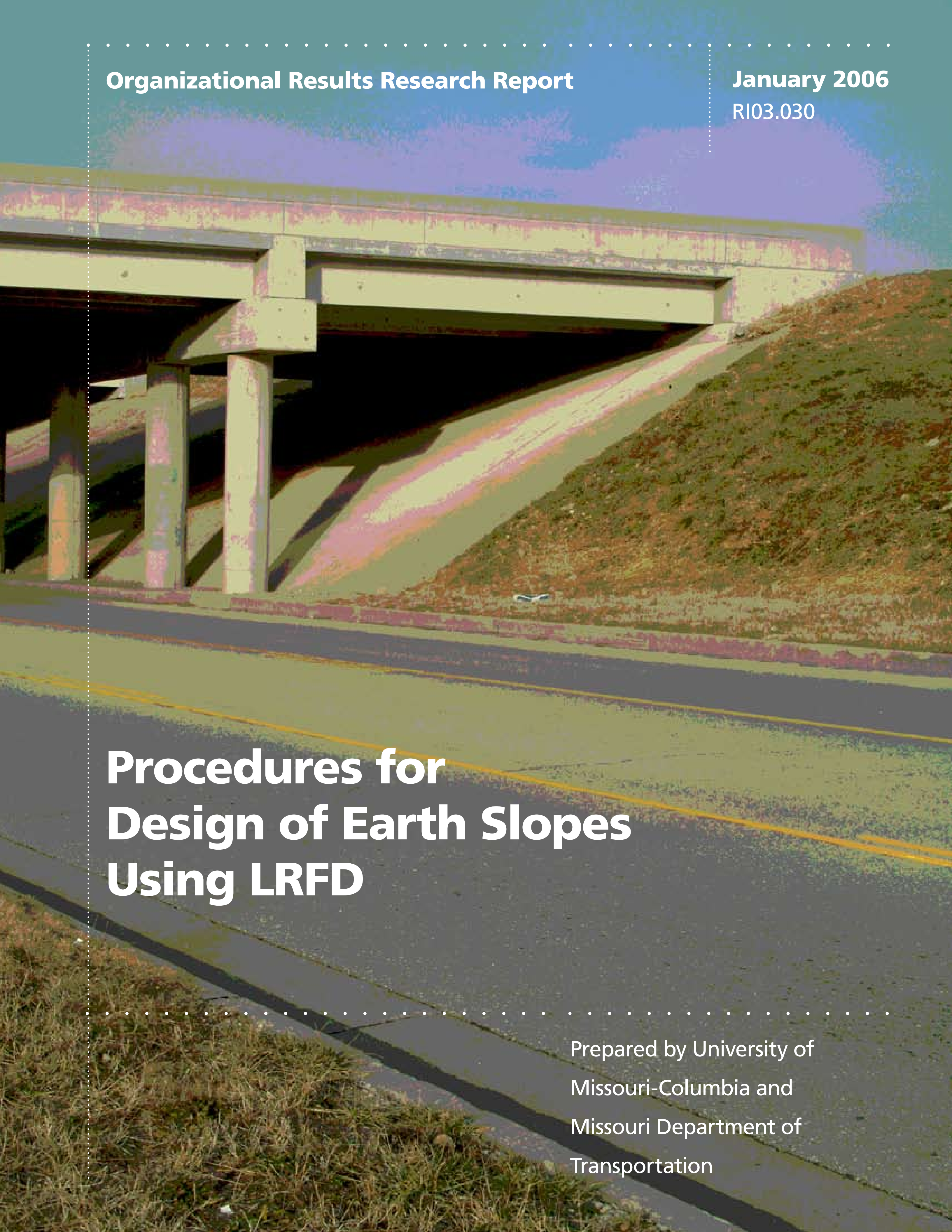
Organizational Results Research Report

January 2006
RI03.030



# Procedures for Design of Earth Slopes Using LRFD

Prepared by University of Missouri-Columbia and Missouri Department of Transportation

# TECHNICAL REPORT DOCUMENTATION PAGE

<table>
<tr><td>1. Report No. OR 06-010</td><td>2. Government Accession No.</td><td colspan="2">3. Recipient's Catalog No.</td></tr>
<tr><td colspan="2" rowspan="2">4. Title and Subtitle<br>Procedures for Design of Earth Slopes Using LRFD</td><td colspan="2">5. Report Date<br>December 2005</td></tr>
<tr><td colspan="2">6. Performing Organization Code</td></tr>
<tr><td colspan="2">7. Author(s)<br>J. Erik Loehr, Cynthia A. Finley, and Daniel Huaco</td><td colspan="2">8. Performing Organization Report No.</td></tr>
<tr><td colspan="2" rowspan="2">9. Performing Organization Name and Address<br>University of Missouri – Columbia<br>310 Jesse Hall<br>Columbia, Missouri 65211</td><td colspan="2">10. Work Unit No.</td></tr>
<tr><td colspan="2">11. Contract or Grant No.<br>RI03-030</td></tr>
<tr><td colspan="2" rowspan="2">12. Sponsoring Agency Name and Address<br>Missouri Department of Transportation<br>Research, Development and Technology Division<br>P. O. Box 270-Jefferson City, MO 65102</td><td colspan="2">13. Type of Report and Period Covered<br>Final Report</td></tr>
<tr><td colspan="2">14. Sponsoring Agency Code</td></tr>
<tr><td colspan="4">15. Supplementary Notes<br>The investigation was conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration.</td></tr>
</table>

16. Abstract


This report describes a proposed procedure and process for implementation of LFRD for slope stability analysis applications, including evaluation of overall stability of earth retaining structures. Two sets of load and resistance factors are present. The first is based on establishing load and resistance factors to produce designs that are similar to existing methods. These factors are a necessary first step to ensure prudent implementation of the technique while allowing designers to become familiar with the technique prior to full implementation. The second set of load and resistance factors, established through "probabilistic calibrations", will produce the true benefits of the LRFD approach so that consistent levels of reliability are achieved for all sites.

The principal conclusions from the project include:

1. The procedure proposed for performing slope stability analysis using LRFD concepts is feasible and provides significant potential for producing consistent levels of safety for slopes located on a broad range of sites and significant cost savings.
2. Load and resistance factors established by matching to current ASD procedures serve as an effective preliminary step for adoption of LRFD for slope stability analysis. However, use of these load and resistance factors will limit the benefits that can be achieved through adopting LRFD.
3. Load and resistance factors established through probabilistic calibrations demonstrate that appropriate resistance factors are highly sensitive to the level of uncertainty present at a given site. As such, full realization of the benefits of LRFD requires that probabilistically calibrated load and resistance factors be eventually adopted.
4. Several impediments to implementation of probabilistically calibrated load and resistance factors exist that prevent current implementation of these factors. However, these impediments can be realistically addressed within a reasonable period of time.

The primary recommendation from this work is to adopt a staged implementation plan to implement the proposed LRFD procedure for design of earth slopes. The plan consists of the following steps:

1. Begin implementation of general LRFD procedure using load and resistance factors established by matching to current ASD procedures as provided in the report.
2. Simultaneously engage in activities necessary to transition to load and resistance factors established through probabilistic calibrations, also provided in this report. These activities should minimally include:
   a. Development of appropriate procedures for quantifying the uncertainty in the respective input parameters from information currently obtained in routine site investigations;
   b. Evaluation of current bias in key parameters following current site investigation and testing procedures; and
   c. Evaluation/verification of probabilistic load and resistance factors for a series of actual project cases, especially those involving complex stratigraphy and pore pressure conditions.
3. As these activities are completed, adopt revised load and resistance factors based on probabilistic calibrations.

The estimated period of time to complete this plan is two years. Additional recommendations for future work to expand or improve upon the proposed procedure are also provided in the report.


<table>
<tr><td colspan="2">17. Key Words<br>slope stability analysis, load and resistance factor design, reliability-based design</td><td colspan="2">18. Distribution Statement<br>No restrictions. This document is available to the public through National Technical Information Center, Springfield, Virginia 22161</td></tr>
<tr><td>19. Security Classification (of this report)<br>Unclassified</td><td>20. Security Classification (of this page)<br>Unclassified</td><td>21. No. of Pages<br>80</td><td>22. Price</td></tr>
</table>

Form DOT F 1700.7 (06/98)

Final Report for RI03-030

# Procedures for Design of Earth Slopes Using LRFD

December 19, 2005

Dr. J. Erik Loehr

Dr. Cynthia A. Finley

Daniel R. Huaco

Civil & Environmental Engineering

University of Missouri – Columbia

Columbia, Missouri, 65211-2200

Phone 573-882-6380

Fax 573-882-4784

eloehr@missouri.edu

http://www.geotech.missouri.edu/

# Executive Summary

The American Association of State Highway and Transportation Officials (AASHTO) is currently working to progressively convert from traditional Allowable Stress Design (ASD) methods to more modern Load and Resistance Factor Design (LRFD) methods. This conversion began with the adoption of the *LRFD Highway Bridge Design Specifications* in 1994 and has progressed since then to incorporate more applications, including many geotechnical applications such as shallow and deep foundations, earth retaining structures, and culverts. As a part of this conversion, State Departments of Transportation, including the Missouri Department of Transportation (MoDOT), are mandated to convert to LRFD methods by October 1, 2007. In converting to LRFD, state departments of transportation have the latitude to adopt AASHTO standard LRFD procedures or to develop their own methods and specific load and resistance factors appropriate for conditions and procedures utilized by the respective organizations.

The principal motivation behind transitioning to LRFD is to produce consistent levels of reliability (or safety) across a broad range of design cases, regardless of the level of uncertainty associated with the loads and resistances for the particular case. The fundamental promise of transitioning to LRFD is that by applying appropriate conservatism (and thus funding) where needed but not where not needed, substantial cost savings can be realized agency wide. This promise simply cannot be realistically achieved using existing design approaches. While estimates of expected cost savings are speculative at this time, savings of millions of dollars per year are easily achievable if effective implementations of LRFD are adopted. LRFD also offers the potential for continuously improving the accuracy and efficiency of designs over time without requiring wholesale changes to design procedures.

A significant amount of research has been performed in recent years to develop LRFD methods, including specific load and resistance factors for various aspects of design, and to facilitate conversion to LRFD from ASD procedures. The results of this work have been published and are widely available for use by states developing their own LRFD procedures. However, one aspect of the existing body of work that is severely lacking is design of earth slopes, including consideration of the overall stability of earth retaining structures. The work described in this report addresses this deficiency by providing MoDOT with guidance and procedures for reliable and cost effective design of earth slopes and retaining structures using the LRFD approach.

This report describes a proposed procedure and process for implementation of LFRD for slope stability analysis applications, including evaluation of overall stability of earth retaining structures. Two sets of load and resistance factors are presented. The first is based on establishing load and resistance factors to produce designs that are similar to existing methods. These factors are a necessary first step to ensure prudent implementation of the technique while allowing designers to become familiar with the technique prior to full implementation. The second set of load and resistance factors, established through "probabilistic calibrations", will produce the true benefits of the LRFD approach so that consistent levels of reliability are achieved for all sites.

The primary recommendation from this work is to adopt a staged implementation plan to implement the proposed LRFD procedure for design of earth slopes. The plan consists of the following steps:

1. Begin implementation of general LRFD procedure using load and resistance factors established by matching to current ASD procedures as provided in the report.
2. Simultaneously engage in activities necessary to transition to load and resistance factors established through probabilistic calibrations, also provided in this report. These activities should minimally include:
    a. Development of appropriate procedures for quantifying the uncertainty in the respective input parameters from information currently obtained in routine site investigations;
    b. Evaluation of current bias in key parameters following current site investigation and testing procedures; and
    c. Evaluation/verification of probabilistic load and resistance factors for a series of actual project cases, especially those involving complex stratigraphy and pore pressure conditions.
3. As these activities are completed, adopt revised load and resistance factors based on probabilistic calibrations.

The estimated period of time to complete this plan is two years. Additional recommendations for future work to expand or improve upon the proposed procedure are also provided in the report.

# Table of Contents

# List of Figures

# List of Tables

# 1. Introduction

The American Association of State Highway and Transportation Officials (AASHTO) is currently working to progressively convert from traditional Allowable Stress Design (ASD) methods to more modern Load and Resistance Factor Design (LRFD) methods. This conversion began with the adoption of the *LRFD Highway Bridge Design Specifications* in 1994 (AASHTO, 1994) )and has progressed since then to incorporate more applications, including many geotechnical applications such as shallow and deep foundations, earth retaining structures, and culverts. As a part of this conversion, State Departments of Transportation, including the Missouri Department of Transportation (MoDOT), are mandated to convert to LRFD methods by October 1, 2007. In converting to LRFD, state departments of transportation have the latitude to adopt AASHTO standard LRFD procedures or to develop their own LRFD methods and specific load and resistance factors appropriate for conditions and procedures utilized by the respective organizations.

A significant amount of research has been performed in recent years to develop LRFD methods, including specific load and resistance factors for various aspects of design, and to facilitate conversion to LRFD from ASD procedures. The results of this work have been published and are widely available for use by states developing their own LRFD procedures (e.g. FHWA, 2001; Paikowsky, 2004). However, one aspect of the existing body of work that is severely lacking is design of earth slopes, including consideration of the overall stability of earth retaining structures. The work described in this report addresses this deficiency by providing MoDOT with guidance and procedures for reliable and cost effective design of earth slopes and retaining structures using the LRFD approach.

## 1.1. Project Objectives and Scope

The primary objective of this project was to develop procedures for analysis of earth slopes, including analysis of the overall stability of earth retaining structures, using the LRFD approach and to establish specific load and resistance factors for application of these procedures based on current MoDOT site investigation, laboratory testing, design, and construction practices. A secondary objective was to simultaneously establish baseline probabilistic parameters needed to apply LRFD methods to other geotechnical applications.

The methodology adopted to achieve these objectives was to draw upon previous work in developing LRFD methods for geotechnical applications and reliability-based methods for slope stability analyses to the extent possible to develop a method for LRFD of earth slopes. A series of seven general tasks were undertaken to accomplish the project objectives:

1. Literature review and review of current LRFD procedures;
2. Establish appropriate design cases and loading conditions;
3. Evaluation and development of appropriate reliability-based analysis method;
4. Characterization of probabilistic input variables;
5. Establish appropriate target reliability levels for established design cases;
6. Development and calibration of specific load and resistance factors; and
7. Documentation of project efforts.

Progress on each of these tasks has been conveyed to MoDOT in a series of quarterly progress reports. This document is therefore not organized in accordance with project tasks. Rather, the document is organized in a manner deemed better suited for presentation of the developed procedures as a whole to facilitate implementation of the products of the project.

## 1.2. Content and Organization of Report

This report is organized into eight different chapters describing different aspects of the development of LRFD procedures for design of earth slopes. This chapter provides general information regarding the objectives and scope of the project as well as an outline of the contents of this report. Chapter 2 provides background information necessary to establish the context for transitioning to LRFD as well as to provide general background information necessary to understand the remainder of the report. In Chapter 3, the general procedure developed to implement the LRFD approach for application to slope stability analysis is presented and contrasted with existing procedure for design following ASD concepts. Two alternative methods for establishing, or “calibrating”, appropriate load and resistance factors are also described.

