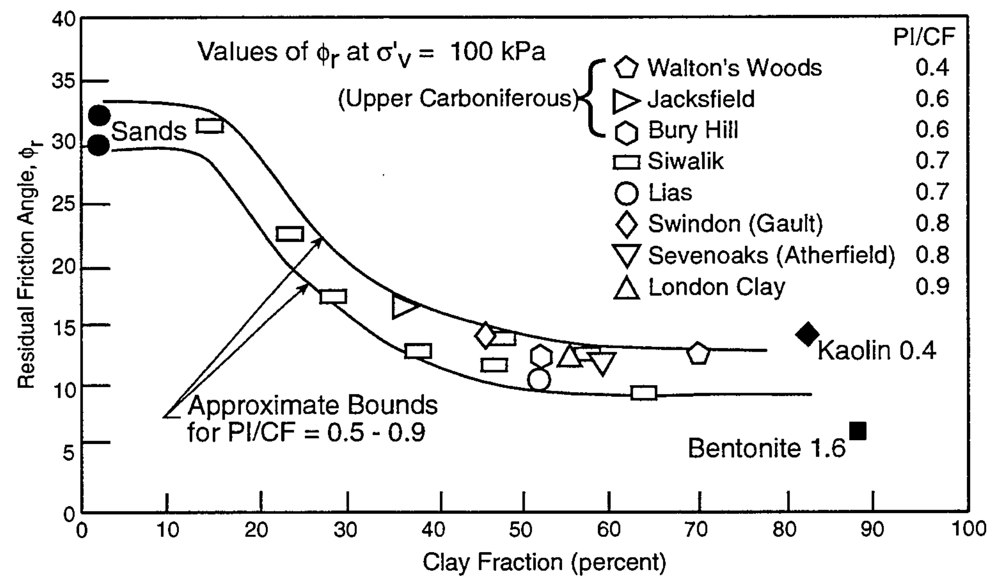


Figure 2.10 Relationship between residual friction angle and plasticity index (Bovis, 1985)

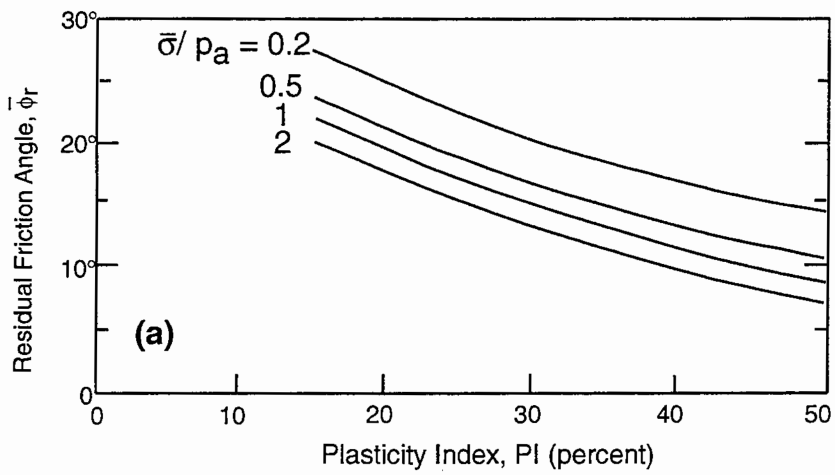


Residual Friction Angle/Clay Size Fraction Relationship (Skempton 1964)



Field Residual and Ring Shear Tests on Sands, Kaolin, and Bentonite (Skempton 1964)

Figure 2.11 Relationship between residual friction angle and percent clay fraction (Skempton, 1985)



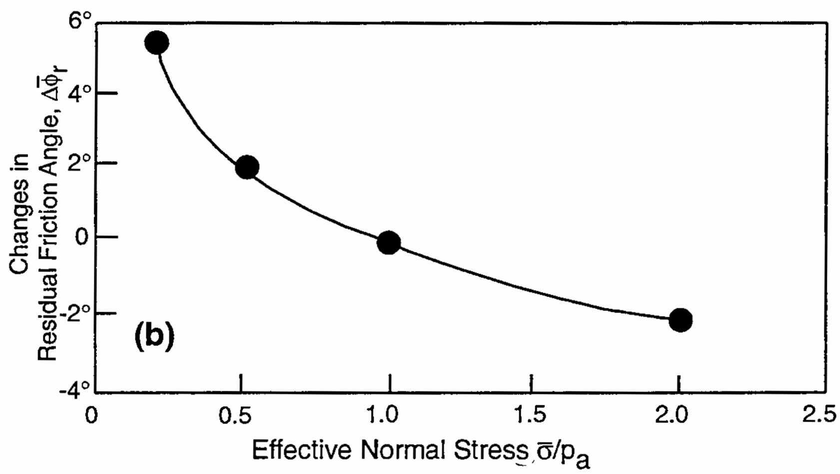


Figure 2.12 Relationship between residual friction angle and effective normal stress for soils at the Amuay landslide sites (Lambe, 1985)

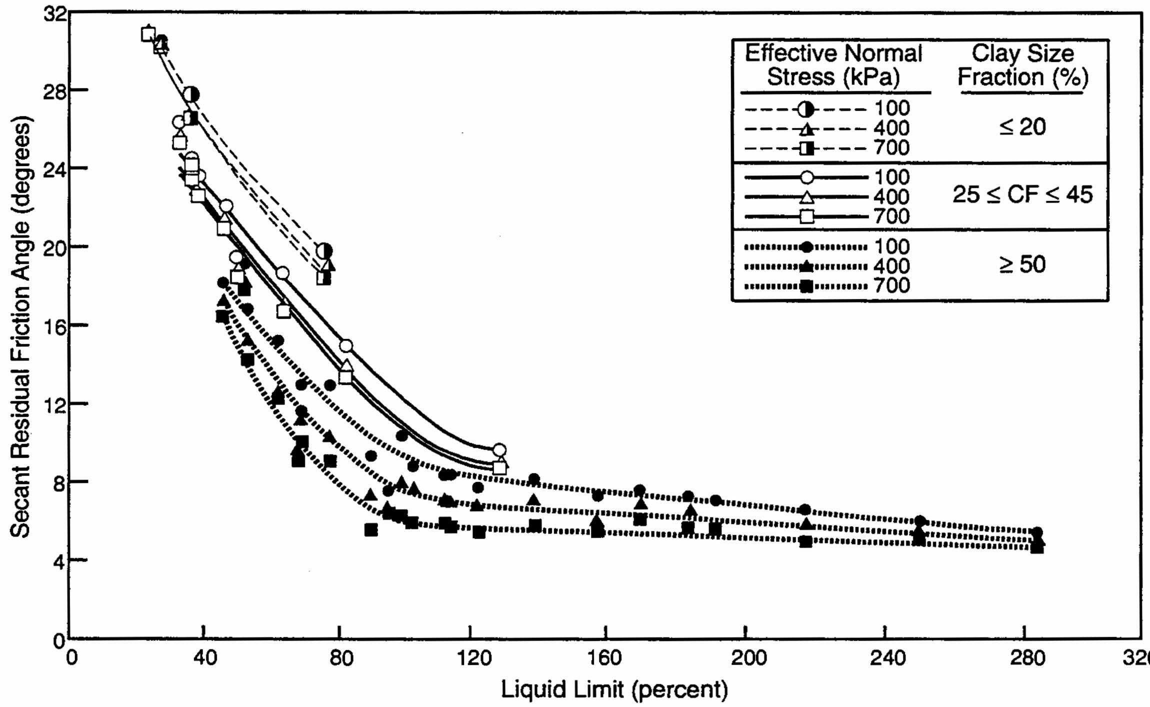


Figure 2.13 Relationship between drained residual friction angle and liquid limit (Stark and Eid, 1994)

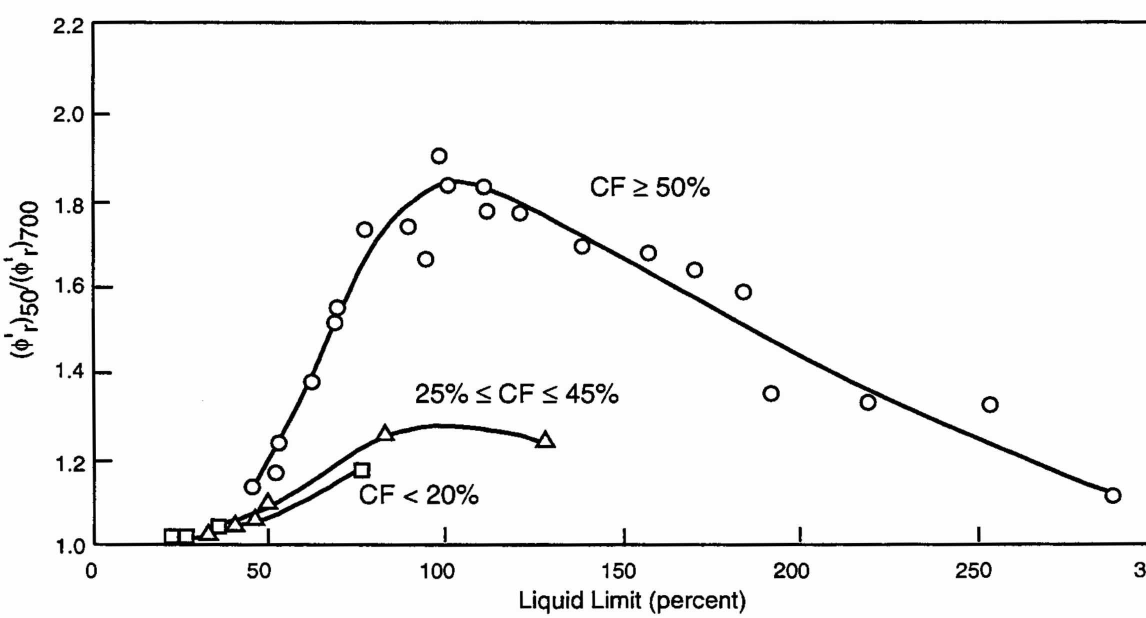


Figure 2.14 Variation of secant residual friction from effective normal stresses of 50 to 700 kPa as a function of liquid limit (Stark and Eid, 1994)

Most of the clays from the embankments that experienced failure in Texas have liquid limits of at least 60, and the clay size fraction is generally at least 50 percent (Stauffer and Wright, 1984). These soils lie in the range where Stark and Eid indicate the residual shear strength envelope is nonlinear. The observed failures in the highway embankments in Texas involved slides that rarely exceeded 7 feet in depth in embankments that have heights ranging from 10 to 30 feet. The range of effective normal stresses in these slopes is approximately 10-50 kPa, which is below the range of normal stresses (50-700 kPa) reported by Stark and Eid.

2.7 COMPARISON WITH DATA FROM TEXAS EMBANKMENT SLOPE FAILURES

A significant amount of shear strength data has been developed for compacted soils obtained from embankments in Texas. In the following sections these data are reviewed and compared with the correlations discussed earlier.

2.7.1 Peak Effective Stress Shear Strength Correlations

Gourlay and Wright (1984) performed laboratory tests on soil samples obtained from two of the embankments studied by Stauffer and Wright (1984). Consolidated undrained triaxial compression tests with pore water pressure measurements were performed on compacted specimens of what is believed to be clay from the Beaumont formation in the Houston, Texas, area. Table 2.1 summarizes index properties and peak effective stress shear strength parameters for Beaumont clay. Gourlay and Wright (1984) used color as an additional descriptor for the clays that they examined. Red clays were part of the soils found in an embankment at the IH 610 and Scott Road intersection in Houston; grey clays were identified in embankments both at the IH 610 and Scott Road location and at an embankment at the SH225 and SH146 intersection, also in the Houston area. Both of these soils are believed to be from the same Beaumont formation, with the red clays being more plastic.

Stauffer and Wright (1984) calculated factors of safety for slope stability for the two embankments (IH 610 and Scott Road and SH225 and SH146 embankments) based on the effective stress shear strength parameters reported from laboratory tests performed by Gourlay and Wright (1984) for Beaumont clay. The computed factors of safety were much higher than unity for the embankments that failed. Stauffer and Wright's (1984) analyses showed that the shear strengths measured in the laboratory on compacted specimens significantly overestimated the strength that was developed in the field. They found that the effective stress friction angle backcalculated from the observed slides agreed with that obtained in the laboratory, while the backcalculated cohesion ranged between 7 and 25 psf, and was much lower than that measured in the laboratory.

In an effort to understand the discrepancy observed by Stauffer and Wright (1984) between field and laboratory strengths, several additional laboratory testing programs were subsequently carried out at The University of Texas at Austin. The first was reported by

Green and Wright (1986). They performed triaxial compression tests to measure effective stress shear strength parameters on three types of specimens: (1) undisturbed specimens taken from slopes which had failed, (2) specimens prepared by consolidating soil from a slurry in the laboratory, and (3) specimens prepared by "packing" (remolding) soil into a special mold in the laboratory. In addition, peak and residual shear strengths were determined on conventional compacted specimens. All specimens came from the red Beaumont clay. Results of the various tests are summarized in Table2.2.

Table 2.1 Gourlay and Wright (1984) laboratory test results

Parameter	Beaumont Clay (Red)	Beaumont Clay (Grey)
PI %	52	38
LL	72	55
Clay size %	69	50
Activity	0.75	0.77
$\bar{c}$	270 psf	390 psf
$\bar{\phi}$	20.0 degrees	19.7 degrees

Table 2.2 Green and Wright (1986) laboratory test results

Type of Specimen	Test	$\bar{c}$, psf	$\bar{\phi}$, degrees	Strength Condition
Consolidated from a slurry	CU	110	24	Peak
Remolded "Packed" Specimens	CU	80	22	Peak
Compacted	DS	260	21	Peak
Compacted	DS	0	14	Residual
Undisturbed	CU	140	22	Peak

Although the cohesion determined from these tests varied depending on the type of specimen and how it was prepared, the peak effective stress friction angle varied only slightly (21–24 degrees). The additional laboratory tests performed by Green and Wright (1986) on compacted samples also reaffirmed the values reported earlier by Gourlay and Wright (1984). None of the tests explained the difference between field and laboratory strengths.

As a consequence of Green and Wright's observations, Rogers and Wright (1986) performed an additional series of tests on specimens that were subjected to various amounts of wetting and drying prior to shear. Rogers and Wright's work showed that repeated wetting and drying can significantly reduce the effective stress cohesion intercept; no significant changes in the effective stress friction angle were observed. Stability analyses performed using strengths measured on specimens subjected to wetting and drying showed significantly lower factors of safety for the two embankments mentioned earlier; however, the computed factors of safety were still much greater than unity. Subsequent research by Kayyal and Wright (1991) reaffirmed Rogers and Wright's findings and provided insight into the role of pore water pressures in explaining the discrepancies that existed between the field and laboratory shear strength data. Pore water pressures are discussed further in Section 2.8.

The data in Tables 2.1 and 2.2 suggest the difficulty that may be encountered in correlating the effective stress friction angle to the plasticity index or the liquid limit for the red and the grey Beaumont clay. Both soils exhibited almost the same friction angle, but had different Plasticity Indices and Liquid Limits. Based on the results shown in Figures 2.9 and 2.13 (Stark and Eid [1994, 1995]), it is possible for two soils to have the same friction angle but different liquid limits, provided the soils have different clay size fractions. Unfortunately, results reported in the literature usually report clays having clay size fractions greater than 50 percent as one group of soils with no further distinction.

Values of peak effective stress friction angle were estimated for the red and grey Beaumont clay using several of the correlations presented earlier. These values are shown in Table 2.3. The values were based on the index properties reported by Gourlay and Wright (1984) and on an effective normal stress of 50 kPa (approximately 1,000 psf). This normal stress (50 kPa) represents approximately the highest normal stresses expected along the potential failure surfaces in the two embankments mentioned earlier.

The friction angles obtained in Table 2.3 are secant friction angles, while the friction angles reported by Gourlay and Wright (1984) are tangent friction angles (see Figure 2.5). To illustrate the difference between these two friction angles (secant, tangent) consider the tangent strength parameters ($\overline{\phi}$ =21 degrees, $\overline{c}$ = 260 psf) reported by Green and Wright (1984) for compacted specimens, and an effective normal stress of 1,000 psf. For these values the equivalent secant friction angle is 33 degrees. If pore water pressures reduce the normal stresses to something less than 1000 psf, the secant value of the friction angle may be noticeably higher than 33 degrees. Thus, the secant friction angles shown in Table 2.3 actually fall well below the values obtained from laboratory data. This might be due to the fact that the correlations were developed based on normally consolidated clays, rather than on compacted specimens, which would behave more like overconsolidated clays at the normal stresses (1,000 psf) being considered.

Table 2.3 $\overline{\phi}$ *correlation results for the red and grey Beaumont clays*

Correlation Source	**Beaumont Clay (Red)**	**Beaumont Clay (Grey)**
Mitchell (1976)	25 degrees	27 degrees
Ladd (1977)	22 – 28 degrees	25- 30 degrees
Kanji (1974)	10 - 25 degrees	15- 28 degrees
Stark and Eid (1995)	27 degrees	28 degrees

2.7.2 *Shear Strength Correlations for Fully Softened Strengths*

Kayyal and Wright (1991) subsequently developed fully softened shear strength envelopes for the Beaumont clay and another clay soil obtained from an embankment near Paris, Texas. Kayyal and Wright (1991) found that the effective stress shear strength envelope exhibited pronounced nonlinearity at low stresses and the cohesion intercept was negligible. Index properties for the two soils tested by Kayyal and Wright (1991) are summarized in Table 2.4. The effective stress friction angles presented in Table 2.4 are secant values calculated from the curved Mohr-Coulomb envelopes with an effective normal stress of 50 kPa (approximately 1,000 psf). Effective stress friction angles for the Paris and Beaumont clays were estimated using the existing correlations for fully softened strengths; these angles are summarized in Table 2.5. Effective stress failure envelopes corresponding to the correlations are also plotted in Figures 2.15 and 2.16 for the Beaumont and Paris clays, respectively. Also plotted on these figures are the measured peak and fully softened shear strength envelopes measured by Kayyal and Wright (1991). It can be seen that all the correlations give strengths lower than any of the strengths obtained from the laboratory. However, Stark and Eid's correlation, although still conservative, results in values in reasonable agreement with the measured fully softend strength.

2.7.3 *Residual Shear Strength Correlations*

To examine potential values for residual strengths for soils like those used in Texas embankments, the correlations for residual strengths were used to calculate $\bar{\phi}_r$ for Beaumont and Paris clays. The soil index properties previously shown in Table 2.4 for the Paris and Beaumont clays were used for this purpose. Table 2.6 lists the $\bar{\phi}_r$ values obtained from the various charts and correlations. The residual friction angles obtained from Stark and Eid (1994) are for an effective normal stress of 100 kPa (or 1 atmosphere), which is much higher than the range of stresses expected in the Paris and Beaumont embankment slopes. If the friction angles shown for Stark and Eid (1994) are increased by several degrees to account for the nonlinearity in the residual strength envelope and the higher values at lower normal stresses, the values may agree more closely with the average values obtained by the approach recommended by Kulhawy. Stark and Eid's data agree with Skempton's data for normal stresses of about 1 atmosphere.

Residual failure envelopes for the Beaumont clay obtained using the correlation listed in Table 2.6 are plotted in Figure 2.17. Plotted on the same figure is the residual shear strength for compacted specimens of the Beaumont clay reported by Green and Wright in 1986 (see Table 2.2). As can be seen in Figure 2.17, Stark and Eid's correlation results in a residual strength very similar to that which was measured by Green and Wright.

As expected, the measured fully softend strength is much greater than any of either the measured or correlated residual strengths of the Beaumont clay (Figure 2.18a). Similarly, residual strengths estimated for the Paris clay from the various correlations are shown to be significantly less than that of the measured fully softend strength (Figure 2.18b).

2.8 ANALYSES OF CASE HISTORIES USING ESTIMATED SHEAR STRENGTHS

To further establish the validity of the correlations for shear strength, strengths were estimated from the correlations and then used to perform stability analyses for a number of the Texas highway embankment slopes that had failed. The slopes examined are the ones reported by Stauffer and Wright (1984) and Kayyal and Wright (1991). A list of these slopes and relevant data are presented in Table 2.7.

Table 2.4 Soil data related to Paris and Beaumont embankments

Parameter	Paris Clay	Beaumont Clay
PI %	58	52
LL	80	73
Clay size %	58	48
Activity	1.0	1.1
Secant Friction Angle (for σ= 50 kPa or 1000 psf)	30°	35°

Table 2.5 $\bar{\phi}$ correlation results for the Paris and Beaumont clays

Correlation Source	Paris Clay	Beaumont Clay
Mitchell (1976)	24 degrees	25 degrees
Ladd (1977)	23 - 28 degrees	22- 27 degrees
Kanji (1974)	11 – 23 degrees	12- 24 degrees
Stark and Eid (1995)	27 degrees	27 degrees

Table 2.6 Residual friction angle for the Paris and Beaumont embankments

Reference	Paris Clay	Beaumont Clay
Voight (1973) and Bovis (1985)	11.5 degrees	10.5 degrees
Skempton (1964, 1985)	9-14 degrees	10-15 degrees
Modified as per Kulhawy and Maine (1990) (average values)	13-18 degrees (15.5 degrees)	14-19 degrees (16.5 degrees)
Stark and Eid (1994); using 1 atmosphere	12 degrees	13 degrees

Table 2.7 Data of case histories used to examine recommended correlations

Case No.	Site	District No.	Soil	Total Unit Weight, pcf	Slope Ratio	Slope Angle, deg.	Slide Depth, ft	PL %	LL %	PI %	Clay Size Fraction %
1	Loop 286 @ Missouri Pacific RR Overpass, SW quadrant (north side), Lamar Co.	1	Grey Clay	112	2.9	19.0	8	18.4	48.3	29.9	43.2
2	Loop 286 @ SH 271 Interchange, NW quadrant, Lamar Co.	1	Light Brown Clay	112	2.5	21.8	4	18.8	58.1	39.3	55.3
3	Loop 286 @ Missouri Pacific RR Overpass, NW quadrant, Lamar Co.	1	Tan Clay	112	2.7	20.3	10	24.8	71	46.2	67.1
4	Loop 286 @ FM 79, SW quadrant, Lamar Co.	1	Tan Clay	112	2.3	23.5	4	25.2	73.8	48.6	68.7
5	Loop 286 @ Missouri Pacific RR Overpass, SW quadrant (south side), Lamar Co.	1	Tan Clay	112	2.8	19.7	6	27.5	75.8	48.3	69.9
6	U.S. 59 @ FM 525, NE quadrant, Harris Co.	12	Light Tan Clay	120	2.4	22.6	3	11.3	42	30.7	40.1
7	U.S. 59 @ Shepard St., SE quadrant, Harris Co.	12	Tan Clay	120	3.1	17.9	3.5	13.1	45.4	32.3	37
8	IH 610 & Richmond, SW quadrant, Harris Co.	12	Grey Clay	120	2.7	20.3	5	15.5	53.7	38.2	45.5
9	SH 225 @ Southern Pacific RR Overpass, SE quadrant, Harris Co.	12	Grey Clay	120	2.6	21.0	4	18	53.8	35.8	53.6
10	SH 225 @ Southern Pacific RR Overpass, SE quadrant, Harris Co.	12	Grey Clay	120	3.1	17.9	3	18	53.8	35.8	53.6
11	IH 610 & Scott St., NE Quadrant, Harris Co.	12	Red Clay	120	2.6	21.0	3.5	16.3	54.5	38.2	49.45
12	IH 45 @ SH 146, south side, Harris Co.	12	Brown/ Grey Clay	120	3.1	17.9	3.5	16.5	56.7	40.2	48.3
13	IH 610 & SH 225, SE quadrant, Harris Co.	12	Dark Grey Clay	120	2.7	20.3	2	15.9	58.1	42.2	46.9
14	IH 10 & Crosby, NW quadrant, Harris Co.	12	Tan Clay	120	2.6	21.0	5	16	61.2	45.2	53.2
15	SH 225 @ SH 146, NE quadrant, Harris Co.	12	Grey Clay	120	3.4	16.4	3.5	20.7	62.8	42.1	65.5
16	SH 225 @ SH 146, NW quadrant, Harris Co.	12	Red Clay	120	3.1	17.9	2.4	22.6	63.2	40.6	75
17	IH 45 @ FM 2351, NE quadrant, Harris Co.	12	Red/Br own	120	2.5	21.8	2.5	19	67.8	48.8	59.4

Case No.	Site	District No.	Soil	Total Unit Weight, pcf	Slope Ratio	Slope Angle, deg.	Slide Depth, ft	PL %	LL %	PI %	Clay Size Fraction %
			Clay								
18	IH 45 @ SH 146, SE quadrant, Harris Co.	12	Red/Brown Clay	120	3	18.4	3	18.9	68.7	49.8	57.8
19	SH 225 @ SH 146, SW quadrant, Harris Co.	12	Brown Clay	120	3	18.4	4.3	21.2	70.4	49.2	63.3
20	SH 225 @ Scarborough, SE quadrant, Harris Co.	12	Dark Grey Clay	120	2.1	25.5	3	18.2	70.9	52.7	60
21	SH 225 @ Southern Pacific RR Overpass, NW quadrant, Harris Co.	12	Brown/Tan Clay	120	3.1	17.9	2.5	16.6	71.2	54.6	61.2
22	IH 45 @ College St., NE quadrant, Harris Co.	12	Grey/Olive Clay	120	3	18.4	2	17.9	88.8	70.9	67.4
23	SH 225 @ Southern Pacific RR Overpass, SW quadrant, Harris Co.	12	Tan Clay	120	2.4	22.6	5	29.6	97	67.4	86.4
24	U.S. 87 @ Loop 175, NW quadrant, Victoria Co.	13	Tan Clay	120	2.2	24.4	5	19.4	60	40.6	60.1
25	U.S. 290 5 miles east of IH 35, NW quadrant, Travis Co.	14	Grey Clay	120	2.5	21.8	6	23.2	62.5	39.3	61.8
26	U.S. 77 @ SH 21, SW quadrant, Lee Co.	14	Gold/Tan Clay	120	3.4	16.4	4	27.2	64.4	37.2	52.7
27	U.S. 77 @ SH 21, NW quadrant, Lee Co.	14	Gold/Brown Clay	120	2.9	19.0	3	33.2	86.7	53.5	75.5
28	U.S. 79 @ U.S. 95, SE quadrant, Williamson Co.	14	Tan Clay	120	2.3	23.5	6	23.1	92.7	69.6	76.4
29	Houston Embankment, Kaayal & Wright (1991)	12		114	2.5	21.8	4	21	73	52	47.3
30	Paris Embankment, Kaayal & Wright (1991)			107	3	18.4	5	22	80	58	58

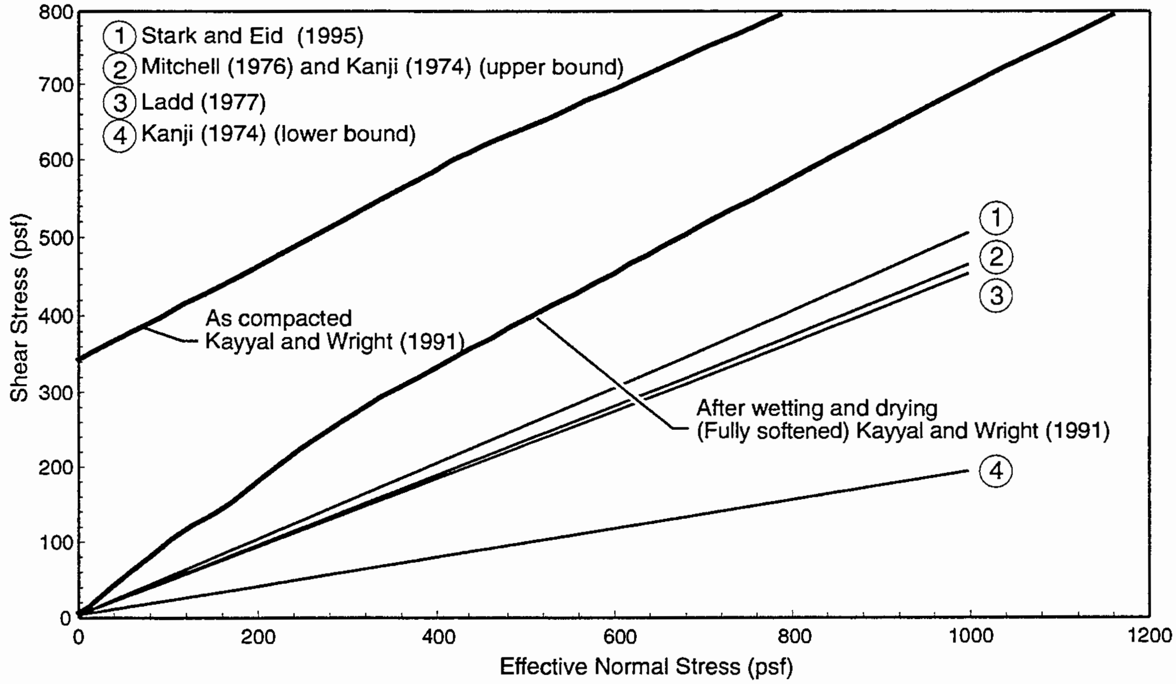


Figure 2.15 Effective stress Mohr-coulomb failure envelopes for Beaumont clay

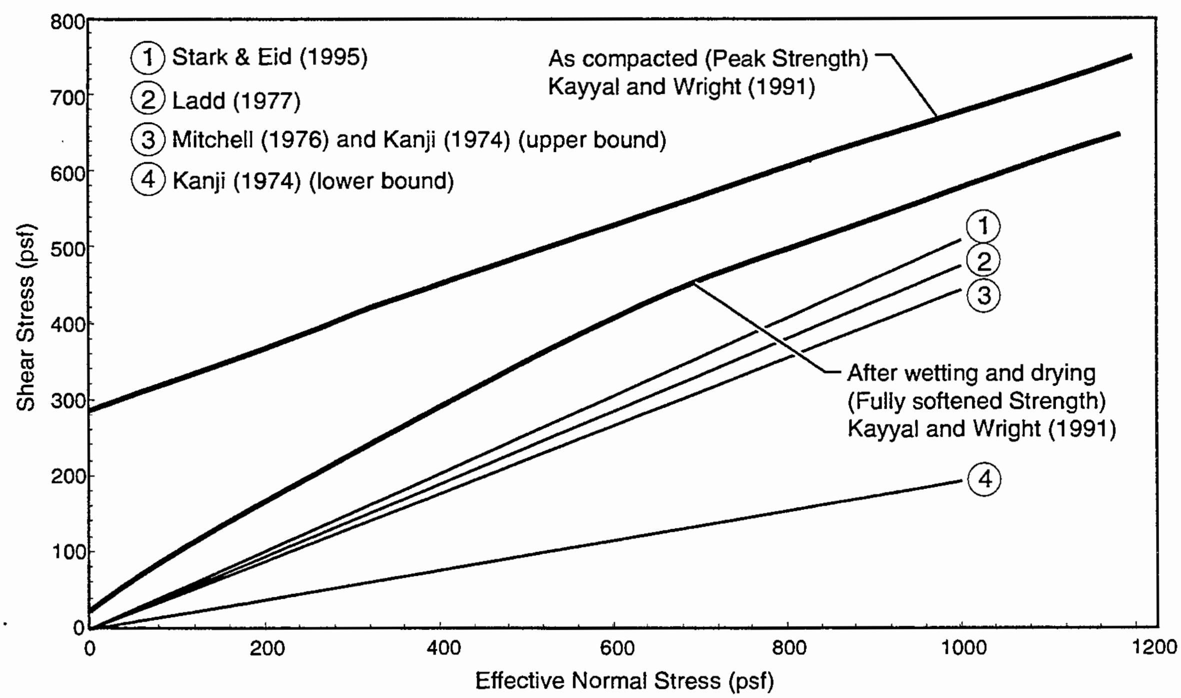


Figure 2.16 Effective stress Mohr-coulomb failure envelopes for Paris clay

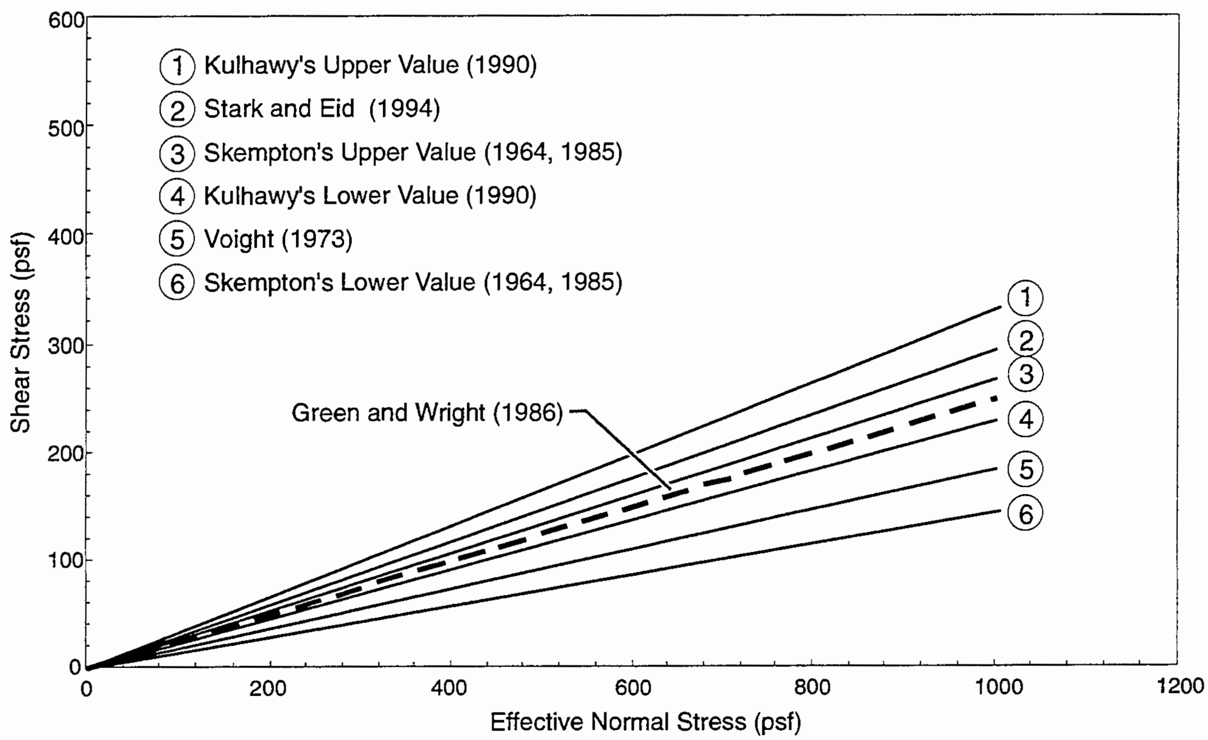


Figure 2.17 Residual strength envelopes for Beaumont clay

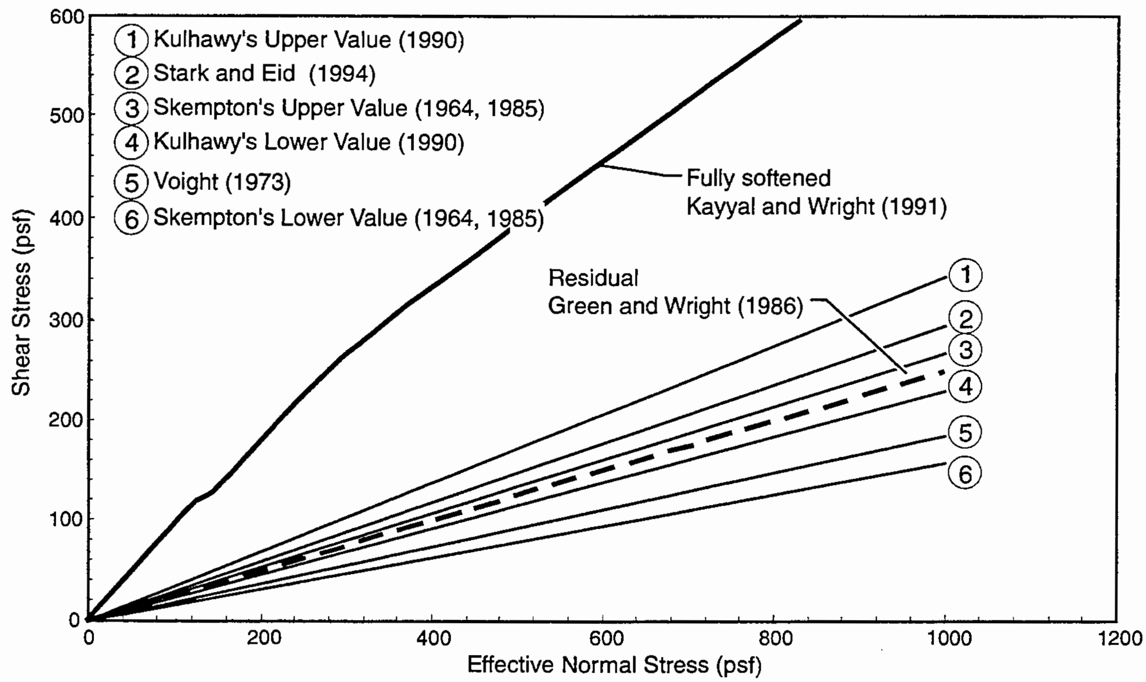


Figure 2.18a Fully softened and residual strength envelopes for Beaumont clay

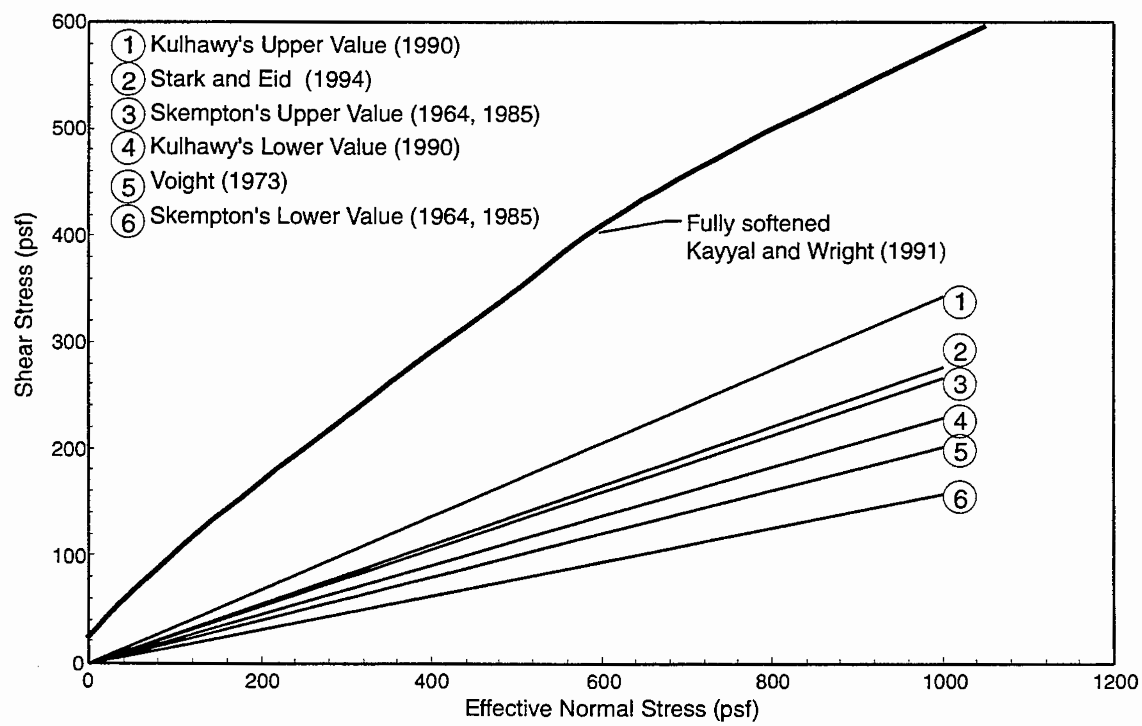


Figure 2.18b Fully softened and residual strength envelopes for Paris clay

An important unknown for each of the slopes that failed is the pore water pressure at the time of failure. Kayyal and Wright (1991) suggested that positive pore water pressures, as large as those that would exist with a water level at the ground surface, might have existed at failure. Consequently, in exploring the validity of the correlations for slopes in Texas, two extreme cases of pore water pressure were examined: (1) zero pore water pressure, and (2) pore water pressures represented by a water table at the ground surface (Figure 2.19). These two cases are expected to bracket fully the probable extremes of pore water pressures in the slope at failure.

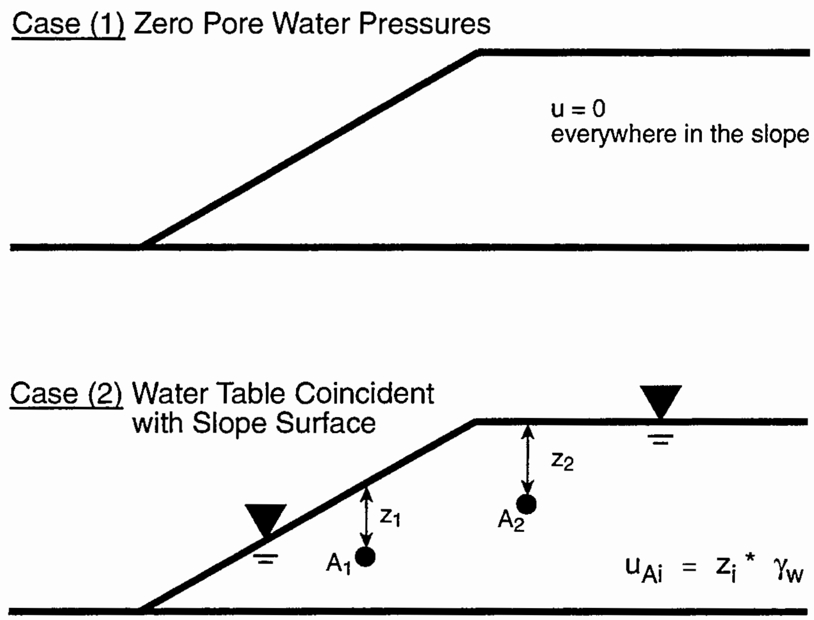


Figure 2.19 Two extreme pore water pressures conditions considered for stability analyses of case histories

Stark and Eid's correlation for fully softened strengths was used to estimate the shear strength parameters. Their correlation was shown in Section 2.7.2 to produce strengths closest to the measured strengths and is believed to be based on as extensive an amount of data as any of the correlations presented. The lowest value for effective stress for which Stark and Eid presented correlations is 50 kPa. This value (50 kPa) is much larger than the values (6 to 40 kPa) expected for typical embankments slope failures in Texas. Because the shear strength envelope shows significant curvature at effective stresses near 50 kPa, it was judged appropriate to adjust the values from Stark and Eid's correlation to account for the lower stresses.

To adjust values of the strength for lower stresses, previous work by Duncan et al. (1989) was used. Duncan et al. (1989) suggested that the values of the friction angle decrease in proportion to the logarithm of the confining pressure. To adjust the friction angles from Stark and Eid's correlation, values corresponding to various liquid limits (40, 60, 80, 100 and 120) and a clay size fraction higher than 50 percent were read from Stark and Eid's chart (Figure 2.9) for the three values (50, 100 and 400 kPa) of effective stress reported. The values for the friction angle were then replotted versus the logarithm of effective normal stress as shown in Figure 2.20. Straight trend lines shown in Figure 2.20 were extended (extrapolated) to cover the range (6 to 40 kPa) of effective stresses expected in Texas embankment slopes. The range of effective stresses was computed using the maximum effective stress estimated for each slide and the two extremes of pore water pressures discussed above. Details of how the maximum effective stress was computed are illustrated in Figure 2.21. For zero pore water pressures, the average maximum effective stress for all slides examined in this report is approximately 25 kPa. For the case of a water table coincident with ground surface, the average maximum effective stress is approximately 10 kPa. The values of liquid limit represented in Figure 2.20 were selected to cover the range of liquid limits (42 to 97) for the slopes considered. Using the line drawn in Figure 2.20, values of friction angle corresponding to various selected liquid limits were estimated for two new levels of effective stresses (10 and 25 kPa). The new values are replotted in Figure 2.22 along with Stark and Eid's data for 50, 100, and 400 kPa stress levels.

Using the information presented in Table 2.7, maximum effective stresses were computed and a friction angle was estimated from Figure 2.22 for each of the slopes. Stability analyses were then performed to compute a factor of safety using the estimated friction angle with zero cohesion. The factors of safety from the stability analyses are summarized for each slope in Figure 2.23. These results show that Stark and Eid's correlation with a water table at the ground surface would predict failures for all of the embankments that failed. On the other hand, with the assumption of zero pore water pressures, none of the failures would have been predicted. Although the assumption of a ground water table at the ground surface may be overly conservative in some cases, especially when coupled with the apparent conservatism of Stark and Eid's correlations, unless more detailed investigations of ground water levels and shear strength are performed, such conservative assumptions may be necessary.

2.9 CONCLUSIONS

Various correlations between shear strength and simple index properties (Atterberg limits, grain size) reported in the literature have been examined to determine their applicability to typical slopes in Texas. Emphasis was placed on long-term stability and drained shear strengths, which are of primary interest. Data in the literature were supplemented by data and experience obtained from other research on slope failures at The University of Texas.

Based on these studies, the recommendations presented in the next two sections are made. Conclusions and recommendations in the first section apply to the design of new slopes. The second section deals with strength correlations related to design of slopes that have experienced previous sliding.

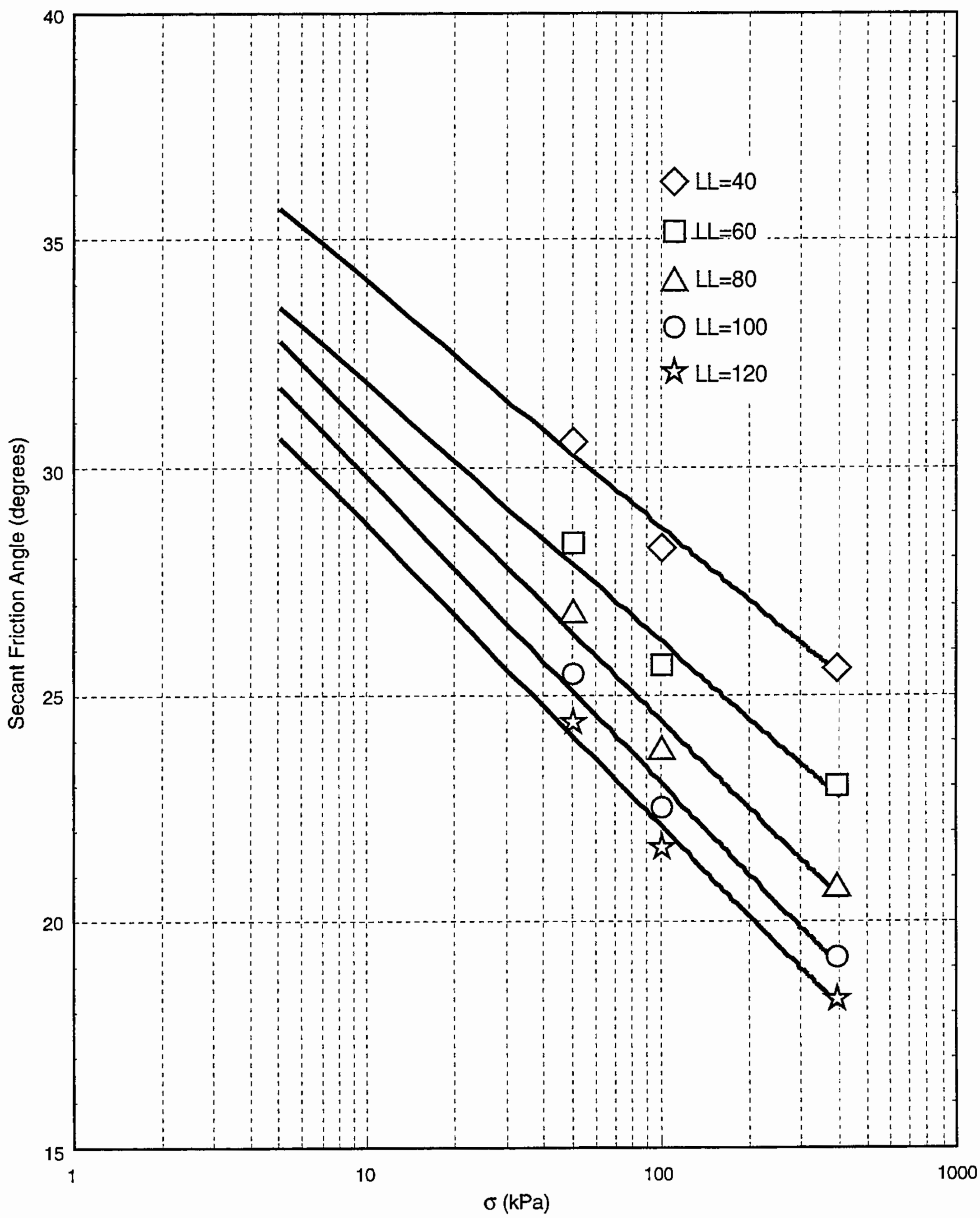


Figure 2.20 Estimated "trend" lines relating fully softened secant friction angle to effective stress for selected values of liquid limit

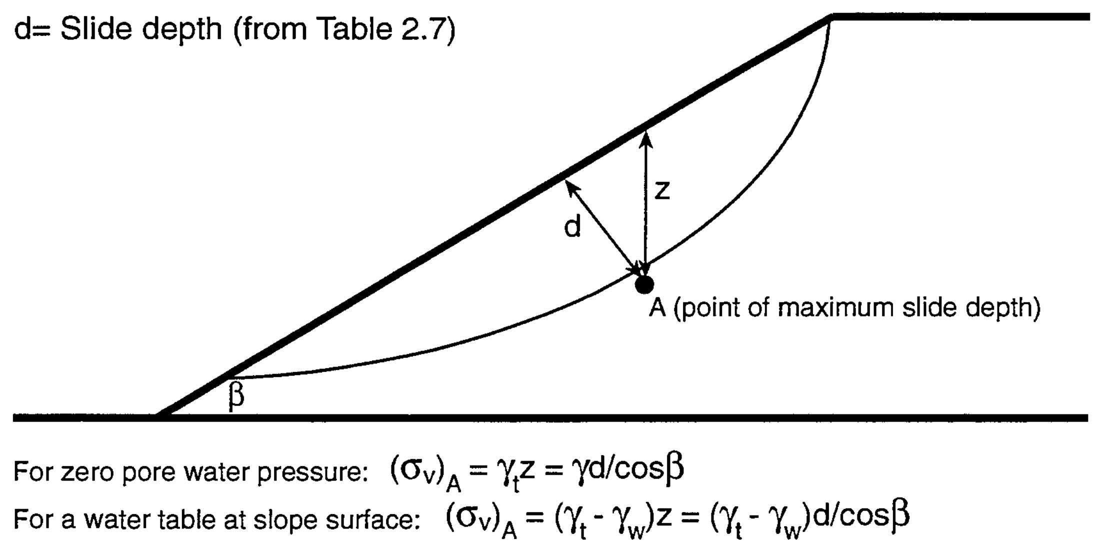


Figure 2.21 Approximation of maximum effective vertical stress along a slide

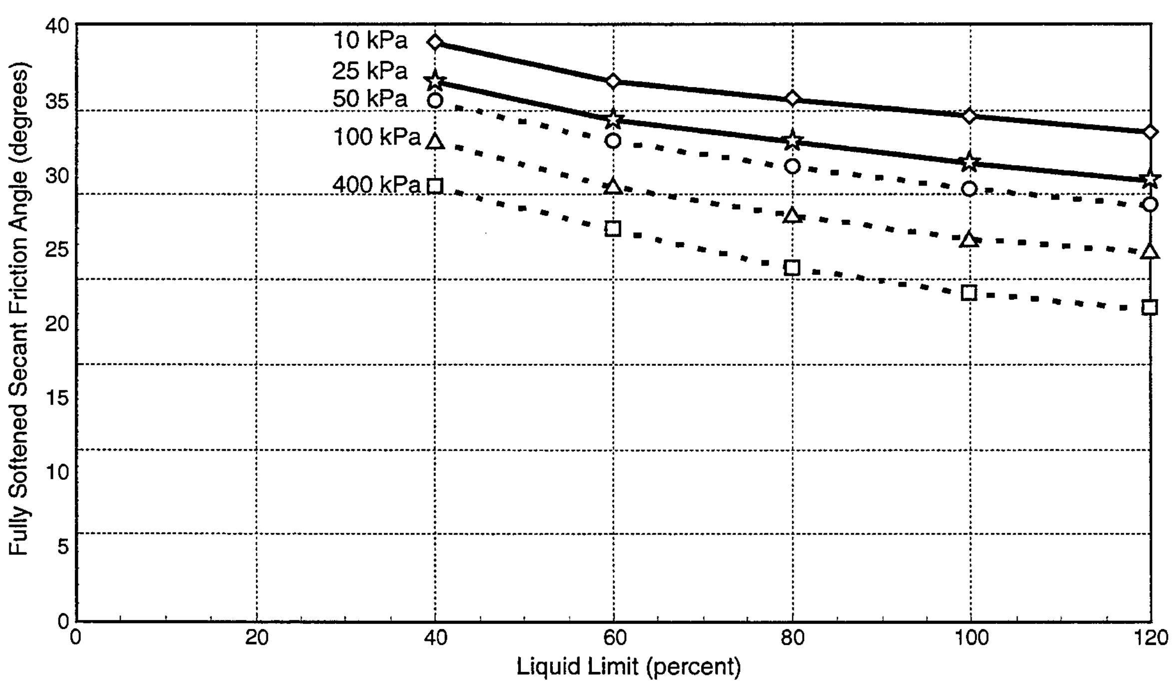


Figure 2.22 Correlation between secant friction angle for fully softened strength and liquid limit for selected values of effective stress

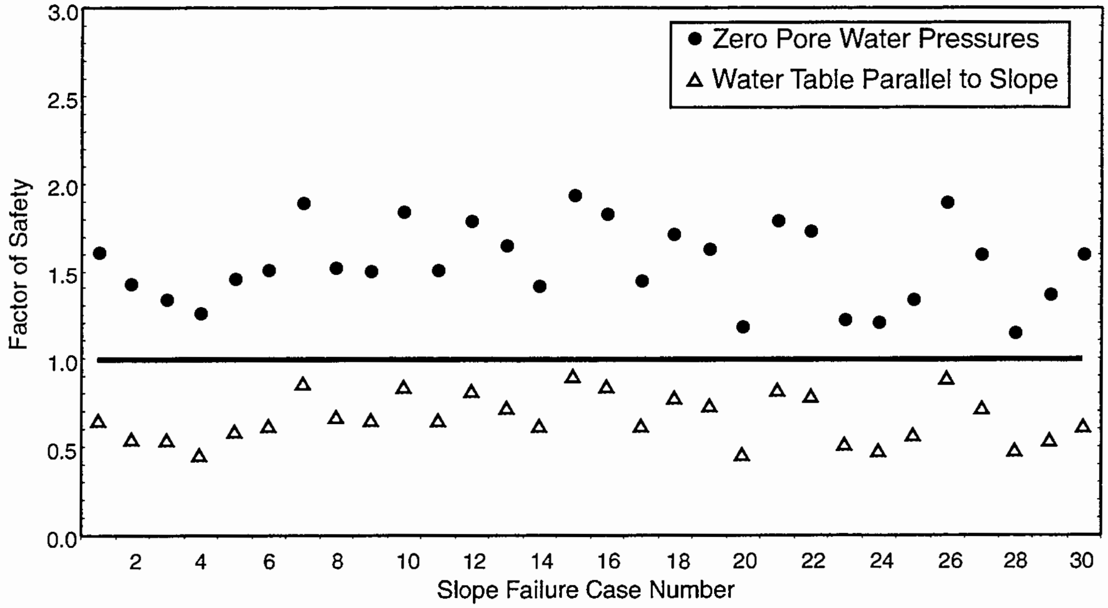


Figure 2.23 Factors of safety for case histories using shear strength estimated from Stark and Eid's correlation and two extreme cases of pore water pressures

2.9.1 Fully Softened Effective Shear Strength Correlations

Fully softened strengths are recommended for the design of new slopes and slopes that have not failed previously. Stark and Eid's correlation for fully softened shear strengths appears slightly conservative, but reasonable, and, therefore, is recommended. Stark and Eid's correlation is shown in Figure 2.22, along with the extension developed in this study to include effective stresses lower than those originally reported.

While Stark and Eid's correlation appears reasonable for obtaining fully softened effective stress shear strength parameters, significant positive pore water pressures must also be considered for design of slopes. Calculation of safety factors for a number of case histories showed that even using Stark and Eid's conservative correlation for shear strength, slope stability would be overestimated if pore water pressures were assumed to be zero. In the absence of more complete laboratory strength tests and field studies, it is recommended that when these correlations are used, a water table coincident with the surface of the slope be assumed. The cost of this high degree of conservatism may well warrant more detailed investigations and testing.

2.9.2 Residual Shear Strength Correlations

It is recommended that the residual shear strength, rather than the peak shear strength, be used for slopes that have experienced prior sliding. Very little data are available covering recurring slides in Texas. There is also very little data on residual shear strength for Texas soils. Based on the limited data presented by Green and Wright (1986), shown in Figure 2.17, it appears that Stark and Eid's (1994) correlation agrees well with the data and, therefore, it is again recommended for estimating residual strengths. Procedures like those described in Section 2.8 were used to extend Stark and Eid's (1995) chart for residual shear strengths to lower stress levels. The extended correlations presented in Figure 2.24 are recommended for making estimates of residual strength for long-term stability analysis of slopes that have experienced prior sliding.

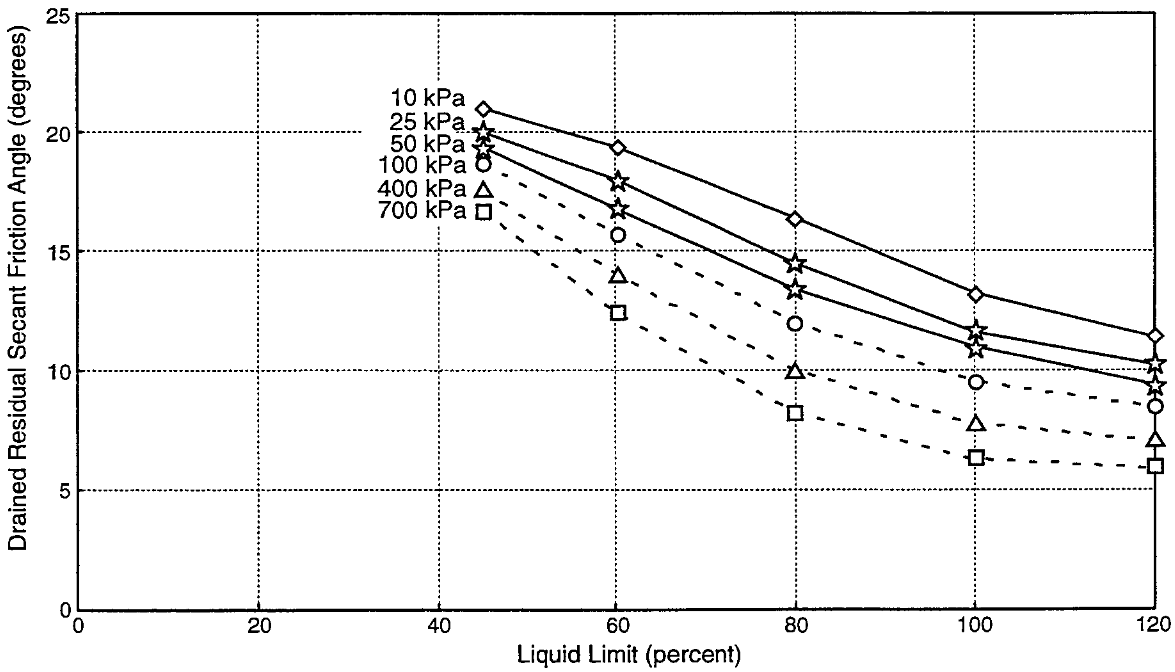


Figure 2.24 Correlation between drained residual secant friction angle for selected values of effective stress

CHAPTER 3. SUGGESTED PROCEDURE FOR ESTIMATING SHEAR STRENGTH PARAMETERS FROM INDEX PROPERTIES

3.1 INTRODUCTION

In Chapter 2, various correlations between shear strength parameters and index properties were examined. In this chapter, specific procedures and steps are presented for using these correlations to estimate shear strengths for the design of typical highway slopes in Texas. Based on extensive field investigations and laboratory studies of failures of highway slopes in Texas, the majority of slopes that have failed by sliding possess the following common characteristics:

(1) The soils are highly plastic clays—liquid limits > 50.

(2) The slopes fail a number of years (usually at least 10) after construction and, thus, the applicable shear strengths are the drained shear strengths expressed in terms of effective stress shear strength parameters ($\bar{c}$ and $\bar{\phi}$).

(3) The highly plastic clays have essentially no cohesion intercept ($\bar{c} = 0$) at the time of failure (a number of years after construction).

Based on these observations, only effective stress friction angles $\bar{\phi}$ and clays having liquid limits of at least 50 are considered. The effective stress cohesion is assumed to be zero. Because of the lack of experience with slope failures in low plasticity soils, no recommendations are made for such soils; however, the information presented in Chapter 2 may be useful for making preliminary estimates of shear strength parameters of low plasticity soils.

As discussed in Chapter 2, different shear strengths are applicable for slopes that have not failed and for slopes that have failed at least once. Thus, separate procedures are presented for each.

3.2 SHEAR STRENGTH PARAMETERS FOR DESIGN OF NEW SLOPES

The applicable shear strength for slopes in highly plastic clays that have not failed previously is the fully softened shear strength. Although the Mohr-Coulomb envelope for fully softened shear strengths is typically curved, the shear strength can be expressed using a "secant" friction angle, $\bar{\phi}_s$, and the equation,

$$s = (\sigma - u)\tan\bar{\phi}_s \tag{3.1}$$

where σ and u are the total normal stress and pore water pressure on the failure surface, respectively. The secant friction angle depends on the effective normal stress $(\sigma - u)$ and the liquid limit of the soil. The relationship between the fully softened secant friction angle and the liquid limit and effective normal stress was presented in Chapter 2 (Fig. 2.20) and is shown again in Figure 3.1. To determine the secant friction angle, the following steps are used:

1. Either determine from laboratory tests or estimate from past experience the liquid limit for the clay.

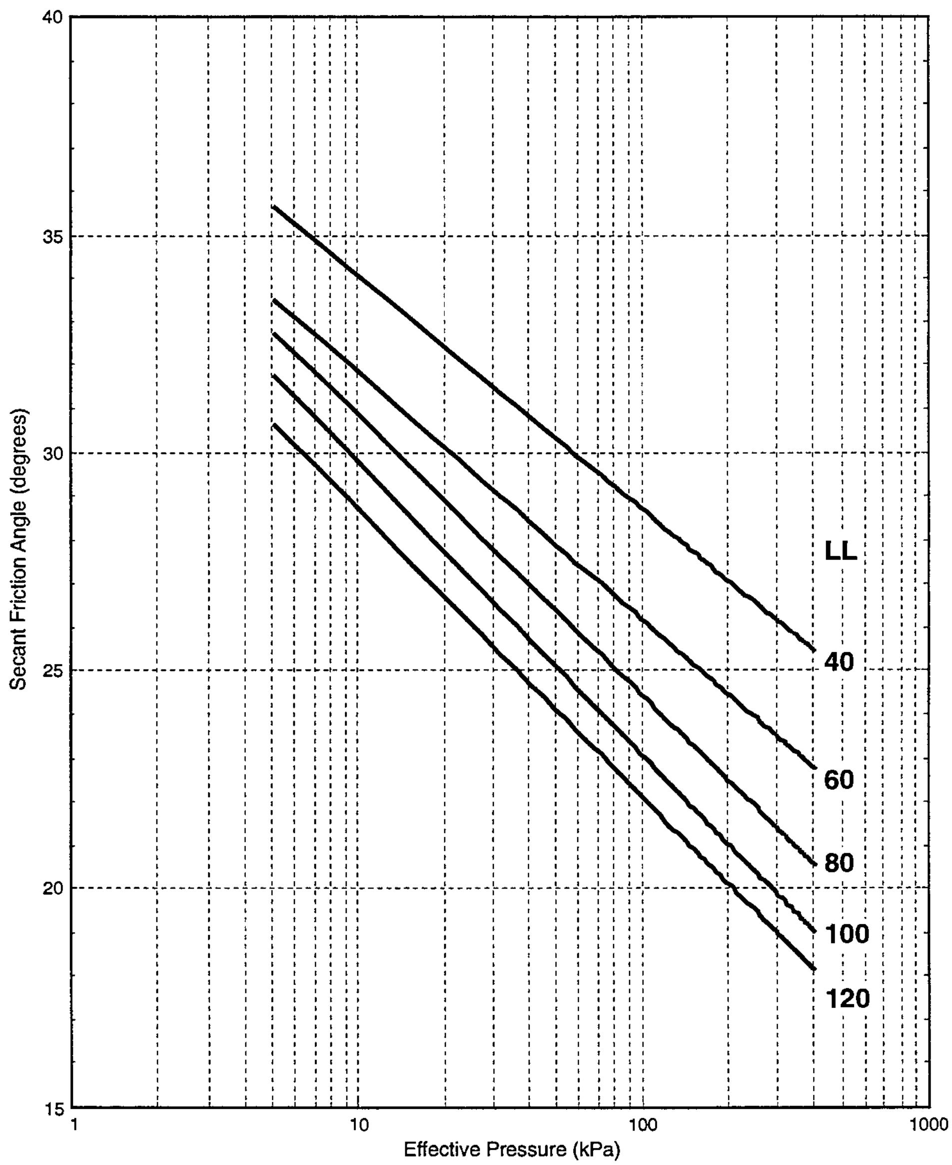


Figure 3.1 Correlation between fully softened secant friction angle and effective stress for selected values of liquid limit

2. Estimate the nominal effective normal stress ($\bar{\sigma}$) along potential sliding surfaces. Typically slides are shallow. A good approximation is that the slide depth will be 20 percent of the slope height. Thus,

   Slide depth = 0.2 x H

   where H is the slope height.
   The effective normal stress can be estimated to be approximately 2.5 kPa (50 psf) per foot of slide depth. Thus, the nominal effective stress along the slide surface would be:

   $$\bar{\sigma} = (2.5)(0.2)H = 0.5H.kPa$$

   For a 20-foot-high slope, the effective stress would be 10 kPa (0.5x20). This is the maximum effective stress along the slide surface. The average effective stress will be lower. However, the maximum effective stress is recommended (it gives a smaller, more conservative estimate for $\bar{\phi}$).

3. Once the liquid limit and effective normal stress are estimated, the secant friction angle can be estimated from Figure 3.1. Say, for example, that the liquid limit is 80 and the effective normal stress has been estimated to be 10 kPa. Then, from Figure 3.1, the friction angle is estimated to be approximately 31 degrees.

The friction angle estimated in the manner described above can be used in slope stability analyses for new slopes and for existing slopes that have not failed previously.

3.3 SHEAR STRENGTH PARAMETERS FOR DESIGN OF REMEDIAL MEASURES

Once a slope has failed, the shear strength is lower than the fully softened strength. The appropriate shear strength for slopes that have previously failed is the residual shear strength. Unless the repair would prevent sliding from occurring along a surface similar to the original slide surface, the lower, residual strength should be used for redesign. Residual shear strengths are estimated in much the same manner as the fully softened strengths, except the correlation chart presented in Figure 3.2 is used. Residual shear strengths are typically significantly lower than peak shear strengths and will result in more costly remedial measures. However, by judicious selection and design of remedial measures, it should be possible to greatly reduce the likelihood of a slide recurring along the same slide surface. In these cases, fully softened, rather than residual, shear strengths can be used for design of the remedial measures. The use of fully softened, rather than residual, shear strengths could result in significant cost savings.

3.4 IMPORTANT PRECAUTIONS

In Chapter 2, it was shown that use of shear strengths estimated through correlations presented in Figure 3.1 (with no pore water pressures) was unconservative and would not have predicted any of the observed failures in Texas highway embankments. Experience with actual slopes that have failed suggests that there are significant positive pore water pressures that must also be considered. Use of the correlations presented in Figures 3.1 and 3.2 with pore water pressures equivalent to those produced by a groundwater table at or very near the ground surface appears to be conservative and is recommended in the absence of further data and investigation.

Direct comparisons between the shear strength envelopes measured in the laboratory for actual soils from embankments in Texas and the shear strength envelopes derived from the recommended correlations showed that the correlations are conservative. However, the correlations may be overly conservative and significant savings may be realized by performing more comprehensive investigations, including laboratory strength testing, to develop design strengths for particular projects. In no event should the correlations be viewed as superior to more detailed investigations.

Only correlations for the long-term strength of highly plastic clays have been presented. Clays of low, rather than high, plasticity are preferable for embankment construction and can be expected to have higher shear strengths. Low, rather than high, plasticity clays should be used whenever possible.

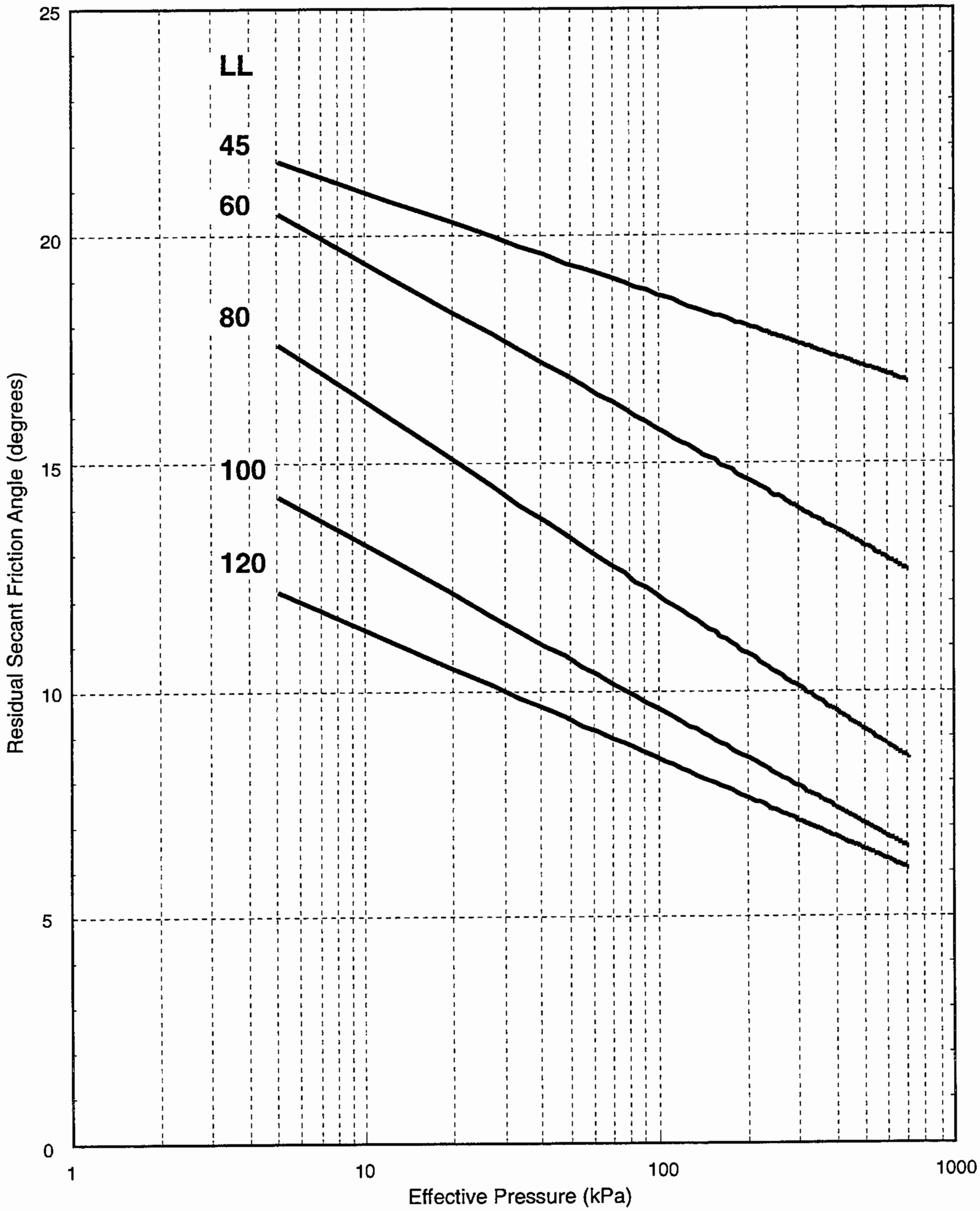


Figure 3.2 Correlation between residual secant friction angle and effective stress for selected values of liquid limit

CHAPTER 4. IDENTIFICATION OF CANDIDATE REMEDIAL MEASURES FOR SLOPE FAILURES

4.1 INTRODUCTION

Identification and selection of remedial measures for slides requires an understanding of the mechanisms that led to the original slope failure, as well as potential remedies that can be used to eliminate or reduce the chance of recurrence of the problem. The primary objective of this chapter is to provide maintenance engineers with a general understanding of the important features of slope failures and a guide for preliminary selection of candidate remedial measures. It is hoped that this chapter provides the basis for TxDOT to eventually develop a manual for identification and selection of measures for slope repair.

This chapter is not intended to provide detailed descriptions of remedial measures and their design; rather, it is intended to serve as a starting point for selecting remedial measures that warrant more detailed study. More detailed coverage of remedial measures, including their design, can be found in numerous publications, including the recent works by Koerner (1994), Abramson et al. (1996) and the Transportation Research Board (1996).

4.2 CLASSIFICATION AND DESCRIPTION OF SLOPE FAILURES

Before embarking on the design and construction of any remedial measure, several distinguishing characteristics of a slope failure should be identified. These include whether the slope failure was short term or long term and whether the slope was an excavated or a fill slope.