Load and resistance factors established for design of earth slopes using the LRFD technique are presented in Chapter 4 and 5. Chapter 4 first presents load and resistance factors established to produce designs that are similar to those that would result from conventional ASD procedures. While such efforts may be questioned on the basis of “Why bother?”, this process is an important one in that it provides baseline load and resistance factors that are known to be consistent with current design, and serves as an important intermediate step towards achieving the most notable advantages of LRFD. Load and resistance factors established by probabilistic calibrations are then presented in Chapter 5. These load and resistance factors are intended to produce more consistent levels of safety (reliability) across a broad range of sites and design conditions, which is the primary objective of transitioning to LRFD.

Chapter 6 and 7 describe practical issues associated with implementation of LRFD. Results of a series of analyses performed on actual site data are presented in Chapter 6 to serve as a basis for understanding typical probabilistic parameters associated with slope stability analysis. In Chapter 7, issues associated with implementation of LRFD procedures are presented and discussed and the pros and cons of adopting the alternative sets of load and resistance factors are discussed. Finally, Chapter 8 provides a summary of the report, along with a series of major conclusions drawn from the results of this work and a series of recommendations for implementing LRFD for slope stability applications and improving upon the currently recommended load and resistance factors over time.

# 2. Background

In this chapter, the basis and motivation for transitioning to Load and Resistance Factor Design (LRFD) from the more traditional Allowable Stress Design (ASD) is described to convey to potential users the primary reasons for the transition. MoDOT's current practices for slope stability analysis are then summarized and discussed to serve as a basis for the changes proposed in this report.

## 2.1. Basis and Motivation for LRFD

In traditional ASD, uncertainty in the loads and resistances for a particular application is generally accounted for through the use of a single factor of safety (*F*). Such methods have been used for many years with a great deal of success, primarily due to the fact that acceptable values for the factor of safety have been empirically developed over time. However, ASD methods suffer from the fact that uncertainty attributed to a number of different input parameters must be "lumped" together into a single factor. In many instances the real uncertainties of these different parameters may vary substantially; ASD methods may therefore produce designs that have significantly different levels of conservatism (safety) despite having similar overall factors of safety. The LRFD approach seeks to address this limitation by applying different "load" and "resistance" factors to each variable in a problem so that a more consistent degree of safety can be achieved for different components of a design, and for designs of different structures at different sites. The LRFD approach also has the significant advantage of providing the designer with direct information regarding which aspects of a design involve the most uncertainty, thereby indicating areas where "better" data are warranted and empowering the designer with information needed to address such uncertainty. In turn, this may lead to development of more reliable and cost effective designs in many cases.

The principal difficulty with converting to LRFD methods is that specific load and resistance factors must be developed for various types of design and for various loading conditions. As discussed in more detail in subsequent chapters, two alternative approaches may be used to develop load and resistance factors. The first approach is to calibrate load and resistance factors based on current ASD methods. This approach has the advantage of being relatively simple while at the same time assuring that a reasonable level of safety is maintained (based on long-term calibration of acceptable factors of safety for ASD). The disadvantage of this approach is that it limits the benefits that can be realized by LRFD because it is tied to ASD methods. The second approach is to establish target levels of reliability for different designs based on reliability-based analyses and historical precedent, and then develop appropriate load and resistance factors to produce the target levels of reliability based on knowledge of the uncertainties in the different parameters. This approach has the advantage of allowing one to develop more appropriate load and resistance factors that are specifically targeted to the uncertainty in specific design input parameters. This approach also has the advantage of allowing load and resistance factors to be more easily adjusted over time as more information regarding the levels of uncertainty is obtained. However, the second approach suffers from the disadvantages that: (1) information needed to establish factors for some parameters is not routinely available at present, and (2) procedures for reliability-based analysis of the particular application must be available. As a result, most load and resistance factors current being utilized were developed using some combination of the two approaches. Future work is expected to provide information that will allow many of the current load and resistance factors to be modified in time as more appropriate data is collected and made available to researchers working in the area.

At present, baseline LRFD methods and specific load and resistance factors have been developed for the Federal Highway Administration (FHWA) for several geotechnical engineering applications including design of structural foundations and earth retaining structures (FHWA, 2001). However, LRFD methods for analysis and design of earth slopes are not currently available. One possible reason for the delay in applying LRFD to design of slopes is that the factor of safety normally used for ASD (deterministic design) of earth slopes is different than that normally used for foundations and retaining structures. The factor of safety for foundations and retaining structures is generally defined as

$$F = \frac{R}{P} \tag{2.1}$$

where *F* is the factor of safety, *R* is the available resistance, and *P* is the applied load. This definition of the factor of safety is consistent with that normally used in structural applications and is normally used because the uncertainty in the problem is essentially equally shared between the applied load and the available resistance. Because this factor of safety is similar to that used in structural applications, it is a relatively straightforward process to extend LRFD methods for structures, which are more established, to geotechnical applications, where LRFD is relatively new. In contrast, the factor of safety most commonly applied for slope stability studies is defined with respect to the shear strength of the soil as

$$F = \frac{s}{\tau} \tag{2.2}$$

where *s* is the shear strength of the soil and $\tau$ is the mobilized shear stress (the shear stress required to maintain equilibrium under the imposed geometry and loading conditions). The reason for using a different factor of safety for slope stability studies is because the majority of the uncertainty in the analyses is in the shear strength of the soil rather than the loads (which are relatively well known in most cases). Extension of LRFD methods utilized for structures is therefore more complex, although the basic ideas are the same.

Despite these difficulties, there is no inherent reason that LRFD approaches cannot be used for design of earth slopes with resulting improvement in consistency, effectiveness, and efficiency. In fact, a significant amount of research has been performed to evaluate and develop reliability-based design (RBD) procedures for earth slopes (e.g. Wolff, 1996; USACE, 1999; Wolff et al., 2004; Duncan et al., 1999), most for the U.S. Army Corps of Engineers (USACE). This research has identified and begun to address many of the impediments to development of suitable RBD methods. While RBD and LRFD are not synonymous, they are closely related and the existing body of research provides a firm basis for extension of RBD methods to LRFD.

### 2.2. Current MoDOT Procedures for Slope Stability Analysis

As is the case for other geotechnical applications, applications for slope stability analysis must consider a variety of design/loading conditions to ensure that a slope will remain stable throughout its effective design life. At a minimum, the design conditions to be evaluated for most slopes include evaluation of both the short-term (generally undrained) stability of the slope both during and shortly after construction as well as the evaluation of the long-term (generally fully drained) stability of the slope long after construction. Generally speaking, each of these conditions is evaluated based on the anticipated "worst-case" conditions that are expected to occur throughout the respective periods. In certain cases, additional "special" design/loading cases, such as sudden-drawdown conditions or seismic loading conditions, must also be evaluated.

Prior to developing the new LRFD procedures for stability analysis of earth slopes and retaining structures, project investigators consulted with MoDOT Soils and Geology personnel to establish a list of stability cases that are routinely evaluated as well as to establish the general procedures used to evaluate these cases based on current practices. The results of these efforts are summarized in Table 2.1, which shows the general stability cases that are routinely evaluated along with a summary of common procedures used to evaluate these cases.

The stability cases commonly evaluated generally consist of both short-term and long-term stability analyses as conditions warrant. For cohesive soils and soils with significant fines, short-term stability analyses are generally performed using total stress strength parameters, usually defined in terms of an undrained shear strength, $s_u$, and ignoring any strength attributed to a total stress angle of internal friction, $\phi$; long-term stability analyses are performed using effective stress strength parameters, $\bar{c}$ and $\bar{\phi}$, along with pore pressures estimated following established practice. In many cases, the effective stress cohesion intercept, $\bar{c}$, is assumed to be negligible and is taken to be zero. For granular soils without significant fines content, the long-term (fully drained) condition is reached during construction so undrained analyses are not generally performed.

Table 2.1. Summary of general stability cases currently considered by MoDOT personnel along with summary of procedures utilized to evaluate stability for these conditions.

| Stability Case | Analysis Type | Design Parameters | Method for Est. Parameters | Factor of Safety | Comments |
|---|---|---|---|---|---|
| Embankment on Soft Foundation | Short-term, total stress | $\gamma$, $s_u$ | $s_u = q_u/2$ or $s_u = 2/3\, s_{u-\mathrm{TV}}$ | $F$≥1.0 | • if $F$ <1.0 and long-term stability acceptable will control rate of filling; |
| | Long-term, effective stress | $\gamma, u, \bar{\phi}, \bar{c}$ | DS tests on 3- to 5-in. Shelby tube samples | $F$≥1.5 | • cohesion intercept, $\bar{c}$, often neglected |
| | | | Estimated from PI and NAVFAC correlation | $F$≥1.25 | • $\bar{\phi}$ vs PI correlation taken as mean minus 1 std. dev.<br>• cohesion intercept, $\bar{c}$, neglected; |
| Slide remediation | Long-term, effective stress | $\gamma, u, \bar{\phi}, \bar{c}$ | Back-calculation procedure and/or DS tests | $F$ ≥1.25 | cohesion intercept, $\bar{c}$, often neglected |
| Temporary slope | short-term, total stress | $\gamma$, $s_u$ | $s_u = q_u/2$ or $s_u = 2/3\, s_{u-\mathrm{TV}}$ | $F$≥1.10-1.15 | |
| Sudden drawdown | Effective stress | $\gamma, u, \bar{\phi}, \bar{c}$ | • DS tests on 3- to 5-in. Shelby tube samples or estimated from PI and NAVFAC correlation (-1 std. dev.)<br>• pore pressures estimated from high water elevation (HWE) with "buoyed toe" | $F$ ≥1.0 | • cohesion intercept, $\bar{c}$, neglected |
| General stability | Long-term, effective stress | $\gamma, u, \bar{\phi}, \bar{c}$ | DS tests on 3- to 5-in. Shelby tube samples | $F$ ≥1.5 | • some flexibility in required $F$ |
| | | | Estimated from PI and NAVFAC correlation (-1 std. dev.) | $F$ ≥1.25 | • cohesion intercept, $\bar{c}$, neglected |
| Overall stability of retaining walls | Long-term, effective stress | $\gamma, u, \bar{\phi}, \bar{c}$ | DS tests on 3- to 5-in. Shelby tube samples or estimated from PI and NAVFAC correlation (-1 std. dev.) | $F$ ≥1.5 | • cohesion intercept, $\bar{c}$, often neglected;<br>• Min. $F$ is hard requirement<br>• if traffic within ½ $H$, add surcharge of 2-ft of fill for stability analysis (AASHTO requirement) |
| Seismic Stability | Pseudo-static | Varies | Varies | $F$ ≥1.1 | • Not routinely performed at present |

Methods used to estimate design parameters for each of the stability cases shown vary depending on the criticality of the respective conditions and range from crude estimation procedures for preliminary analyses and analyses for conditions generally deemed non-critical to more elaborate site investigation and testing procedures for cases deemed to be more critical. Required factors of safety for the various stability conditions are also seen to vary. The variability in the required factors of safety is, to some extent, also based on the criticality of the respective stability conditions, but also based on the level of conservatism inherent in the methods for estimating design parameters and based on experience with performance of slopes designed for the respective conditions. The result of this is that the required factors of safety are empirically derived to achieve a reasonable level of performance based on experience.

Overall, the procedures summarized in Table 2.1 have proven successful for analysis of slope stability for the cases identified. There have not been occasions where failures have been attributed to flaws in the procedures. However, this success does not necessarily mean that the procedures are producing stability that is adequate to ensure an appropriate level of safety while also minimizing costs, or that application of these same procedures to different cases is producing consistent levels of safety. It is just these issues that the LRFD approach is intended to address so that design and analyses for different sites and situations produce consistent levels of safety so that limited funds are applied in the most cost effective manner possible.

# 3. Implementation of LRFD for Slope Stability Analysis

This chapter describes the general procedure adopted for implementing the LRFD approach for stability analysis of earth slopes and overall stability evaluations for retaining structures. The general procedure used for current ASD analysis is first summarized followed by description of the procedure proposed for LRFD analyses. The similarities and differences between the two approaches are then discussed followed by discussion of several issues pertaining to the adopted LRFD approach. Finally, the two methods available for establishing appropriate resistance factors for performing slope stability analyses using LRFD are described.

## 3.1. General Procedure for Allowable Stress Design of Earth Slopes

In general terms, the procedure conventionally adopted for analysis of slope stability using current ASD procedures consists of the following steps:

1. Establish site geometry and stratigraphy using available geologic information, boring logs, site surveys and plans, and other information available to the designer;
2. Estimate parameters for each respective stratum within the slope using available laboratory test results, empirical correlations, back-calculations, and other available information;
3. Estimate anticipated pore pressure conditions (required only for effective stress analyses) based on available historical records and judgment;
4. Evaluate the factor of safety for the conditions established using appropriate slope stability analysis methods with the parameters established in previous steps as input; and
5. Compare the computed factor of safety to the required or target factor of safety (Table 2.1):
    a. If the computed factor of safety is approximately equal to the required factor of safety, the design is considered acceptable.
    b. If the factor of safety is significantly greater than the required factor of safety, changes to reduce the computed factor of safety may be considered if significant cost savings can be realized.
    c. If the factor of safety is less than the required factor of safety, the designer must consider alternative measures to increase the factor of safety and repeat the procedure until an acceptable factor of safety is achieved.

Steps 1 through 3 are common to all geotechnical design and analysis in one form or another. These steps will remain unchanged with adoption of LRFD procedures. Step 4 is most commonly performed using commercially available slope stability analysis software, usually utilizing limit equilibrium methods such as Bishop's Simplified Procedure or Spencer's procedure, but can also be performed using simpler analytical solutions (e.g. infinite slope solutions) or stability charts. All of these steps generally involve substantial interpretation and judgment by the analyst, a requirement that will not change with adoption of LRFD.

## 3.2. General Procedure Adopted for LRFD of Earth Slopes

The procedure for implementing slope stability analysis using the LRFD approach is, in fact, very similar to the approach outlined for conventional ASD analyses above. The primary changes involved with transitioning to LRFD involve Steps 4 and 5. The revised general LRFD procedure involves the following steps:

1. Establish site geometry and stratigraphy using available geologic information, boring logs, site surveys and plans, and other information available to the designer;
2. Estimate parameters for each respective stratum within the slope using available laboratory test results, empirical correlations, back-calculations, and other available information;
3. Estimate anticipated pore pressure conditions (required only for effective stress analyses) based on available historical records and judgment;
4. Evaluate the factor of safety for the conditions established using appropriate slope stability analysis methods with *factored parameters* as input; and

5. Compare the computed factor of safety to the limit factor of safety (= 1.0):
   a. If the computed factor of safety is approximately equal to 1.0, the design is considered acceptable.
   b. If the factor of safety is significantly greater than 1.0, changes to reduce the computed factor of safety may be considered if significant cost savings can be realized.
   c. If the factor of safety is less than 1.0, the designer must consider alternative measures to increase the factor of safety and repeat the procedure until a factor of safety approximately equal to 1.0 is achieved.