4.2.1 Short-Term vs. Long-Term Failure

Failures that occur during or very shortly after construction of a slope—within at most a few weeks—are termed *short-term* failures. When these failures occur in fine-grained soils, especially clays, the soil has little time to drain. To calculate short-term stability, the applicable shear strength is the undrained shear strength. The undrained shear strength is typically measured in the laboratory using either unconfined compression or unconsolidated-undrained triaxial compression test procedures. The undrained shear strength may also be measured using vane shear tests; vane shear tests may be performed either in-situ in the field or in the laboratory on undisturbed samples. The undrained shear strength may also be estimated from *static* cone penetration tests; *dynamic* cone penetration tests are not recommended for determining the undrained shear strength. The direct shear test device is not suitable for determining undrained shear strengths because of nonuniformities in stress and the inability to control drainage in the test.

Failures that occur some time after construction, where the soil has had time to either consolidate or swell since construction, are termed long-term failures. The shear strength that is used to assess stability is termed the drained shear strength, although the term *drained* does not mean that water has entirely drained from the soil. The drained shear strength can be measured using either consolidated-drained (CD) triaxial test procedures or consolidated-undrained (CU) triaxial test procedures with pore water pressure measurements.

Many years may be required for a slope to reach the fully drained condition applicable to long-term stability. Probably many failures that do occur some time after construction are neither undrained (short-term), nor fully drained (long-term) failures. However, if a slope fails some time after construction, it is very likely that the eventual fully-drained, long-term condition is the most critical condition. Even though the slope has not reached the fully drained condition, it is appropriate to design for the long-term condition.

4.2.2 Embankment (Fill) vs. Excavated (Cut) and Natural Slopes

Whether a slope was created by compacting soil fill or by excavating natural ground is important for several reasons. First, compacted soils often perform differently from natural soils. Also, the properties of compacted fills can be controlled and their composition can be observed at the time of construction. Natural soils may vary widely depending on the geological processes that formed the soil deposits. Often the subsurface stratigraphy can only be inferred based on knowledge of the geology and limited soil borings.

One of the most important distinctions between fill and excavated slopes is that one (fills) generally involves increasing the load on the soil, while the other (cuts) involves removal of load. Increase in load causes the soil to consolidate and become stronger with time; decrease in load causes the soil to swell and become weaker with time. Thus, foundation soils beneath earth fills usually increase in strength with time. If a failure is a result of a foundation with inadequate strength, the failure will usually occur during or shortly after construction. Failure of embankments caused by weak foundations are usually short-term failures. In contrast most excavated and natural slopes involve unloading and the soil strength gradually decreases over time. Long-term stability is most critical for these slopes; the slope may be stable at the time it is constructed, but over time as the soil becomes weaker, the slope may eventually fail. In the case of embankments on strong foundations where the failure is confined to the fill material itself, either the short-term or long-term condition may be the most critical, depending on the type of soil. For relatively non-expansive soils and high embankments (high loads) where the soil tends to consolidate with time, short-term stability is typically most critical. However, for embankments constructed of expansive soils and low embankments where the soil is likely to expand with time, the long-term stability is often most critical. Most of the embankments failures observed in Texas have been in expansive soils and have been long-term failures. There are, however, a few short-term failures of embankments that have occurred for embankments constructed on soft ground along the Gulf Coast of Texas.

4.2.3 Slope vs. Foundation Failures

The terms *slope failures* and *foundation failures* are used to distinguish between failures that are confined entirely to the portion of the slope above the toe of the slope and failures that involve at least a portion of the soil beneath the slope (Figure 4.1). Short-term failures of embankment slopes often involve weaker foundation soils and, thus, are categorized as foundation failures. In contrast, many long-term slope failures involve relatively shallow slides that are confined entirely to the slope. Almost all long-term failures are confined to the slope, particularly for homogeneous slopes and foundations. The only significant exception is when a particularly weak layer of highly plastic clay exists in the foundation. In such cases long-term failures may involve the foundation.

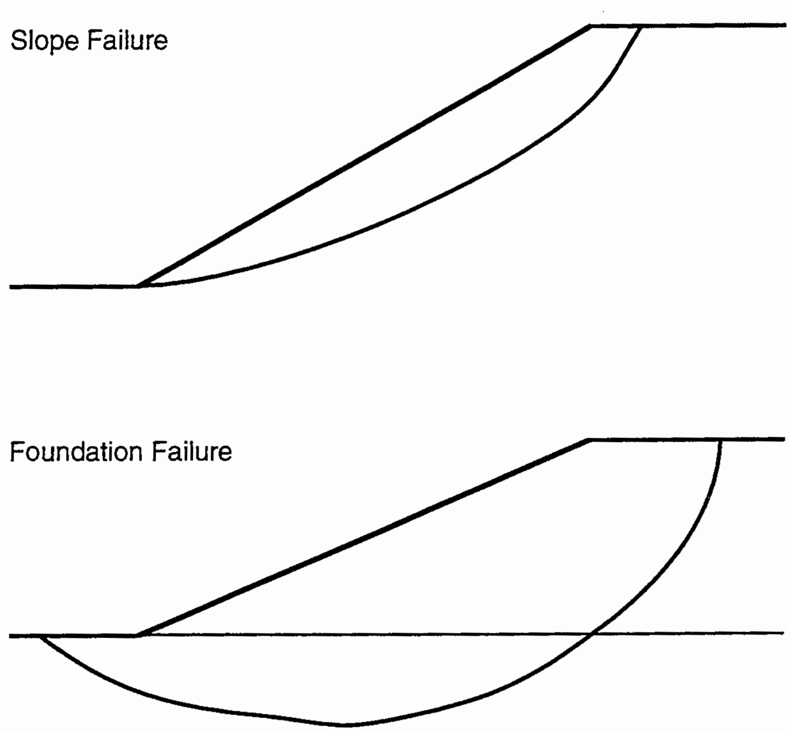


Figure 4.1 Illustration of slope vs. foundation failure

4.2.4 Significance of Classification

The type of failure, including whether the failure is short term or long term, whether the slope is a fill or cut slope, and whether the failure is a slope or foundation failure, is particularly important in selecting remedial measures. For example, reducing the slope height is typically not effective for remedying long-term failures, but is effective as a temporary or permanent remedy for short-term failures. In contrast, flattening a slope may greatly improve long-term stability but has much less effect on short-term stability. Before considering any remedial action, the slope failure should be classified in accordance with the categories described above. Failures should be categorized with regard to:

(1) Short-term versus long-term failure

(2) Fill slopes (embankments) versus excavated and natural slopes

(3) Slope versus foundation failures

4.3 REMEDIAL MEASURES

A number of remedial measures exist for improving the stability of earth slopes. Some of these are best suited as remedial measures, while others are better suited as preventive measures. Selection of most of these depends at least in part on the category of failure, as discussed in the previous section.

A number of remedial measures are listed in Tables 4.1a through 4.1w. A name and description are given for each remedy, along with a statement of the slope category to which the method is well or poorly suited. Finally, for each remedy, supplemental comments on the design and any special precautions that should be taken are noted. In developing the guidelines shown in Table 4.1a through 4.1w, coverage of each method was restricted to no more than one page of text for conciseness and ease of reference. In many cases, information on much more detailed design procedures can be found in the references cited at the beginning of this chapter.

4.4 POTENTIALLY DETRIMENTAL ACTIVITIES

Some maintenance and related activities, including even some remedial measures, may worsen the stability of slopes. Activities that may worsen the stability are listed and described in Tables 4.2a through 4.2f. Possible preventive actions are also presented.

Table 4.1a Remedial measures

Remedial Measure	Cement Stabilization with Recompaction
Description	Soil is excavated, pulverized, mixed with portland cement and recompacted in layers much like cement-stabilized soils are constructed for pavements. The stabilized soil should include all the slide mass, plus additional soil beyond the slide zone.
Applicability	Applicable primarily to relatively shallow slides, i. e., long-term failures confined to the slope itself.
Limitations	A large amount of soil may need to be removed to a location where mixing can be done and then transported back to the site. The depth of stabilization required to achieve significant improvement may be large.
Comments	Used much less than lime. Will work in soils containing little or no clay, where lime will not work. However, these are not generally the soils that cause problems. Stabilization may need to extend large distances behind the slope and below the crest to achieve significant improvement in stability. There is a danger of sliding recurring if only the slide material is stabilized. Care should be exercised that an impervious barrier to seepage is not created by construction of the stabilized zone. A permeable drainage layer may need to be placed beneath and behind the stabilized soil to intercept and remove any groundwater.

Table 4.1b Remedial measures

Remedial Measure	Concrete Slope Paving (Riprap)
Description	Concrete slabs or mats are constructed on the face of the slope.
Applicability	Until recently, concrete slope paving has not been considered beneficial to slope stability, and in some cases, where water could not drain freely from beneath the concrete, the concrete has actually been judged to be detrimental. However, there is some evidence that the concrete reduces moisture fluctuations and the effects of repeated wetting and drying and can help improve long-term stability. Thus, concrete paving may improve long-term stability of slopes with highly plastic clays.
Limitations	Probably primarily beneficial in reducing some of the softening in high PI clays. Not likely to prevent moisture increases in the soil beneath riprap. Can be detrimental if adequate drainage is not provided to allow water to escape from behind the concrete slab.
Comments	Concrete slope paving can have detrimental effects; even the beneficial effects are not yet fully known.

Table 4.1c Remedial measures

Remedial Measure	Conventional Reinforced Concrete Retaining Walls
Description	A conventional concrete wall is constructed to support the unstable slope.
Applicability	Primarily where the underlying foundation is at least as strong as the slope, but otherwise should work in virtually any case.
Limitations	Very expensive. Requires time to construct. Because of cost the wall should be carefully designed; time and expense of design should also be considered in total cost. Soil may need to stand exposed and unsupported during construction, or additional temporary support must be provided.
Comments	Seldom used because of high cost. Overall (global) stability must be checked in addition to stability of the wall itself. Walls may need a deep foundation (drilled piers, piles) to prevent sliding of the wall and global failure surfaces from extending beneath walls.

Table 4.1d Remedial measures

Remedial Measure	Drilled Piers (Drilled Shafts)
Description	Cast-in-place concrete piers are constructed in the slope to pin the soil in place.
Applicability	Provide short-term as well as long-term improvement in stability, but see limitations below.
Limitations	Very limited applicability due to possibility of soil moving (flowing) around the piers unless piers are very closely spaced (see also Slide Suppressor Walls). May have difficulty operating the required construction equipment on the slope, especially if the slope is only marginally stable.
Comments	Piers must resist large lateral loads. A proper design analysis should be performed to ensure that piers have adequate resistance. Specialty contractors have had considerable success with use of large numbers of closely spaced micro-piles. These tend to be custom designed installations that need to be evaluated on a cost basis job-by-job.

Table 4.1e Remedial measures

Remedial Measure	Drilled Pier Slide Suppressor Walls
Description	A precast concrete panel is placed behind a drilled pier and buried entirely within the final slope profile to form a wall. This method was originally developed in the San Antonio, Texas, area.
Applicability	Applicable where a single wall is sufficient to prevent sliding without failure of the slope above or below the wall.
Limitations	Relatively expensive. Soil could still fail by sliding entirely above or below the wall. It may be difficult to get necessary equipment on the slope to construct piers. Relatively large amounts of soil may need to be removed from the immediate area of the slope and stockpiled during construction.
Comments	Piers must resist large lateral loads. A proper design analysis should be performed to ensure that piers have adequate resistance. TxDOT has successfully repaired numerous slides in the San Antonio area with such walls.

Table 4.1f Remedial measures

Remedial Measure	Driven Piles
Description	Piles are driven into the slope to pin the soil in place.
Applicability	Useful for improving short-term stability where an immediate improvement in stability is needed. Should work for long-term stability, but other methods may be more cost effective.
Limitations	Very limited applicability owing to possibility of soil moving (flowing) around the piles unless piles are very closely spaced. May have difficulty operating the required construction equipment on the slope, especially if the slope is only marginally stable to begin with.
Comments	Piles must resist significant lateral load. A proper design analysis should be performed to ensure that the piles have adequate resistance. Specialty contractors have had considerable success with use of large numbers of closely-spaced micropiles. These tend to be custom designed installations that need to be evaluated on a cost basis job-by-job. Timber piles placed in prebored holes have proven effective in one case. Various discarded materials (steel guardrails, poles, etc.) have been used successfully with no design. Such applications involve large uncertainties and typically are evaluated on a case-by-case basis based on local experience. No assurance can be given that such measures will work without proper design.

Table 4.1g Remedial measures

Remedial Measure	Horizontal Drains
Description	Drainage holes are drilled into the face of the slope and a perforated or slotted drainage pipe is inserted into the hole. Although termed *horizontal,* the drains are actually sloped slightly upward (10–15 degrees from horizontal) into the slope.
Applicability	Generally only applicable to improving long-term stability. Only applicable to slopes which have high groundwater levels where water will drain under gravity alone. (Drainage of water from the foundation can improve short-term stability of embankments on weak foundations; however, other types of drains are required for this purpose.)
Limitations	Drains must intercept water internally in the slope so that water can be carried to face. May not work well in (natural) soils that are highly stratified, such that water flows predominately horizontally along pervious strata. Many of the clay soils in Texas have water carried either by thin, nearly horizontal, sand and silt seams or layers of fractured and jointed limestone. Drains will probably not work well in these settings unless they penetrate (intersect) each of the water-bearing strata. Specialized drilling equipment and trained personnel are required for installation.
Comments	Drains require periodic maintenance and flushing to avoid buildup of sediments and/or vegetation.

Table 4.1h Remedial measures

Remedial Measure	Lime Slurry Injection
Description	Lime-water slurry is injected into the in-place soil to strengthen it. Usually done by specialty contractors.
Applicability	Theoretically applicable only to soils containing significant amounts of clay mineral that can react with the lime to form the necessary cementing compounds. Applicability to stabilizing any type of slide is questionable. To be effective lime must be thoroughly dispersed throughout the soil. Most clays are sufficiently impermeable that lime will only penetrate through open cracks and fissures, leaving intact material largely unmodified.
Limitations	As noted above, lime injection is questionable due to difficulty in mixing thoroughly with the soil. Must have clay minerals present to interact with lime and form cementing agents. Even when lime is effective, benefits will be delayed by the time required for lime to react with the clay to form cementing compounds.
Comments	Little evidence that this method works to repair slides. Stabilization may need to extend large distances behind the slope and below the crest to achieve significant improvement in stability. There is a danger of sliding recurring if only the slide material is stabilized.

Table 4.1i Remedial measures

Remedial Measure	Lime Stabilization with Recompaction
Description	Soil is excavated, pulverized and recompacted in layers much like lime-stabilized soils are constructed for pavement applications. The stabilized soil should include all the slide mass plus additional soil beneath the slide zone.
Applicability	Applicable primarily to relatively shallow slides, i. e., long-term failures confined to the slope itself.
Limitations	A large amount of soil may need to be removed to a location where mixing can be done and then transported back to the site. Great depths of stabilization required to achieve significant improvement in stability. Must have clay minerals present to interact with lime and form cementing agents. Time is required for the lime to react with the clay and produce the expected strength gain.
Comments	Used successfully to repair many slides. Stabilization may need to extend large distances into the slope and to achieve significant improvement in stability. There is a danger of sliding recurring if only the slide material is stabilized. Compaction to high dry-unit weights is desirable to improve resistance to long-term effects of wetting and drying. Care should be exercised to ensure that an impervious barrier to seepage is not created by construction of the stabilized zone. A permeable drainage layer may need to be placed beneath and behind the stabilized soil to intercept and remove any groundwater.

Table 4.1j Remedial measures

Remedial Measure	Mechanically Stabilized Earth (MSE) Walls
Description	A wall is constructed using various mechanical forms of reinforcement to strengthen the soil backfill behind the wall.
Applicability	Applicable to most cases where conventional reinforced concrete retaining walls are applicable. Generally preferable to have a foundation with strength at least comparable to the strength of the overlying slope and one which can support the bearing loads of a wall.
Limitations	For most walls a select backfill material is preferred and/or required; the highly plastic clays that are the problem with many slopes in Texas are generally unsuitable as backfill. Generally restricted to slopes with foundations at least as strong as the overlying slope material. Most MSE walls can probably not be supported by deep (pile or pier) foundations.
Comments	Typically more economical than conventional reinforced concrete retaining walls. Walls are sufficiently expensive to justify detailed engineering design. Many of these walls are proprietary and have specialized procedures for design.

Table 4.1k Remedial measures

Remedial Measure	Regrading/Drainage Control
Description	The slope is regraded to provide better drainage of surface water away from the face and behind the crest of the slope.
Applicability	Applicable primarily to long-term stability only.
Limitations	Possibly limitations of right-of-way. Negligible improvements in short-term stability.
Comments	One of the most cost-effective ways of improving stability, especially where surface water infiltration is the cause of instabilities. Effects are not immediate. Some time is required for any water that has already infiltrated the slope to drain from the slope. If large volumes of surface water are involved, paved drainage ditches and channels may be necessary to prevent erosion and possible development of low spots as a result. Any regrading of the slope, including flattening and reducing the slope height, should ensure that the slope is graded so that surface water can drain freely away from the slope.

Table 4.11 Remedial measures

Remedial Measure	Regrading/Reducing Slope Height
Description	The slope is regraded to a lower height.
Applicability	Reduction of slope height is generally most effective in materials whose strength is characterized as primarily cohesive ($f \approx 0$, $c >> 0$), i.e., for cases where the undrained shear strength is applicable. Slope flattening is generally most effective for stabilizing short-term, foundation failures in fill slopes where the slope rests on a much weaker layer.
Limitations	Minimum or no improvement in long-term stability is usually achieved by reducing the slope height
Comments	If regrading involves placement and compaction of soil near the toe of the slope, care must be taken to ensure that the new soil is not less permeable than the underlying soil, such that it might impede drainage. Regrading should be done so that water can drain freely away from the face and behind the crest of the slope.

Table 4.1m Remedial measures

Remedial Measure	Regrading/Slope Flattening
Description	The slope is regraded to a flatter slope.
Applicability	Slope flattening is generally most effective in materials whose strength is primarily frictional (f >> 0, c ≈ 0), i.e., for cases where the drained shear strength is applicable.
Limitations	Sufficient right-of-way must be available to flatten the slope. Slope flattening is not effective when the slope is underlain by strata with much lower drained strengths than the overlying slope.
Comments	If regrading involves placement and compaction of soil near the toe of the slope, care must be taken to ensure that the new soil is not less permeable than the underlying soil, such that the new soil might impede drainage. Regrading should be done so that water can drain freely away from the face and behind the crest of the slope. In no case should regrading be done such that soil is added at the top of the slope near the face.

Table 4.1n Remedial measures

Remedial Measure	Sand Drains
Description	Vertical drain holes are made in the foundation and filled with sand to provide a shorter drainage path for water to escape. Holes may be drilled, jetted, or formed by driving a steel pipe mandrel and then filling the hole with sand.
Applicability	Only applicable to improving short-term stability of embankments on weak foundations.
Limitations	Relatively expensive. Speeds consolidation, but some time is still required to realize benefits of increased shear strength.
Comments	Detailed testing and analyses should be performed to calculate the required drain spacing and expected rates of consolidation. Largely replaced by wick drains using synthetic materials.

Table 4.1o Remedial measures

Remedial Measure	Slope Planting/Vegetation
Description	Vegetation ranging from grasses to larger plants and trees may be planted on the surface of the slope.
Applicability	Vegetation is generally not considered effective in stabilizing slides more than about a foot in depth. However, there are cases where vegetation has been more effective in stabilizing slopes. Persons knowledgeable in the beneficial (as well as detrimental) effects of vegetation should be consulted before any form of vegetation or plant is considered as a remedial measure.
Limitations	Difficult to quantitatively assess the beneficial effects of vegetation except possibly for shallow erosion protection. Not usually considered a viable alternative for preventing or repairing slides more than a foot or so deep. Decaying roots from dead vegetation may provide a path for water to infiltrate the slope, thus producing a detrimental side effect.
Comments	Difficult to judge effectiveness.

Table 4.1p Remedial measures

Remedial Measure	Soil Nailing
Description	Holes are drilled and a steel bar (dowel) is inserted and grouted to reinforce the soil. Typically wire mesh and concrete (shotcrete) are applied to the exposed slope face to form the wall face; more permanent wall facing may be added later using either precast or cast-in-place wall segments. Construction is performed in stages from the top down. Standard practice is for nails to be grouted under a gravity head only with no pre-stressing of the steel tendon. However, there are cases where pressure grouting and/or tensioning of the nail have been used to reduce wall movements.
Applicability	Generally applicable to new, excavated slopes. Nail forces are mobilized as wall excavation proceeds.
Limitations	Primarily used for new, excavated slopes. The applicability of soil nailing to existing slopes where excavation does not occur to load the nails is not well-understood and nails should not be used or used very cautiously for such cases. Not suitable for slopes with weaker foundations.
Comments	Relatively new compared with other forms of slope stabilization and wall construction. For thicker walls secondary nails may be needed for support of the facing during construction. Design procedures are evolving. FHWA has recently developed procedures for design of soil nails.

Table 4.1q Remedial measures

Remedial Measure	Soil Removal and Recompaction
Description	The problem soil is removed and recompacted in the slope.
Applicability	This method provides no permanent improvement in stability. However, many of the embankment slope failures observed in highly plastic clay soils have occurred anywhere from ten to thirty years after the embankments were built. Normally the steeper the embankment, the sooner it fails. Embankments inclined at 2:1 to 2.5:1 typically fail sooner, e.g., after 10-15 years, while flatter embankments, inclined at say 3:1 to 3.5:1, may only fail 20–30 years after construction. By reconstructing the embankment to its near-original condition, the cycle of strength loss with time is restarted and additional life of 10–30 years may be expected.
Limitations	Only a temporary measure. Restricted to long-term failures. Must have suitable area to stockpile excavated soil before recompaction.
Comments	Depth and breadth of material removal and reconstruction should extend well beyond the original slide plane or failure may occur just beneath the reconstructed zone.

Table 4.1r Remedial measures

Remedial Measure	Soil Replacement
Description	The problem soil involved in the failure is excavated and replaced with a select material.
Applicability	Many slope failures occur in highly plastic clays. Removal of the offending soil and replacement with either a low-plasticity or nonplastic soil will improve stability in two ways: First, low-plasticity or nonplastic soil will probably be more permeable and allow better drainage of water from the slope. Second, even with the same moisture conditions, the lower-plasticity or nonplastic soil should be stronger.
Limitations	This is generally applicable only to long-term slope failures; not applicable to short-term or foundation failures (except as described elsewhere for vibro-replacement).
Comments	As with stabilization employing lime and cement additives, any new material must not only replace the slide material, but must also extend beyond the limits of the original slope failure to achieve significant benefits.

Table 4.1s Remedial measures

Remedial Measure	Steel Sheet Piling
Description	Steel sheet piling is driven at or near the crest of the slope to restrain soil behind the slope face.
Applicability	Generally this measure is used as a temporary support measure while more substantial structures, e.g., reinforced concrete, MSE, or other proprietary wall systems are being constructed.
Limitations	Primarily used as temporary measure; no knowledge of application as a permanent remedial measure for slope failures.
Comments	Typically used as temporary measure.

Table 4.1t Remedial measures

Remedial Measure	Transverse “Buttress” or “Slot” Drains
Description	A series of trenches is excavated transversely into the face of the slope and filled with a coarse drainage material.
Applicability	Effective for improving the long-term stability of slopes with high groundwater levels.
Limitations	Limitations are not well known. Probably restricted to improving long-term stability against shallow sliding where significant amounts of groundwater are present.
Comments	Any lowering of groundwater level by drainage will improve slope stability. Difficult to analyze spacing and depth requirements. No known cases of use in Texas and, thus, there is little experience with performance.

Table 4.1u Remedial measures

Remedial Measure	Vertical Interceptor Drains
Description	A trench or closely spaced row of vertical drainage wells is constructed at or behind the crest of the slope to intercept water before in reaches the vicinity of the slope face.
Applicability	Effective for improving long-term stability of slopes where there are large amounts of groundwater. Lowering of groundwater levels can provide significant improvements in long-term slope stability.
Limitations	For all but small slopes (less than 10 feet high) it is difficult to excavate trenches to the required depth without elaborate bracing and trench support, especially if large amounts of groundwater are present. Unless the trenches extend to a depth at least near the bottom (toe elevation) of the slope, they may not be effective in capturing the flow before it reaches the slope face. It is also necessary for the water to be able to drain (escape) freely from the bottom of the drainage trench. This typically requires a drainage pipe or supplemental trench to carry the water around the ends of the slope to a suitable discharge point.
Comments	Slope stability analyses can be used to estimate the magnitude of the effect of lowering the groundwater levels on the slope's stability.

Table 4.1v Remedial measures

Remedial Measure	Vibro-Replacement
Description	A special-purpose mechanical vibrator (vibro-flot) with an integral jetting system is used to force relatively coarse gravel and rock into an otherwise fine-grained foundation. A portion of weaker foundation soils is replaced by stronger soil that provides a drainage path for water to escape in a process similar to that provided by the sand and wick drains described elsewhere.
Applicability	Primarily applicable to improving short-term stability of embankments on weak foundations.
Limitations	Relatively expensive. It is difficult to estimate increases in strength unless the soil is fully drained. Large amounts of water are typically used and the construction site can become very messy.
Comments	Analyses can and should be performed to compute the expected increase in stability produced by this technique. The analyses involve greater uncertainty than more conventional slope stability analyses and, thus, they are not as reliable.

Table 4.1w Remedial measures

Remedial Measure	Wick Drains
Description	Synthetic wick drains are punched vertically into soft soil foundations to provide a shorter drainage path for water to escape. This allows soil strength to increase more rapidly than it otherwise would and, thus, provides benefits of increased strength earlier. Often drains are used during construction so that some strength gain can even be realized during construction. In such instances, staged construction can be used to build embankments higher than would otherwise be possible with no drainage during construction.
Applicability	Only applicable to improving short-term stability of embankments on weak foundations.
Limitations	Relatively expensive. It is difficult to estimate increases in strength unless soil is fully consolidated under the applied loads.
Comments	Wick drains have largely replaced sand drains for this purpose. Because of cost, detailed testing and analyses should be performed to estimate suitable drain spacing and rates of consolidation. Some controversy exists regarding whether to use undrained shear strengths and total stresses or effective stress strength parameters and effective stress procedures for analyses.

Table 4.2a Activities

Activity	Adding load to the crest of the slope

Potential Detrimental Effect

Placement of soil or other materials at or near the crest of the slope adds driving forces and reduces the stability of the slope. Operation of vehicles near the crest of the slope may cause localized sloughing, but unless the vehicle is carrying unusually large loads, vehicle loads do not normally significantly affect slope stability. For example, a vehicle producing a distributed load equivalent to the AASHTO bridge loading of 240 psf is only equivalent to about 2 feet of additional soil. Normally, 2 feet of additional soil should represent a small load compared to the weight (height) of the slope itself.

Possible Preventive Action

Placement and stockpiling of soil near the crest of the slope should be avoided. Generally, if material is to be placed within a distance of 1–2 times the slope height from the crest, a stability analysis should be performed to determine the effects.

Table 4.2b Activities

Activity	Covering a slope with impervious cover

Potential Detrimental Effect

Placement of impervious cover on a slope may restrict the escape of groundwater from the slope, resulting in rises in water pressures and decreases in stability for the slope. Impervious cover may consist of concrete slope paving (riprap) that does not have sufficient underdrainage and weep holes, or fine-grained fills, including some stabilized soils. Even simple regrading and recompaction of slide material near the face of the slope could reduce the permeability of the soil such that water is trapped in more pervious soil behind the regraded soil

Possible Preventive Action

When concrete riprap is placed as slope cover, a suitable drainage blanket and weep holes should be constructed to allow water that collects behind the riprap to escape. Coarse rock slope protection is much more desirable than concrete with regards to drainage characteristics.

When slope repair consists of stabilizing material near the slope face with additives, such as lime and Portland cement, care should be taken to ensure that the underlying, unstabilized soil can drain. Unless the stabilized material can be verified as having a higher permeability than the underlying soil, a suitable drainage layer should be placed behind and beneath the stabilized soil. Provisions must be included for water to freely escape from the drainage layer.

It should be noted that riprap by itself is not necessarily detrimental and may have a favorable effect in reducing the cyclic fluctuations in moisture in the soil. However, positive water pressures must not be allowed to develop beneath the riprap.

Table 4.2c Activities

Activity	Mowing grass

Potential Detrimental Effect

The operation of mowers and similar wheeled equipment on slopes can lead to rutting. Rutting can provide a place for water to pond on the surface of the slope and eventually seep into the underlying soil, thus reducing stability.

Possible Preventive Action

It is particularly important to not operate mowers on slopes during wet periods or when the soil has been weakened owing to rainfall.

Table 4.2d Activities

Activity	Rapid removal of water adjacent to slope (Rapid drawdown)

Potential Detrimental Effect

Although internal pore water is generally detrimental to slope stability there are instances where sudden removal of external water adjacent to a slope may actually decrease the stability. If water is removed fast enough that the water does not have time to drain from the adjacent soil, the slope may fail due to the removal of the water. Removal of water removes lateral support from the surface of the slope and if drainage does not occur at the same time to increase the strength, the slope becomes less stable. This is the reason that slopes often fail when river or lake levels are lowered after periods of high water.

Possible Preventive Action

If water has been standing adjacent to a slope for some time, the water should not be suddenly removed, e.g., by pumping, unless it is known that the soil will drain and/or a complete slope stability analysis has been performed. Most clays will not be able to drain significantly at the rate at which water levels are normally lowered through pumping.

Table 4.2e Activities

Activity	Removal of support from the toe of the slope

Potential Detrimental Effect

Removal of soil in the vicinity of the toe of the slope will reduce stability in two ways: (1) the removal of soil results in removal of support for the remaining soil, and (2) the removal of soil unloads the remaining soil so that it can swell (expand), thus reducing the shear strength of the soil.

Possible Preventive Action

Soil should only be removed from the slope when it has been determined that the stability will not be adversely affected. Often it is necessary to remove soil near the toe when the slope has failed and repair is being undertaken. In such cases if the soil failed due to excess moisture, it may be possible to safely remove soil from the toe provided that the soil has had time to dry, and its strength has significantly increased owing to the drying. However, even when this is done, the soil may begin to lose strength once soil is removed from the toe. Even though the slope may be stable immediately after the soil is removed, it may become weaker with time and later collapse.

Table 4.2f Activities

Activity	Planting and maintaining vegetative growth

Potential Detrimental Effect

Vegetation can have both beneficial and detrimental effects on slope stability. Dead and decaying plant roots can provide a path for water to enter the slope, thus weakening the soil and reducing slope stability.

Possible Preventive Action

Care should be exercised in planting vegetation to improve slope stability. Local experience with specific climatic conditions and types of vegetation should be relied on to determine if vegetation is beneficial or detrimental.

CHAPTER 5. ISSUES RELATED TO MOBILIZED FORCES IN GEOSYNTHETIC REINFORCEMENT

5.1 INTRODUCTION

Geosynthetic (geogrid) reinforcement offers considerable promise for repair of slides. The geosynthetic reinforcement improves stability through the development of longitudinal, tensile forces in the reinforcement. Thus, the development of these tensile forces is essential for the reinforcement to be beneficial. This chapter addresses issues related to the forces that can reasonably be expected to develop in reinforcement for typical Texas highway embankments constructed of high PI clays.

Considerable advances have been made in the last 15 years in understanding how geosynthetics behave in slopes and how they contribute to slope stability. Numerous design procedures have also been developed for use of geosynthetics in slopes including procedures developed in this study and presented in a companion report (Fippin and Wright, 1997). However, almost all of the design methods explicitly or implicitly assume that the slope would be *unstable at the time of construction* if the slope is not reinforced. If the slope is unstable during construction, the reinforcement plays an active role during construction and significant forces are mobilized in the reinforcement. However, if the unreinforced slope is stable during construction (but later becomes unstable) the role of the reinforcement and the forces developed during construction are much more uncertain.

TxDOT has many slopes constructed of high PI clays that are stable without reinforcement when built, but require some strengthening for long-term stability. It is not clear that the procedures that have been developed are applicable to these slopes of interest. Many of the slopes which have failed in Texas are very stable during construction; computed factors of safety for the conditions during and immediately following construction are sometimes in excess of 4. In these cases, any reinforcement in the slope most likely would mobilize much less of its capacity. Reinforcement of slopes constructed of high PI clays requires special consideration.

5.2 PREVIOUS WORK

Cuenca (1989) studied the development of reinforcement forces in reinforced slopes, and showed that the reinforcement might potentially have little effect. However, his results were preliminary in nature and not conclusive. One of the objectives at the outset of this study was to extend Cuenca's work and better understand the magnitude of the reinforcement forces that can be mobilized in a slope.

Given the periods of time (10 to 30 years) before most of the slopes of interest become unstable and need reinforcement it seems impractical to conduct field experiments to measure the development of forces over time in the reinforcement. Consequently, numerical simulation, employing finite element procedures, was chosen to estimate both the short- and long-term development of reinforcement forces. During the course of this research, it was determined that extensive and special soil laboratory tests would have to be performed to appropriately simulate field conditions for the long-term stability case. Accordingly, for the current project efforts were focused on understanding the issues and parameters that might affect the short-term mobilization of the reinforcement forces during construction. It should be noted that construction deformations of embankments are generally believed to be much larger than postconstruction deformations. Wilson (1973) reported measured horizontal deformations of various dams founded on competent foundations. He reported deformations measured both during and for some time after construction. He showed that deformations during construction represented a major portion of the total deformations that occurred.

The remainder of this chapter describes an investigation of reinforcement forces developed during construction. Chapter 6 includes recommendations for future research to better determine the long-term development of reinforcement forces.

5.3 CONSTRUCTION DEFORMATIONS

During construction of a reinforced embankment, deformations occur. These deformations cause strains to develop in the soil, which in turn develop forces in the reinforcement. For constructibility reasons, the reinforcement is generally placed horizontally in the fill. Thus, the horizontal strains are of primary interest. A finite element program that simulates staged embankment construction was used to estimate the magnitude of the horizontal strains produced during and immediately after construction. For these calculations the presence of reinforcement is ignored. If the reinforcement were included in the analyses, the horizontal strains would be less than those computed without the reinforcement. Thus, the horizontal strains (or forces) computed in this manner represent an upper limit of the strains (or forces) that might be developed in the reinforcement during construction. The significance of this upper bound will be discussed later in this chapter.

A series of sensitivity studies was performed to examine the influence of various parameters and assumptions on the magnitude of the horizontal strains during construction. The Taylor Marl was the soil chosen for the sensitivity studies. This soil was chosen because it tended to produce the largest horizontal deformations when compared with other soils that were examined in this study. Thus, the effects of variations in parameters were expected to be accentuated. The results of the sensitivity studies are presented in Appendix A.

5.3.1 Embankment Geometry Considered for Analyses

To establish the magnitude of horizontal strains that might develop in slopes during construction, analyses were performed for a range of slope heights, slope inclinations, and crest widths. Five different slope heights, ranging from 10 to 50 feet, were used to cover the range of embankment heights of interest. Slope inclinations of 1:1 (horizontal: vertical), 2:1, 3:1, and 4:1 were used. Almost all of the highway embankments examined in Texas during our previous research efforts had slope inclinations that were in the chosen ranges. Three different crest widths (50, 100, and 150 feet) were also chosen to represent an appropriate range of crest widths.

5.3.2 Embankment Soils Considered for Analyses

Five different soils with different properties were selected for the parametric studies. A brief description and basic soil properties for each soil are shown in Table 5.1. Properties for all the soils except the Taylor Marl were obtained from Duncan et al. (1980). Duncan et al. (1980) present data for over 80 soils. The soils chosen from Duncan et al. (1980) were selected because they represent a wide range of clay soil types, shear strength, and stress-strain parameters, and because they contain complete sets of data for parameters needed for the constitutive stress-strain model used in this study. The parameter values for the last two soils listed in Table 5.1 were termed by Duncan et al. as conservative values. They are considered conservative in the sense that they are typical of the lower values of strength and modulus and the higher values of unit weight for each type of soil, based on the data contained in Duncan's report.

Table 5.1 Index and strength properties for different soils used to compute construction deformations

Soil No.	Soil Type	PI	γ (pcf)	c (psf)	ϕ	Reference
1	Clayey Sand (Don Pedro Dam)	11	135	5000	26	Duncan et al. (1980)
2	Clayey Gravelly Sand (Proctor Dam)	18	137	3600	4	Duncan et al. (1980)
3	Taylor Marl	84	121	400	15	Cuenca (1989)
4	SC (Conservative Values)		125	300	33	Duncan et al. (1980)
5	CL (Conservative Values)		120	100	30	Duncan et al. (1980)

Because Cuenca's studies suggested that construction deformations in high PI soils might be small, an attempt was made in these studies to establish an upper-bound on deformations by selecting some sets of soil properties that would produce the largest deformations. For this reason the two sets of conservative values shown in Table 5.1 were included in this study. These two soils are expected to yield the largest deformations in their class of soils.

The properties used for Taylor Marl were reported by Cuenca (1989). Cuenca chose Taylor Marl because it represents a highly plastic clay from Texas. Based on the Atterberg limits and effective stress shear strength parameters presented by Cuenca, it is believed that the Taylor Marl would yield larger deformations than most of the other Texas soils examined previously. Taylor Marl should also aid in providing an upper bound on horizontal strains.