Comparison of the procedure listed above with the procedure listed previously for conventional ASD analysis reveals two simple differences in the procedures. The first difference occurs in Step 4, where factored parameters are used as input for the slope stability analyses for the LRFD procedure whereas unfactored parameters are used for the traditional ASD procedure. The second difference occurs in Step 5, where instead of comparing the computed factor of safety to a required or target factor of safety, the computed factor of safety is compared to a limit value (= 1.0) indicating stability or instability. In this respect, the LRFD procedure is indeed more straightforward than current procedures in that the analysis target or limit is consistent for all stability cases for the LRFD procedure whereas the analysis target for conventional ASD procedures varies from one application to another. The result of these differences is simply that, for LRFD procedures, uncertainties in the analyses are accounted for through factoring of the input parameters whereas for ASD procedures the uncertainty is accounted for through a single factor of safety. By factoring individual input parameters, it is possible to more appropriately apply conservatism to the individual parameters involved in the analysis, and therefore to effect more consistent levels of safety across a broad range of cases. Both load and resistance factors in LRFD and factors of safety in ASD are intended to account for uncertainties involved in the respective analyses. They are simply different methods for accounting for these uncertainties.

Inspection of Table 2.1 reveals that there are five common parameters typically used as input for slope stability analyses[1]. These parameters include the soil (total) unit weight, $\gamma$, undrained shear strength, $s_u$, Mohr-Coulomb shear strength parameters, *c* and $\phi$ (or *c*' and $\phi'$ in the case of effective stress analyses), and the pore water pressure, $u$. For the proposed implementation, pore water pressures are considered as deterministic values that are not factored. Thus, procedures for handling pore water pressures, or piezometric lines and other constructs used to model pore water pressures, remain unchanged and should be estimated following procedures identical to those used for traditional ASD procedures. The same is true of site stratigraphy and slope geometry. The remaining input parameters are factored and established by first estimating appropriate nominal, or mean values for the required parameters following conventional approaches (laboratory tests, field tests, empirical correlations, etc). These nominal values are then "factored" using appropriate "load factors" or "resistance factors" to produce factored parameters that are subsequently used as input for slope stability analyses.

Unlike most other existing applications of LRFD, there is some difficulty in establishing which parameters are considered "loads" and which parameters are considered "resistances" when considering the application of slope stability. In general, load factors are applied to parameters that tend to reduce stability while resistance factors are applied to parameters that tend to promote stability. Thus, load factors tend to be greater than or equal to 1.0 so as to account for the potential for the "loads" to be greater than expected. Likewise, resistance factors tend to be less than or equal to 1.0 to account for the potential for the "resistance" to be less than expected. For slope stability problems, the dominant "load" frequently arises from the self weight of the soil. Thus, it is logical to consider the unit weight of the soil as a "load" and to apply load factors to this parameter. Conversely, the dominant resistance in slope stability problems arises from the shear strength of the soil. Thus, it is logical to consider the shear strength (or parameters used to compute the shear strength) as resistances and to apply resistance factors to these parameters. While examples where this logic fails can be demonstrated, the logic nevertheless holds for the vast majority of cases and thus the proposed implementation adopts the

[1] Other parameters such as reinforcement loads, surcharge loads, etc. may also be required in some cases but are not considered here

convention that soil unit weight is taken to be a load and soil shear strength (or shear strength parameters) is taken to be a resistance.

In the present implementation, factored values of soil unit weight are computed as

$$\gamma^* = \chi \cdot \gamma \tag{3.1}$$

where $\gamma^*$ is the factored unit weight to be used in stability analyses for the LRFD procedure, $\chi$ is the load factor and $\gamma$ is the unfactored nominal unit weight[2]. As is common practice, the value of $\chi$ will generally be greater than or equal to 1.0. Similarly, factored values for soil shear strength are computed as

$$s^* = \psi \cdot s \tag{3.2}$$

where $s^*$ is the factored shear strength, $\psi$ is the resistance factor, and $s$ is the unfactored shear strength. For cases where the shear strength is to be represented by an undrained shear strength, $s_u$, Eq. 3.2 becomes

$$s_u{}^* = \psi \cdot s_u \tag{3.3}$$

where $s_u^*$ is the factored undrained shear strength and $s_u$ is the unfactored, nominal undrained shear strength. For cases where the soil shear strength is represented using a Mohr-Coulomb failure envelope and shear strength parameters $c$ and $\phi$ for total stress analyses or $\bar{c}$ and $\bar{\phi}$ for effective stress analyses, Eq. 3.2 can be effected by simply applying the resistance factor to the respective parameters independently. In other words, the factored shear strength can be specified by inputting factored parameters $c^*$ and $\phi^*$ computed as follows

$$c^* = \psi \cdot c\,; \quad \tan\phi^* = \psi \cdot \tan\phi \tag{3.4}$$

for total stress stability analyses or

$$\bar{c}^* = \psi \cdot \bar{c}\,; \quad \tan\bar{\phi}^* = \psi \cdot \tan\bar{\phi} \tag{3.5}$$

for effective stress stability analyses where $c$ and $\phi$ are the unfactored shear strength parameters defined in terms of total stresses, $\bar{c}$ and $\bar{\phi}$ are the unfactored shear strength parameters defined in terms of effective stresses, and the superscript * again indicates factored values of the respective parameters.

### 3.3. Methods for Calibration of Resistance Factors

To implement the LRFD procedure described above, appropriate values for the load factors, $\chi$, and the resistance factors, $\psi$, must be established. The process used to establish appropriate values for these parameters is referred to as calibration. As discussed previously, two alternative approaches may be used for calibration. The first approach is to calibrate load and resistance factors so that they produce designs that are essentially identical to those produced using current ASD methods. The second approach is to perform reliability-based calibrations wherein target levels of reliability are established for different types of structures or loading conditions and reliability-based analyses are subsequently used to determine the required load and resistance factors to produce the target levels of reliability.

Regardless of the method used for calibration of load and resistance factors, the procedure for implementing LRFD is identical. So while there are distinct advantages and disadvantages of LRFD procedures established using these two different forms of calibration, application of the procedures does not require different approaches for the different calibrations. As such, the LRFD approach offers the potential for a continuous process of improving the accuracy and efficiency of designs over time without requiring wholesale changes to design procedures. This point must be emphasized because early implementations of LRFD procedures, which are often established based on the first calibration

[2] Many references in the literature adopt notations for load and resistance factors that unfortunately conflict with common notations for several soil parameters. The notation adopted here is therefore somewhat unusual, but considered necessary for clarity of presentation.

procedure, are often criticized for failing to effect real improvements to design procedures because they simply produce designs similar to ASD procedures only through a different process. While the criticism is accurate, it is the shift to a more adaptable design process that is the true advantage for transitioning to LRFD from ASD.

In the present work, both calibration procedures have been utilized to develop resistance factors for slope stability analysis applications. Calibrations performed by matching to historical ASD procedures are presented in Chapter 4, while calibrations performed using reliability-based procedures are presented in Chapter 5. The relative merits of the respective calibrations are then presented and discussed in Chapter 7, where recommendations for both immediate and future implementations are made.

# 4. Calibration of Resistance Factors by Matching Historical Design Procedures

This chapter presents results of calibrations performed to develop load and resistance factors that will produce designs that are consistent with current procedures. While these calibrations are relatively simple, and at some level seem unjustified, it is critical that these calibrations be performed to establish baseline load and resistance factors to assure that a reasonable level of safety is maintained (based on long-term calibration of acceptable factors of safety for ASD). The disadvantage of this approach is that it limits the benefits that can be realized by using LRFD because it is tied to ASD methods, but this limitation can be subsequently remedied over time by adopting load and resistance factors calibrated using reliability-based procedures as they are developed and verified.

## 4.1. Load factors

One key aspect of LRFD procedures is that both load and resistance factors are “coupled” to produce appropriate levels of safety. Thus, the process of calibration is in general non-unique. Conceptually one could provide the entire margin of safety exclusively through the load factors, exclusively though resistance factors, or through some combination of load and resistance factors. The preferred combination of load and resistance factors is the one that applies a margin of safety that is consistent with the knowledge of the respective loads and resistances. However, this is complicated by the need to limit the complexity of design to a reasonable level and to maintain some degree of consistency across design for different applications. As a result of these issues, final implementations of LRFD generally represent some compromise of these competing demands.

Current implementations of LRFD for applications involving the stability of spread footings and classical retaining walls, both of which require evaluation of overall slope stability, have recommended that stability be evaluated under what is referred to as the Service I limit state (FHWA, 2001). In this limit state, the load factors applied to the unit weight of the soil is generally taken to be equal to 1.0. This approach is somewhat crude in the sense that some of the uncertainty in these designs can certainly be attributed to uncertainty in the unit weight of the soil. However, as a practical matter, uncertainty in unit weight is generally very small when compared to uncertainty in other parameters so adopting a $\chi$=1.0 is a reasonable simplification. To be consistent with these existing design specifications, and as a reasonable first approximation, calibrations for resistance factors have been performed utilizing a load factor of $\chi$=1.0 for the unit weight of the soil. Alternative load factors may be developed at a later time to better account for potential uncertainty and variability in soil unit weight, but it should be noted that this will also require re-calibration of resistance factors at the same time since the two types of factors are coupled.

## 4.2. Resistance Factors

Given that the load factor for the soil unit weight is taken as 1.0, resistance factors for the respective parameters based on calibration to existing ASD procedures (Table 2.1) can be established from the following logic. Starting with the basic definition of the conventional ASD factor of safety as defined in Eq. 2.2, the design criterion to be evaluated in conventional ASD design from Section 3.1 is

$$F_{req'd} \leq \frac{s}{\tau} = F_{computed} \tag{4.1}$$

Where $F_{req'd}$ is the minimum allowable factor of safety established for the respective design condition and $F_{computed}$ is the computed factor of safety representing the stability of the slope. The analogous criterion for LRFD analyses is

$$\chi \cdot \tau \leq \psi \cdot s \tag{4.2}$$

Rearranging Eq. 4.2 results in

$$\frac{\chi}{\psi} \leq \frac{s}{\tau} \tag{4.3}$$

which, from Eq. 4.1, produces the follow relation between $F_{req'd}$ and the resistance factor for shear strength, $\psi$

$$\chi = \psi \cdot F_{req'd} \tag{4.4}$$

or, taking $\chi$=1.0 and solving for $\psi$

$$\psi = \frac{1}{F} \tag{4.5}$$

Thus, calibration by matching to existing ASD procedures simply requires application of resistance factors for shear strength equal to the inverse of the required factors of safety for the respective design cases. Resistance factors established in this manner for the design cases shown in Table 2.1 are shown in Table 4.1.

### 4.3. Discussion

The resistance factors for shear strength listed in Table 4.1 vary from a low of 0.67 to a high of 1.0. When applied to nominal values of shear strength parameters, which are then input into slope stability analyses as factored values, the results of the analyses should produce a factor of safety (output from slope stability methods) approximately equal to 1.0 for acceptable designs as described in Chapter 3. As illustrated in the example problem provided in Section 4.4, the resulting designs (i.e. slope angle, slope height, etc.) will be identical to those that would be achieved using current design and analysis practices. While this fact begs the question “why switch to LRFD?”, the fact of the matter is that the switch to LRFD is motivated primarily by the ability to use alternative resistance factors developed using reliability-based calibrations as described in Chapter 5. The resistance factors shown in Table 4.1 are simply baseline values to be used temporarily while reliability-based calibrations are performed and verified, and to serve as a baseline upon which to evaluate the reasonableness of factors established using such calibrations.

An important point to note at this point is, aside from being coupled to the adopted load factors as described above, the resistance factors presented above are also dependent on the specific current practices for site investigation, laboratory testing, test interpretation, method of analysis, level of construction monitoring/instrumentation, etc. As such, Table 4.1 also includes these requirements for establishing nominal values for the unfactored shear strength parameters to highlight this dependence. Deviations from these standard practices may result in the resistance factors becoming inapplicable in much the same way as target factors of safety would become inapplicable. A good example of this is the practice shown for evaluation of the short-term stability of embankments on soft foundations or for evaluation of stability under sudden drawdown conditions. Current practice requires a minimum factor of safety of 1.0 for both of these cases, which corresponds to resistance factors of 1.0. On the surface, both of these imply no margin of safety in the design. However, closer inspection of the specific procedures generally adopted in these cases indicates that strength parameters selected for these analyses have significant conservatism imposed prior to performing stability analyses. For the case of short-term stability of embankments on soft foundations, the undrained shear strength may be estimated as 2/3 of the undrained shear strength measured using Torvane tests. Alternatively, the undrained shear strength may be estimated as one-half of the unconfined compressive strength. In this case, there is no obvious conservatism being applied to the undrained shear strength since the undrained shear strength is theoretically equal to one-half of the unconfined compressive strength for saturated soils. However, these values still have some conservatism because of sampling disturbance, lack of confining stress, and other factors and, as a result, experience has shown that use of these values has produced acceptable performance (even if it is fortuitous). Similarly, for the sudden drawdown case, strength parameters are estimated based on correlation with Atterberg limits, but the correlation used represents the mean value minus one standard deviation. In essence, these practices are already using a form of LRFD by utilizing reduced resistances as a means for introducing a margin of safety into design. Use of other practices for establishing these parameters could potentially negate this margin of safety regardless of whether ASD or LRFD procedures are followed.

Table 4.1. Resistance factors for shear strength determined by matching to existing ASD procedures.

| Stability Case | Analysis Type | Shear Strength Parameter(s) | Method for Est. Parameters | Resistance Factor, $\psi$ | Comments |
|---|---|---|---|---|---|
| Embankment on Soft Foundation | Short-term, total stress | $s_u$ | $s_u = q_u/2$ or $s_u = 2/3\, s_{u-\mathrm{TV}}$ | 1.0 | |
| | Long-term, effective stress | $\bar{\phi}$, $\bar{c}$ | DS tests on 3- to 5-in. Shelby tube samples | 0.67 | • cohesion intercept, $\bar{c}$, often neglected |
| | | | Estimated from PI and NAVFAC correlation | 0.80 | • $\bar{\phi}$ vs PI correlation taken as mean minus 1 std. dev.<br>• cohesion intercept, $\bar{c}$, neglected; |
| Slide remediation | Long-term, effective stress | $\bar{\phi}$, $\bar{c}$ | Back-calculation procedure and/or DS tests | 0.80 | cohesion intercept, $\bar{c}$, often neglected |
| Temporary slope | short-term, total stress | $\gamma$, $s_u$ | $s_u = q_u/2$ or $s_u = 2/3\, s_{u-\mathrm{TV}}$ | 0.87-0.91 | |
| Sudden drawdown | Effective stress | $\bar{\phi}$, $\bar{c}$ | • DS tests on 3- to 5-in. Shelby tube samples or estimated from PI and NAVFAC correlation (-1 std. dev.)<br>• pore pressures estimated from HWE with "buoyed toe" | 1.0 | • cohesion intercept, $\bar{c}$, neglected |
| General stability | Long-term, effective stress | $\bar{\phi}$, $\bar{c}$ | DS tests on 3- to 5-in. Shelby tube samples | 0.67 | • some flexibility in required $F$ and therefore $\psi$ |
| | | | Estimated from PI and NAVFAC correlation (-1 std. dev.) | 0.80 | • cohesion intercept, $\bar{c}$, neglected |
| Overall stability of retaining walls | Long-term, effective stress | $\bar{\phi}$, $\bar{c}$ | DS tests on 3- to 5-in. Shelby tube samples or estimated from PI and NAVFAC correlation (-1 std. dev.) | 0.80 | • cohesion intercept, $\bar{c}$, often neglected;<br>• if traffic within ½ $H$, add surcharge of 2-ft of fill for stability analysis |
| Seismic Stability | Pseudo-static | Varies | Varies | 0.91 | • Not routinely performed at present |

### 4.4. Example Application

To demonstrate application of the LRFD procedure for slope stability analysis and to simultaneously demonstrate the equivalence of results obtained using traditional procedures and LRFD procedures, the stability of the slope shown in Figure 4.1 was evaluated using both procedures. The example slope is a 2.5H:1V slope that is 30-ft in height. The slope is assumed to be homogeneous and the foundation of the slope is assumed to be firm soil or rock that prevents sliding along surfaces passing into the foundation. The properties of the slope in terms of both total and effective stresses are shown in Figure 4.1. The shear strength parameters are assumed to be determined from unconfined compression tests and direct shear tests on 3-in diameter Shelby Tube samples, following established MoDOT practices. In all cases, the computed factors of safety were determined using slope stability charts, but similar results would be obtained using commonly available slope stability analysis software. A target value of the factor of safety of 1.5 was also adopted for ASD analyses of both the short-term and long-term stability conditions.