5.3.3 Soil Constitutive Model

The constitutive model used to predict the stress-strain response of the soil during construction (undrained conditions) is the hyperbolic stress-strain model developed by Kondner (1963) and extensively modified by Duncan and Chang (1970). This model is employed in incremental finite element computations to calculate tangent moduli. The relationship between incremental stress and strain in matrix form for the plane strain condition used in the finite element analyses, can be written as:

$$\begin{Bmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\tau \end{Bmatrix} = \frac{3K_t}{3K_t - E_t} \begin{bmatrix} 3K_t + E_t & 3K_t - E_t & 0 \\ 3K_t - E_t & 3K_t + E_t & 0 \\ 0 & & E_t \end{bmatrix} \begin{Bmatrix} \Delta\xi_x \\ \Delta\xi_y \\ \Delta\gamma_{xy} \end{Bmatrix} \tag{5.1}$$

where E_t and K_t are the tangent Young's modulus and bulk modulus of the soil. The stress strain relationship expressed by Equation 5.1 is implemented directly into the two-dimensional finite element program. Nonlinearity, stress-dependency, and inelasticity, which are some of the important characteristics of soils, are easily incorporated by adjusting the values of E_t and K_t at each loading stage as described by Duncan and Chang (1970). Expressions for E_t and K_t are given below:

$$E_t = \left[1 - \frac{R_f(1-\sin\phi)(\sigma_1 - \sigma_3)}{2(c.\cos\phi + \sigma_3 \sin\phi)}\right]^2 K_i P_a \left(\frac{\sigma_3}{P_a}\right)^n \tag{5.2}$$

$$K_t = K_b P_a \left(\frac{\sigma_3}{P_a}\right)^m \tag{5.3}$$

where $(\sigma_1\text{-}\sigma_3)$ is the principal stress difference, σ_3 is the minor principal stress, k_i and n are dimensionless material constants, p_a is atmospheric pressure, c is cohesion, ϕ is friction angle, R_f is a constant termed the failure ratio, and k_b and m are dimensionless parameters used to define the stress-dependent bulk modulus. In finite element computer codes, the equivalent values of tangential Poisson's ratio are restricted to be between 0 and 0.5. This imposes certain additional restrictions on values of the tangential bulk modulus. These restrictions can be satisfied by limiting values of K_t to the range between $E_t/3$ and $17E_t$, which corresponds to values of Poisson's ratio between 0 and 0.49. The soil parameters required in addition to c and ϕ to define the nonlinear hyperbolic stress-strain relation for the selected soils are summarized in Table 5.2.

5.3.4 Fill Construction Simulation and Boundary Conditions

A finite element computer program (EDDC) developed previously at The University of Texas was used to compute construction deformations. This code simulates the typical placement of successive layers of embankment material that is illustrated in Figure 5.1. For computational convenience, a constant thickness of 0.61 m (2 ft) was used for all construction layers.

Table 5.2 Hyperbolic model parameters for different soils used to compute construction deformations

Soil No.	Soil Type	k_i	n	R_f	k_b	m	Reference
1	Clayey Sand (Don Pedro Dam)	3900	.1	.9	12000	.99	Duncan et al. (1980)
2	Clayey Gravelly Sand (Proctor Dam)	510	.4	.6	250	0	Duncan et al. (1980)
3	Taylor Marl	76.3	.8	.8	122.3	.45	Cuenca (1989)
4	SC (Conservative Values)	150	.6	.7	75	.5	Duncan et al. (1980)
5	CL (Conservative Values)	60	.5	.7	50	.2	Duncan et al. (1980)

All the embankments were assumed to rest on perfectly rough, rigid foundations. This assumption was satisfied by restricting horizontal and vertical movements to zero at the bottom of the first layer of finite elements. A vertical line of symmetry at the centerline of the embankment was used for the finite element mesh. Neither shear force nor horizontal displacement was assumed along the vertical line of symmetry (see Figure 5.2).

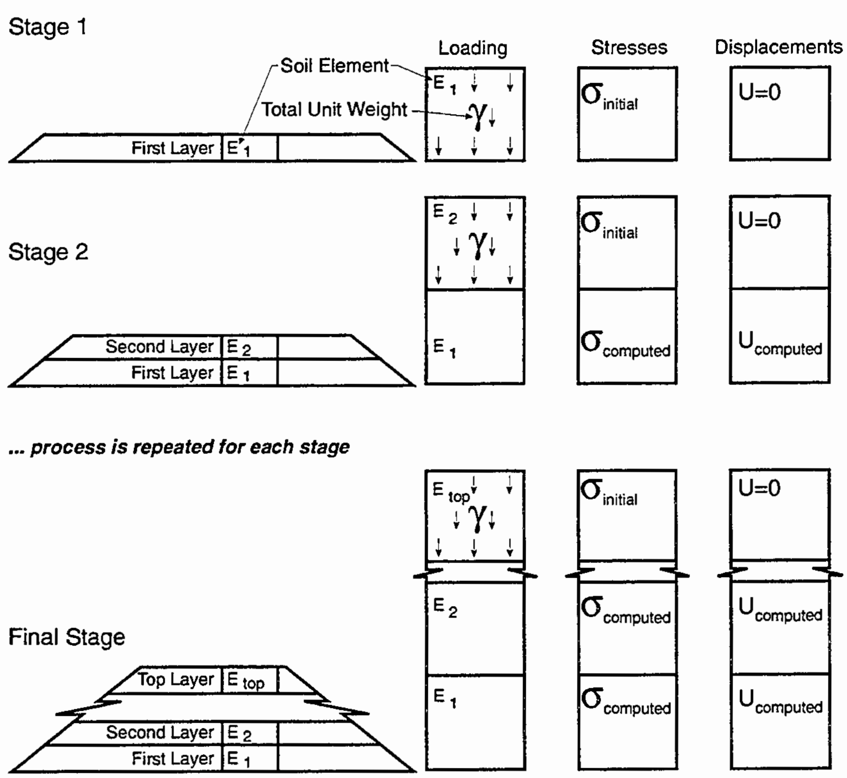


Figure 5.1 Illustrations of fill staged construction as simulated in finite element computations

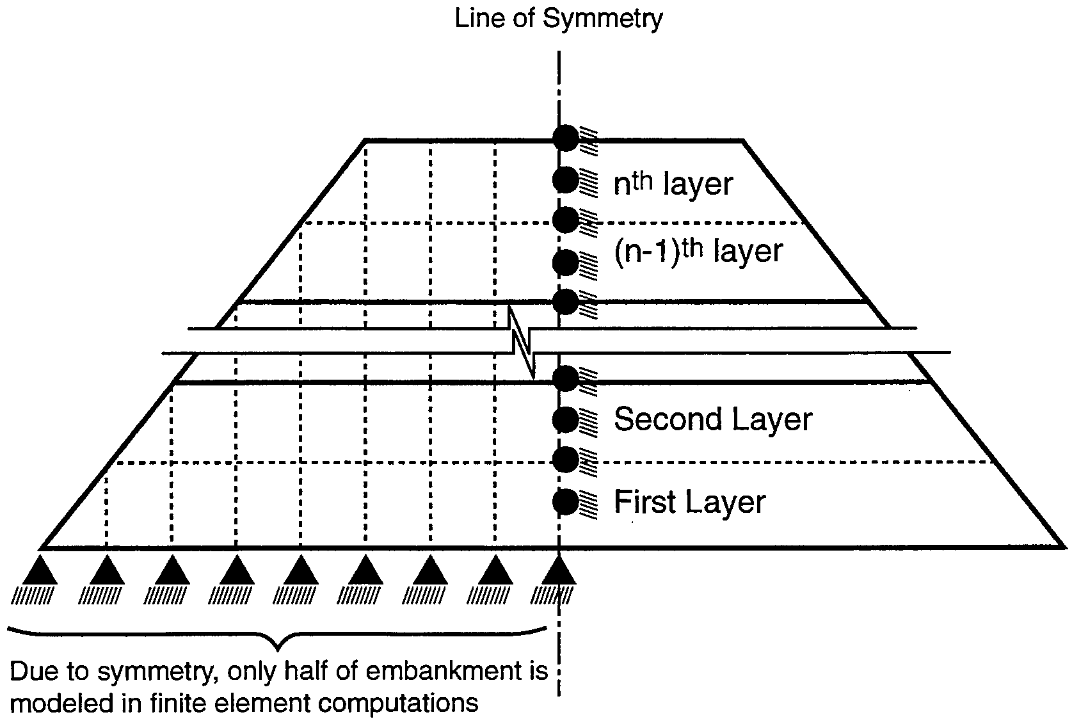


Figure 5.2 Typical boundary conditions used for finite element computations of construction deformations

5.3.5 Typical Strain Profiles and Location of Maximum Horizontal Strains

Typical contours of horizontal strain and displacement calculated in the finite element analyses are shown in Figure 5.3. The results shown in Figure 5.3 are for a 9.14-m (30-ft) high embankment with a 2:1 side slope inclination and 15.24-m (50-ft) crest width. The soil properties used are those used for the Taylor Marl (Soil #3 in Table 5.2). The maximum horizontal strain always occurred within a well-defined zone. The zone of maximum horizontal strain was generally located below the top of the slope face (Point A in Figure 5.3), at or slightly above midheight of the slope.

Figure 5.4 presents profiles of horizontal strains taken at selected elevations that could correspond to typical lines of reinforcement in the slope. These elevations were chosen arbitrarily for illustration purposes. The profiles of strain might also represent strains in the reinforcement if reinforcement was placed at the associated horizontal elevations and deformed with the soil, i.e., with no slippage at the soil-reinforcement interface.

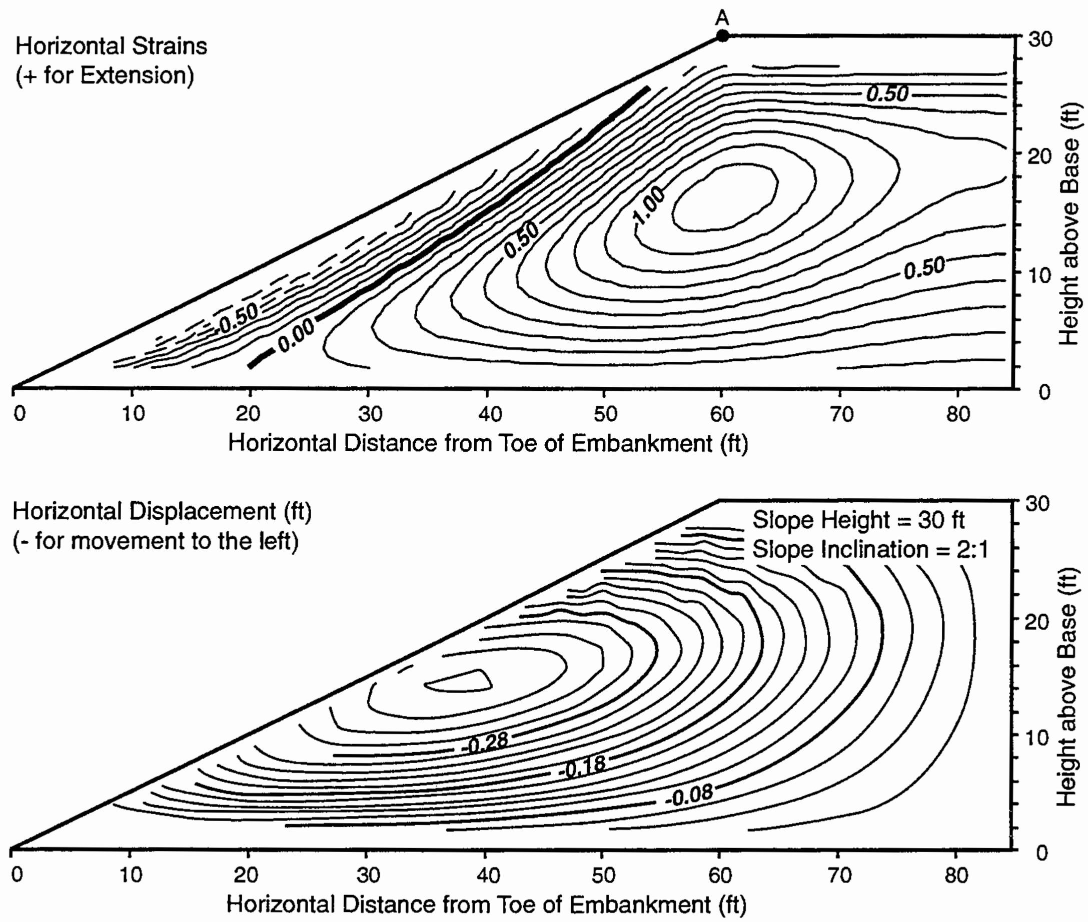


Figure 5.3 Typical horizontal strains and displacements

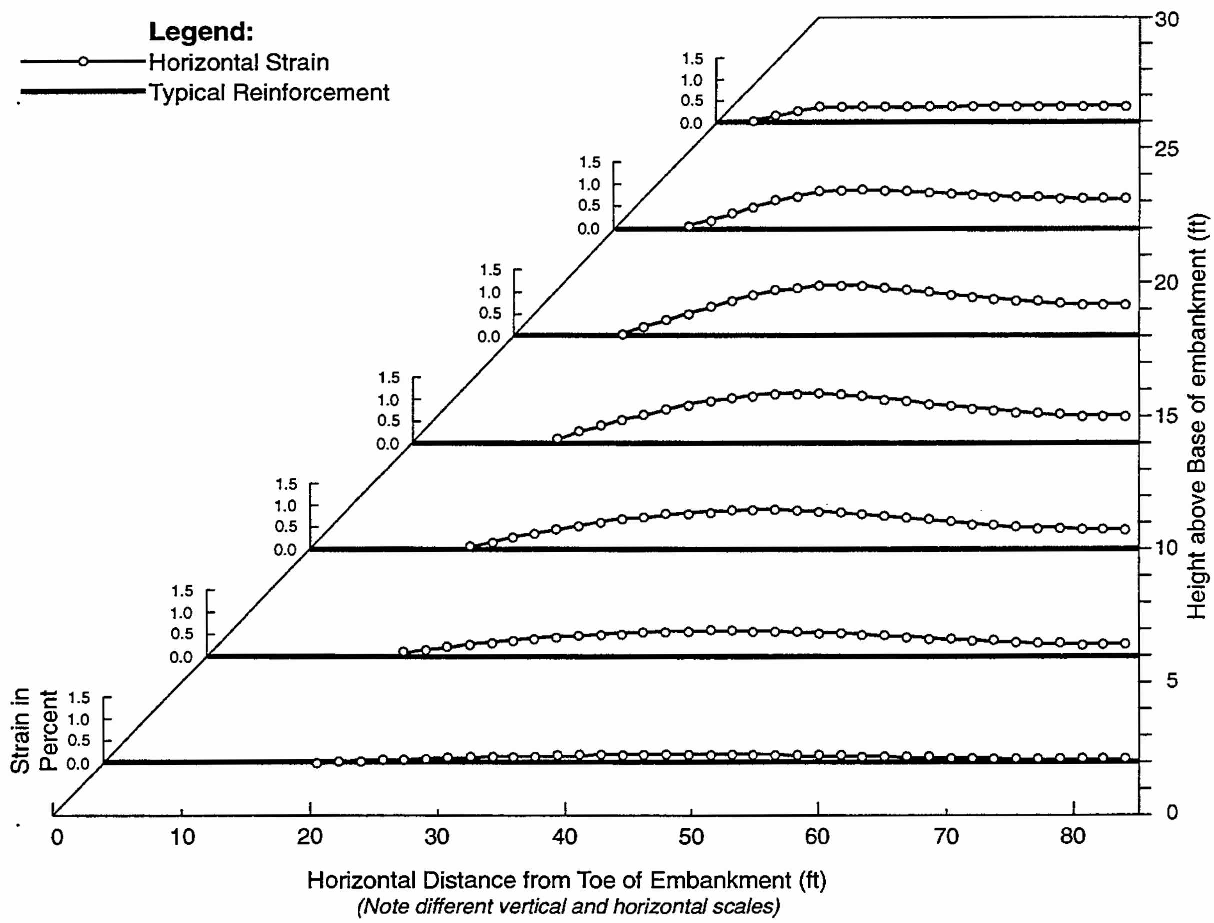


Figure 5.4 Horizontal tensile strain profiles along typical reinforcement locations

5.3.6 Trend of Factor of Safety versus Maximum Horizontal Strains

Maximum horizontal strains were determined from the finite element analyses for various combinations of slope height and inclination and from the various sets of soil properties listed in Tables 5.1 and 5.2. These various combinations are shown in Table 5.3. The smallest practical crest width of 50 feet was chosen for all cases. Conventional slope stability computations were also performed for all the cases considered to compute a factor of safety for short-term (end-of-construction) stability. The computer program UTEXAS3 (Wright, 1991) and Spencer's procedure were used for all the stability computations. Maximum horizontal strains from the finite element computations were then plotted versus the factors of safety, as shown in Figure 5.5. Stability analysis for the case involving soil number 3 with a 50-foot-high embankment and 1:1 side slopes showed that the factor of safety was less than unity. As expected for this case, numerical instability was encountered when attempts were made to analyze it using the finite element method; accordingly, the case was omitted from inclusion in Figure 5.5.

The relationship shown in Figure 5.5 between maximum horizontal strain and factor of safety during or immediately after construction is definitive. A clear trend can be seen toward decreasing maximum horizontal strain with increasing factor of safety. A band is shown in Figure 5.5 within which maximum strains at end-of-construction for embankments constructed of high PI clays are believed to exist.

Table 5.3 Combinations of slope height, slope inclination, and soils used in the finite element analyses

		SLOPE INCLINATION			
		1:1	**2:1**	**3:1**	**4:1**
SLOPE HEIGHT IN FEET	**10**	Soil #1 Soil #2 Soil #3 Soil #4 Soil #5	Soil #1 Soil #2 Soil #3 Soil #4 Soil #5	Soil #1 Soil #2 Soil #3 Soil #4 Soil #5	Soil #1 Soil #2 Soil #3 Soil #4 Soil #5
	20		Soil #1 Soil #2 Soil #3		
	30	Soil #1 Soil #2 Soil #3 Soil #4 Soil #5	Soil #1 Soil #2 Soil #3 Soil #4 Soil #5	Soil #1 Soil #2 Soil #3 Soil #4 Soil #5	Soil #1 Soil #2 Soil #3 Soil #4 Soil #5
	40		Soil #1 Soil #2 Soil #3		
	50	Soil #1 Soil #2 Soil #3	Soil #1 Soil #2 Soil #3	Soil #1 Soil #2 Soil #3	Soil #1 Soil #2 Soil #3

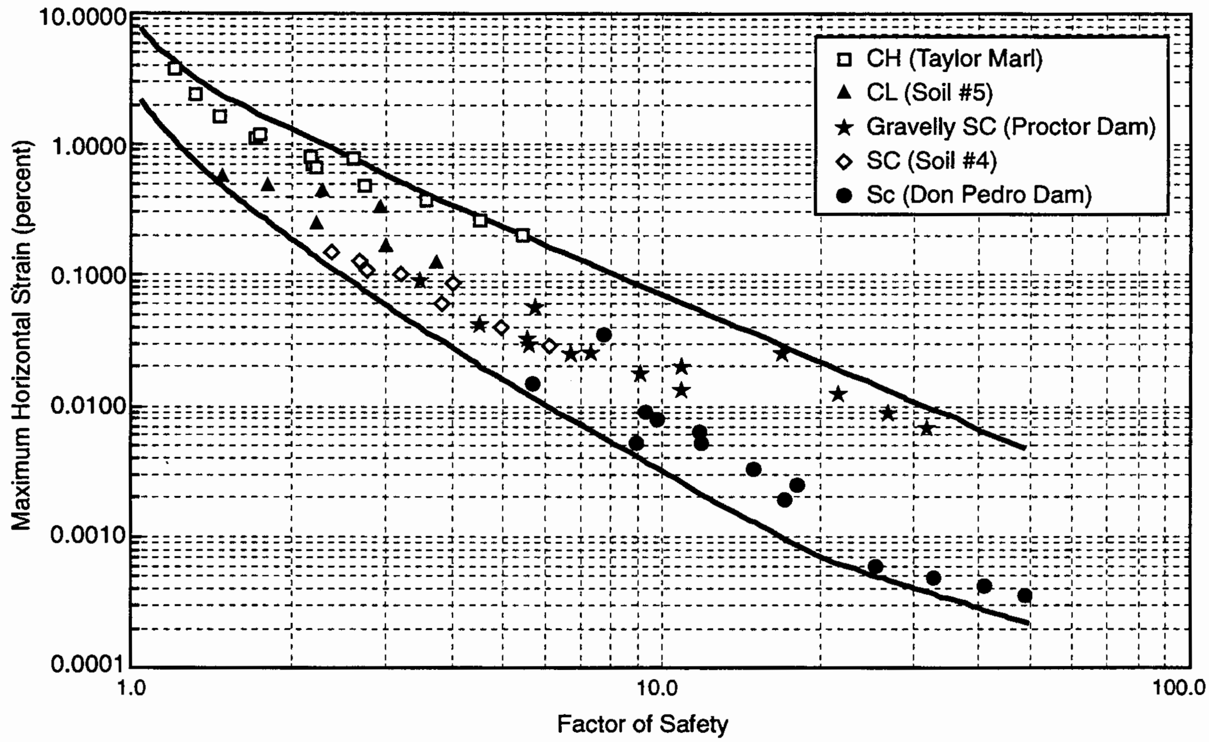


Figure 5.5 Maximum horizontal strain vs. safety factor neglecting forces in reinforcement

5.4 EVALUATION OF EFFECTS OF MOBILIZED FORCES IN REINFORCEMENT DURING CONSTRUCTION ON LONG-TERM STABILITY

To examine the effect of forces mobilized in the reinforcement during construction on eventual long-term slope stability, a set of calculations was performed for a hypothetical slope using estimated mobilized reinforcement forces. Reinforcement was selected using existing design procedures and then mobilized forces were estimated for the reinforcement using results of the finite element analyses. The mobilized forces were then used to compute a factor of safety for comparison with the factor of safety originally assumed for the reinforcement design. Calculations and results are described in further detail in the following sections.

5.4.1 Embankment Geometry

The embankment selected for study has a slope inclination of 2:1, a height of 30 feet, and a crest width of 50 feet. Thirty feet represents an upper height limit for typical Texas embankments, while a 2:1 slope inclination represents the steepest slope typically found.

5.4.2 Embankment Soil and Material Properties

The embankment was assumed to be constructed of compacted Taylor Marl; the properties reported by Cuenca (1989) and briefly mentioned in Section 5.2 of this report were used. A total unit weight of 121 pcf (= 95 percent of Standard Proctor) was assumed for the computations.

The undrained shear strength parameters for Taylor Marl used for the short-term stability analyses are c = 400 psf and ϕ = 15 degrees (Table 5.1). These strength parameters along with the hyperbolic model parameters summarized in Table 5.2 were used in the short-term deformation analysis.

For the long-term stability analyses, effective stress shear strength parameters were estimated using the correlation presented in Chapter 3. Using the chart in Figure 5.21 and values of 110 and 50 kPa (1000 psf) for liquid limit and effective stress, respectively, the estimated fully softened effective stress friction angle for Taylor Marl was estimated to be 25 degrees. Effective stress cohesion ($\bar{c}$) was assumed to be zero for long-term stability.

5.4.3 Stability Computations for Unreinforced Slope

The first set of stability computations consisted of short-term and long-term analyses of the embankment with no reinforcement. All computations were carried out using the computer program UTEXAS3 and Spencer's procedure of slices. A water table coincident with the surface of the slope was assumed for long-term conditions following the recommendation in Chapter 2.

The computed factor of safety of the slope for end-of-construction using the strength parameters described in Section 5.3.2 was 1.70. The computed long-term safety factor was 0.33 with the water table at the surface of the slope. The long-term factor of safety was even below 1.0 (0.93) if the pore water pressures were assumed to be zero. Thus, the unreinforced embankment was clearly not stable for the long-term conditions, though apparently stable during construction.

5.4.4 Reinforcement Design

The next step was to determine the required reinforcement for long-term stability. This was done using the charts and procedures presented in the companion report (Fippin and Wright, 1997).

The design of reinforcement using the procedures presented by Fippin and Wright and others requires first that the mobilized friction angle be determined. Using a target long-term factor of safety of 1.5, the mobilized friction angle for Taylor Marl was computed to be 17.3 degrees ($=\tan^{-1}[\tan 25°/1.5]$). With this mobilized friction angle (17.3°) and a slope angle of 26.6 degrees (2:1 slope) a total required reinforcement force of 21,780 pounds was determined using the appropriate charts. The required length of reinforcement, also determined using the charts, was found to be 72 feet. Details of these calculations are presented in Appendix B.

The reinforcement selected for design is a TENSAR geogrid type UX1500HS manufactured by Tensar Earth Technologies, Inc. Material properties were obtained from the manufacturer's literature. The recommended allowable strength for the UX1500HS geogrid is 2,762 pounds per foot. The allowable strength accounts for the various partial reduction factors for construction damage, biological degradation, creep, and joints that are typically used in determining a geosynthetic material's long-term strength.

Using the required total force (21,780 pounds) and the allowable force per layer (2,762 pounds), the number of reinforcement layers was determined by dividing the total force by the allowable force. The vertical spacing of the various reinforcement layers was determined using a procedure suggested by Schmertmann and also presented by Fippin and Wright (1991). The final layout of the reinforcement is shown in Figure 5.6. The height above the toe and length of each layer of reinforcement are presented in Table 5.4.

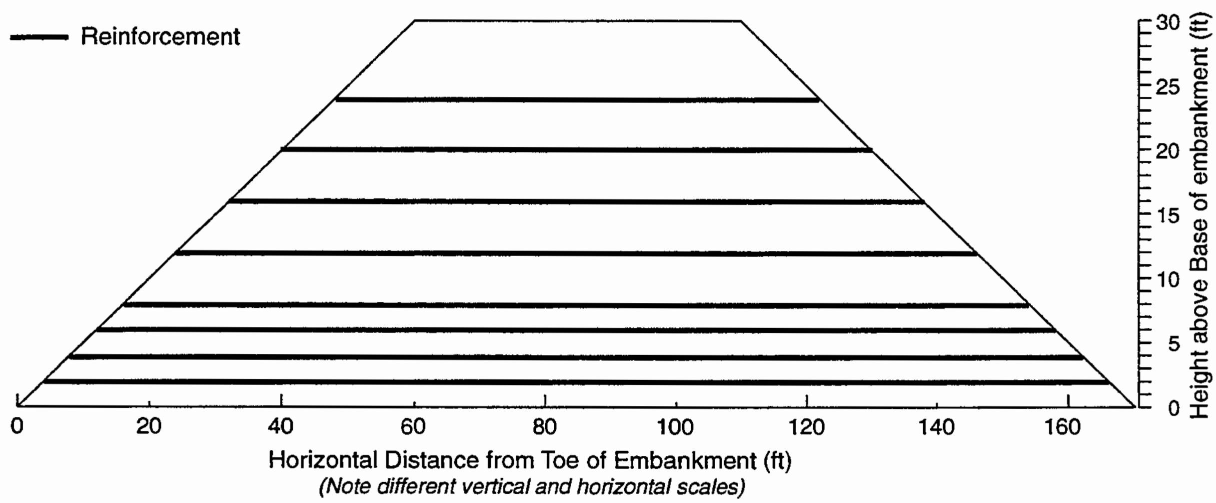


Figure 5.6 Reinforcement layout

Table 5.4 Reinforcement location and length

Reinforcement Line No.	Elevation (Height above base of embankment) in feet	Length in feet
1	2	72
2	4	72
3	6	72
4	8	72
5	12	72
6	16	72
7	20	72
8	24	72

5.4.5 Stability Computations for Reinforced Slope

Once a reinforcement layout was established, additional stability calculations were performed in which the effects of reinforcement were included. In the first set of these calculations, the manufacturer's recommended allowable tensile forces were assumed to be developed in the reinforcement. The forces were assumed constant along the length of the reinforcement, as shown in Figure 5.7. Stability analyses were conducted using UTEXAS3 with circular failure surfaces and Spencer's procedure. The reinforcement was modeled as horizontal layers with tensile forces.

An additional set of stability computations was performed in which the forces in the reinforcement were computed using the horizontal strains computed from the finite element deformation analyses. The reinforcement forces were computed as

$$F = \varepsilon M \tag{5.4}$$

where M is a tensile modulus for the reinforcement in pounds per foot, and ε is the strain. This assumes that the stress-strain behavior of the reinforcement is linear, which is reasonably true for strains of less than about 2 percent (see Figure 5.8). A tensile modulus of 95,000 pounds per lineal foot was used to compute the forces. Profiles of horizontal strains and associated forces along the reinforcement layers are shown in Figures 5.9 and 5.10. These forces were used in UTEXAS3 to again compute the stability. The forces actually used in the computations varied linearly between selected points on the profiles, as shown in Figure 5.11.

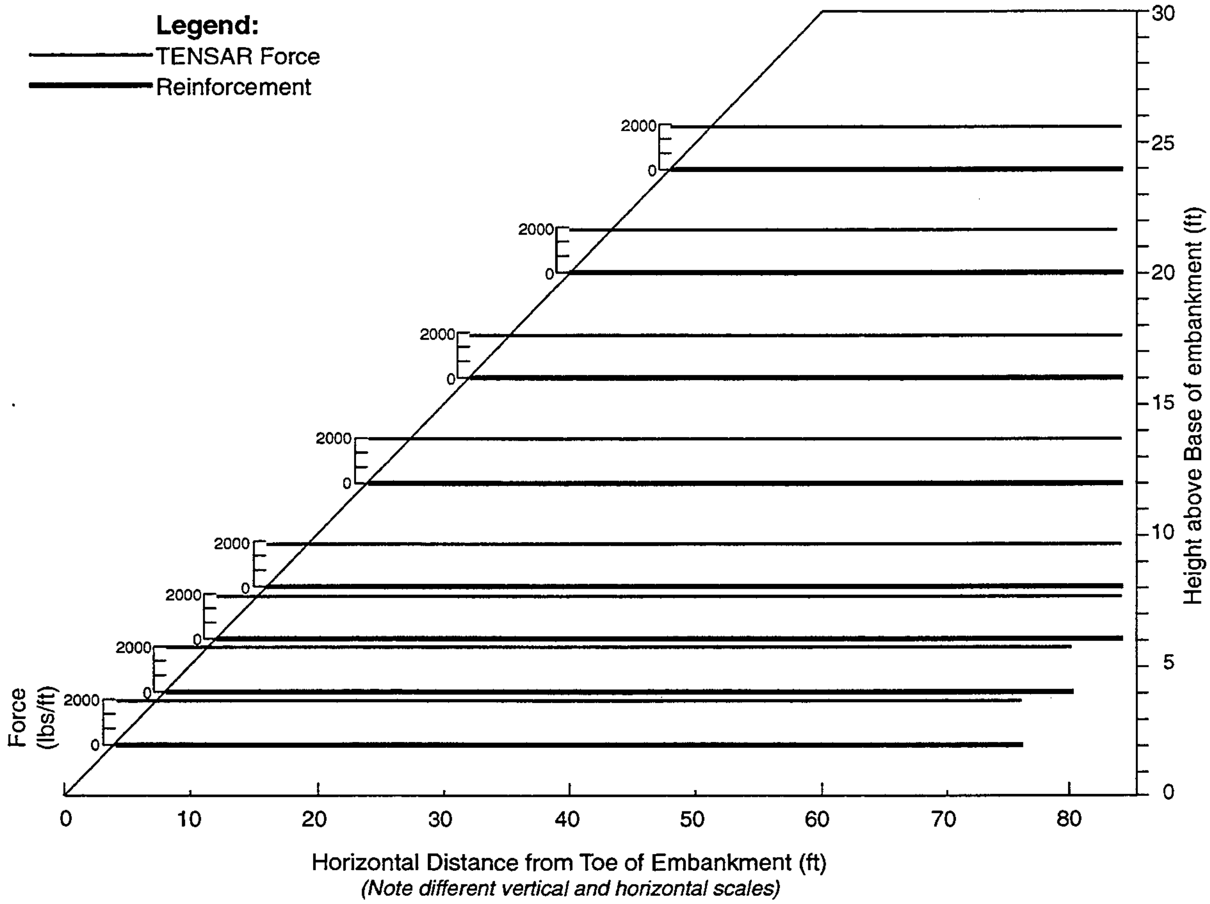


Figure 5.7 TENSAR recommended tensile forces for reinforcement

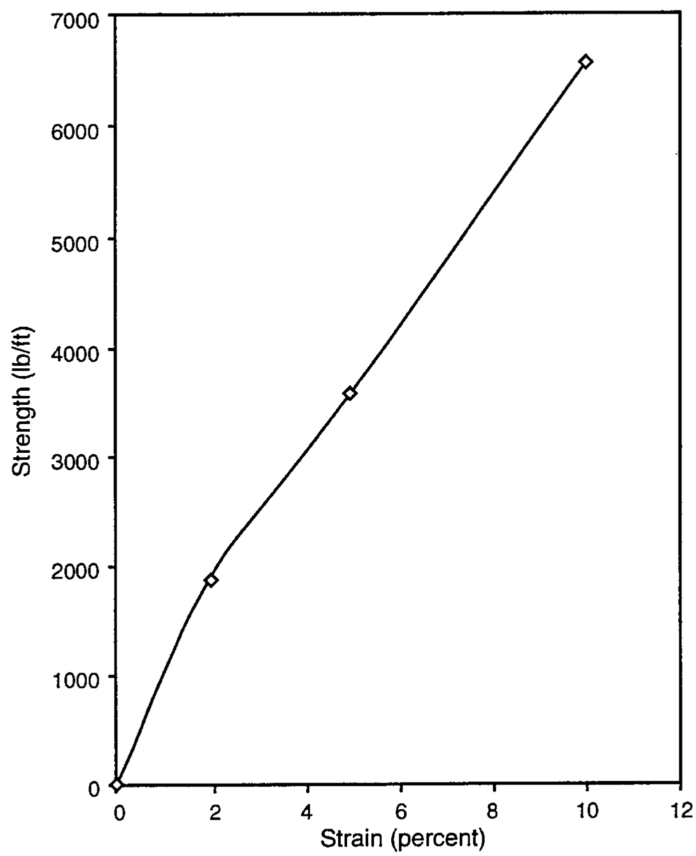


Figure 5.8 Tensile strength for TENSAR UX500HS geogrid

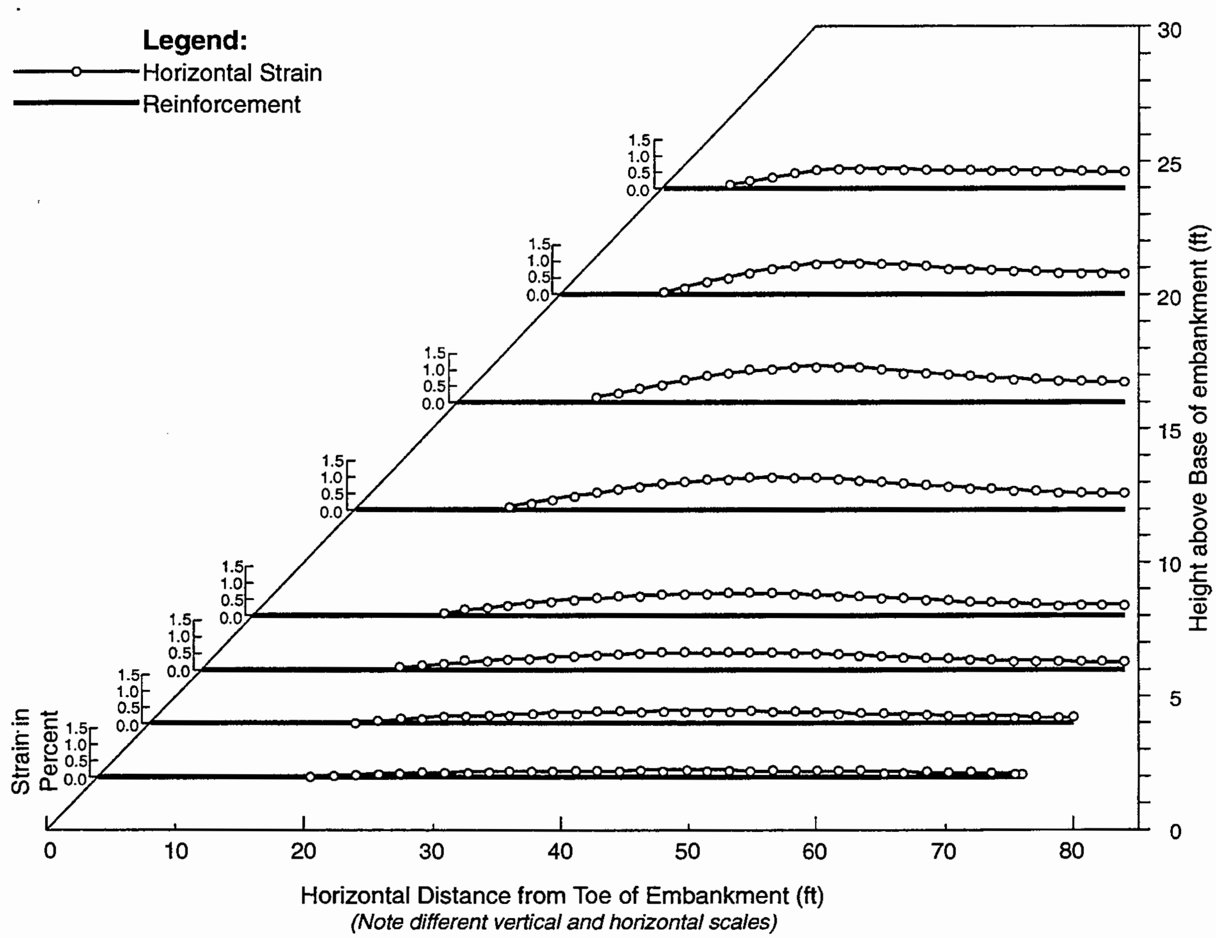


Figure 5.9 Horizontal tensile strain profiles along reinforcement locations

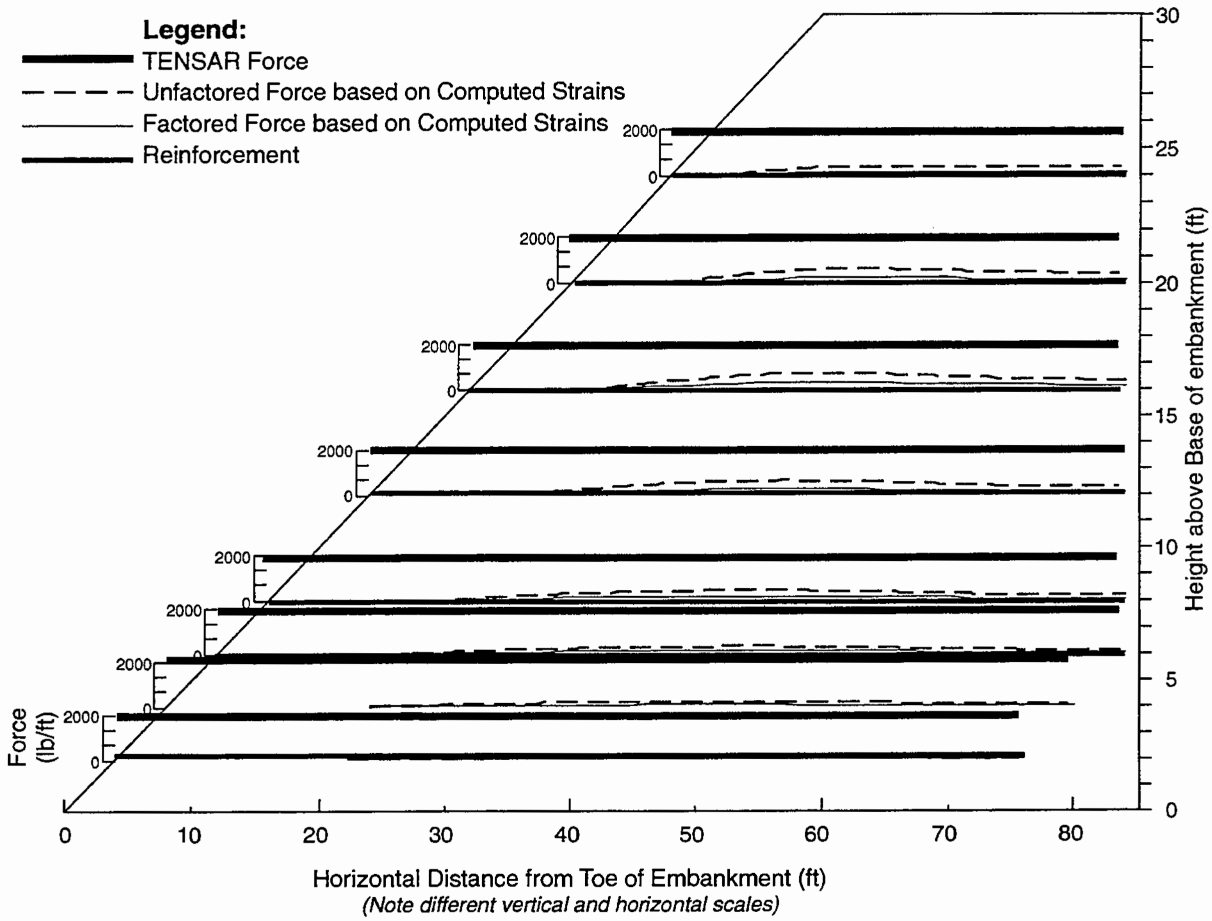


Figure 5.10 Factored and unfactored tensile forces based on computed strains during construction