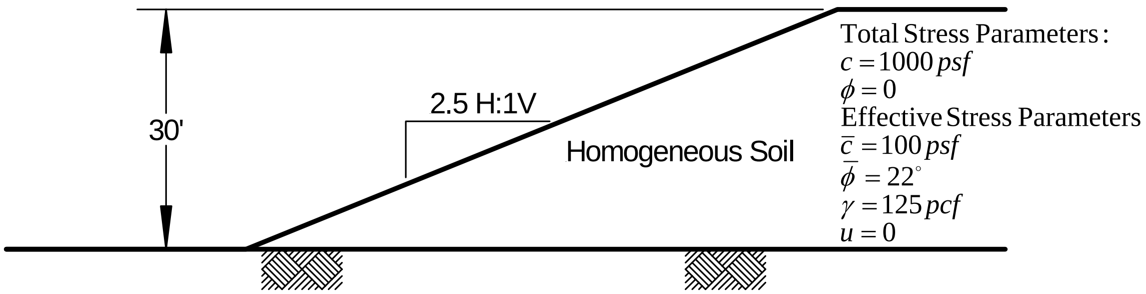



Figure 4.1 Slope conditions used for example problem.

4.4.1. Evaluation of Short-Term Stability Using ASD Approach

The stability of the slope under short-term, undrained conditions was evaluated using the total stress strength parameters $c$=1000 psf and $\phi$=0. Using these parameters as input into the slope stability analysis, the resulting factor of safety was found to be 2.4. This value is well above the target value of the factor of safety of 1.5, which indicates the design is acceptable. The high value of the factor of safety also suggests that the slope could potentially be steepened if short-term stability controls the design.

4.4.2. Evaluation of Short-term Stability Using LRFD Approach

Using the LRFD approach, stability was evaluated in a similar manner to that used for ASD analyses, except that *factored* strength parameters were used as input into the analyses and the resulting factor of safety as compared to a value of 1.0 rather than to a target value of 1.5. The resistance factor associated with a target ASD factor of safety of 1.5 is 0.67 (=1/1.5). Thus, the factored shear strength parameters are (from Eq. 3.4):

$$c^* = \psi \cdot c = 0.67 \cdot 1000\,psf = 670\,psf \quad (4.6)$$

$$\tan\phi^* = \psi \cdot \tan\phi = 0.67 \cdot 0 = 0 \quad (4.7)$$

Inputting $c$=670-psf and $\phi$=0 into the stability analysis produces a factor of safety of 1.61, which is well above the limiting value of 1.0. This again indicates the design is acceptable and that the slope could potentially be steepened if short-term stability controls the design.

4.4.3. Evaluation of Long-term Stability Using ASD Approach

The stability of the slope under long-term, fully drained conditions was evaluated using the effective stress strength parameters $\bar{c} = 100\,psf$ and $\bar{\phi} = 22^\circ$ and zero pore pressures. Using these values as input, the computed factor of safety for the long-term stability condition is 1.44. This value is just slightly below the target value of 1.5, which indicates that the stability can be considered marginally acceptable. This result also shows that the long-term stability will govern design.

4.4.4. Evaluation of Long-term Stability Using LRFD Approach

Long-term stability using the LRFD approach was again evaluated in a similar fashion except that factored shear strength parameters were used and the limit value of the computed factor of safety is 1.0.

Again using a resistance factor of 0.67, the values of the factored shear strength parameters are as follows (from Eq. 3.5):

$$\bar{c}^* = \psi \cdot \bar{c} = 0.67 \cdot 100\,psf = 67\,psf \qquad (4.6)$$

$$\tan\bar{\phi}^* = \psi \cdot \tan\bar{\phi} = 0.67 \cdot \tan(22^\circ) = 0.271 \quad \Rightarrow \bar{\phi}^* = \tan^{-1}(0.271) = 15^\circ \qquad (4.7)$$

Inputting the factored shear strength parameters of $\bar{c} = 67\,psf$ and $\bar{\phi} = 15^\circ$ into the stability analysis produces a factor of safety of 0.96. This value is just slightly below the limit value of 1.0, which again indicates that the stability is marginal.

# 5. Probabilistic Calibration of Resistance Factors

This chapter describes the methods and procedures used to establish load and resistance factors probabilistically along with the resulting load and resistance factors. First, the objective of probabilistic calibration is described and contrasted to that of calibration by matching to conventional ASD design. Next, several concepts necessary for understanding of reliability-based analyses are described, followed by description of the specific methods selected for reliability-based analysis of slope stability. Finally, the analyses utilized to establish the probabilistically calibrated load and resistance factors are described and the load and resistance factors are presented, generally in the form of “resistance factor relations”.

## 5.1. Objectives of Probabilistic Calibration

In contrast to calibration by matching to traditional design procedures where the objective is to produce similar results using different procedures, the objective of probabilistic calibration is establish load and resistance factors that will produce consistent levels of reliability (or safety) across a broad range of design cases. The latter objective is really the fundamental promise of transitioning to LRFD design – applying appropriate conservatism (and thus funds) where needed but not where not needed – and one that cannot be effectively reached using existing ASD procedures.

Stated more precisely, the objective of probabilistic calibration is to develop combinations of load and resistance factors that will produce a consistent reliability, or probability of failure, regardless of the uncertainty or variability of the values of the necessary design parameters. This objective can be more clearly appreciated by considering a simple conceptual example of an embankment that is to be constructed on a particular site. If that site has soils with highly variable soil properties and is poorly characterized because of limited site investigation, the properties and conditions needed for input into slope stability analyses will be poorly defined and have a large degree of uncertainty. Furthermore, the resulting uncertainty in stability will also be relatively high because of the uncertainty in the inputs. In such a case, it is necessary to design with a higher level of conservatism to overcome the uncertainty present so as to produce a design that has an appropriate level of safety (reliability). Conversely, if the site is relatively uniform and is well characterized with an extensive site investigation and testing program, the properties and conditions needed for input into slope stability analyses will be relatively well defined and have much less uncertainty. In this case, there is less uncertainty about the stability of the slope and therefore less conservatism is needed to produce an appropriate level of safety or reliability. Designs with acceptable levels of safety can be realized in both cases. However, different approaches and resulting designs will be required to accomplish this, and generally with substantial cost implications. In the former example, the limited nature of the site investigation would likely produce cost savings, but these cost savings are offset (either partially or completely) by the high level of conservatism needed to compensate for the lack of information. Conversely, in the latter example, the extensive site investigation would undoubtedly require additional costs, but these costs would again be offset by reduced costs for construction. Which of these approaches is preferable will vary from case to case, and it is the designer’s task to select the most appropriate approach. The LRFD method is specifically intended to facilitate designer’s ability to address such dilemmas while maintaining consistency in design – i.e. using greater conservatism in cases where uncertainty is great and less conservatism in cases where uncertainty is less, but in all cases producing appropriate *and consistent* levels of safety or reliability. While it is possible to achieve the same objective using ASD procedures by requiring different target factors of safety depending on the level of uncertainty present, the LRFD approach allows this process to be performed more explicitly and effectively.

## 5.2. Basic Probability Concepts

Several simple probability concepts will be used throughout the remainder of this chapter and subsequent chapters so they are presented succinctly here to facilitate understanding and subsequent presentation. Readers are also encouraged to consult available textbooks (e.g. Baecher and Christian, 2003) for further discussion of these concepts.

The basic premise of reliability-based analysis (which may form the basis for LRFD) is that the stability of a slope, as commonly represented by a factor of safety[3], is never known with certainty but instead takes on the form of a probability distribution, or probability density function (pdf), as shown conceptually in Figure 5.1. Position along the abscissa (x-axis) represents the values of factor of safety that may exist for a particular slope under a particular set of conditions, while position along the ordinate axis (y-axis) represents the likelihood that the stability of the slope is represented by a particular factor of safety (higher values indicating more likelihood that the particular factor of safety is correct). The probability density function provides a measure of the uncertainty in the actual stability of the slope (again as usually measured using a factor of safety). Distributions with larger spreads along the abscissa have greater uncertainty than distributions with lesser spreads. The uncertainty in stability arises from uncertainty in the input parameters of these analyses, which in turn arises from lack of complete or certain information about conditions that exist, or will exist in the slope.

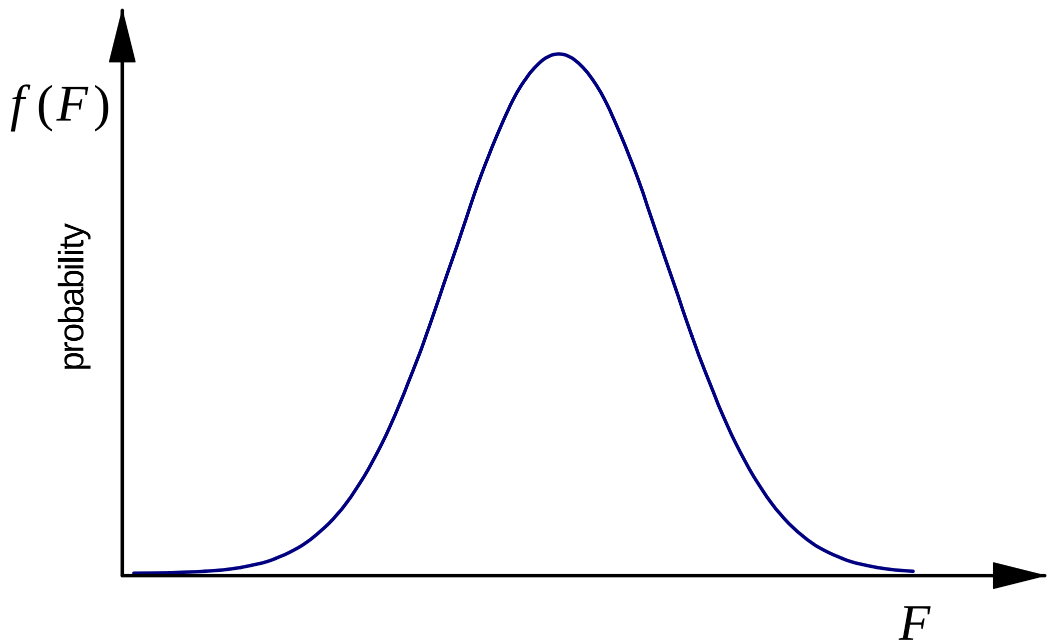



Figure 5.1. Conceptual distribution of factor of safety.

While probability density functions are a useful and general way to depict the uncertainty in stability or other parameters of interest, these distributions are also frequently discussed in terms of a series of parameters that serve to describe the distribution numerically. Several common parameters that will be subsequently used are illustrated on a conceptual probability density function of the factor of safety, $F$, in Figure 5.2. The mean of a parameter, $\mu$, is a measure of the central tendency of the distribution, which for normally distributed variables is coincident with the most likely value from the distribution. The standard deviation, $\sigma$, on the other hand is a measure of the dispersion or breadth of the distribution with higher values for standard deviation representing greater uncertainty or variability and lower values of standard deviation representing less uncertainty or variability. These two parameters are most commonly used to describe probability density functions. A third common parameter, denoted the coefficient of variation or $COV$, is determined as the ratio of the standard deviation to the mean value. The $COV$ is really just a normalized, dimensionless form of the standard deviation that also represents the degree of spread or uncertainty in the distribution. The standard deviation is measured in the same units as the mean value while the $COV$ is dimensionless and is generally expressed as a percentage of the mean value.

[3] Other measures of stability, such as the safety margin, can also be used in the same context. However, the factor of safety is used to represent stability throughout this report because of its familiarity to designers.

Several additional probabilistic quantities are also frequently encountered when specifically referring to the distribution of the factor of safety, or distributions of other parameters indicating stability (but not distributions of input parameters). These quantities are illustrated in Figure 5.3. For the distribution shown, the mean or most likely value of the factor of safety ($\mu_F$) is greater than the limit value of the factor of safety ($F$<1.0 indicating instability), which indicates that the slope is more likely to be stable than not. However, because of the uncertainty in the factor of safety, there is some likelihood that the actual factor of safety is less than 1.0. This likelihood is represented using the ***probability of failure***, $p_f$, which is computed as the area under the pdf where $F$<1.0 as indicated by the horizontally hatched region in Figure 5.3. An alternative measure of stability is the ***reliability***, $r$, which is the likelihood that the actual factor of safety is greater than or equal to 1.0. This parameter represents the area beneath the pdf for $F$≥1.0 as indicated by the vertically hatched region in Figure 5.3. Since the total area under the pdf is equal to 1.0, the probability of failure and reliability are directly related as

$$p_f = 1 - r \tag{5.1}$$

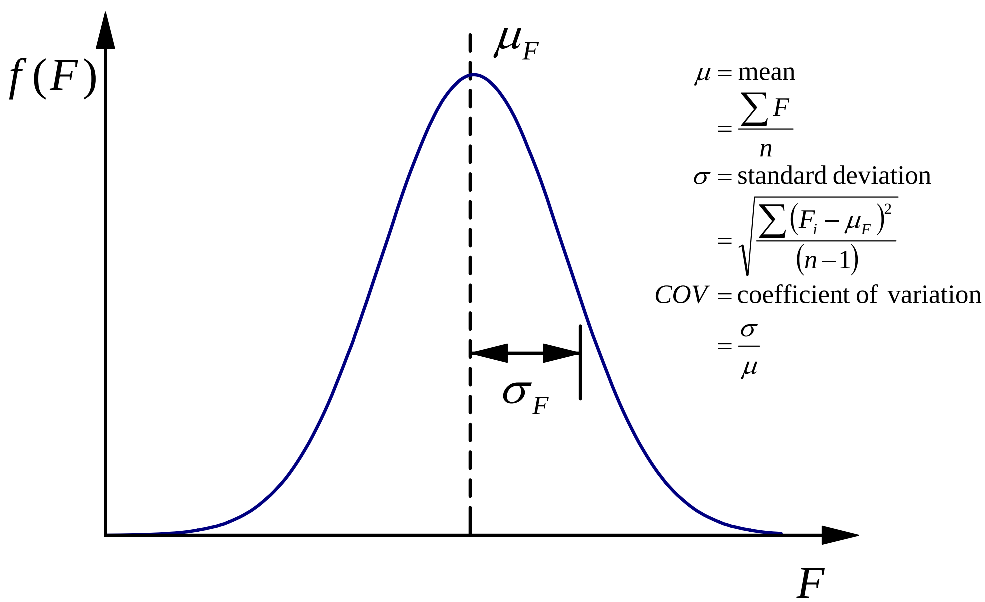



Figure 5.2. Sample distribution illustrating common probabilistic parameters used to described probability density functions.