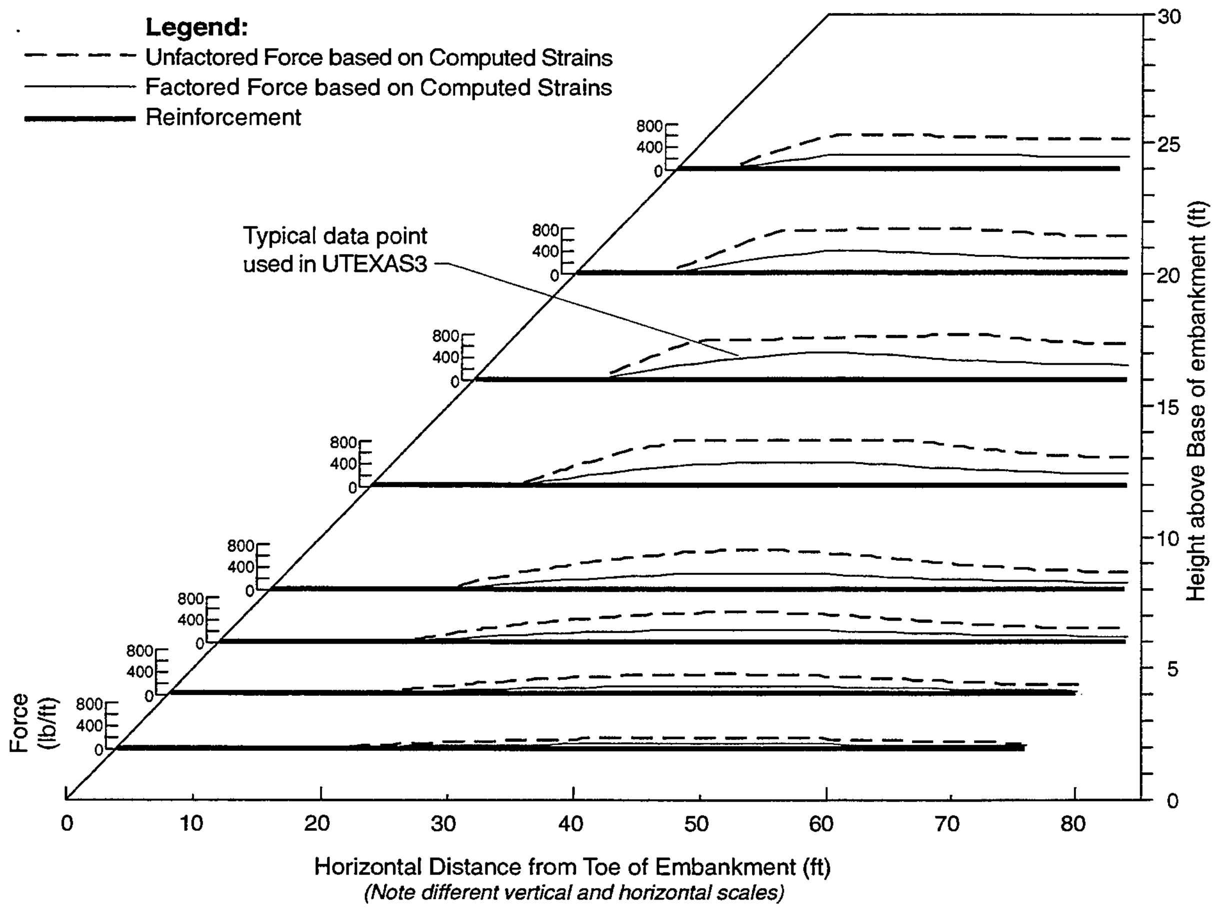


Figure 5.11 Tensile force profiles showing data points used in UTEXAS3

For the first set of stability calculations, where the manufacturer's allowable forces were used, the factor of safety for the critical circle was found to be 1.47. The factor of safety for the same circle using the forces obtained from the finite element analyses was found to be 0.71. An additional stability computation was performed without the reinforcement. This computation resulted in a factor of safety of 0.67 for the same circular surface. The circular failure surface used for these analyses and the corresponding factors of safety are shown in Figure 5.12. The factor of safety obtained using actual strains is obviously much less than the desired factor of safety (1.5), and is only 6 percent higher than the factor of safety for the unreinforced slope (0.67). Based on these results, it appears that use of standard recommended forces and procedures for reinforcement of embankments like those of interest (constructed of high PI clays) might lead to unsafe designs.

The computed strains and analyses described above consider only deformations that occur during construction of the slope. Additional strain may occur over time as a result of swelling of the soil and reductions in strength caused by increases in pore water pressures and softening effects. These strains will increase the forces and, thus, reduce the differences between factors of safety computed using actual forces and the conventional design values. However, experience with properly designed and stable slopes on competent foundations indicates that additional deformations over time seldom exceed construction deformations to a significant degree. Furthermore, the conditions selected to compute the strains during construction were specifically chosen to produce an upper limit on strains: A relatively high embankment with steep slopes was chosen, a soil with below average strengths was selected, and the effects of the slippage at the soil reinforcement interface was ignored.

5.5 SUMMARY AND CONCLUSIONS

The finite element computations presented were performed using a range of soil types and embankment geometry to establish horizontal strains that might develop in embankments during construction. Horizontal strains for embankment heights up to 30 feet and side slopes not exceeding 2:1 are less than 1 percent. These strains do not include any restraining effect of reinforcement during construction and, thus, represent an upper bound for strains during construction; the presence of reinforcement could make the horizontal strains even smaller. A clear trend exists between the factor of safety of a slope during construction and the maximum horizontal strain that the slope might experience. The results indicate that even for highly plastic clays, like the Taylor Marl, the maximum short-term horizontal strains may not exceed 1 percent for slopes with factors of safety greater than 1.3 during construction.

The maximum strains considered throughout this chapter are those that occur in a well-prescribed zone located away from the face of the slope. In contrast, failures in embankment slopes along Texas highways are shallow in nature, and generally well away from the zone of maximum strains. Thus, the contribution of reinforcement to stability of the slope in the zone where strengthening is needed may be even less.

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

6.1.1 Strength Correlations

Various correlations between shear strength and simple index properties reported in the literature were examined to determine their applicability to typical slopes in Texas. Data in the literature were supplemented by data and experience from other research on slope failures at The University of Texas. The purpose of this effort was to develop simple means to estimate the shear strength of the soil based on correlations with results of relatively simple index property tests, which maintenance personnel at TxDOT can obtain. The same correlations can also be useful for making preliminary estimates for design of new slopes.

Stark and Eid's correlations, although slightly conservative, were found to be the most promising. Step-by-step procedures and charts for estimating effective stress friction angle were presented. The procedures and charts were based on extensions of Stark and Eid's work to include stress levels of effective stresses lower than those that were originally reported. Specific procedures and recommendations have been presented for estimating strength for both new slopes as well as for slopes that have experienced previous sliding. Safety factors were calculated for a number of Texas embankments that failed using the recommended correlation. These calculations showed that significant positive pore water pressures must be assumed to explain the failures. Thus, it is also recommended that a water table coincident with the surface of the slope be assumed when the correlations are used. Only correlations for the long-term strength of highly plastic clays have been examined. Furthermore, these correlations should not supplant more thorough field and laboratory testing when funds are available.

6.1.2 Remedial Measures

Various remedial measures used to repair slopes have been examined in Chapter 4. Each measure was presented along with comments on the design, application, limitations, and any special precautions that should be taken for each measure. The purpose of this effort was to provide a guide for preliminary selection of remedial measures. A number of maintenance activities that may be detrimental to slope stability were also presented with suggestions on how such actions can be avoided.

6.1.3 Mobilized Reinforcement Forces

Finite element computations were performed using a range of soil types and embankment geometry to establish the horizontal strains that might develop in an embankment during construction. This effort was aimed at evaluating the mobilized forces in horizontal, geogrid reinforcement. It was found that, for high PI soils and embankment geometries like those encountered along Texas highways, maximum computed horizontal strains were less than or near 1 percent. If reinforcement were included in the computations, the effects or presence of reinforcement would even further diminish the magnitude of the computed horizontal strains. A clear trend was also shown to exist between the factor of safety of a slope during construction and the maximum horizontal strain that the slope might experience during or immediately after construction. For the typically high factors of safety during construction, which are common for Texas embankments, the *maximum* horizontal strains for an unreinforced embankment may be well below 1 percent.

Results of slope stability calculations for a typical high PI clay embankment with reinforcement showed that the factor of safety based on mobilized forces was significantly less than 1 when the reinforcement was designed using conventional approaches and accepted design values for reinforcement forces. Using *actual* mobilized forces in the reinforcement the factor of safety was increased by only 6 percent from a value of 0.33 for the unreinforced slope. Thus, conventional procedures for estimating reinforcement forces for design do not seem applicable to slopes in high PI clays and should not be used.

6.2 RECOMMENDATIONS FOR FUTURE RESEARCH

Future research aimed at a better understanding of the long-term performance of slopes and development of reinforcement forces in such slopes would be useful. Such research would need to better define the nonlinear stress-strain properties of soil, including modeling the effects of softening caused by repeated wetting and drying. Once these effects are better understood, finite element models can be developed and used to explore the long-term response of reinforced slopes and the development of forces in the reinforcement.

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APPENDIX A

RESULTS OF FINITE ELEMENT SENSITIVITY STUDIES

APPENDIX A. RESULTS OF FINITE ELEMENT SENSITIVITY STUDIES

The objective of the sensitivity studies was to examine the influence of various parameters and certain assumptions (usually made in performing finite element analyses for earthen embankments) on the magnitude of horizontal strains during construction. Taylor Marl was the soil chosen for all sensitivity studies. This soil was chosen because it tended to produce the largest horizontal deformations of the soils studied. Thus, effects of parameter variations were expected to be accentuated. Table A.1 lists, in addition to the slope height and slope inclination, all the parameters that were studied and the associated geometric ranges of the finite element computation series.

Typically, in checking the sensitivity of a technical solution to the variation of one parameter, all other parameters involved in the sensitivity analyses are kept constant. Unless otherwise noted the following values are the constant/default values that were used in the sensitivity analyses while varying any parameter.

- Crest width = 50 feet
- Slope height = 30 feet
- Slope inclination = 2:1
- Total unit weight = 121 pcf
- Initial total horizontal stresses for any construction layer = 0.75 x Total vertical stresses
- Number of rows of finite elements per construction layer = 1
- Solution scheme for nonlinear equations: Newton's method

These constant/default values were chosen partly based on the studies described below, and partly because they represent the most practical values from the range considered. An effort was made to use values that would be reasonable and would yield higher strains to establish an upper magnitude for the horizontal strains in an embankment during construction.

A.1 TYPICAL HORIZONTAL STRAIN PROFILES

Horizontal strain is the only engineering quantity examined and compared in all sensitivity analyses. The following paragraph describes and illustrates the approach used to compare different results of the analyses.

Figure A.1 presents typical contours of horizontal strains and displacements in an embankment immediately after construction. The results shown in Figure A.1 are for an embankment with 30-foot height, 2:1 slope inclination, and 50-foot crest width. Also shown on Figure A.1 are elevation lines of 20, 40, 60, and 80 percent of the slope height. Figure A.2 presents profiles of the horizontal strains taken at these elevations. It is at these elevations that results of different sensitivity analyses are compared. These elevations were chosen to cover most of the horizontal strains experienced in each analyzed case. These profiles of strain might also represent forces in the reinforcement if reinforcement was placed at the associated horizontal elevations.

For comparison and plotting purposes, the horizontal distance from the toe of the embankment into the embankment was normalized with respect to the horizontal distance from the toe to the outside edge of the embankment's crest of each embankment (see Fig. A.1).

Table A.1 List of parameters and geometric ranges considered for parametric studies

Parameter	Parameter Values	Slope Inclinations	Slope Heights
Crest Width	50, 100, and 150 feet	2:1	10, 20, 30
Finite Element Size	1, 2, and 4 rows per construction layer	2:1	10, 20, and 30 feet
Solution Scheme for Nonlinearity	Newton's versus 2-Step Cycle Method	2:1	10, 20, 30, 40 and 50 feet
Total Unit Weight	100, 120, and 150 pcf	1:1, 2:1, and 3:1	10, 20, 30, 40 and 50 feet
Lateral Pressure Coefficient	0.5, 0.75, and 1.0	1:1, 2:1, 3:1, and 4:1	10, 20, 30, 40 and 50 feet

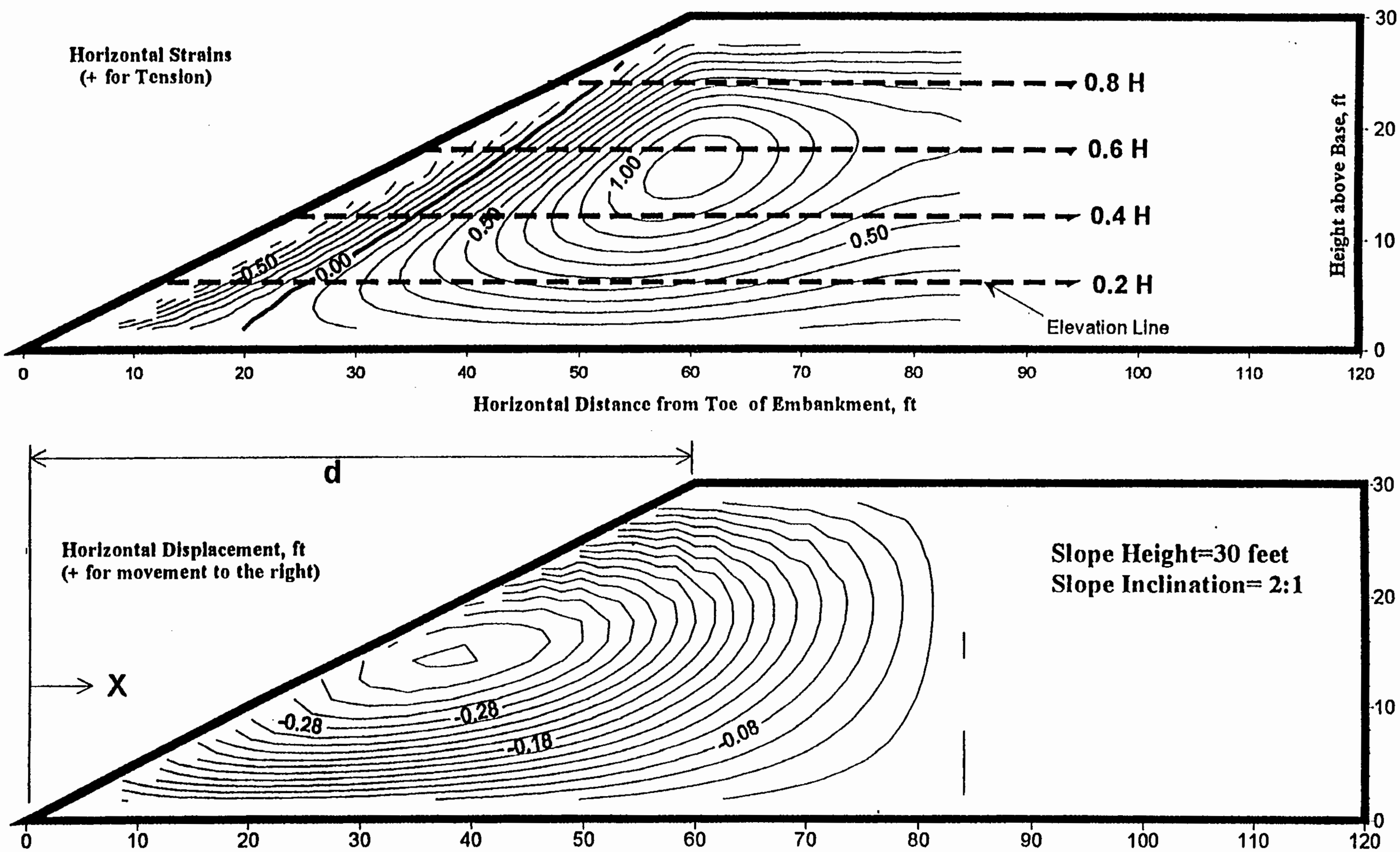

Figure A.1 Typical Horizontal Strains and Displacements with Elevation Lines for Strain Profiles

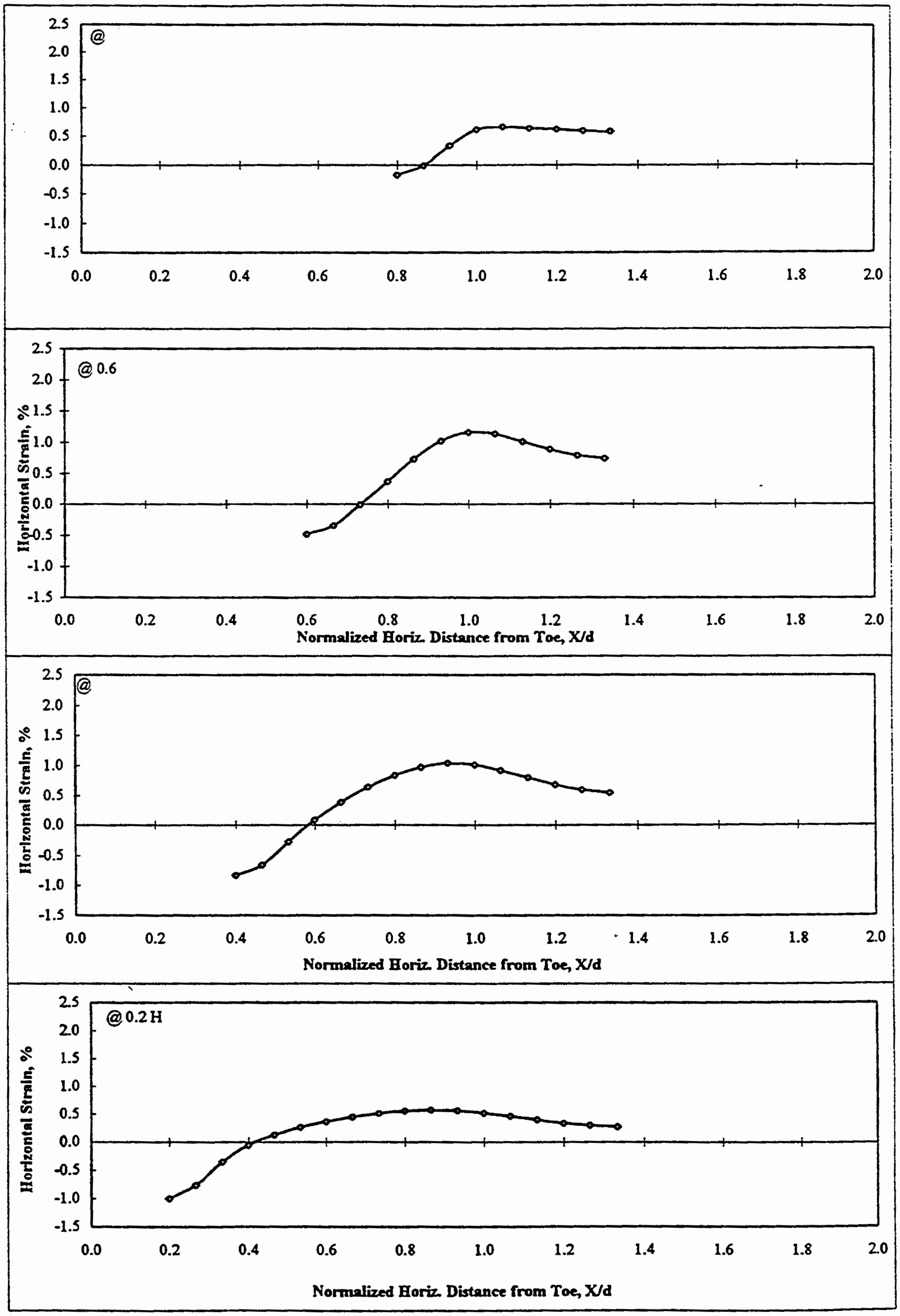

Figure A.2 Profiles of Horizontal Strains for Various Elevation Lines for 2:1 Slope and 30-foot Slope Height

A.2 EFFECTS OF SLOPE HEIGHT

As expected, peak strains increased with an increase in slope height. Figures A.3 through A.5 illustrate the effects of varying the slope height, keeping the slope inclination constant. Distribution of horizontal strain was found to be similar in all cases for both the 40 and 60 percent elevation lines with location of peak horizontal strain remaining almost constant. Figures A.3 through A.5 indicate that although the distribution of horizontal strains along the remaining elevation lines are also similar, the location of peak horizontal strain appears to shift toward the slope face in the lower part of the embankment and away from the slope face in the upper part of the embankment.

Data for the 2:1 slope was replotted in Figure A.6 with the horizontal strains normalized with respect to slope height. The same data again was replotted in Figure A.7, but normalized with respect to maximum horizontal strain. Normalization with respect to slope height appears to be a useful process except in the upper profile (at 80 percent of the slope height) where both normalization methods lead to high variation in the normalized profiles; a phenomenon that might be attributed to the possibility/fact that the finite element method does not yield accurate displacements near the top of the embankment. Figure A.8 presents similar results for the 3:1 slope. As can be shown in both Figures A.6 and A.8, for heights up to 30 feet, the normalization with respect to slope height appears to be very useful and adequate for purposes of estimating horizontal profiles at any elevation level in the embankment for a known inclination. This finding is explored further at the end of the following section.

A.3 EFFECTS OF SLOPE INCLINATION

Finite element computations were also performed on embankments of fixed height while varying the slope inclination. Figures A.9 and A.10 present the horizontal strain profiles for two cases of slope heights of 10 and 30 feet. The slope inclination was varied from 1:1 to 4:1. Again, as expected, the horizontal strains increased with an increase in slope inclination. The effects of varying the slope inclination on the distribution of strains and location of peak horizontal strains are similar to the effects produced when increasing the height of the slope as described in the previous section. In other words, the results show similar distribution with location of peak strain moving toward the face and away from the face of the slope, in the upper and lower portions of the embankments, respectively.

Values of the horizontal strains for the cases shown in Figures A.9 and A.10 were normalized by multiplying them by the respective slope ratio (1/cotangent). The results are presented in Figures A.11 and A.12. As can be seen, this normalization does not appear to be effective in terms of strain distribution, but seems to capture or nearly capture the peak horizontal strain along each profile.

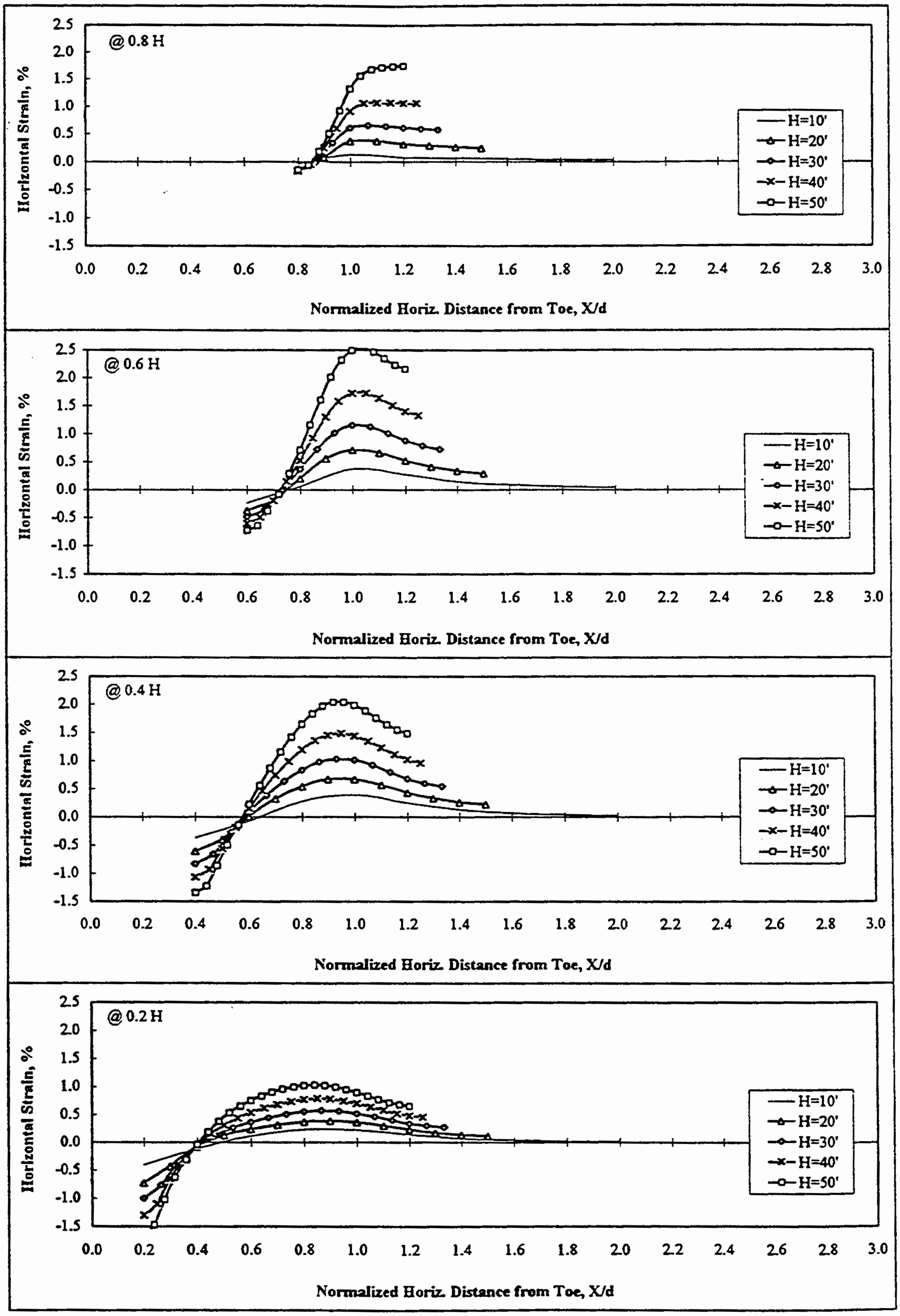

Figure A.3 Profiles of Horizontal Strains Along Various Elevation Lines for 2:1 Slope

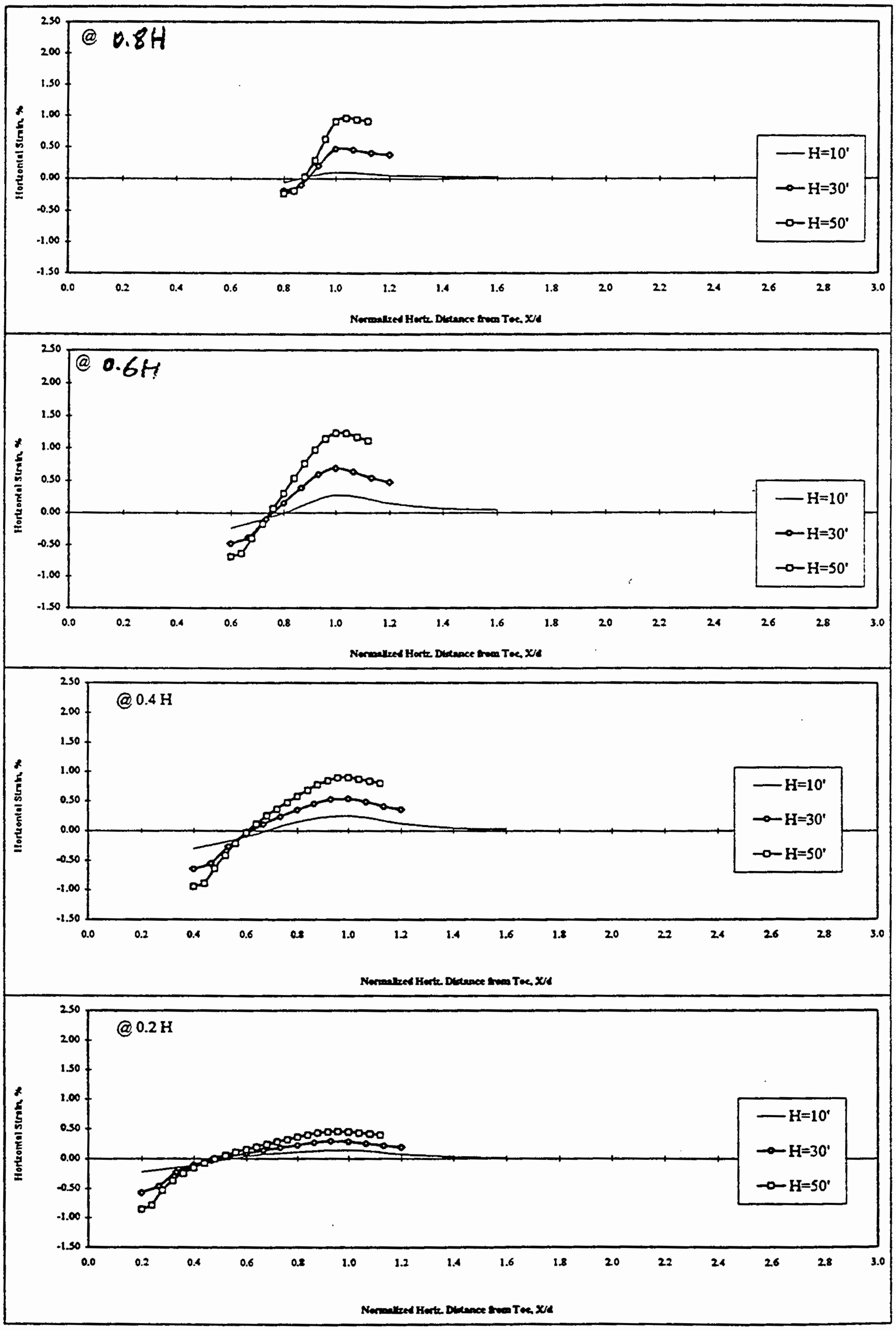

Figure A.4 Profiles of Horizontal Strains Along Various Elevation Lines for 3:1 Slope

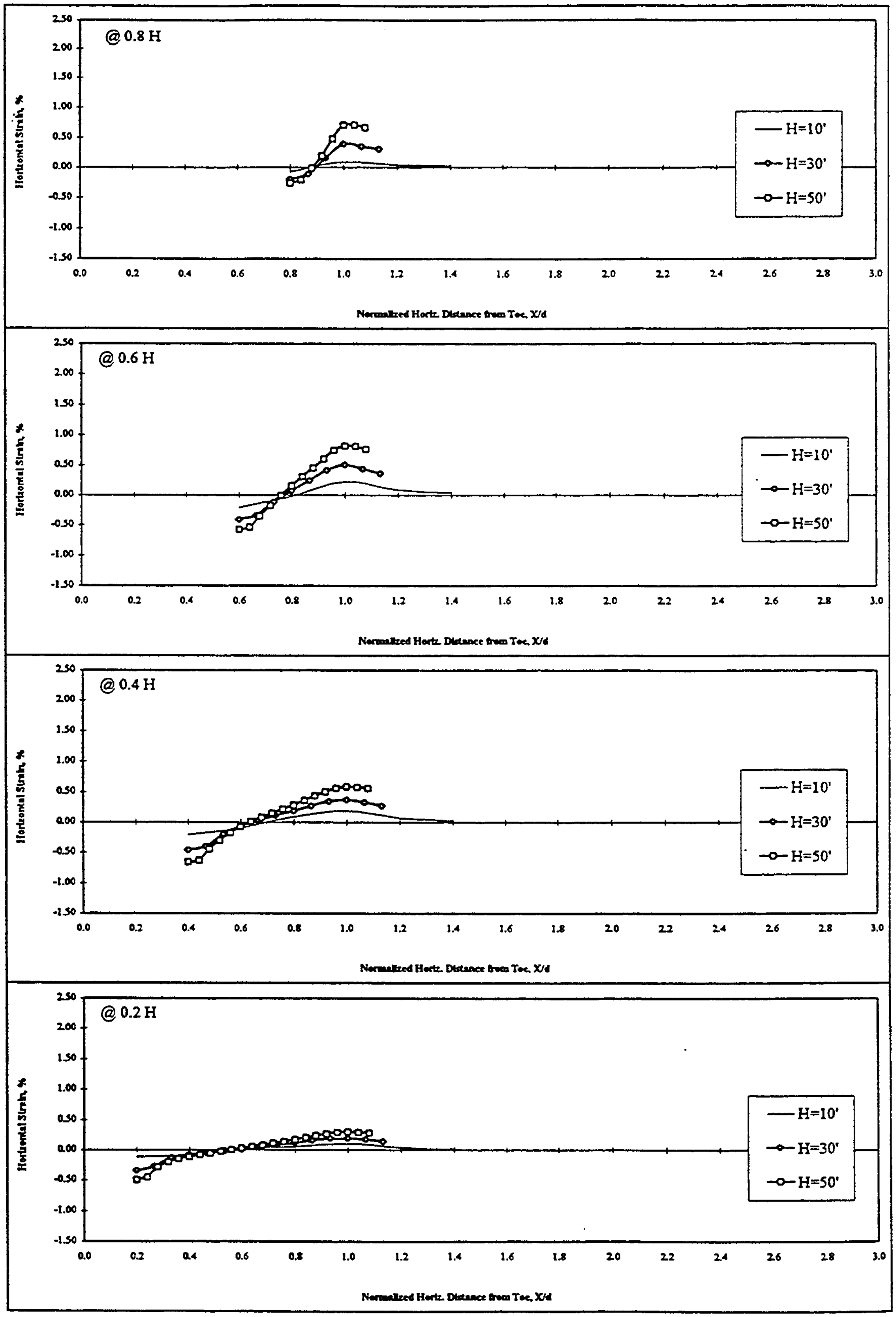

Figure A.5 Profiles of Horizontal Strains Along Various Elevation Lines for 4:1 Slope

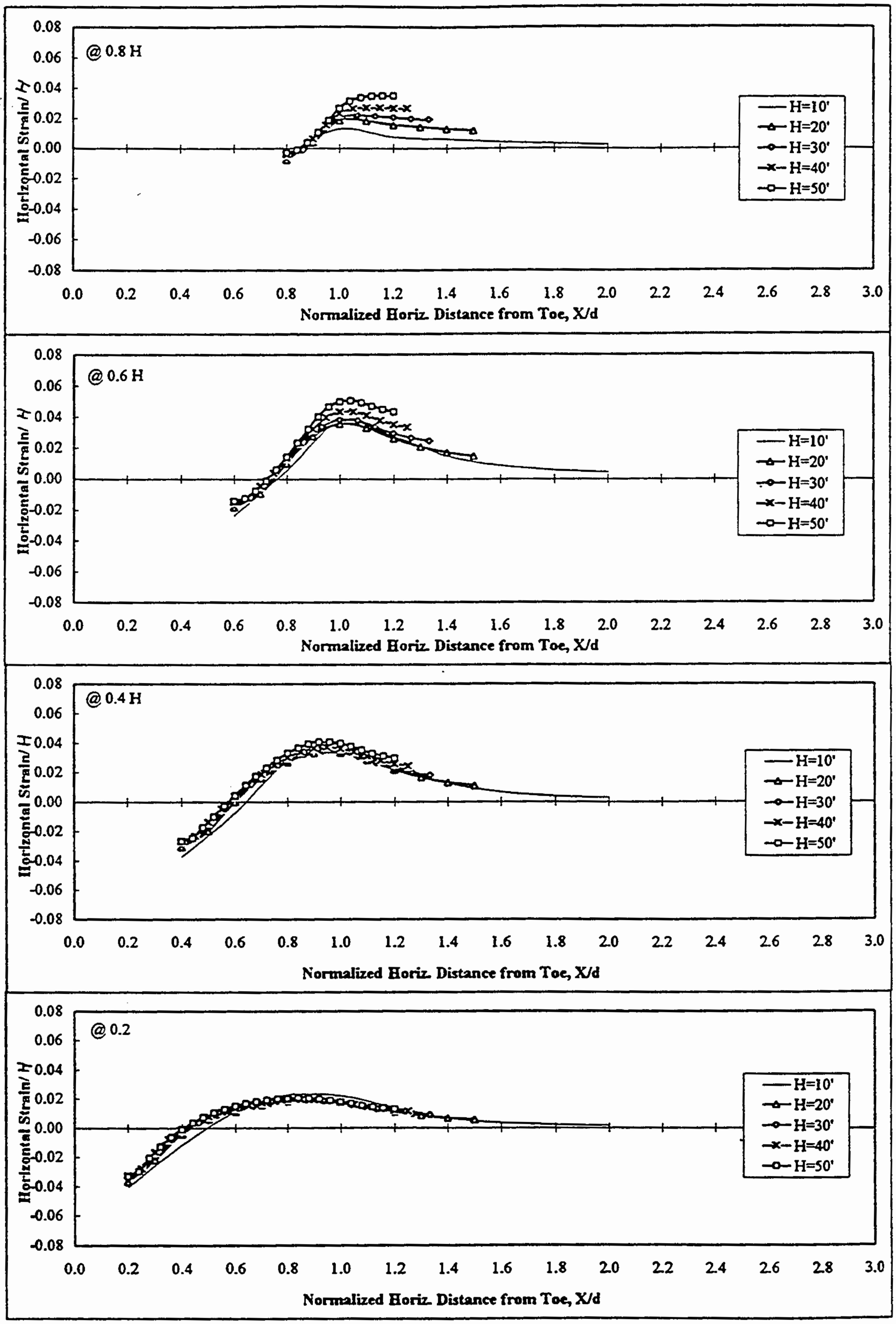

Figure A.6 Profiles of Normalized Horizontal Strains with Respect to Slope Height for 2:1 Slope

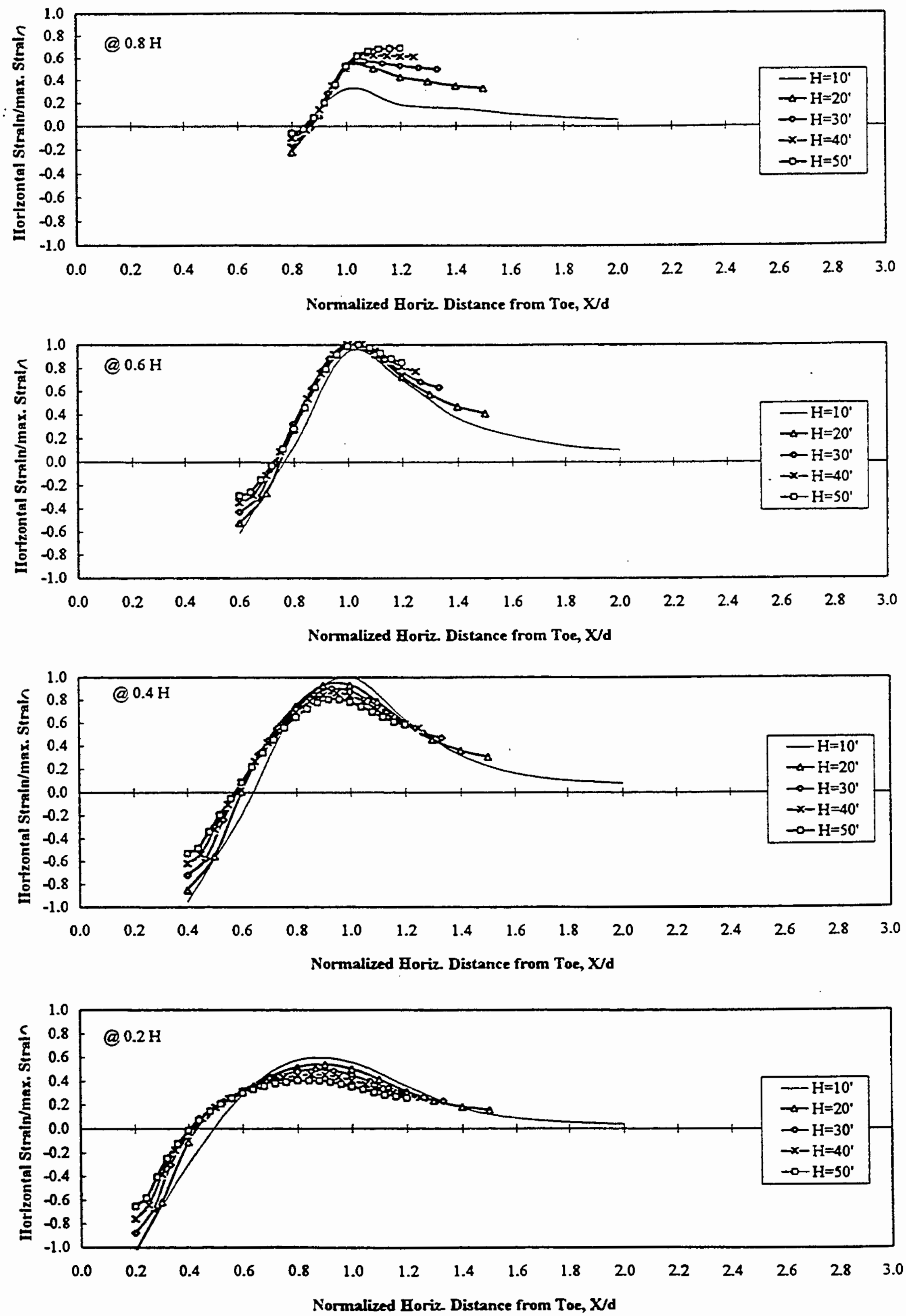

Figure A.7 Profiles of Normalized Horizontal Strains with Respect to Maximum Horizontal Tensile Strain for 2:1 slope

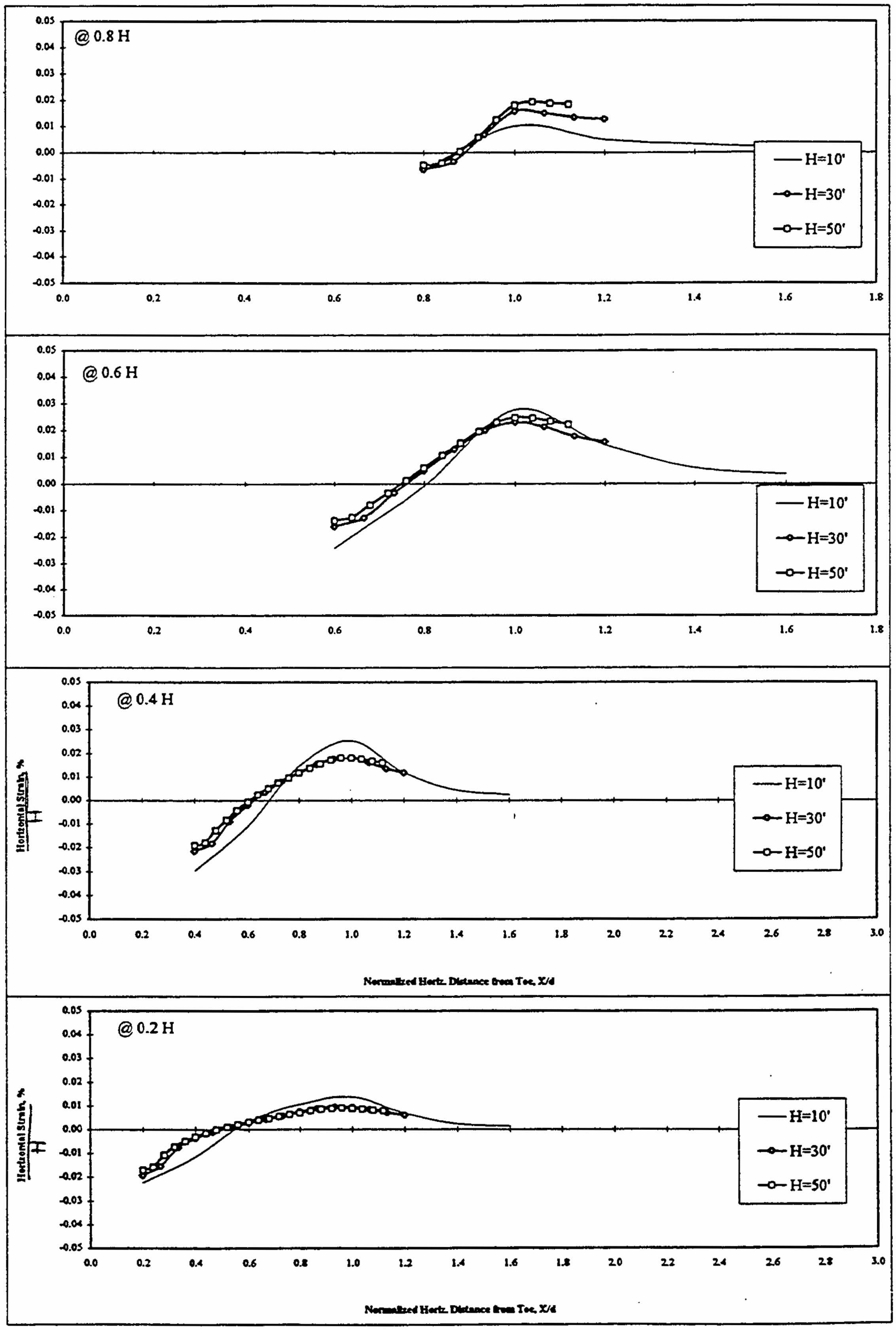

Figure A.8 Profiles of Normalized Horizontal Strains with Respect to Slope Height for 3:1 Slope

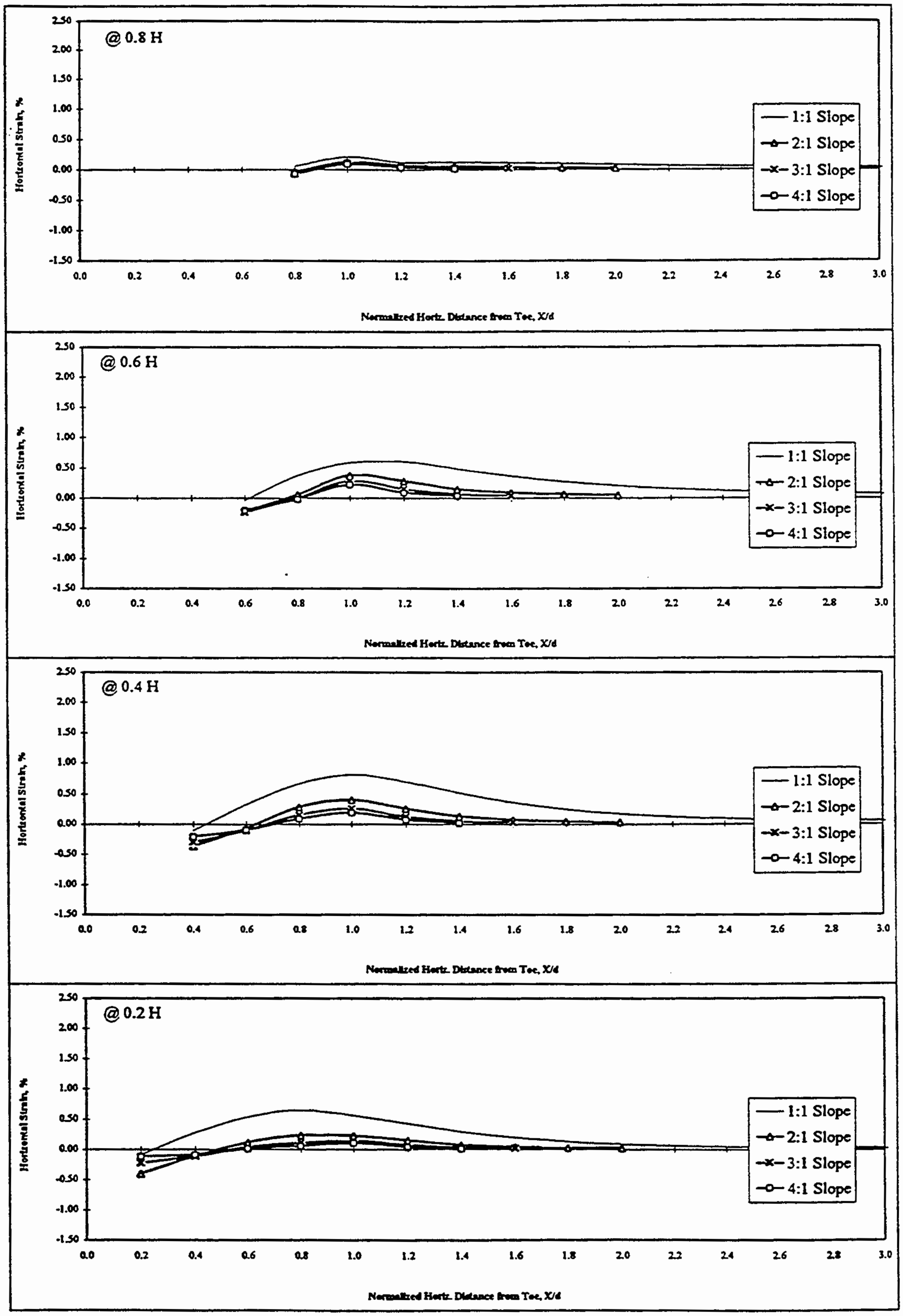

Figure A.9 Profiles of Horizontal Strains Along Various Elevation Lines for 10-foot Height Slopes

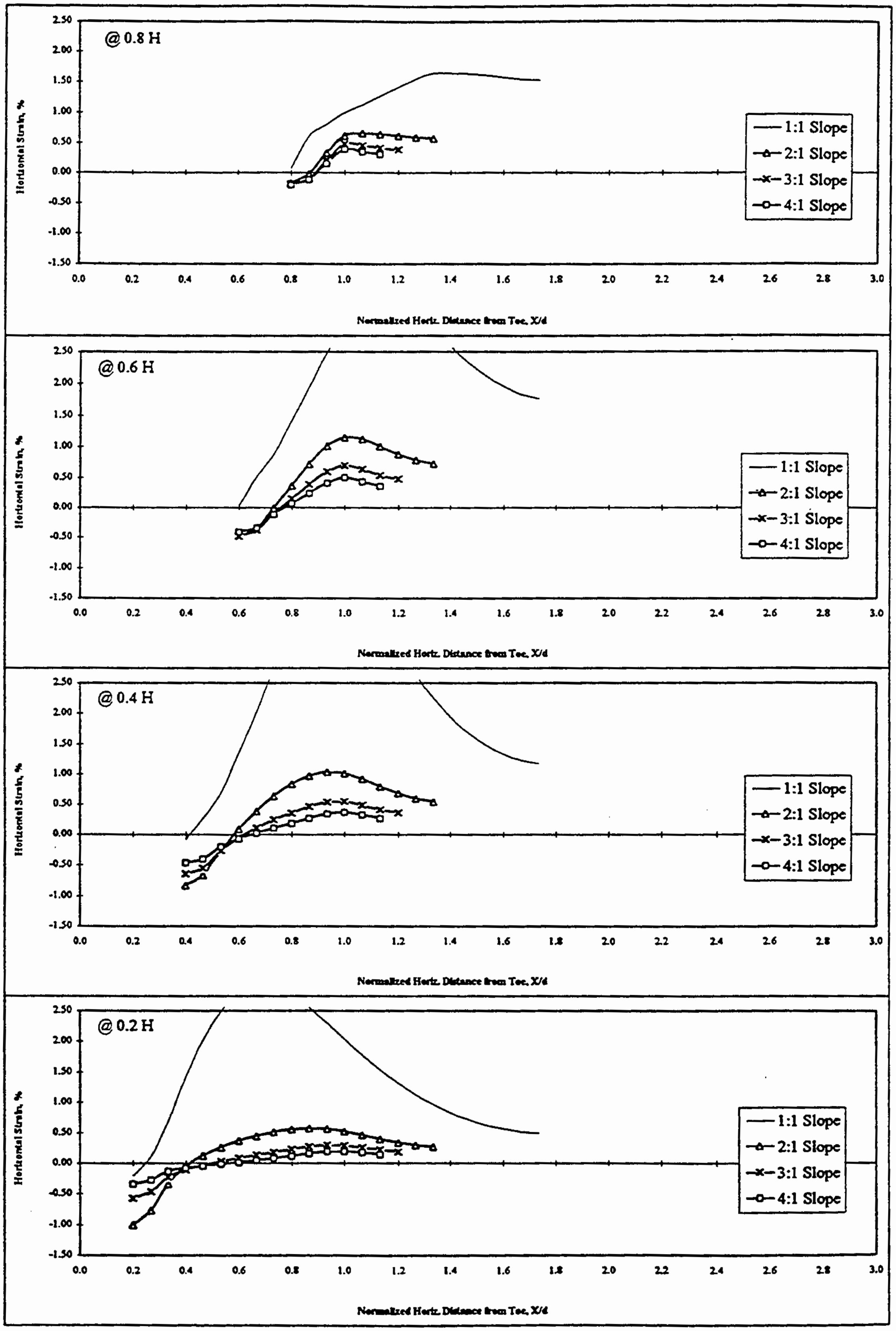

Figure A.10 Profiles of Horizontal Strains Along Various Elevation Lines for 30-foot Height Slopes

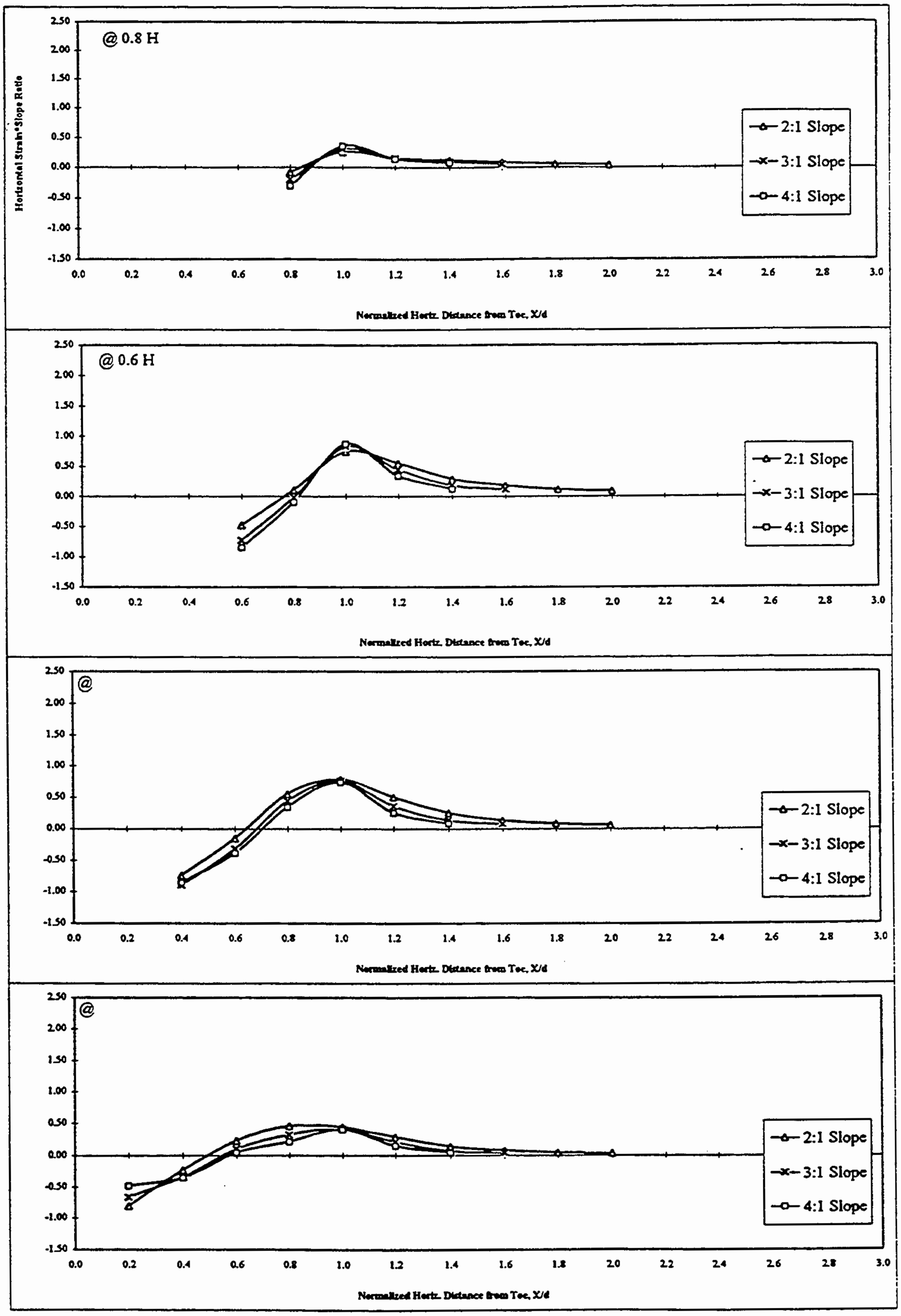

Figure A.11 Profiles of Normalized Horizontal Strains with Respect to Slope Ratio for 10-foot Height Slopes

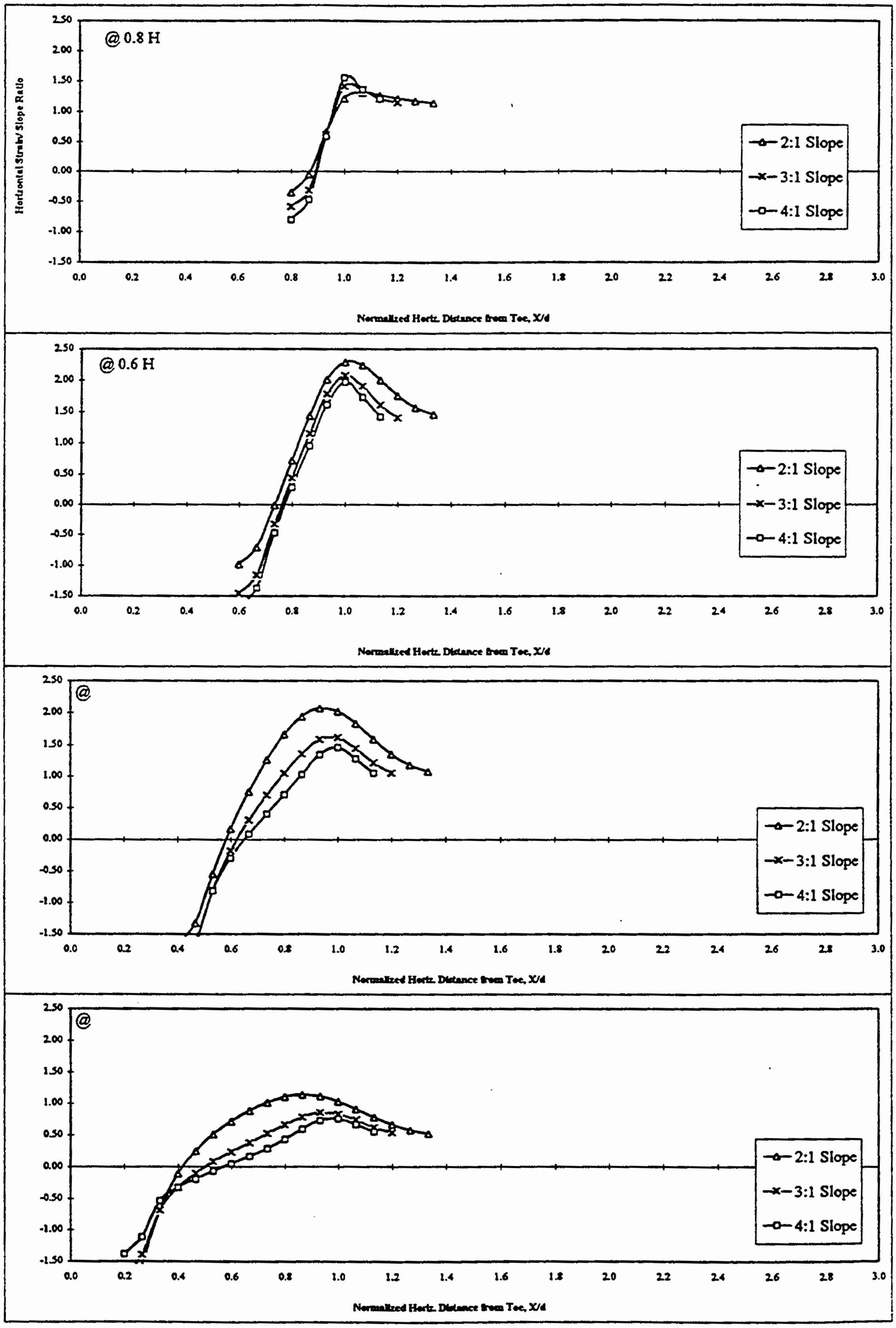

Figure A.12 Profiles of Normalized Horizontal Strains with Respect to Slope Ratio for 30-foot Height Slopes

By combining the two normalization schemes, it is possible to obtain approximated strain profiles in an embankment that are independent of slope height and slope angle. Such result may be useful for developing charts or some means for estimating horizontal strain magnitude and distribution to compute mobilized forces in an embankment after its construction. To illustrate this, the strains obtained from the finite element computations for 2:1, 3:1 and 4:1 slopes were multiplied by the slope ratio and divided by the slope height and replotted versus the normalized horizontal distance from the toe in Figure A.13. A data band was plotted to cover the range of values obtained. The lower part of the band would be recommended for use because it results in lesser (hence conservative) values for the reinforcement forces to be relied upon.

A.4 EFFECTS OF EMBANKMENT CREST WIDTH

To examine the effect of crest widths on the location and magnitude of horizontal strains, three different crest widths of 50, 100, and 150 feet were used in the analysis of a 2:1 slope and three different slope heights (10, 20, and 30 feet). The steeper the slope the higher the effect of the crest width on the numerical solution; the 2:1 slope was considered to be representative of the steeper slopes built by TxDOT.

Results of the finite element computations are shown in Figures A.14 through A.16. As shown in these figures, for embankment heights of up to say 20 feet, the effect of the crest width is negligible. For higher embankments (30 feet), the effect is also insignificant for width cases higher than 100 feet. However, peak horizontal strains computed using a crest width of 50 feet were found to be 15 to 25 percent higher in the upper half of the embankment than those computed using the 100- and 150-foot width.

Based on the studies described above, it was decided to conduct the remainder of the parametric studies using a crest width of 50 feet. This width was chosen to remain consistent with the objective of establishing the upper, but realistic, magnitude of the maximum horizontal strains in embankments during construction.

A.5 INFLUENCE OF FINITE ELEMENT SIZE AND NUMBER OF CONSTRUCTION LAYERS

It is always desirable to simulate the fill placement by using a similar number of layers in the computer analysis as that in the actual field. Actual fill layer thickness during the construction of an embankment may vary between 6 and 18 inches. For example, for a 50-foot embankment, the actual number of layers that would be required in the simulation can vary between 75 to 100. The use of such a high number of layers may impose practical limitations involving computer time, cost, storage, and memory. Previous investigators (Clough and Woodward, 1966, and Kulhawy et al., 1969, and Clough and Duncan, 1969) have shown that the number of construction layers have an effect, but they also showed that increasing that number in the computer simulation has a diminishing effect. They concluded that a total of seven to eight layers appear to yield practical results for computation of displacements (hence strains), and the use of higher numbers of construction layers leads to insignificant improvement in the solution.

In this study, all construction layers have been given a fixed thickness of 2 feet. The 2-foot thickness was chosen because of (1) its proximity to the actual layer thickness (0.5 to 1.5 feet), (2) computer memory requirements, limiting the maximum number of layers that can be used in each case, and (3) the fact that it provides for most of the cases studied a number of layers that is higher than that which is recommended by previous investigators.

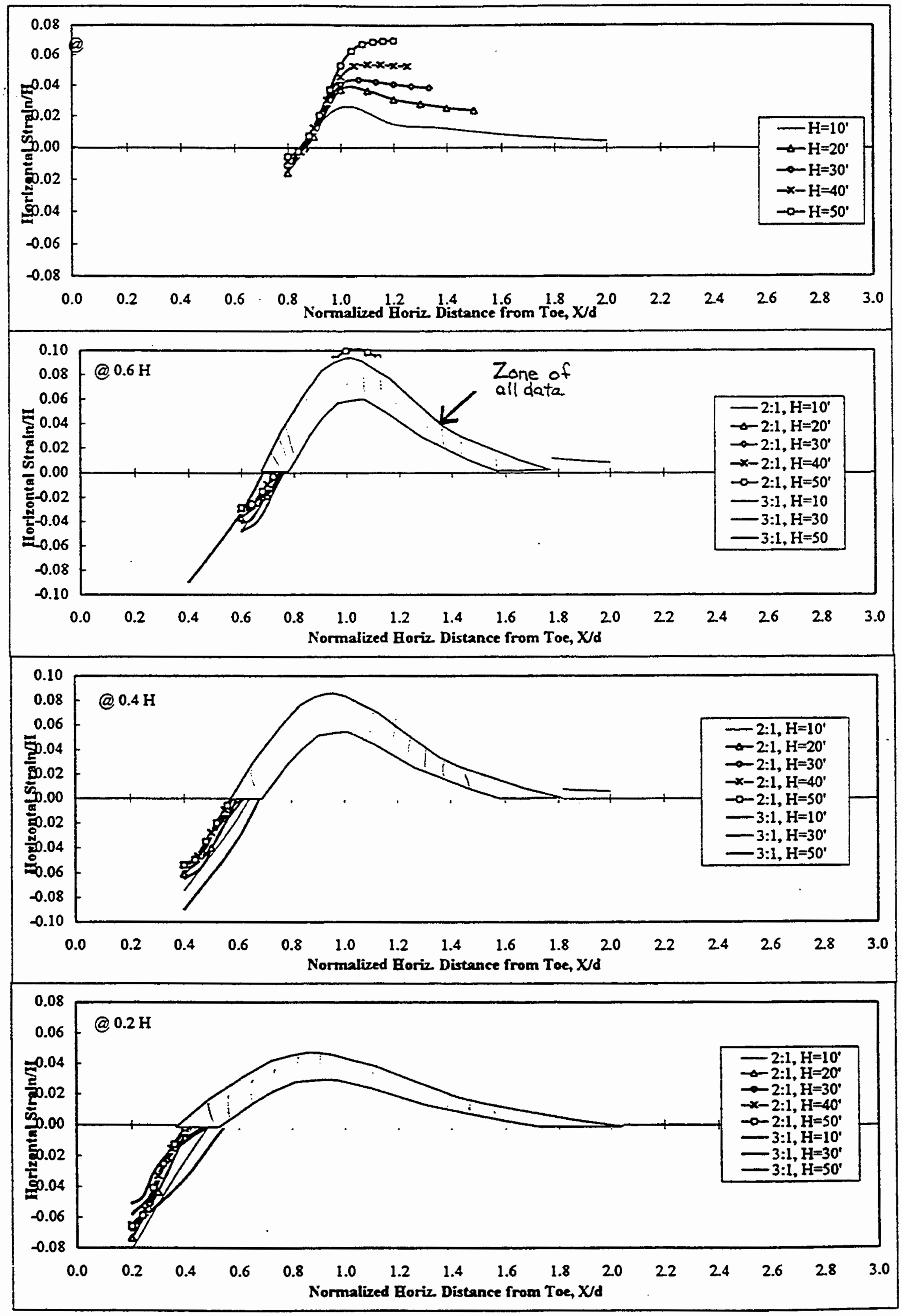

Figure A.13 Profiles of Normalized Horizontal Strains with Respect to Slope Height and Ratio

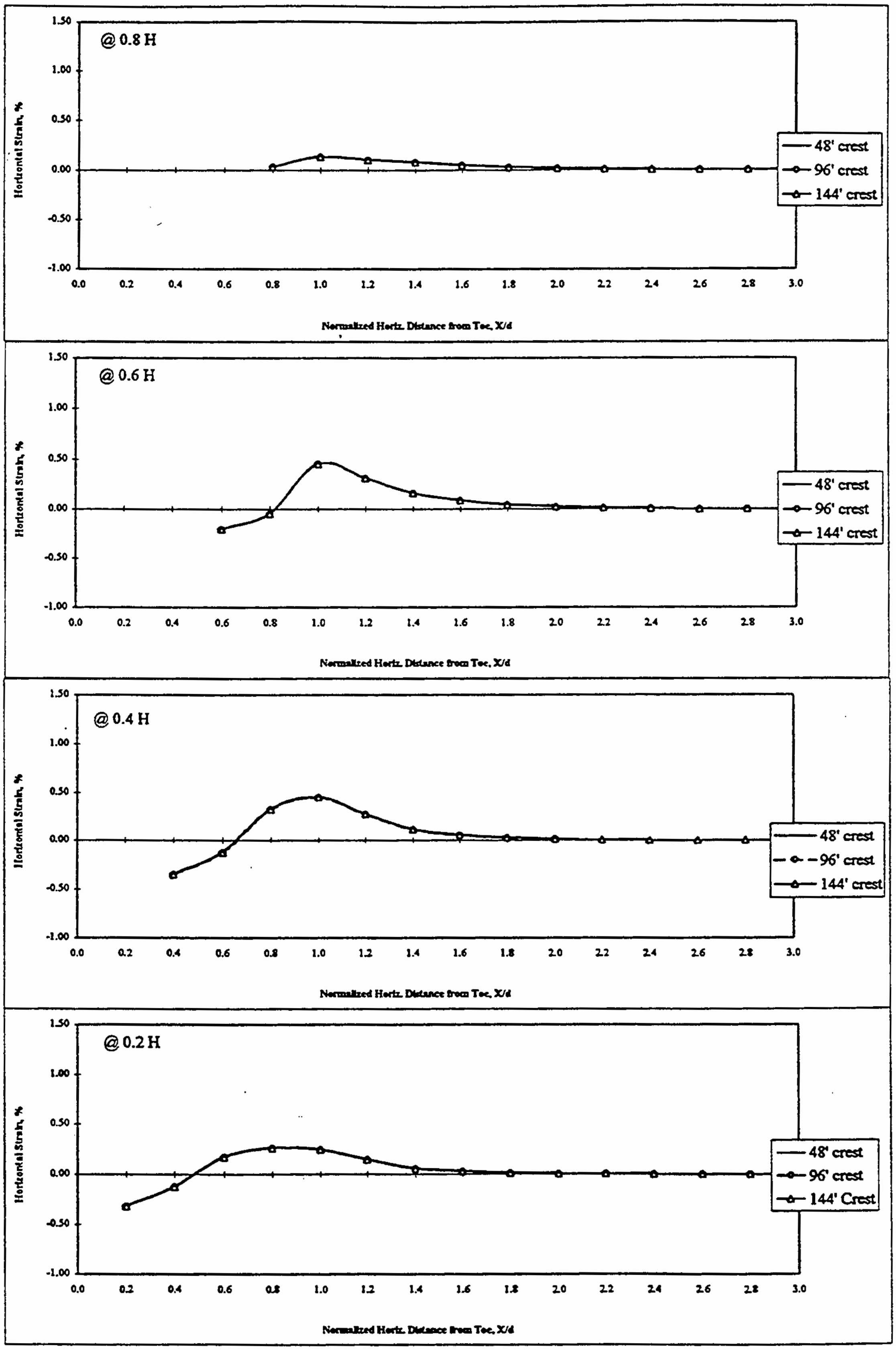

Figure A.14 Profiles of Horizontal Strains Showing Effects of Crest Width Variation for 2:1 Slope and a Slope Height of 10 Feet.