Either measure of safety can be used with the same effect. The probability of failure will generally be used in subsequent sections of this report. A third measure that is frequently referenced in much of the LRFD literature pertaining to geotechnical applications is the reliability index, $\beta$. The reliability index is a measure of the distance between the mean value of the factor of safety, $\mu_F$, and the limit value of the factor of safety, $F$=1.0, measured in units of standard deviations so that $\beta$ is the number of standard deviations between the limit value and the mean. The reliability index is also directly related to both the reliability and the probability of failure *if the distribution of the factor of safety is known*. It is important to emphasize that reliability and the reliability index are not identical as these parameters have at times been incorrectly used interchangeably in some of the LRFD literature. The reliability index has been frequently used (and misused) as a measure of safety, primarily because values of the reliability index are generally more appealing as they usually fall between 1 and 4 (much like conventional factors of safety) while the probability of failure is generally a very small number (e.g. 0.001) and the reliability is generally very close to 1.0 (e.g. 0.999), which renders them more difficult to evaluate when comparing two small values or two values very close to 1.0.

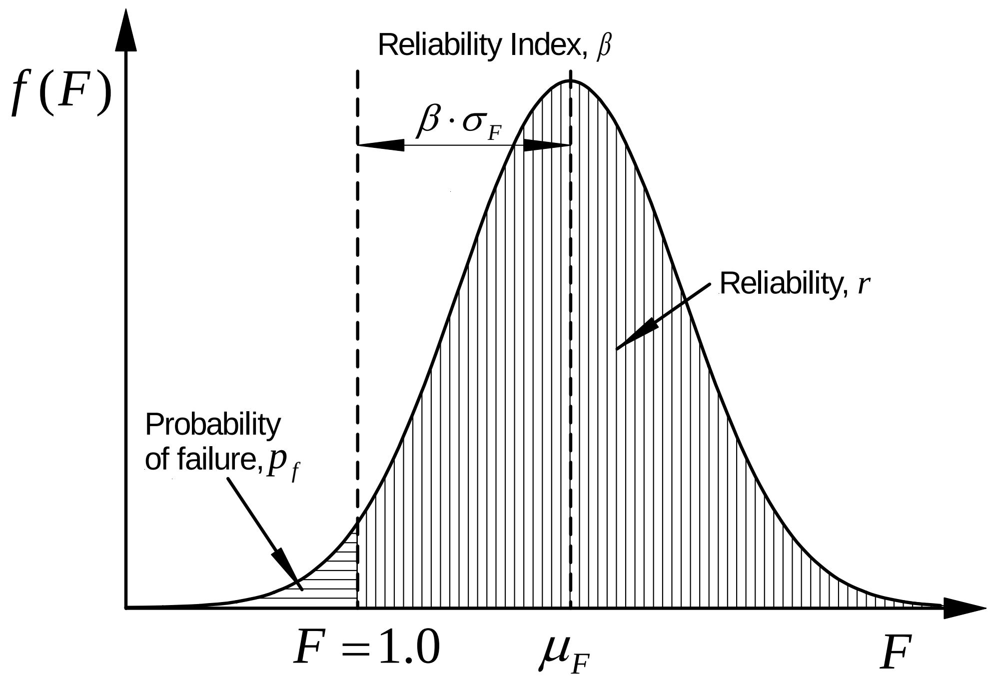



Figure 5.3. Sample distribution of factor of safety illustrating commonly used measures of safety for earth slopes.

### 5.3. Reliability-based Analysis Method

Regardless of whether one chooses to represent the level of safety of a particular design using the probability of failure, the reliability, or the reliability index, the most critical and intensive task for calibration of load and resistance factors is performing the reliability-based analyses required to establish the uncertainty in the factor of safety from the uncertainties in the respective input parameters. Once this is done, any of the probabilistic safety measures can be used to develop appropriate load and resistance factors with equal accuracy, although slightly different computations are involved

Several different methods have been utilized for reliability-based analysis of earth slopes in the past including First-Order, Second Moment (FOSM) methods such as the Taylor Series approximation (Duncan, 2000), the Point Estimate method, and Monte-Carlo simulations among others (USACE, 1999). The most popular method among these has been the Taylor Series approximation, primarily because of its simplicity and as a result of its adoption by the U.S. Army Corps of Engineers (USACE, 1999). Work performed to date to develop LRFD load and resistance factors for other geotechnical applications has generally adopted the First-Order Reliability Method (FORM) as it is more invariant than the simpler FOSM method. If applied appropriately, any of the methods can be used effectively to produce appropriate load and resistance factors.

Both the FORM and, to a lesser extent the FOSM method, are somewhat tedious to apply for slope stability analyses using currently available computer software and procedures. Thus, the more general Monte-Carlo method has been utilized for the vast majority of reliability-bases analyses for this project. This decision was greatly facilitated by the recent availability of a commercial slope stability analysis software package, Slide 5.0™, with capabilities for automating the Monte Carlo analyses so that reliable results could be obtained within a reasonable time period. While the Monte-Carlo method requires significant computational effort, it does not require *a priori* assumptions about the form of the distribution of the factor of safety – a significant assumption that is required using the other methods. Monte Carlo analyses were generally performed using between 1000 and 10,000 simulations depending on the value of the target probability of failure. Latin-hypercube sampling was also utilized to improve the efficiency of the computations and sampling was performed using the Park and Miller v.3 random sampling algorithm implemented in software. Factors of safety were generally computed using Spencer's Method, although other appropriate methods were utilized in some instances.

### 5.4. Calibration Procedure

Once the most appropriate reliability-based analysis procedure and the values of load factors were established, the procedure for probabilistic calibration of load and resistance factors consisted of the following conceptual steps:

1. Select an appropriate target probability of failure (or reliability or reliability index);
2. Establish the range of possible conditions for which designs may be performed including the range of slope angles, slope heights, soil strengths, foundation conditions, etc;
3. For selected cases representative of this range of conditions:
   a. Determine several combinations of means and $COV$ (or standard deviations) of the input parameters that produce the target probability of failure (or reliability or reliability index). This is most effectively done by selecting values of $COV$ that span the range of expected $COV$s for the respective parameters and then varying the mean values to produce the target probability of failure.
   b. Establish the deterministic values of the parameters that produce a factor of safety of unity (1.0).
   c. For each combination of means and standard deviations producing the target probability of failure, compute the appropriate resistance (or load) factor for each input parameter as

   $$\psi(COV) = \frac{x_{\det}}{\mu_x(COV)} \tag{5.2}$$

   where $x_{det}$ is the deterministic value of a particular input parameter producing a factor of safety of 1.0 and $\mu_x$ is the mean value of the same parameter producing the target value of the probability of failure. Since $\mu_x$ is dependent on the $COV$ of the parameter, the computed resistance factor is also a function of this $COV$.
4. Repeat the process for alternative target probabilities of failure as needed.

While the procedure described above is relatively straightforward, it poses a number of practical challenges for the application to slope stability analysis. In particular, Step 3a requires a rather laborious process of trial and error to assume means and standard deviations for parameters, compute a probability of failure for those assumed parameters, and then repeat this process until the appropriate probability of failure is achieved. This process is practically challenging because of the intensive computational effort required to accurately compute the probability of failure using the Monte-Carlo technique, often requiring from an hour to as much as a dozen hours to perform a single Monte-Carlo analysis depending on the target probability of failure. In addition, Step 2 is also challenging for reasons described in the following section.

### 5.5. Selection of Slope Conditions for Calibration

Perhaps the most difficult challenge for calibration of load and resistance factors for slope stability applications is establishing an appropriate range of possible conditions to be evaluated to ensure that the developed factors are appropriate for the conditions that will be encountered. Even for homogeneous slopes, the stability of a slope is dependant on seven parameters that include (Bishop and Morgenstern, 1960):

- the slope height, $H$,
- the slope angle, $\beta$,
- the shear strength parameters, $c$ and $\phi$,
- the soil unit weight, $\gamma$,
- the depth to a “hard stratum”, $d$, and
- the pore water pressure, or one or more parameters representing the pore water pressure.

For stratified sites, the number of parameters increases even further. On the surface, this problem seems almost intractable as an essentially infinite number of scenarios may be encountered. However, the problem can be made more tractable by considering the following:

1. Previous investigators (e.g Bishop and Morgenstern, 1960; Janbu, 1954) have demonstrated that the number of variables that must be evaluated can be reduced using several dimensionless parameters so that the total number of variables can be reduced to four for homogenous slopes in general. In particular, Janbu (1954) has shown that the stability of a homogeneous slope can be represented as a function of the dimensionless parameter $\lambda_{c\phi}$ defined as

$$\lambda_{c\phi} = \frac{\gamma \cdot H \cdot \tan\phi}{c} \quad (5.3)$$

   the slope angle, $\beta$, the pore pressure, and the ratio $c/\gamma H$ (Duncan and Wright, 2005) This result was used extensively in developing an appropriate range of stability cases for this project, as described in more detail below.

2. While the stability of a slope depends on a large number of parameters, the variability and uncertainty in the stability, which dictates appropriate load and resistance factors, may be dominated by relatively few of these parameters. In essence, the safety associated with the mean values of parameters is accounted for within the general LRFD procedure described in Section 3.2 where the factors of safety computed using factored values must be approximately 1.0. The role of the load and resistance factors is to account for the uncertainty or variability of the parameters on top of these mean values.

3. Similarly, it seems likely that the uncertainty introduced in stratified slopes with different levels of uncertainty in the various strata can be conservatively assumed to be equal to or less than the uncertainty for the most uncertain stratum. Thus, in situation with stratified soil profiles, one can conservatively assume that the overall uncertainty in the input parameters for each stratum is equal to the greatest uncertainty for any stratum.

Based on these considerations, the general slope geometry depicted in Figure 5.4 was utilized, with appropriate modifications as warranted, as the basis for calibration of the load and resistance factors in this project. This geometry was selected as a relatively typical geometry of slope that is evaluated by MoDOT. Modifications to this geometry generally included varying the slope height and the slope angle, as well as considering a case where the slope was formed as a vertical earth retaining structure to investigate issues that may be associated with evaluating the overall stability of earth retaining structures. Details regarding the modifications to this slope geometry are described in subsequent sections where appropriate.

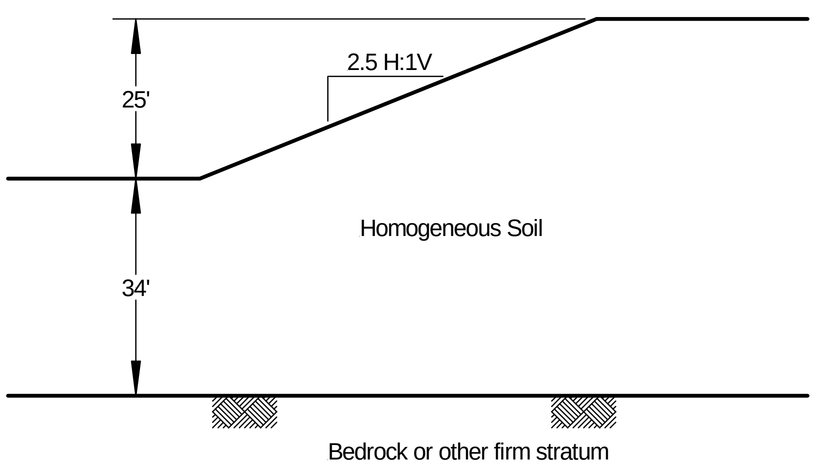



Figure 5.4. General slope geometry utilized for calibration of load and resistance factors in this project.

Beyond the simple geometry, calibrations were performed for a range in values of the parameter $\lambda_{c\phi}$ to account for potential variations in material properties. In general, $\lambda_{c\phi}$ varies from a value of 0 for evaluation of saturated slopes under undrained conditions (i.e. the $\phi$=0 condition) to a value approaching infinity as the Mohr-Coulomb cohesion intercept, $c$, approaches zero. Calibrations were therefore performed for values of $\lambda_{c\phi}$=0 and $\lambda_{c\phi}$=∞. For conditions in which neither $c$ nor $\phi$ can be taken as zero, $\lambda_{c\phi}$ takes on intermediate values. However, results of preliminary analyses indicated that the range of results for different values of $\lambda_{c\phi}$ could be reasonably represented by performing additional calibrations for $\lambda_{c\phi}$ =50 and $\lambda_{c\phi}$ =10 so these values were selected. In all cases, the total unit weight of the soil was taken to be 125 pcf, and except where noted otherwise, the slope height was taken to be 25-ft, so the appropriate values of $\lambda_{c\phi}$ were generally achieved by simply varying the Mohr-Coulomb cohesion intercept and angle of internal friction to produce the appropriate $\lambda_{c\phi}$ value. Finally, since pore water pressures were not included in the proposed implementation of LRFD for slope stability analysis, pore water pressures were simply taken as zero for all calibrations.

In general, there are multiple combinations of the parameters $c$ and $\phi$ that will produce a given $\lambda_{c\phi}$ for given values of the other parameters. As such, the value of $\lambda_{c\phi}$ could be maintained constant when varying the mean values of $c$ and $\phi$ to produce a given target probability of failure (see Step 3c in Section 5.4). However, the value of $\lambda_{c\phi}$ could not be maintained as a constant when performing Monte Carlo simulations to determine the probability of failure, but the variability in $\lambda_{c\phi}$ over the typical ranges of parameters analyzed was generally small compared to the overall range of $\lambda_{c\phi}$ values so this approximation is believed to be acceptable.

### 5.6. Target levels of Reliability

The final issue to be resolved prior to calibration is to establish appropriate target levels of reliability. This issue is a complex one that generally involves evaluation of the potential consequences of failure (e.g. monetary and human risk) and the required investment (i.e. cost) and subsequently striking a balance between the risks and the costs to reduce those risks. Such evaluations are generally beyond the scope of this project. However, it is reasonable to adopt the approach of selecting target probabilities of failure based on historical estimates of reliability for common civil engineering structures. Figure 5.5 shows an example of a diagram illustrating estimated levels of risk posed by common civil facilities. Other similar diagrams are also available for this purpose.

For slope stability analysis cases traditional encountered by state departments of transportation, the potential risks may vary substantially. In cases where risks are relatively low (e.g. largely rural areas where failure is not expected to impact the pavement structure), it may be warranted to accept a higher probability of failure than in cases where risks are higher (e.g. largely urban areas, or cases where failure is likely to impact the pavement structure and pose hazard to users). To provide for a range in potential target reliability (or probability of failure), calibrations were performed for probabilities of failure of $p_f$ =0.1 (1 in 10), $p_f$ =0.01 (1 in 100), and $p_f$ =0.001 (1 in 1000). These values are generally consistent with the “accepted” risk-cost relationship shown in Figure 5.5 considering common civil structures and costs associated with those structures. Future studies involving closer evaluation of potential risks and costs are suggested to further refine target probabilities of failure. Such studies may show a need to develop additional load and resistance factors appropriate for higher or lower target probabilities of failure.