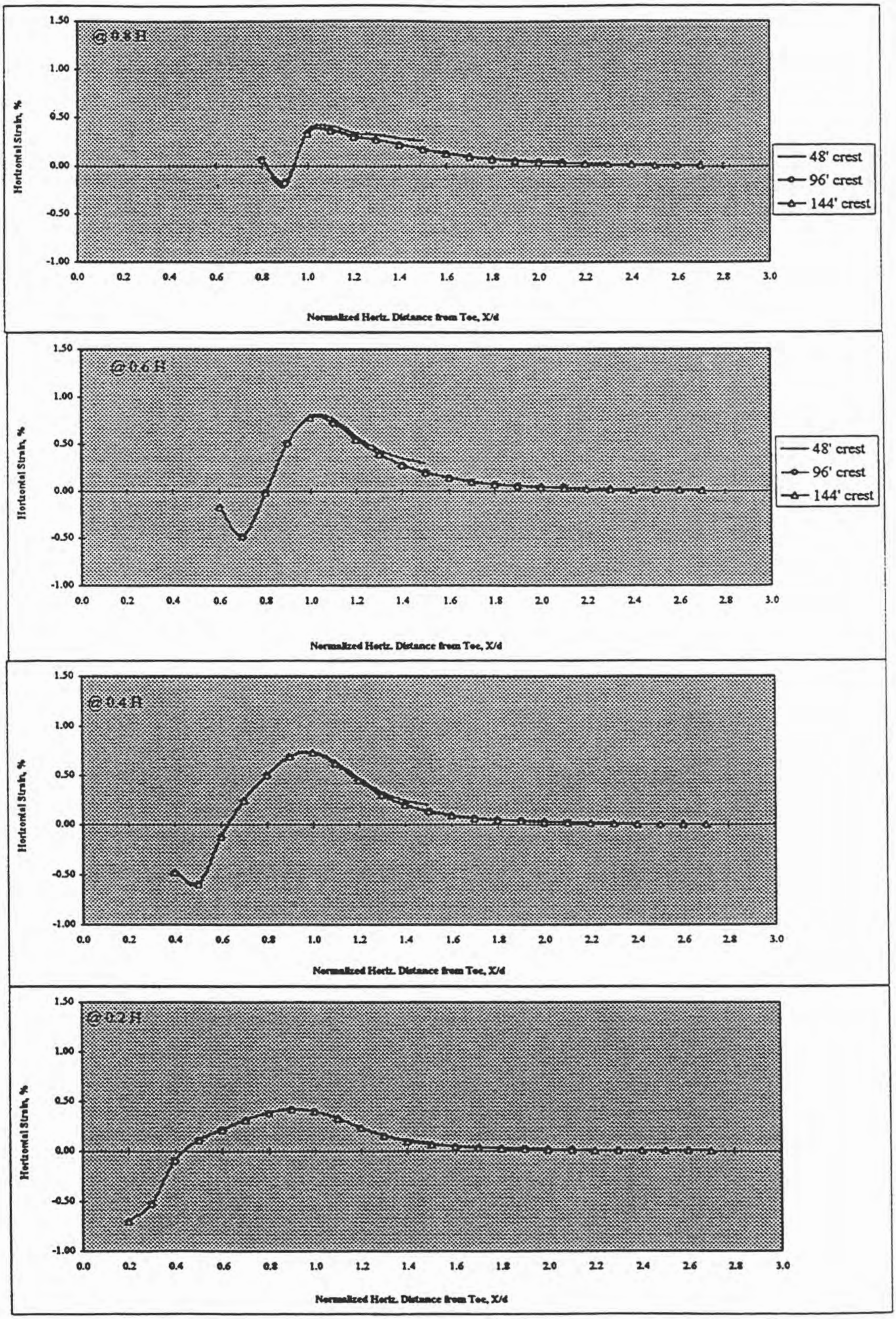

Figure A.15 Profiles of Horizontal Strains Showing Effects of Crest Width Variation for 2:1 Slope and a Slope Height of 20 Feet.

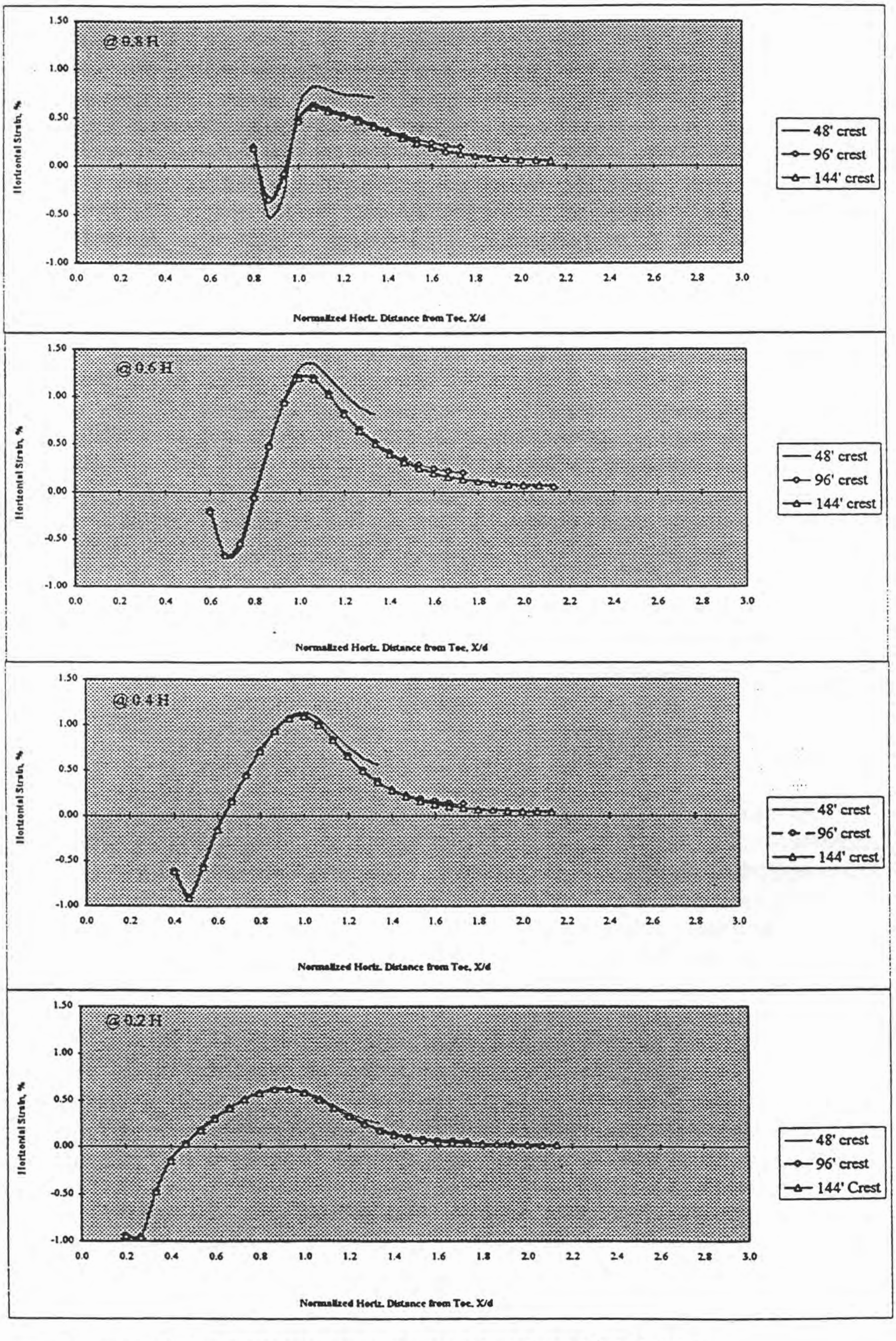

Figure A.16 Profiles of Horizontal Strains Showing Effects of Crest Width Variation for 2:1 Slope and a Slope Height of 30 Feet.

Another important factor in the finite element computation/solution is the size of the finite elements used in each layer. To examine the effects of finite element size, construction layers during each simulation were made to consist of different rows of finite elements. Thus, the size of finite elements was changed by changing the number of finite element rows in each construction layer. One, two, and four rows per layer were used resulting in elements that are two-, one-, and half-a-foot wide, respectively. Finite element computations were performed on a 2:1 slope for different embankment heights (10, 20, and 30 feet). Horizontal strains were computed for each combination of slope inclination, slope height, and a specific number of rows per construction layer. Results of these analyses are presented in Figures A.17 through A.19.

As shown in Figures A.17 through A.19, the difference in horizontal strains seems to be minor throughout the embankment except near the face of the slope for all considered slope heights. However, it should be noted that the effect of the finite element size diminishes as the height of the embankment increases. The major influence seems to be in the area closer to the slope face where the strains change from being tensile to being compressive. It appears that finer meshes result in a better job of catching the transition in that zone. For higher slopes, the only major discrepancy seems to take place in the bottom portion of the embankment and near the toe. The difference in horizontal strain for a fine and a coarse finite element mesh can be as large as 200 percent as in the case for the 10-foot slope height.

Obviously, a finer mesh would be needed to better assess the strain values near the slope face as well as near the toe. However, since the results for the horizontal tensile strains and maximum strains are not greatly affected by the finite element size, all the other parametric studies were performed using one row per construction layer.

A.6 EFFECTS OF INITIAL STRESSES

As discussed in Chapter 2, typical embankment construction involves placement of successive compacted layers of soil. Compaction is accomplished using thin soil lifts and heavy equipment that exert high pressures. These pressures cause an increase in vertical and horizontal pressures on the soil. After the compaction equipment moves away, the vertical pressures decrease to the overburden pressure, while some of the horizontal pressures may remain locked into the soil. Given the variability (layer thickness, soil type, and compaction equipment, etc.) involved in the construction of an embankment, the horizontal stresses after compaction cannot be easily assessed. Nonetheless, these stresses are very important since soil behavior is very much dependent on the soil stress state.

To examine the effects of the magnitude of initial horizontal stresses after compaction of each construction layer on the finite element computations, various values of the lateral pressure coefficient, k, were used. The lateral pressure coefficient, k, is defined as the ratio of horizontal to vertical pressure in a soil element. Values of 0.5 (minimal compaction), 0.75, and 1.0 were used for k. Higher values may be expected in the field, but higher values will result in less computed deformation. Typical results for horizontal strain profiles are presented in Figures A.20 through A.22. These results are from finite element computations for a 2:1 slope and different embankment heights (10, 20, and 30 feet). As expected, the results show that at the end of construction, the horizontal strains decrease with an increase in lateral stress (higher k value). The influence, however, appears to diminish with increase in embankment height, and in all cases, the variation is small for values of k between 0.5 and 1.0. A value of k equal to 0.75 was used for the remaining finite element analyses.

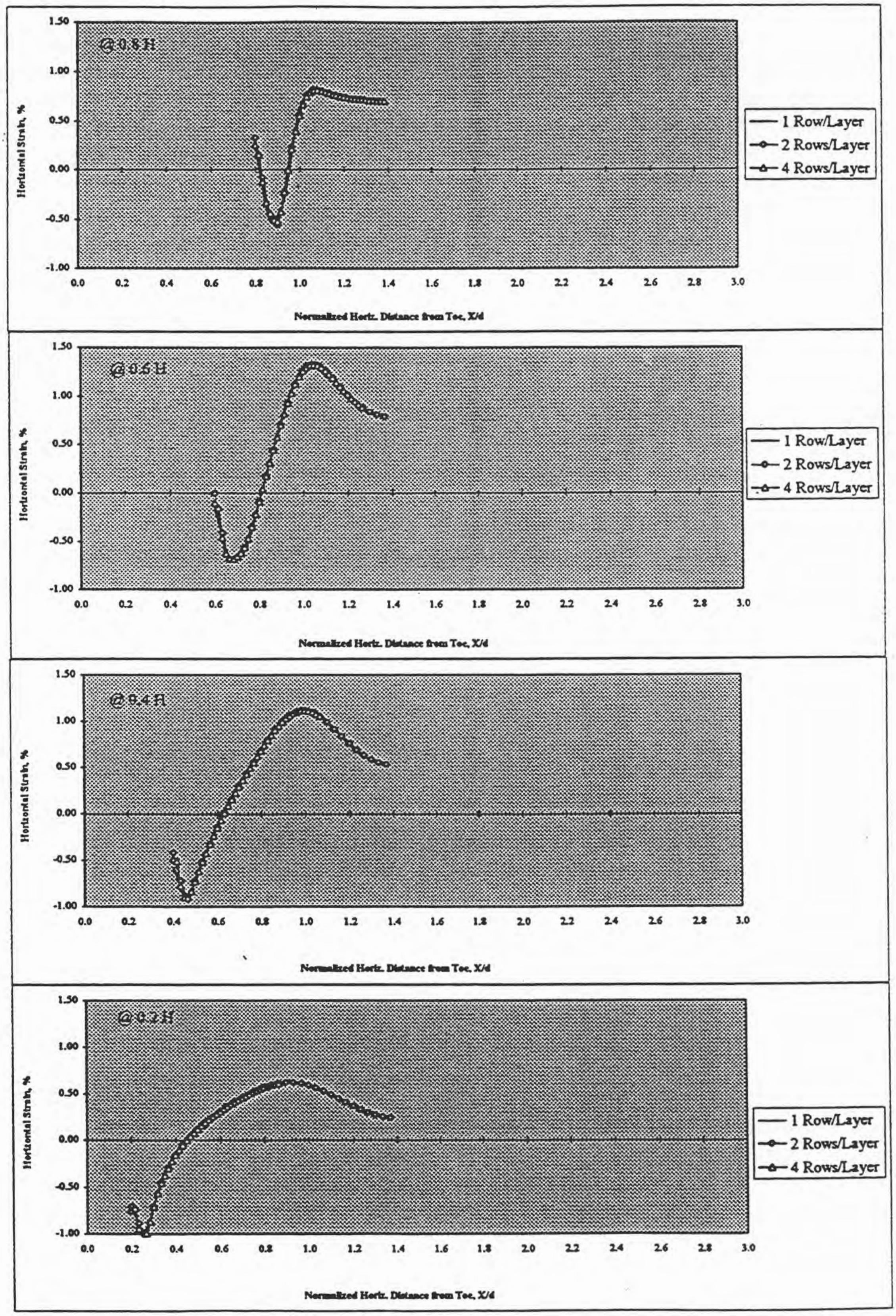

Figure A.17 Profiles of Horizontal Strains Showing Effects of Finite Element Size
for 2:1 Slope and a Slope Height of 30 Feet.

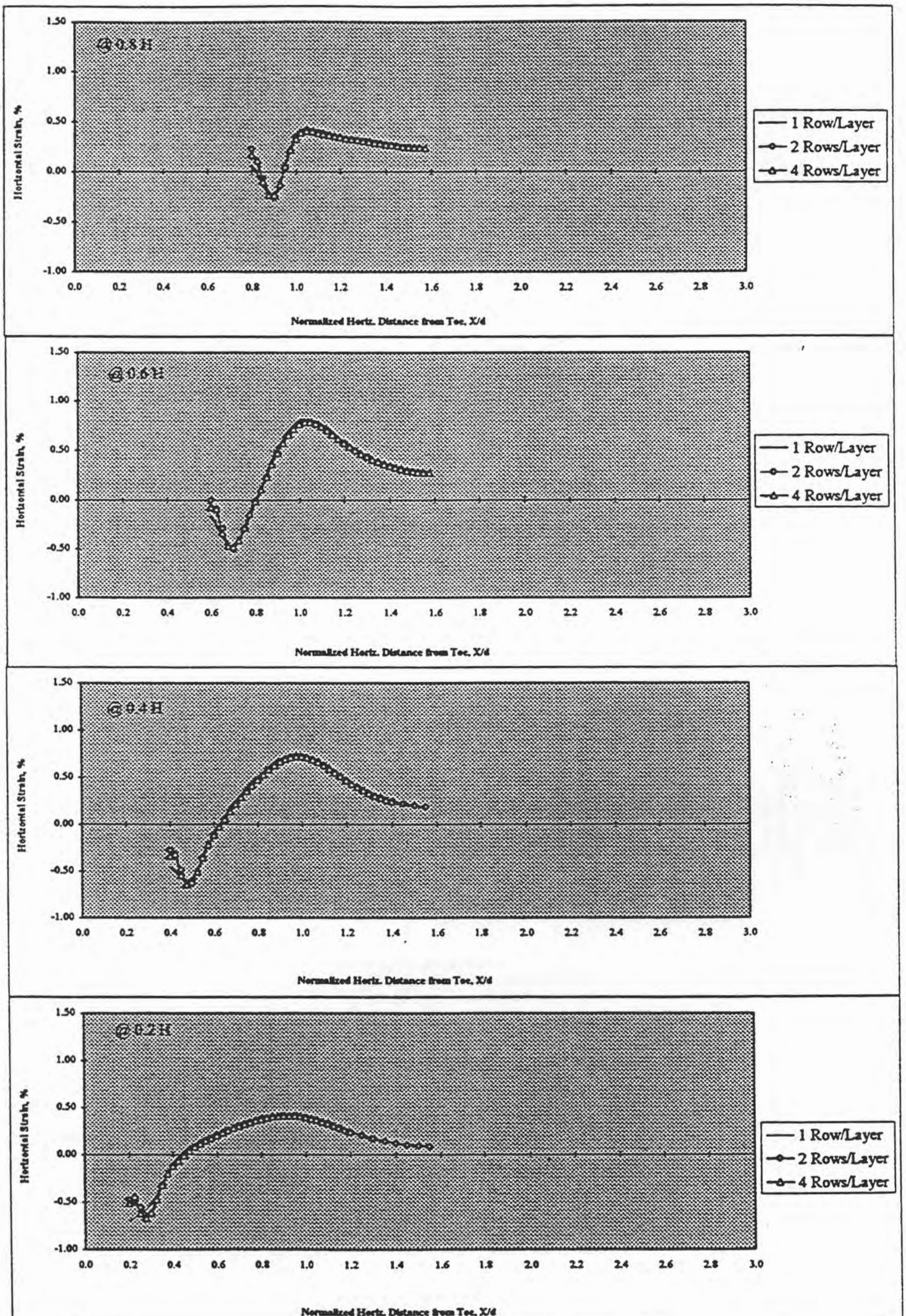

Figure A.18 Profiles of Horizontal Strains Showing Effects of Finite Element Size for 2:1 Slope and a Slope Height of 20 Feet.

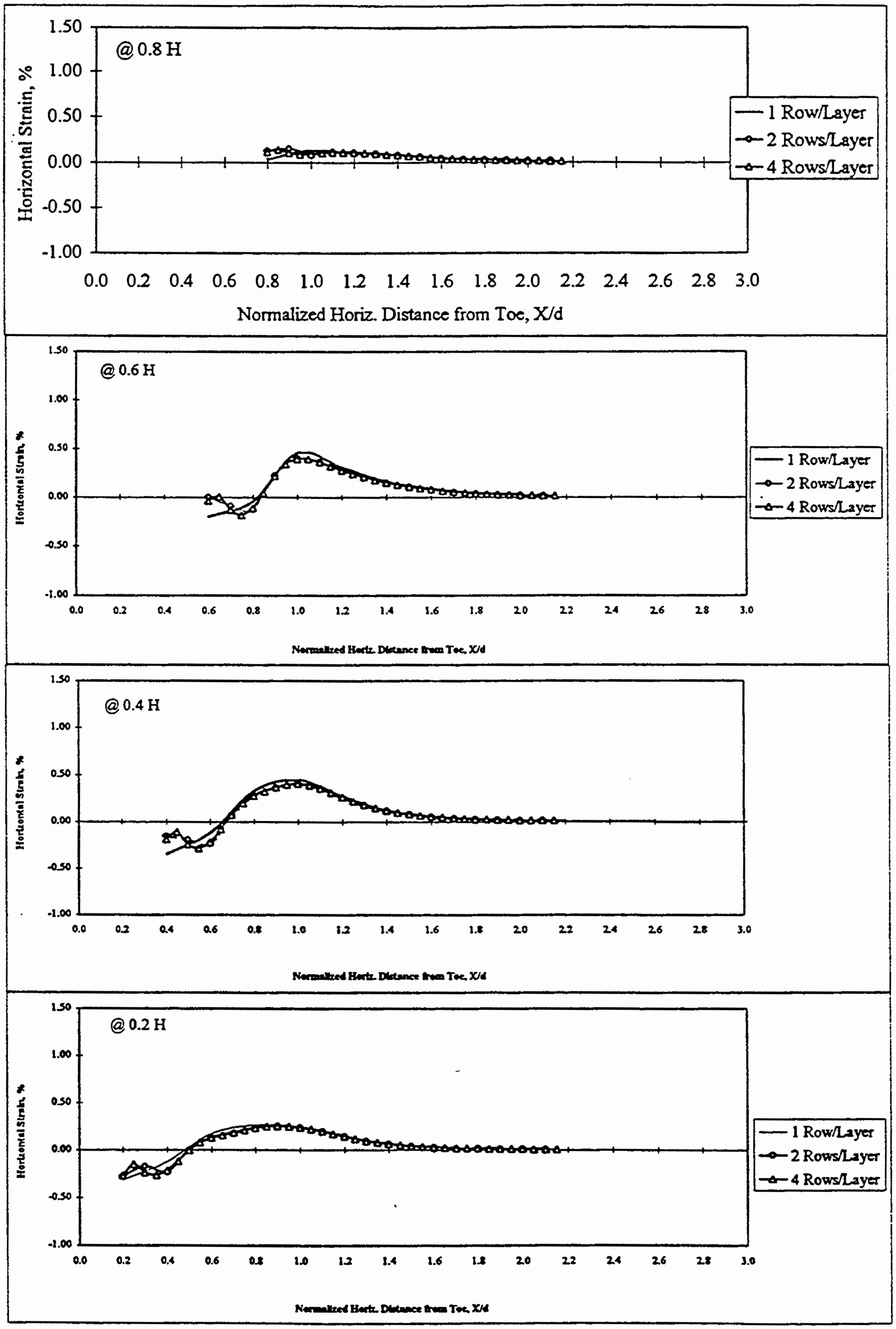

Figure A.19 Profiles of Horizontal Strains Showing Effects of Finite Element Size for 2:1 Slope and a Slope Height of 10 Feet.

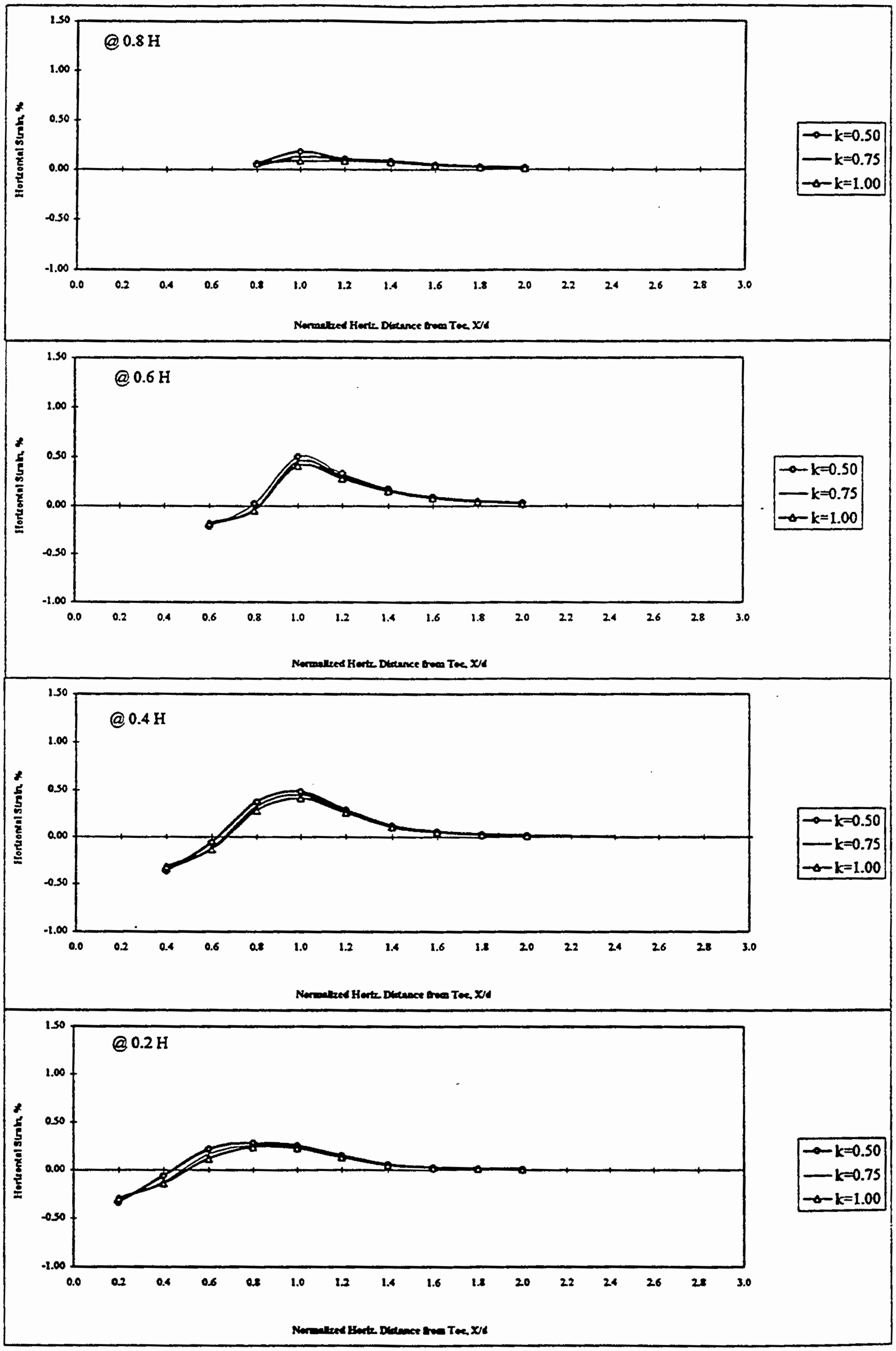

Figure A.20 Profiles of Horizontal Strains Showing Effects of Initial Stresses for 2:1 Slope and a Slope Height of 10 Feet.

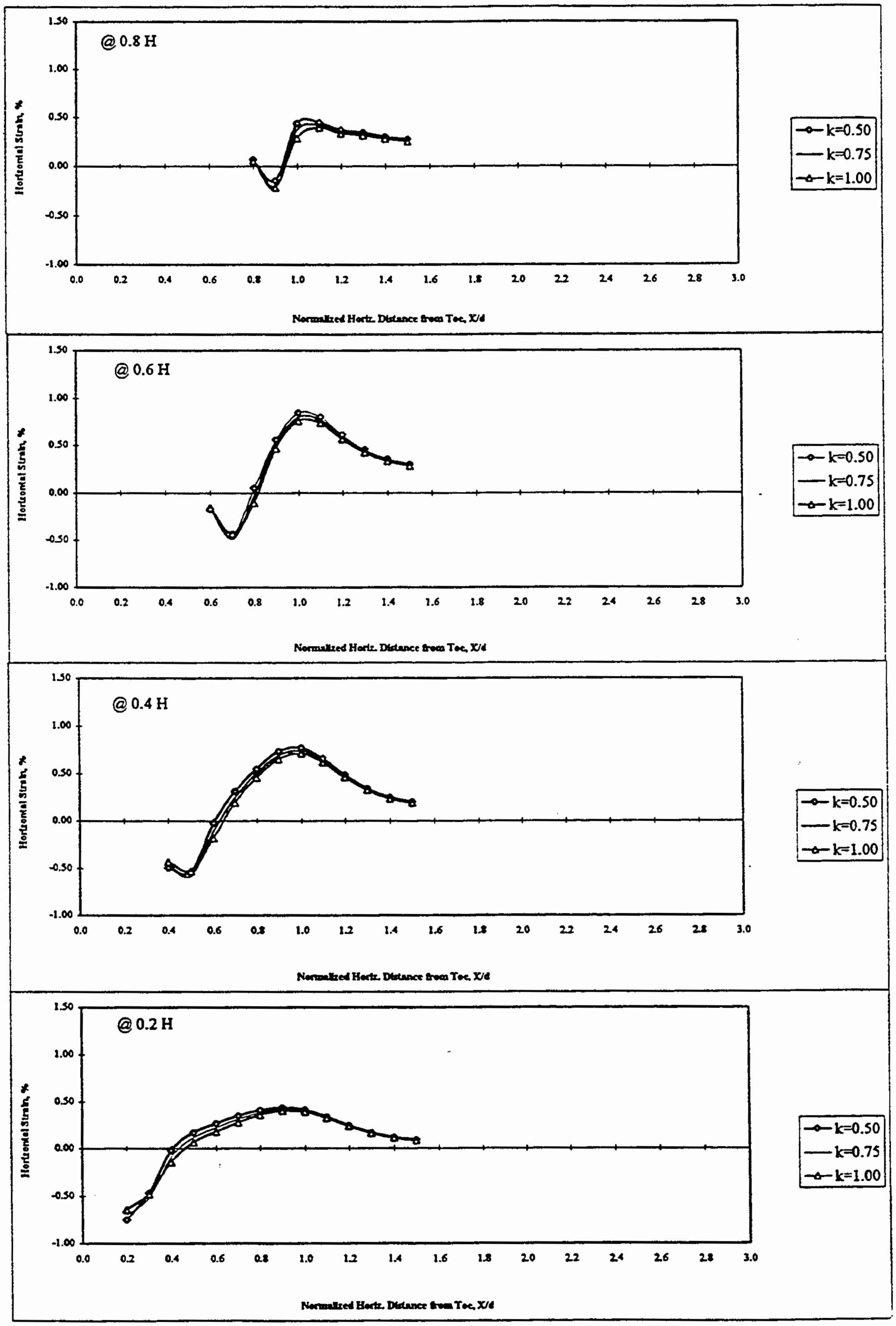

Figure A.21 Profiles of Horizontal Strains Showing Effects of Initial Stresses for 2:1 Slope and a Slope Height of 20 Feet.

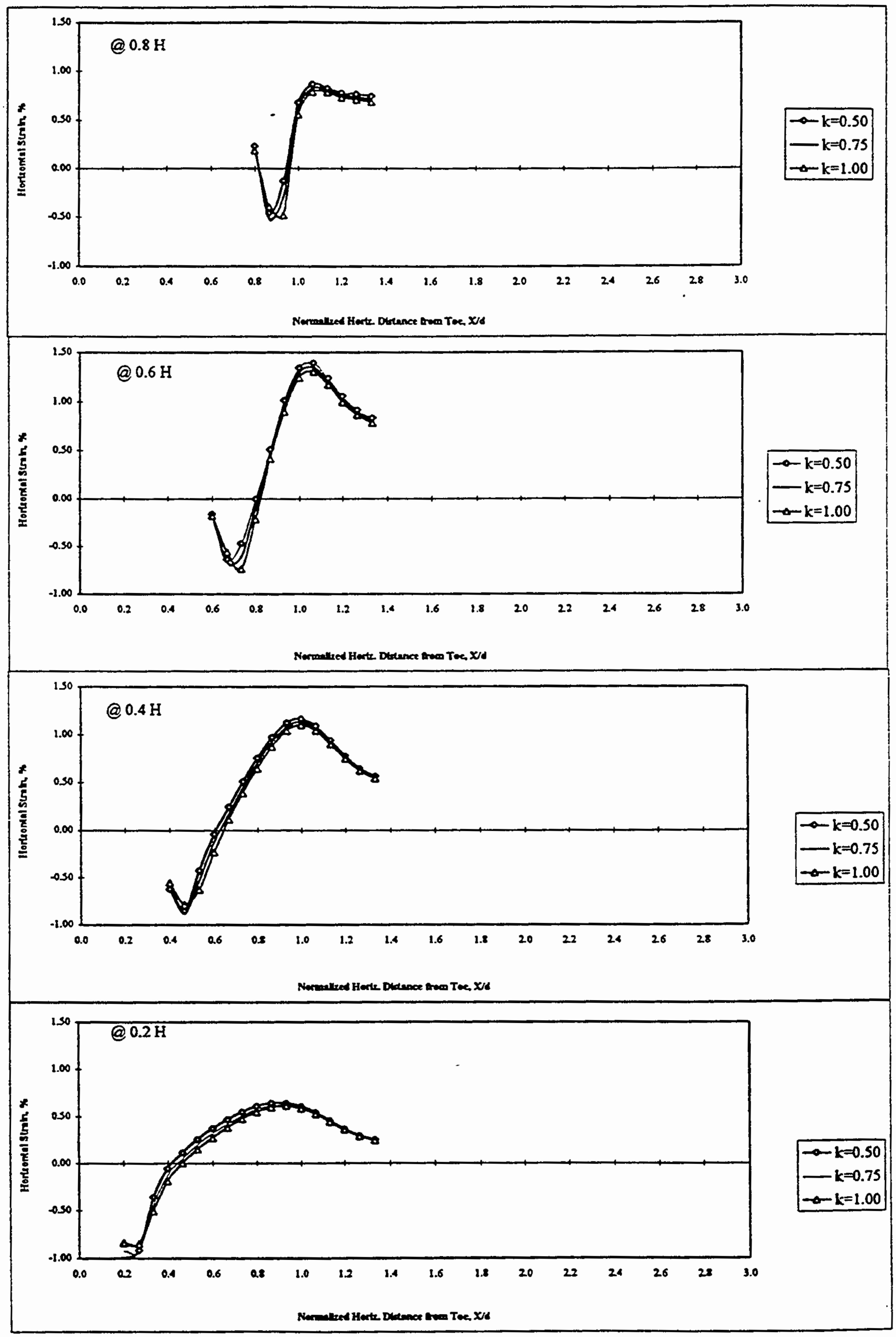

Figure A.22 Profiles of Horizontal Strains Showing Effects of Initial Stresses for 2:1 Slope and a Slope Height of 30 Feet.

A.7 EFFECTS OF VARIATIONS IN TOTAL UNIT WEIGHT

The total unit weight adopted for the Taylor Marl was 121 pcf. Two additional values (100 and 130 pcf) were used to investigate the influence of +10 to –20 percent variation in the density. Typical results are shown in Figures A.23 through A.25. The data shown in these figures are for a 2:1 slope and embankment heights of 10, 20, and 30 feet. The effect of unit weight becomes larger with higher embankments. The difference in maximum horizontal strains was found to be negligible for low slope height, but as large as 25 percent for higher embankments (Figure A.25).

A.8 SOLUTION SCHEME FOR NONLINEAR MODEL

Several computational methods may be used to account for the effects of nonlinear material properties of the soil. A method known as the "two-step-cycle" method has traditionally been used in finite element formulations involving geotechnical problems. When using this method the soil is assumed to behave linearly during each construction increment, with stiffness properties in each element defined initially on the basis of local stress state that existed at the beginning of the increment. However, after the application of each new lift, and the determination of the new stress state, the stiffness properties are re-evaluated based on either the new (at end of increment) stress state (Clough and Woodward, 1967) or an average stress of beginning and end of increment (Kulhawy, Duncan and Seed, 1969). This method has been used in the past owing to its simplicity and because it did not require excessive computer time, memory, or storage. Another method that can be used to account for the nonlinearity of the material is Newton's method. In this method the equilibrium equations are solved iteratively using new stiffness matrices corresponding to the new stresses computed for each iteration. Iteration continues for a specified number of iterations until a specified error tolerance is reached. Newton's method, although superior to all other methods, was almost never used in the past due to the excessive computer power it requires. However, today's computer technology makes it affordable to use such a method and, thus, Newton's method is used for all finite element computations. In this section the two methods are compared to assess the impact of using the simplified "two-step-cycle" method.

To examine the difference between the two methods, a series of finite element computations were made using both methods separately for a 2:1 slope and for different embankment heights (10, 20, and 30 feet). The results are shown in Figures A.26 through A.28. The difference, which can be as large as 35 percent as in the 30-ft height case, seems to increase as the slope height increases. This indicates that the 2-step method deviates further and further from the actual stress-strain curve as we increase the load in the embankment. The 2-step method seems to also overestimate the strains near the face of the slope and underestimate them further inside the slope. In all cases, Newton's method gives a higher peak strain.

Newton's method was used for the remaining of finite element analyses.