### 5.7. Results of Calibration Analyses

Results of analyses performed to establish appropriate resistance factors for the target probabilities of failure and the slope conditions described above are presented in the following sections. Assumptions common to all analyses include:

1. For consistency with current implementation of LRFD for spread footings and earth retaining structures, the load factors to be applied to the soil unit weight were again taken to be equal to 1.0 in all cases (as was assumed for the calibrations matching existing design procedures). Thus, the only remaining factors to be computed are the resistance factors to be applied to the soil shear strength.

2. Coefficients of variation for the Mohr-Coulomb shear strength parameters $c$ and $\phi$ (or $\bar{c}$ and $\bar{\phi}$) were assumed to be identical.

3. The same resistance factor would be applied to both $c$ and $\phi$ (or $\bar{c}$ and $\bar{\phi}$).

For each series of calculations, appropriate shear strength resistance factors are reported as a function of the *COV* for the appropriate shear strength parameters in the form of charts of resistance factor versus *COV*. Different relations are also reported for the different target probabilities of failure considered.

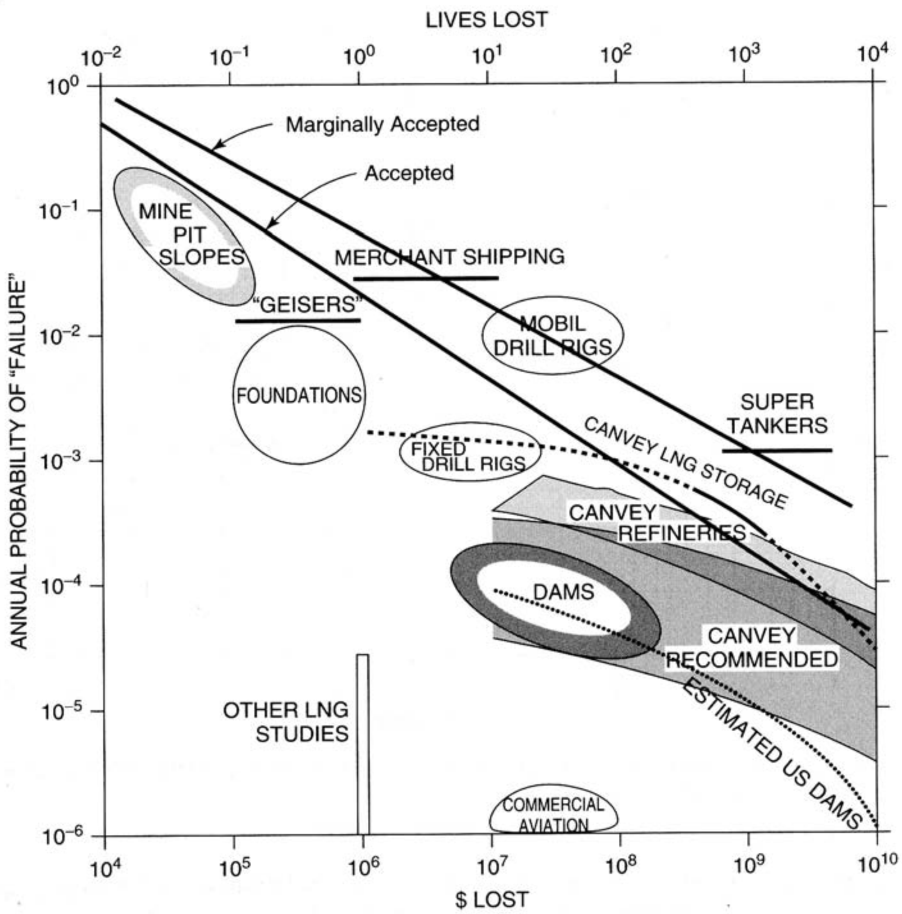



Figure 5.5. Diagram illustrating relationship between annual probability of failure and risk (expressed in terms of $ lost and lives lost) for common civil facilities (from Baecher and Christian, 2003).

5.7.1.Calibrations for $\lambda_{c\phi} = \infty$

The condition of $\lambda_{c\phi} = \infty$ corresponds to any conditions where the Mohr-Coulomb cohesion intercept, $c$ (or $\bar{c}$), is equal to zero. In such cases, the stability of a homogeneous earth slope depends only on the angle of internal friction, $\phi$ (or $\bar{\phi}$), and the inclination of the slope, $\beta$:

$$F = \frac{\tan\phi}{\tan\beta} \tag{5.4}$$

For this simple case, both the mean and standard deviation of the factor of safety can be established analytically, without requiring Monte-Carlo simulations or other probabilistic analyses, if the probability distribution for tan $\phi$ is assumed to be normal. The analytical relation for the mean factor of safety, $\mu_F$, is:

$$\mu_F = \frac{\mu_{\tan\phi}}{\tan\beta} \tag{5.5}$$

while the relation for the standard deviation of the factor of safety, $\sigma_F$, is:

$$\sigma_F = \frac{\sigma_{\tan\phi}}{\tan\beta} \tag{5.6}$$

Given these expressions for the mean and standard deviation of the factor of safety, the combinations of means and standard deviations producing the target probability of failure (i.e. Step 3a) can be computed iteratively using the Microsoft Excel™ function NORMINV.

Figure 5.1 shows the results of the calibration analyses for $\lambda_{c\phi} = \infty$ in graphical form with the coefficient of variation of tan $\phi$ plotted on the horizontal axis and the resistance factor for tan $\phi$ to produce the appropriate target probability of failure plotted on the vertical axis. Relations between the resistance factor, $\psi_{\tan\phi}$, and the coefficient of variation of tan $\phi$ are shown for target probabilities of failure, $p_f$, of 0.1 (1 in 10), 0.01 (1 in 100), and 0.001 (1 in 1000). In all cases, the relations are found to be linear and to indicate rapidly decreasing resistance factors with increasing coefficient of variation. As expected, the required resistance factor is observed to decrease with decreasing target probability of failure for a given coefficient of variation for tan $\phi$. For example, considering a case where the coefficient of variation of tan $\phi$ is 20 percent (0.20), the resistance factor to achieve a probability of failure of 0.1 (1 in 10) is nominally 0.75 while the resistance factor required to achieve a probability of failure of 0.001 (1 in 1000) is 0.40.

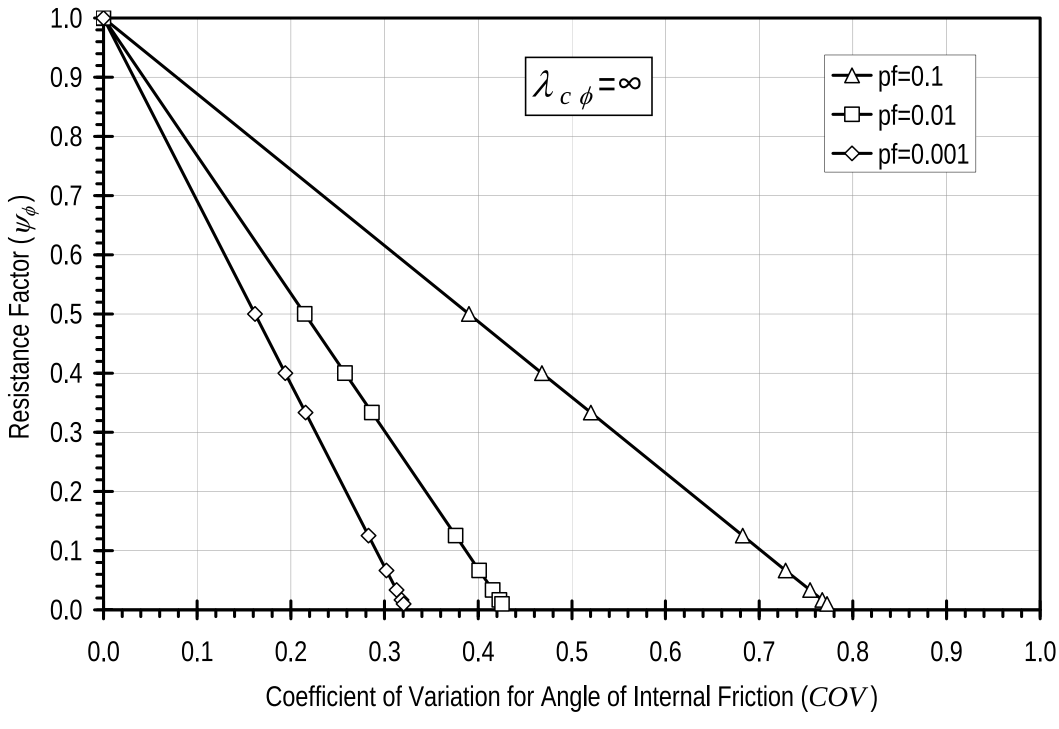



Figure 5.6. Resistance factors relations for tan $\phi$ ($\psi_{\tan\phi}$) for conditions where $\lambda_{c\phi} = \infty$ and $p_f$ = 0.1, 0.01, and 0.001.

5.7.2. Calibrations for $\lambda_{c\phi} = 50$

Figure 5.7 shows the resistance factors determined for conditions where $\lambda_{c\phi} = 50$, for a target probability of failure of 0.01 (1 in 100). The resistance factor relation was again observed to be approximately linear and to show rapidly decreasing resistance factors with increasing uncertainty in the shear strength parameters, $c$ and $\phi$. Recall that these resistance factors are based on the assumption that the uncertainty in both $c$ and $\phi$ is identical and that the same resistance factors are applied to both parameters. The resistance factor relation for $\lambda_{c\phi}$=50 and $p_f$ =0.01 was also observed to be practically identical to the relation establish for $\lambda_{c\phi}$=∞ and $p_f$ =0.01, which leads to the conclusion that the resistance factors for $\lambda_{c\phi}$=∞ are appropriate for values of $\lambda_{c\phi}$ greater than 50. As a result, additional calibrations for $\lambda_{c\phi}$=50 for other probabilities of failure were not performed.

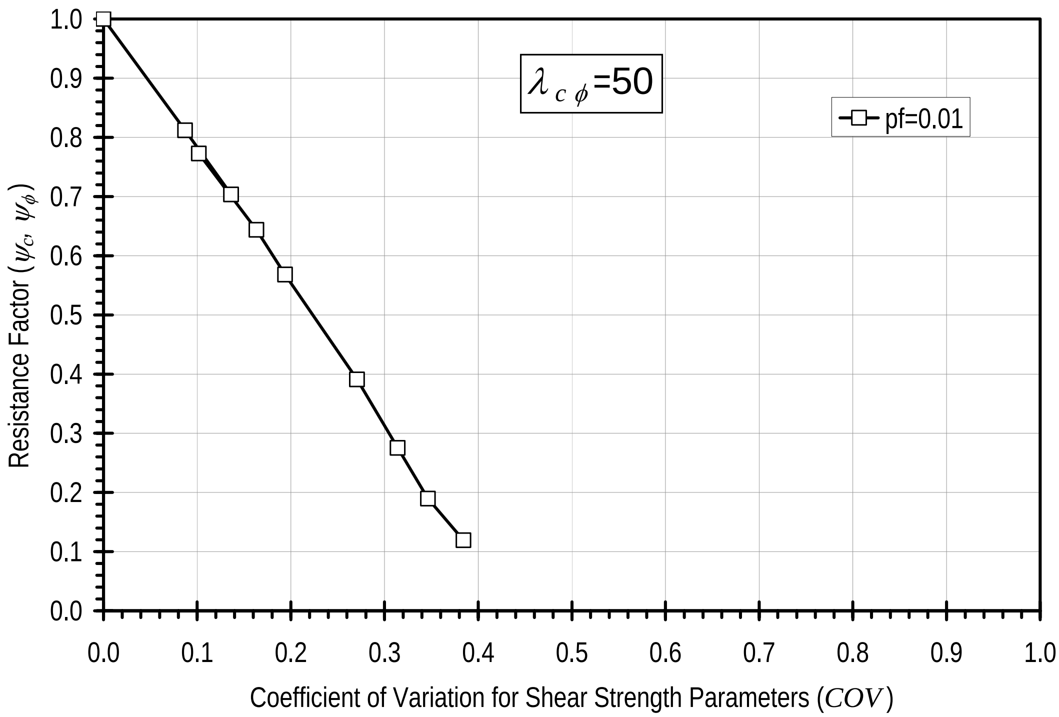



Figure 5.7 Resistance factor relations for $c$ ($\psi_c$) and tan $\phi$ ($\psi_{\tan\phi}$)for conditions where $\lambda_{c\phi}$ = 50 and $p_f$ =0.01.

5.7.3. Calibrations for $\lambda_{c\phi}$ = 10

Figure 5.8 shows the resistance factor relations determined for $\lambda_{c\phi}$ =10 for probabilities of failure of 0.01 and 0.001. The resistance factor relation was again found to indicate rapidly decreasing resistance factors with increasing coefficients of variation. However, in contrast to the relations determined for $\lambda_{c\phi}$ ≥50, the relations for $\lambda_{c\phi}$ =10 were found to be non-linear with the resistance factor tending to decrease at a diminishing rate for very large coefficients of variation. In general, the resistance factors for $\lambda_{c\phi}$ =10 were found to be similar to those found for $\lambda_{c\phi}$ ≥50 at coefficients of variation less than 0.1, but greater than those for $\lambda_{c\phi}$ ≥50 at higher values of coefficients of variation.

5.7.4. Calibrations for $\lambda_{c\phi}$ = 0

Resistance factor relations determined for $\lambda_{c\phi}$ =0 are presented in Figure 5.9 for probabilities of failure of 0.1, 0.01, and 0.001. For these conditions, the relations are found to be even more non-linear that was found for $\lambda_{c\phi}$ =10. In general, the resistance factors determined for $\lambda_{c\phi}$ =0 were greater than or equal to the resistance factors determined for $\lambda_{c\phi}$ ≥10, with the differences being more substantial for larger values of the coefficient of variation.

5.7.5. Sensitivity of Resistance Factors to Slope Angle and Slope Height

The resistance factor relations presented in Figure 5.6 through 5.9 were all established for the slope geometry presented in Figure 5.4. This slope has a height of 25-ft and a slope angle of 2.5(H):1(V). Additional calibration analyses were also performed to evaluate whether the resistance factors determined using this geometry would be appropriate for other slope geometries. In particular, these additional analyses were performed to evaluate the effects of varying the slope height and slope angle.

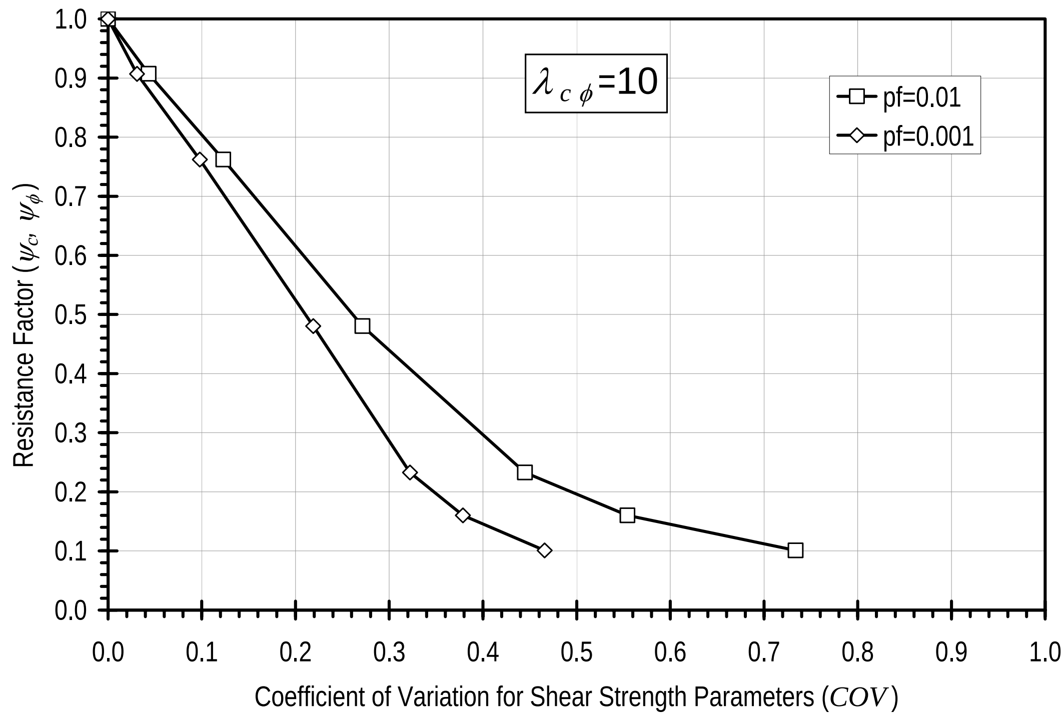



Figure 5.8 Resistance factor relations for $c$ ($\psi_c$) and tan $\phi$ ($\psi_{\tan\phi}$) for conditions where $\lambda_{c\phi}$ = 10 and $p_f$ =0.01 and 0.001.

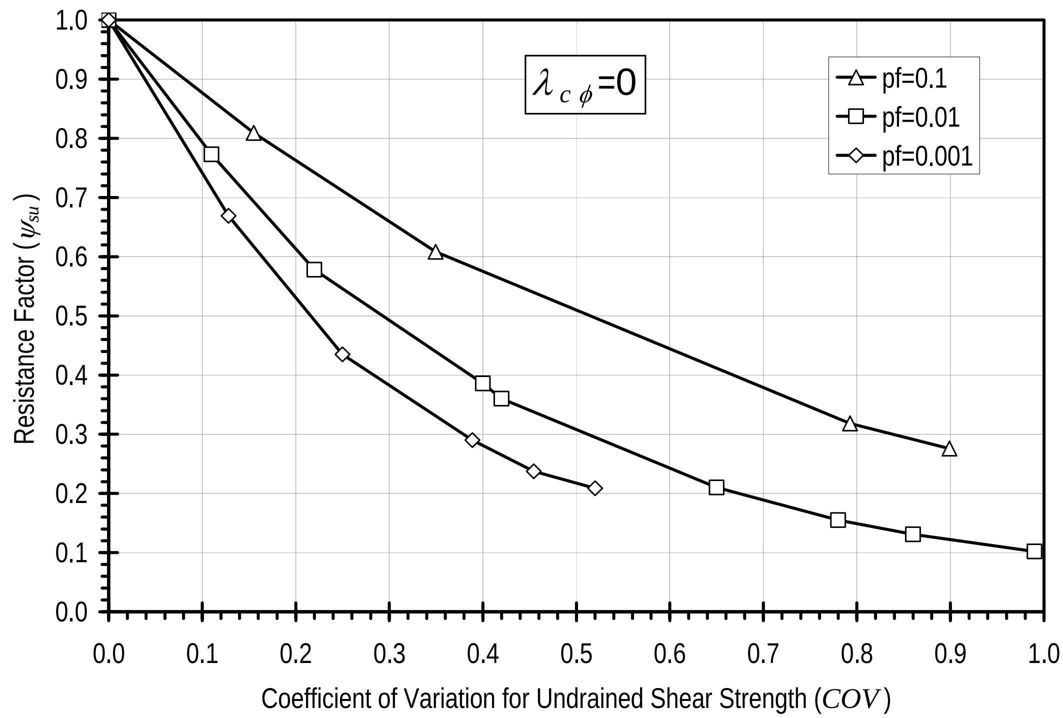



Figure 5.9 Resistance factor relations for undrained shear strength ($\psi_{su}$) for conditions where $\lambda_{c\phi}$ = 0 and $p_f$ = 0.1, 0.01, and 0.001.

Figure 5.10 show the results of calibrations performed for $\lambda_{c\phi}$ =10 and $p_f$ =0.01 (1 in 100) for 25-ft high slopes with slope inclinations of 1.5:1, 2.5:1, and 3.5:1. This range of slope angles is generally representative of the range of slope inclinations routinely encountered by state departments of transportation. The results shown indicate that the effect of slope angle on the computed resistance factors is very limited and certainly within reasonable practical tolerances for computation and selection of resistance factors. Additional evaluations performed for other values of $\lambda_{c\phi}$, including evaluations for vertical slopes, produced similar results. Collectively these results indicate that the computed resistance factors can be utilized for slopes with any slope angle.

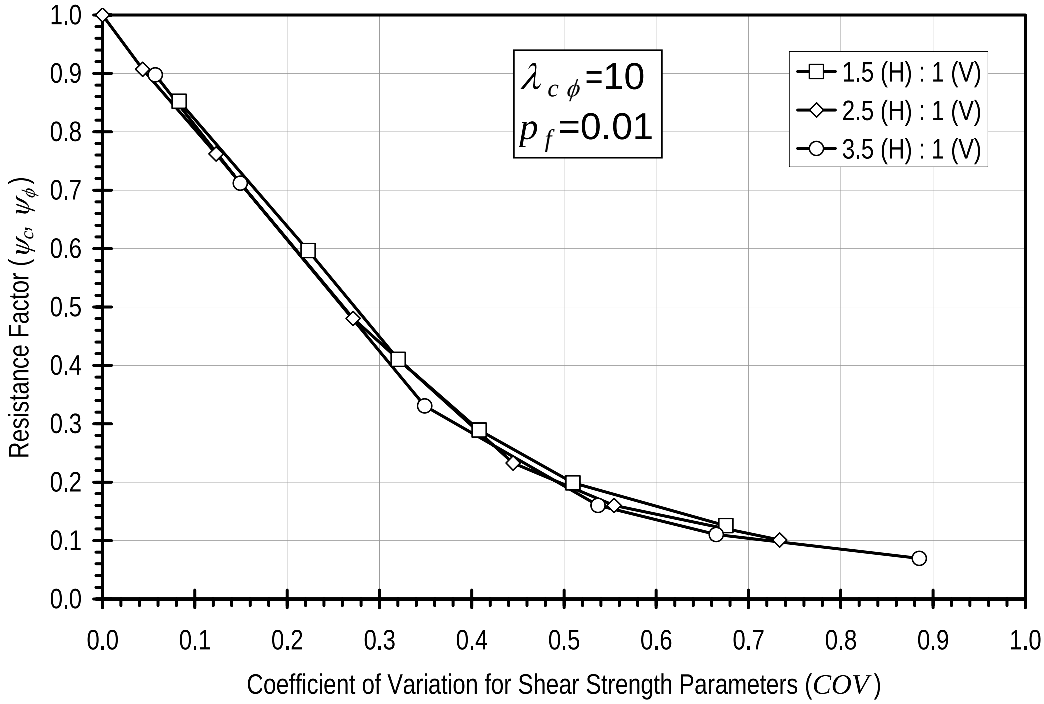



Figure 5.10 Comparison of resistance factor relations for slopes with different slope angles for $\lambda_{c\phi}$=10 and $p_f$ =0.01.

Figure 5.11 shows the results of calibrations performed for $\lambda_{c\phi}$ =0 and $p_f$ =0.01 (1 in 100) for a 2.5:1 slope with slope heights of 12.5-ft, 25-ft, and 50-ft. These results also suggest that the resistance factors provided in Figures 5.6 through 5.9 can be used for slopes of any height, or at least for slope heights of up to approximately 50-ft.

### 5.8. Recommended Presentation of Resistance Factor Relations for Design

The graphs of computed resistance factors presented in Figures 5.6 through 5.9 were developed to facilitate comparison of the resistance factor relations for different target probabilities of failure. While these graphs are useful for this purpose, expected design scenarios are more likely to have a fixed target probability of failure but to require some judgment regarding the appropriate value of $\lambda_{c\phi}$for a particular condition. Furthermore it is likely that many instances will produce values of $\lambda_{c\phi}$ that fall between those for which resistance factor relations have been determined, thus requiring some interpolation between the respective resistance factor relations based on the appropriate value of $\lambda_{c\phi}$ for a particular case. In contrast, interpolation based on a different target probability of failure is unlikely to be necessary.

For these reasons, it is likely that graphs presenting the resistance factor relations for different values of $\lambda_{c\phi}$, but the same value of the probability of failure are likely to be more useful in common design scenarios. Figure 5.12 and 5.13 present just such graphs for probabilities of failure of 0.01 (1 in 100) and 0.001 (1 in 1000), respectively. Since the resistance factor relations for $\lambda_{c\phi}$=50 and $\lambda_{c\phi}=\infty$ are practically identical, only curves for $\lambda_{c\phi}\geq$50, $\lambda_{c\phi}$=10, and $\lambda_{c\phi}$=0 are presented in each graph. Inspection of Figures 5.12 and 5.13 reveals that while there are differences in the relations for different values of $\lambda_{c\phi}$, the relations are close enough to permit interpolation between the respective curves. The figures also indicate that it is generally conservative to assume higher values of $\lambda_{c\phi}$. Finally, the figures also show that the resistance factors for all values of $\lambda_{c\phi}$ are similar for coefficients of variation less than about 0.05 and that differences in the resistance factors for $\lambda_{c\phi}$=10 and $\lambda_{c\phi}$=0 are negligible for coefficients of variation less than about 20 percent for a given probability of failure.

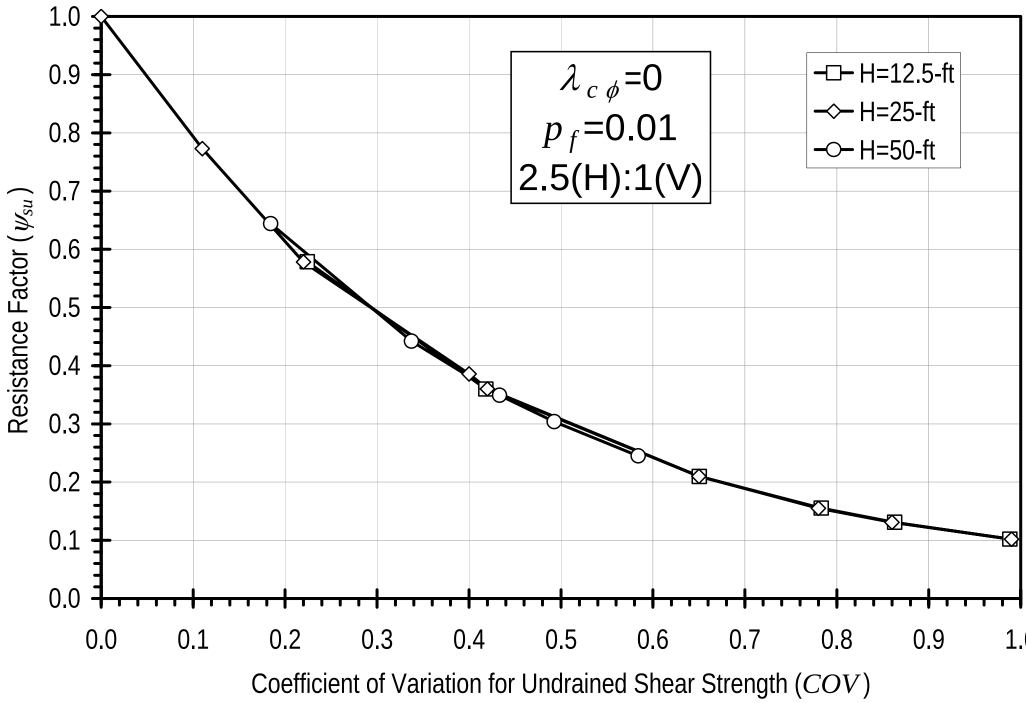



Figure 5.11 Comparison of resistance factor relations for slopes with different slope heights for $\lambda_{c\phi}$=0 and $p_f$ =0.01.

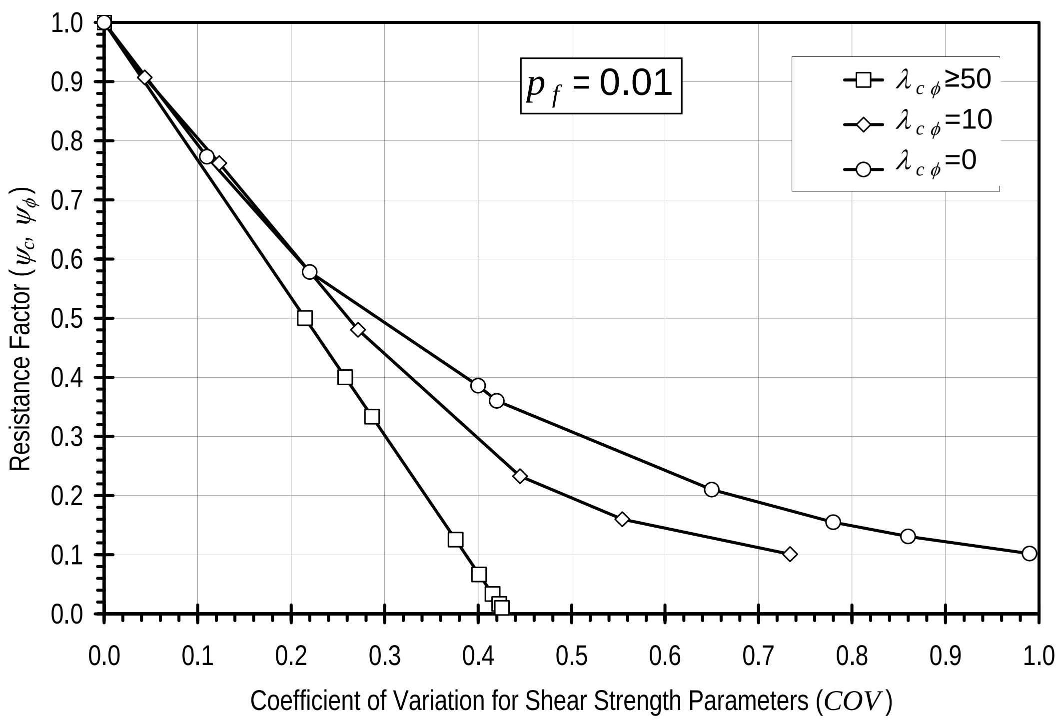



Figure 5.12 Recommended form of “resistance factor chart” for design of earth slopes for probability of failure, $p_f$ =0.01 (1 in 100).