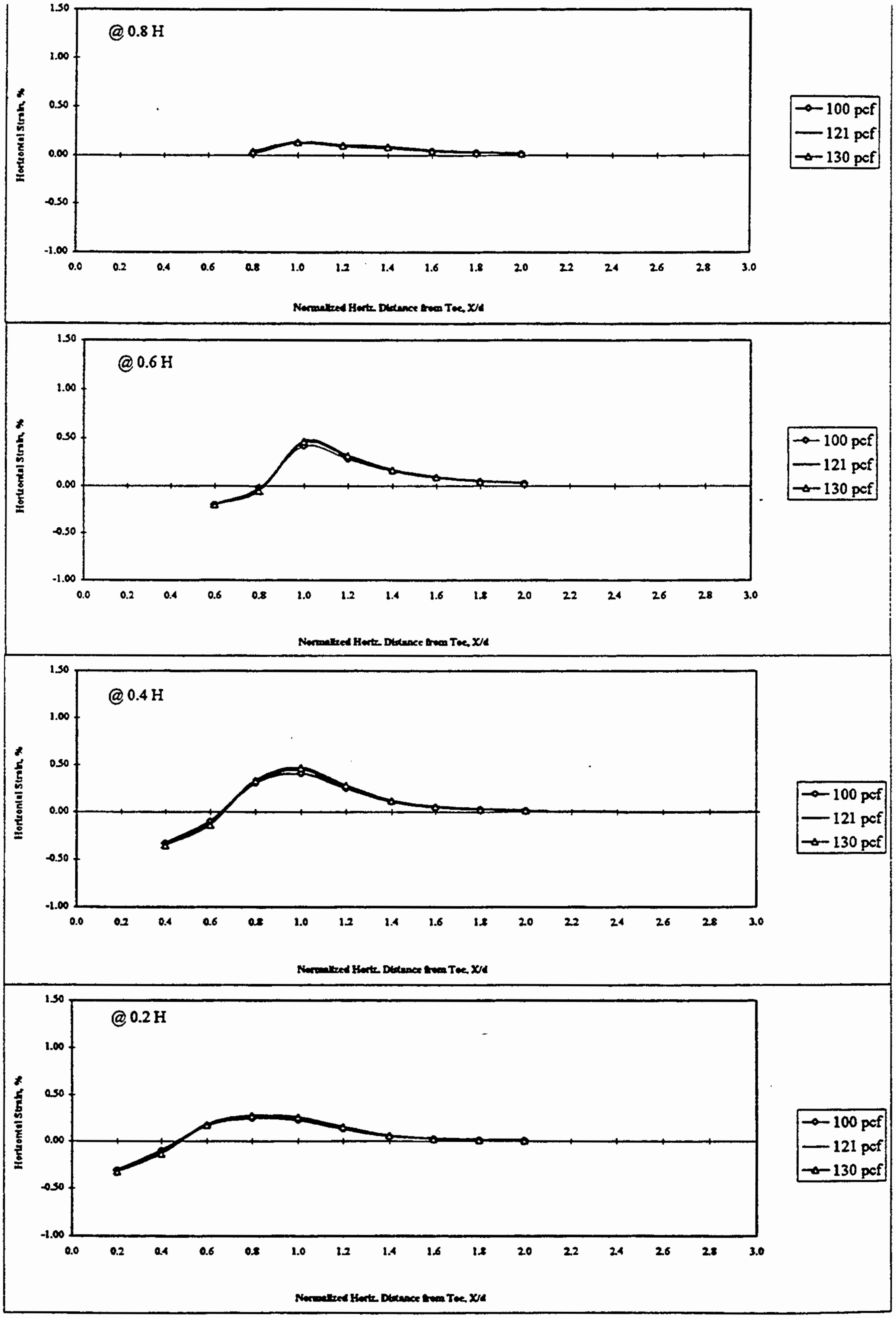

Figure A.23 Profiles of Horizontal Strains Showing Effects of Total Unit Weight Variations for 2:1 Slope and a Slope Height of 10 Feet.

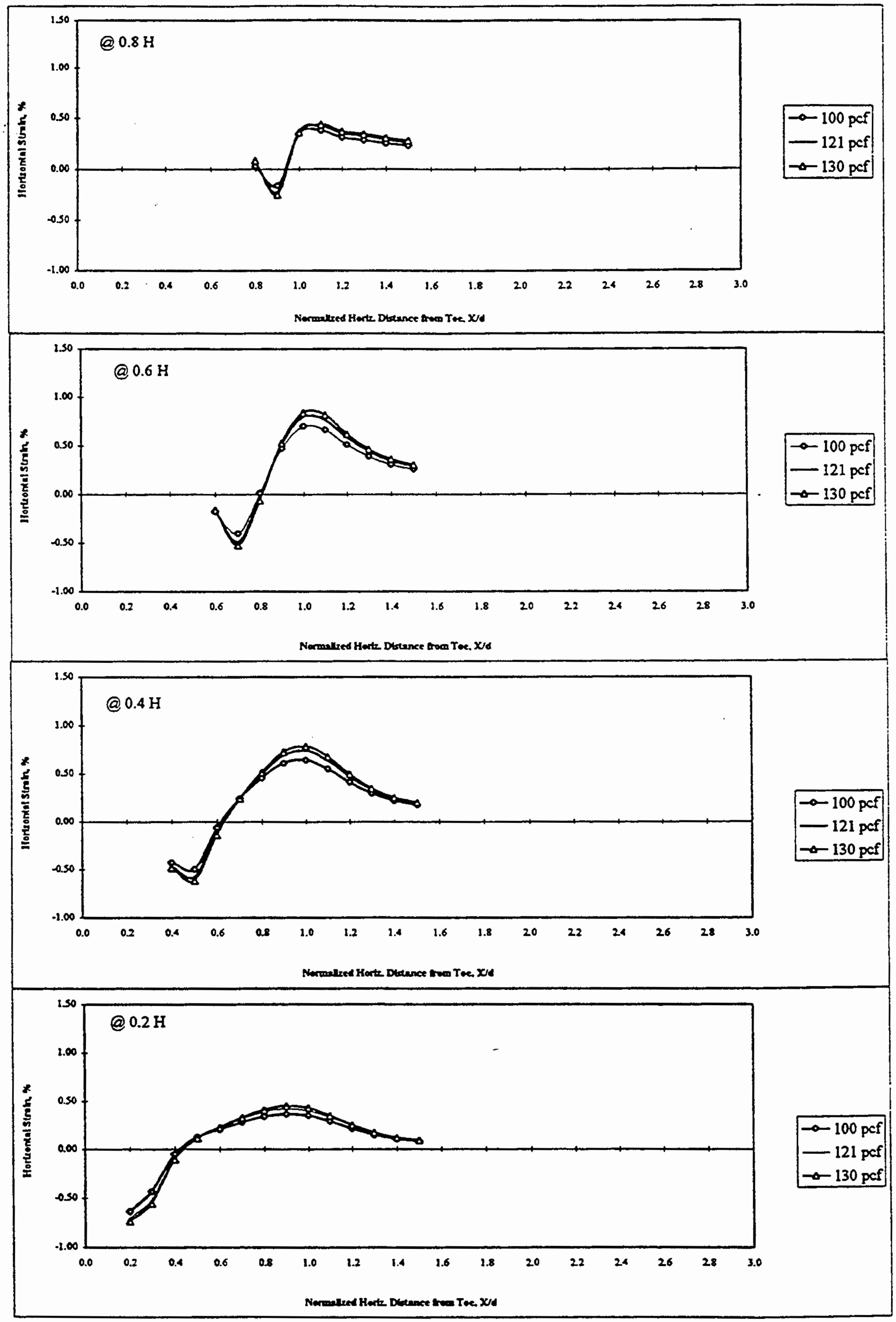

Figure A.24 Profiles of Horizontal Strains Showing Effects of Total Unit Weight Variations for 2:1 Slope and a Slope Height of 20 Feet.

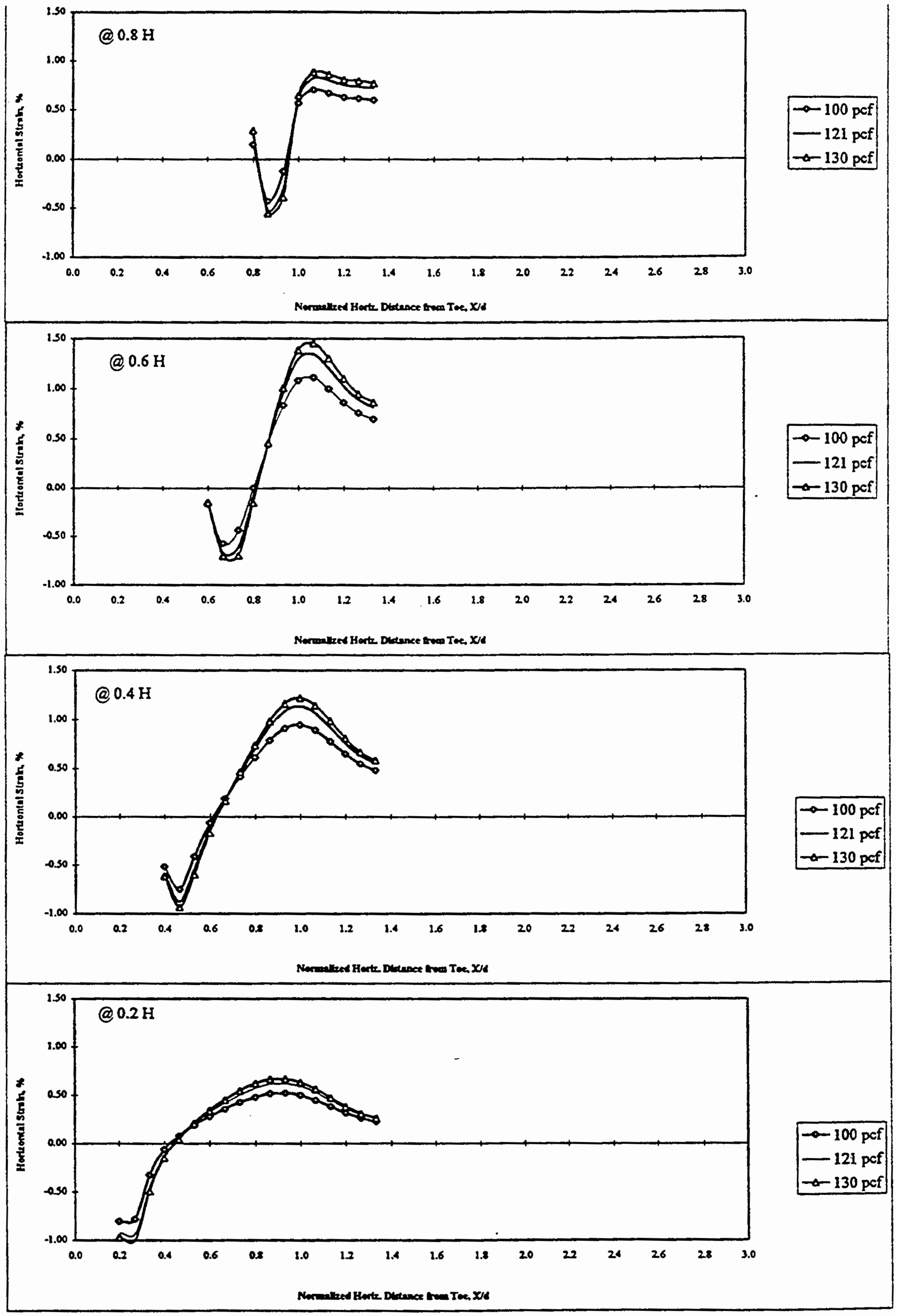

Figure A.25 Profiles of Horizontal Strains Showing Effects of Total Unit Weight Variations for 2:1 Slope and a Slope Height of 30 Feet.

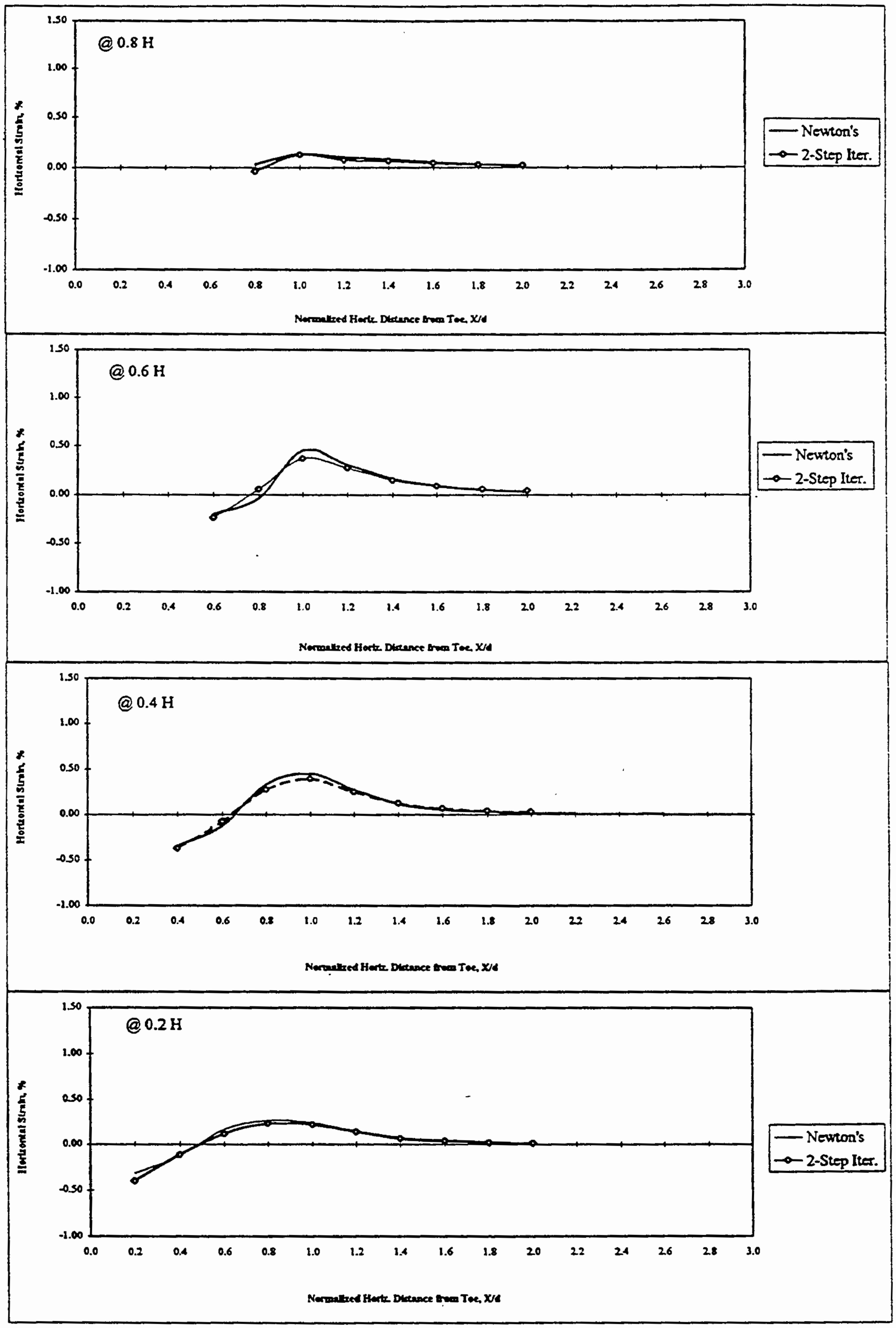

Figure A.26 Profiles of Horizontal Strains Showing Effects of Nonlinear Solution Scheme for 2:1 Slope and a Slope Height of 10 Feet.

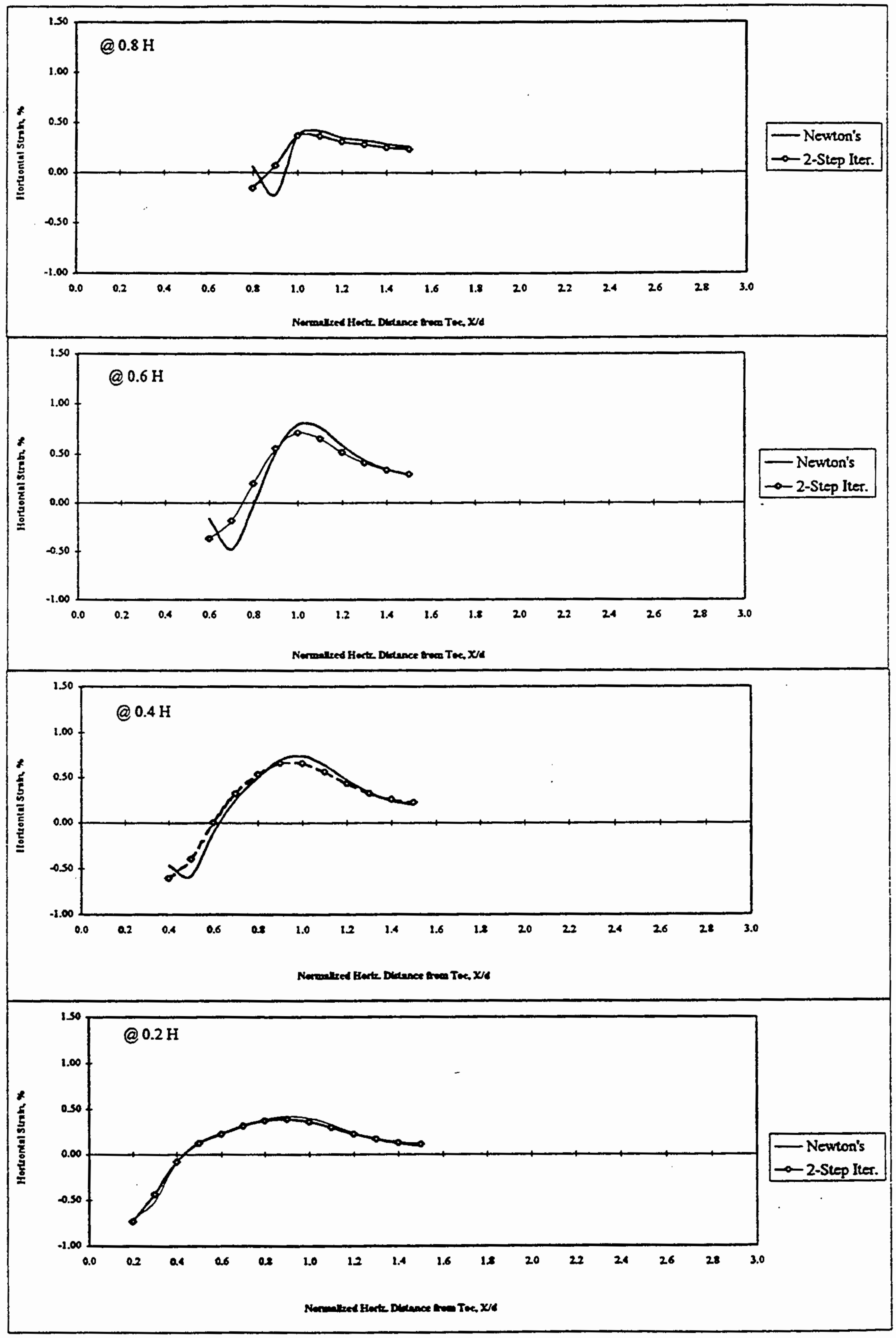

Figure A.27 Profiles of Horizontal Strains Showing Effects of Nonlinear Solution Scheme for 2:1 Slope and a Slope Height of 20 Feet.

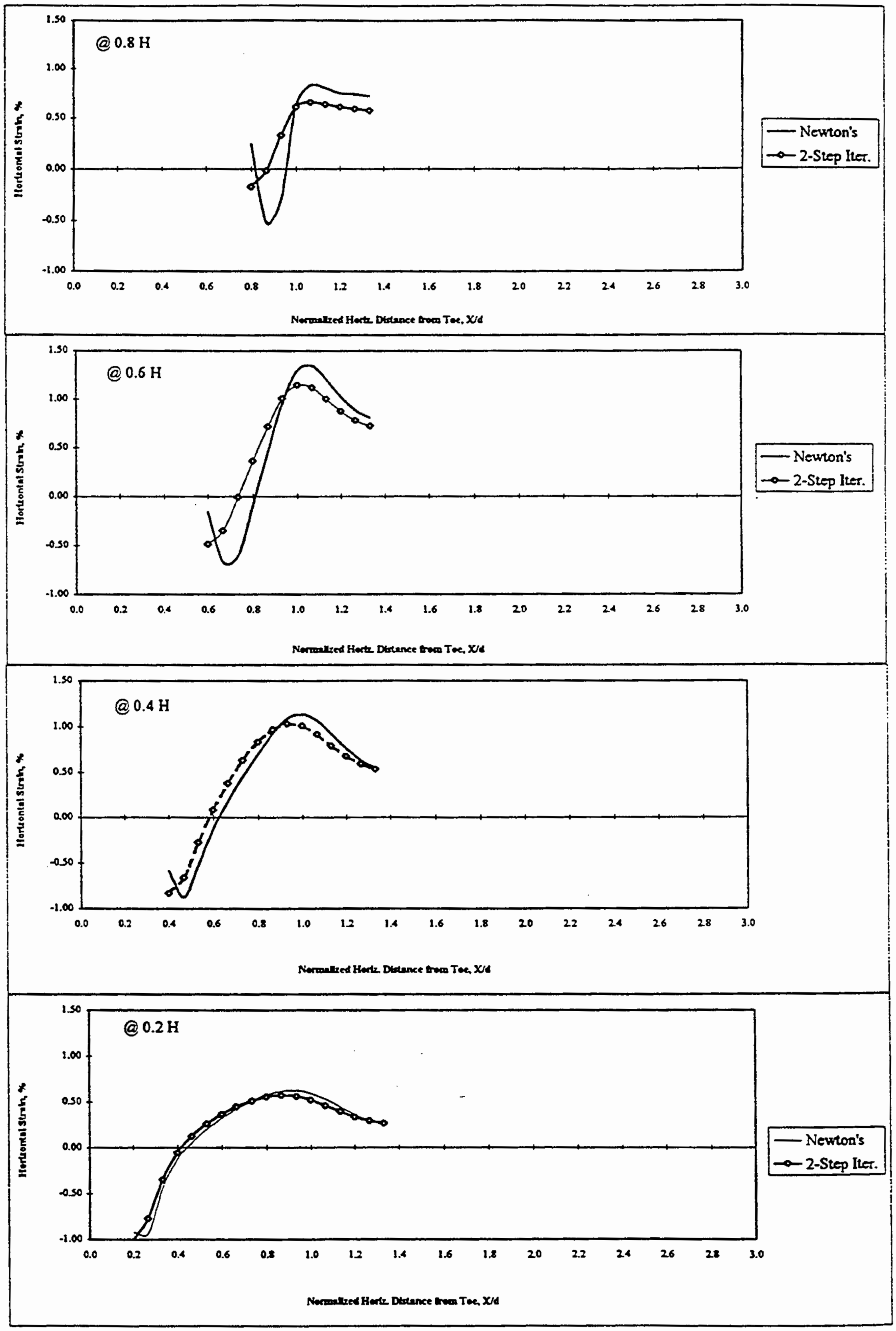

Figure A.28 Profiles of Horizontal Strains Showing Effects of Nonlinear Solution Scheme for 2:1 Slope and a Slope Height of 30 Feet.

APPENDIX B
APPENDIX B. REINFORCED SOIL SLOPE DESIGN USING FIPPIN AND WRIGHT'S CHARTS

APPENDIX B. REINFORCED SOIL SLOPE DESIGN USING FIPPIN AND WRIGHT'S CHARTS

In this appendix, the procedures followed for reinforcement design of the hypothetical embankment in Chapter 5 are summarized. Full procedures and referenced figures are provided in a separate companion report (Fippin and Wright, 1997).

B.1 EMBANKMENT GEOMETRY

Side slope (2:1),	ss= 2
Slope height,	H= 30 feet
Lift thickness,	S_{min}= 6 inches

B.2 SOIL MATERIAL PROPERTIES

Soil type	Taylor Marl, CH
Total unit weight,	γ= 121 pcf
Friction angle,	ϕ= 25 degrees
Cohesion,	c= 0

B.3 ASSUMPTIONS AND OTHER IMPORTANT DESIGN PARAMETERS

Embankment foundation: Rigid
Pore pressure coefficient, r_u= 0.5 (assuming a water table at the slope surface)
Factor of Safety, F= 1.5

B.4 REINFORCEMENT DESIGN

B.1.1 Calculate mobilized soil friction angle

Mobilized soil friction angle, ϕ_m= atan (tan ϕ / F)= 17.3 degrees

B.1.2 Determine geogrid force coefficient

Using: Slope angle, β= atan (1 / ss)= 26.6 degrees

and ϕ_m = 17.3 degrees

Force coefficient, k, is obtained from Figure 4.8 in the companion report (Fippin and Wright, 1997) which is provided again in this appendix as Figure B.1. This chart

(Figure B.1) was selected because it was developed for slopes on foundations that are much stronger than the slope material and for a pore pressure coefficient, r_u, of 0.5, conditions that correspond to the case examined in this appendix.

Thus, Force coefficient, k_{Req} = 0.4

B.1.3 Determine total horizontal geogrid force

Total horizontal geogrid force, $T_t = \frac{1}{2} \gamma H^2 K_{Req} = 21{,}780$ lb/ft

B.1.4 Determine minimum required number of geogrids

Using HX1500HS Tensar geogrid with allowable tensile force, T_a, of 2762 lb/ft:
Minimum number of geogrids, $N_{min} = T_t / T_a = 7.89$ (say 8)

B.1.5 Determine spacing

The vertical spacing of the various reinforcement layers is determined using Schmertmann's equation,

$$s_v = \frac{T_a}{K_{Req}\, \gamma z}$$

(Equation 4.3 in the companion report)

where z is the depth below the crest of the slope. The following is a list of different depths below crest of slope and corresponding elevations (Elev $_{at\ toe}$= 0) and vertical spacing:

z	Elev.	Sv
27.5	2.5	2.08
22.5	7.5	2.54
17.5	12.5	3.26
12.5	17.5	4.57
7.5	22.5	7.61
2.5	27.5	22.83

Based on the spacing obtained above and the need to conform to boundaries between lifts (6-in. lifts), the following general spacing was adopted:

Elev.	Spacing
0.0 – 10.0 ft	2.0 ft
10.0 – 20.0 ft	4.0 ft
20.0 – 30.0 ft	6.0 ft

B.1.6 Determine required geogrid length

Using:	Slope angle,	β= atan (1 / ss)= 26.6 degrees
	and	ϕ_m = 17.3 degrees

Length ratios for global stability (L_{gs}/H) and for sliding stability (L_{ds}/H) are obtained from Figures B.2 and B.3, respectively. Figures B.2 and B.3 are the same as Figures 4.16 and 4.20 in the companion report (Fippin and Wright, 1997) and are provided again in this appendix for convenience. These figures were developed for the same slope conditions described above (rigid foundations and a pore pressure coefficient of 0.5).

Global stability length ratio,	L_{gs} / H = 2.6
Sliding stability length ratio,	L_{ds} / H = 1.35
Largest length ratio,	L / H = 2.6
Minimum reinforcement length,	L = 2.6 H = 72 ft.

B.1.7 Final layout

The final layout of the reinforcement is shown in Figure 5.6. The height above the toe and length of each layer of reinforcement are presented in Table 5.4.

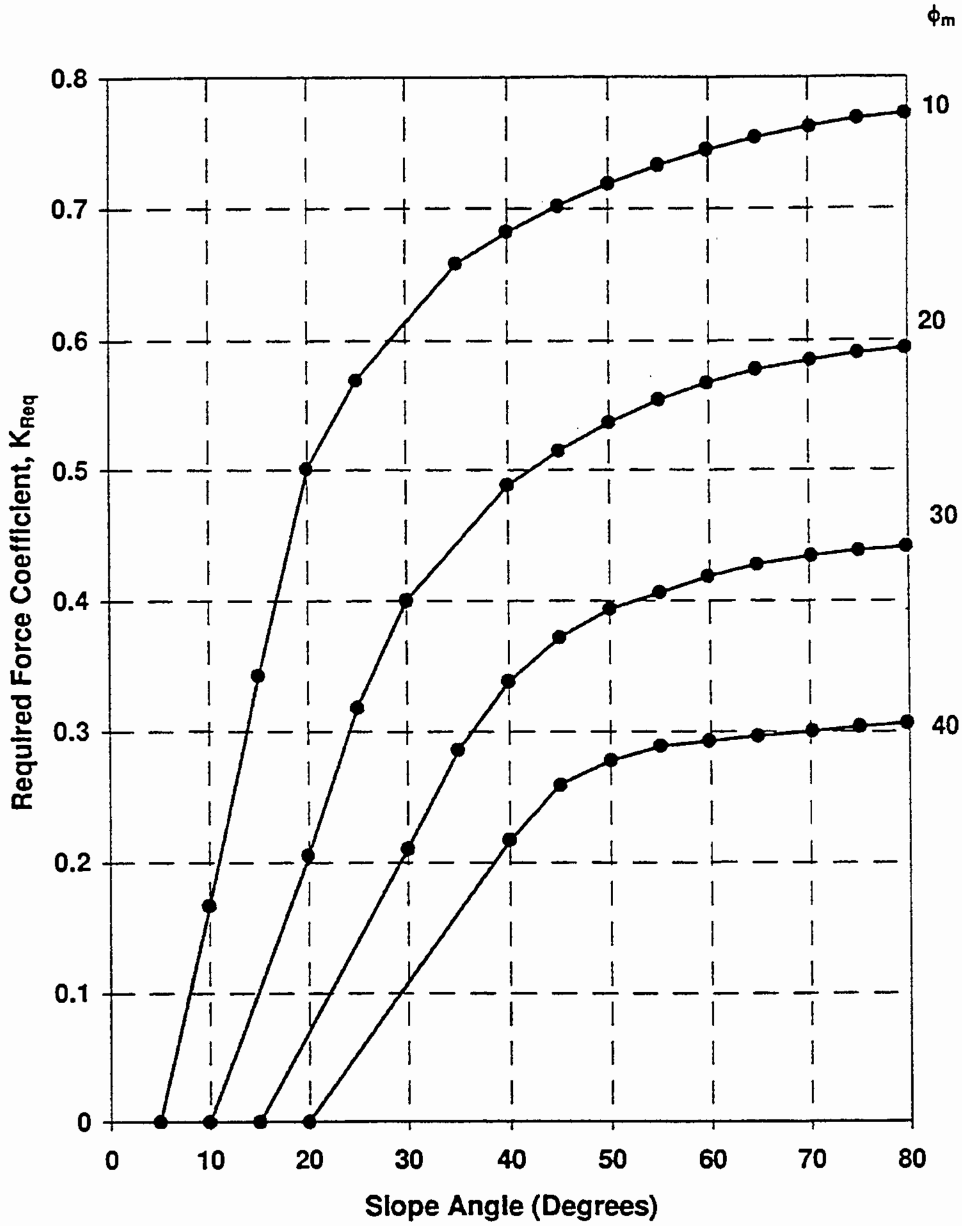

Figure B.1 Chart for Required Reinforcement Force Coefficient (K_{Req}) for Slopes on much Stronger Foundations - $r_u = 0.5$

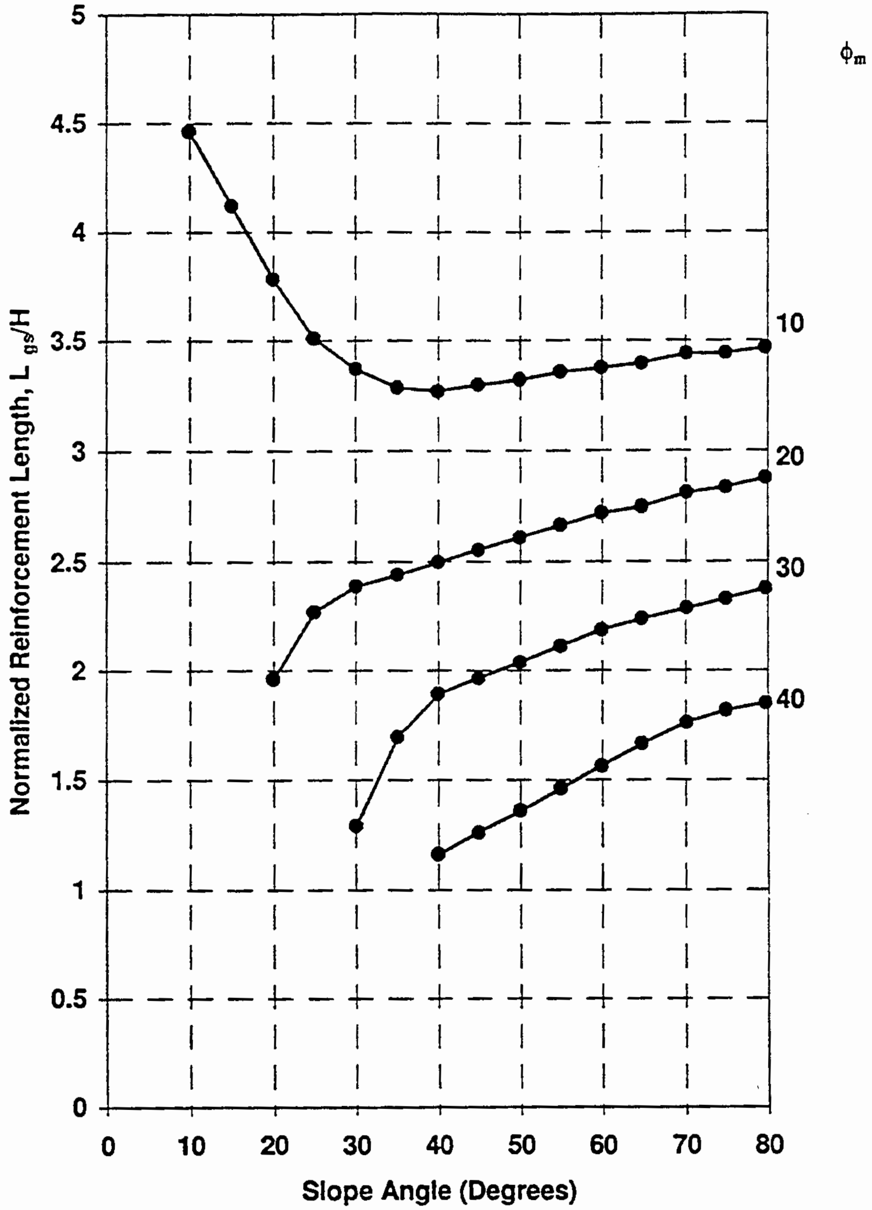

Figure B.2 Minimum Length of Reinforcement for Global Stability of Slopes on Much Stronger Foundations $-r_u = 0.5$

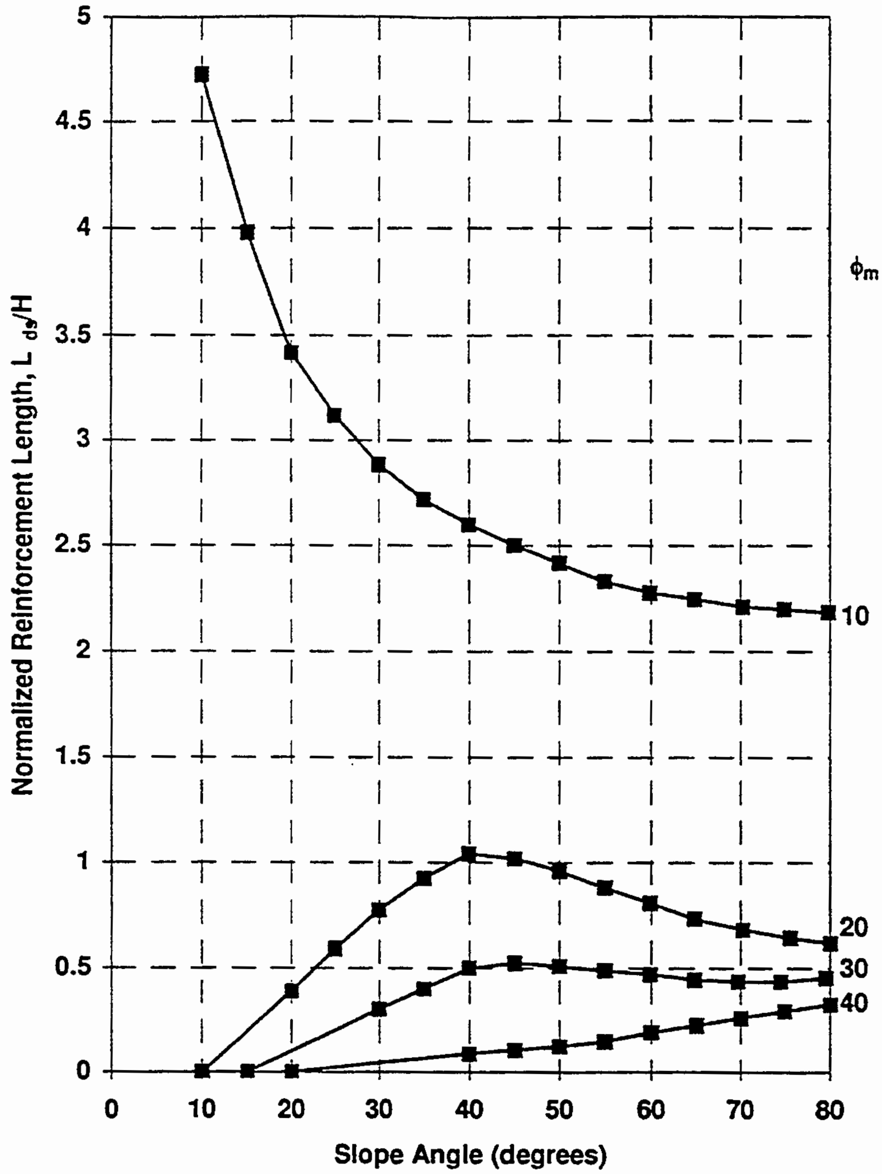

Figure B.3 Minimum Length of Reinforcement for Direct Sliding Stability of Slopes on Much Stronger Foundations – $r_u = 0.5$

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