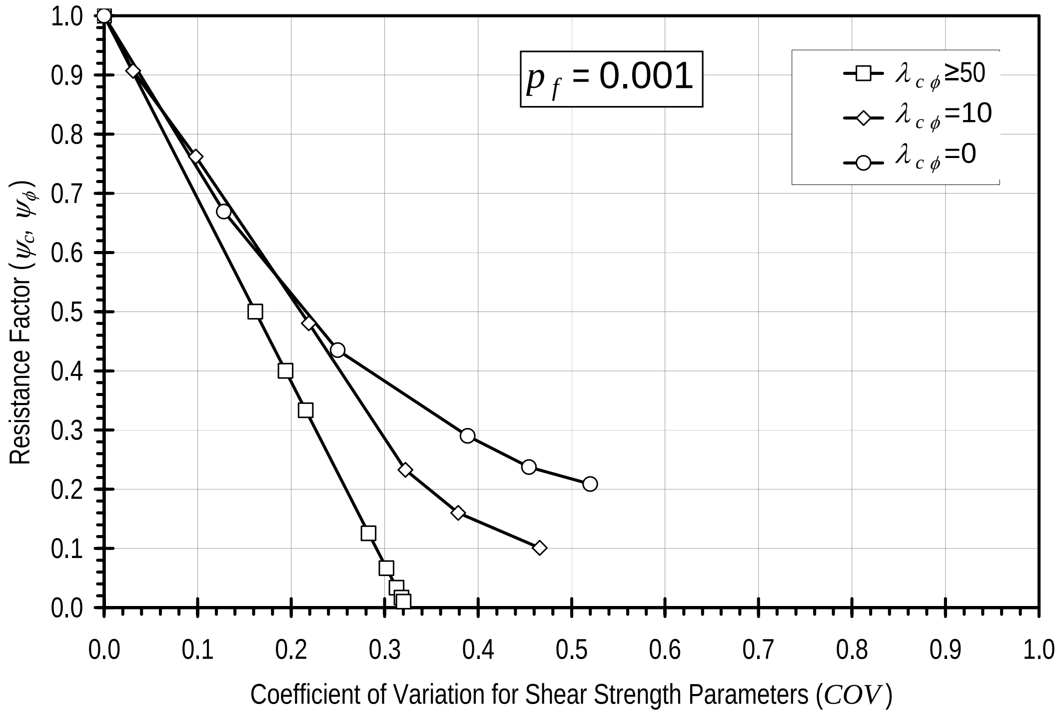



Figure 5.13 Recommended form of "resistance factor chart" for design of earth slopes for probability of failure, $p_f$ =0.001 (1 in 1000).

### 5.9. Example Problem

The example problem described in Section 4.4 and shown in Figure 4.1 was also analyzed using the resistance factors determined from probabilistic calibrations rather than resistance factors determined to match historical ASD methods. The procedure followed is essentially identical to that used for the LRFD analyses described in Section 4.4 except that in the present case the resistance factors are determined from Figure 5.12 or 5.13. The only additional information required for these analyses are estimates of the *COV* of the shear strength parameters, and knowledge of the desired probability of failure for the slope. For this example, it was assumed that the desired probability of failure is 0.01 (1 in 100). It was further assumed that the *COV* of the effective stress strength parameters is 20 percent and that the *COV* for the total stress strength parameters is 50 percent. Both of these values are within the range of values determined for a typical site in St. Louis, Missouri presented in Chapter 6. Since the traditional ASD analyses of the example are identical to those presented in Section 4.4, only the LRFD analyses are presented here.

5.9.1.Evaluation of Short-term Stability Using LRFD Approach (probabilistic factors)

In this case, the *COV* of the total stress strength parameters is 50 percent. From Figure 5.12, for $\lambda_{c\phi}$=0 (f=0) and *COV*=50 percent, the appropriate resistance factor is 0.31. Using this value, the factored shear strength parameters are (from Eq. 3.4):

$$c^* = \psi \cdot c = 0.31 \cdot 1000\,psf = 310\,psf \qquad (4.6)$$

$$\tan\phi^* = \psi \cdot \tan\phi = 0.31 \cdot 0 = 0 \qquad (4.7)$$

Inputting *c*=310-psf and $\phi$=0 into the stability analysis produces a factor of safety of 0.74, which is below the limiting value of 1.0. This indicates that the design is not acceptable (i.e. there is a higher probability of failure than 1 in 100).

5.9.2.Evaluation of Long-term Stability Using LRFD Approach (probabilistic factors)

For the long-term stability condition, effective stress analyses are appropriate. The *COV* for the effective stress strength parameters is 20 percent and $\lambda_{c\phi}$ is approximately 15. Since this $\lambda_{c\phi}$ is reasonably close to 10, the resistance factor relation for $\lambda_{c\phi}$=10 in Figure 5.12 is used, which suggests that the appropriate resistance factor is 0.61. The values of the factored shear strength parameters are thus (from Eq. 3.5):

$$\bar{c}^* = \psi \cdot \bar{c} = 0.61 \cdot 100\, psf = 61\, psf \qquad (4.6)$$

$$\tan\bar{\phi}^* = \psi \cdot \tan\bar{\phi} = 0.61 \cdot \tan(22^\circ) = 0.246 \quad \Rightarrow \bar{\phi}^* = \tan^{-1}(0.246) = 14^\circ \qquad (4.7)$$

Inputting the factored shear strength parameters of $\bar{c} = 61 psf$ and $\bar{\phi} = 14^\circ$ into the stability analysis produces a factor of safety of 0.85. This value is well below the limit value of 1.0, suggesting that the stability of the slope is not acceptable and that the likelihood of failure is greater than 1 in 100.

Results presented in this section for both the short- and long-term stability conditions are contrary to the results obtained for the same stability conditions in Section 4.4. Reasons for the observed differences are discussed in Chapter 7 in relation to the relevance and appropriateness of the resistance factors determined by matching to traditional ASD procedures and those determined using probabilistic analyses.

# 6. Analysis of Site Data

As discussed in previous sections, the proposed procedures for design of earth slopes using LRFD depend on the variation of the soil properties of the slope. It is therefore important to be able to quantify the variation, or uncertainty, in the soil properties used in design. This uncertainty arises from both the inherent variability of soil as a natural material and the variation in the measurement of soil properties. Subsurface investigation and laboratory test results from a site in St. Louis were used as a "typical" site in this study to quantify the uncertainty in soil properties.

## 6.1. Variation in Soil Properties

The variation in soil properties is due to inherent soil variability and to uncertainty in measured soil properties. Inherent soil variability is due to the natural characteristics of soil, whether deposited naturally or placed and modified by human action. The locations of different soil types at a site are never known with certainty, since subsurface exploration only reveals the soil type at discrete points that are usually widely-spaced. There can be many soil types in a single slope, and to make an analysis feasible, the soil types are divided into separate, limited strata. The division between strata need not be horizontal or parallel, and the divisions are rarely obvious or absolute. Given results of subsurface exploration at discrete points, different engineers will construct different soil profiles of a site.

Assigning soil properties to the various soil strata is also difficult, due to the uncertainty in measured soil properties. When samples of soil are taken during an investigation, disturbance occurs as the sample is removed from the subsurface. This disturbance can have a large effect on measured soil strength and other properties. When tests are performed on the soil samples, the test results are not absolute indicators of the soil properties due to the soil disturbance, possible test error, and different interpretations of test results. As with subsurface investigations, soil samples also represent discrete sample points of the soils at the site, and because of the inherent soil variability, properties may vary widely between points, even for the same soil type.

Because of these inherent variations and uncertainty in measurement of soil properties, it is best to consider each soil property as a random variable. A random variable is an uncertainty quantity that can be described by a particular distribution with a mean (expected) value and a standard deviation. In this study, three different continuous distributions were considered to describe the soil properties: uniform, normal, and lognormal distributions. A general example of the probability distribution function (PDF) for each distribution type is shown in Figure 6.1 for a random variable $X$. The normal distribution is a symmetrical, bell-shaped curve with tails that extend to positive and negative infinity. The lognormal distribution is asymmetrical, with a minimum variable valuable of zero and a right-hand tail that extents to positive infinity. The uniform distribution has an equal probability for all variable values between the lower bound, $a$, and the upper bound, $b$.

## 6.2. Analysis of Soil Properties

For this study, subsurface investigation and soil testing results from the new I-70 interchange for the new Mississippi River bridge project in St. Louis (MoDOT Job Number J6U1086) were used as the basis for the analyses. The Missouri Department of Transportation provided the results of the investigation and testing, and project investigators compiled the data and conducted a statistical analysis of the measured soil properties.

The data set contained over 100 soil borings, consisting of descriptions of the soils encountered and corresponding depths. Many of the borings were made by MoDOT crews, but a limited number of borings were made by external crews under contract. Similarly, most of the laboratory testing was performed by MoDOT personnel in the MoDOT Soils and Geology laboratory. However, a limited number of tests were performed by the contractor. Soil properties measured in the field and in the laboratory were available for 1,358 soil samples. The soil properties measured and the numbers of measurements made are shown in Table 6.1.

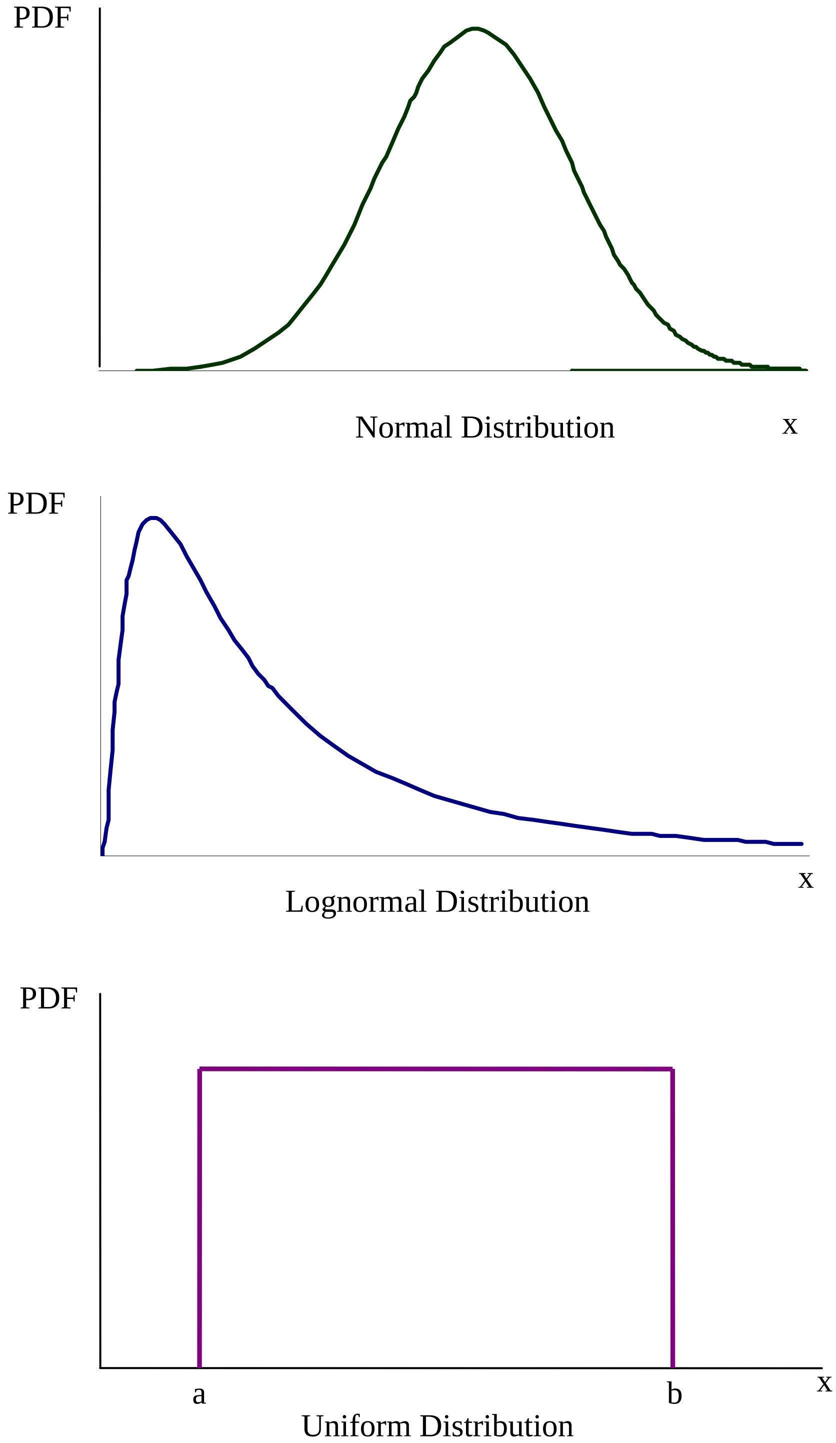



Figure 6.1. Examples of general normal, lognormal, and uniform distributions.

The soil properties were tabulated based on soil type: high-plasticity clay (CH), low-plasticity clay (CL), silt (ML), and silty clay (ML-CL). The soil type for each tested sample was based on the tested soil property results, not the visual classification given in the boring log. For each soil type, the measured values of the soil property were divided into intervals and plotted as histograms. From the shape of each histogram, a distribution type was determined for the soil property. The histograms represent the actual data collected, while the distributions are a theoretical representation of the variation of the soil property. The mean and standard deviation of the data shown in each histogram were also used as the mean and standard deviation for the corresponding distribution.

Table 6.1. Total number of measurements for soil properties.

| Soil Property Measurement | Number of Measurements |
|---|---|
| Liquid Limit | 314 |
| Plasticity Index | 279 |
| Water Content | 476 |
| Pocket Penetrometer Test | 1142 |
| Hand-held Torvane Tests | 667 |
| Unconfined Compressive Strength | 208 |
| Saturated Unit Weight | 111 |
| Effective Stress Friction Angle | 113 |
| Effective Stress Cohesion Intercept | 113 |

Appendices A through D contain the histograms and the corresponding plots of the property distributions. Table 6.2 shows a summary of the mean, standard deviation, coefficient of variation (*COV*), and assigned distribution type for each soil property and each soil type along with published ranges of the *COV* for several parameters. The histograms clearly followed a normal or lognormal distribution pattern for most cases; however, when only a small number of sample points were available to construct a histogram, the distribution pattern is not as clear.

Inspection of Table 6.2, and in particular the *COV* values for the respective parameters, provides useful guidance on the relative variability of the various soil properties analyzed. The property with the least variability is the soil unit weight, which has *COV* ranging from 3 to 5 percent. This low variability is consistent with values reported by Duncan (2000) and supports the adoption of using a load factor of 1.0 for soil unit weight as it has very little influence on the variability in computed factors of safety. The effective stress friction angle is found to have COV ranging from 9 to 19 percent when the cohesion intercept is taken to be zero and from 11 to 27 percent when non-zero cohesion intercepts are used. These values are within or slightly above the range reported by Duncan (2000), and suggest that the effective stress friction angle plays a significant role in the uncertainty in computed factors of safety. *COV* values for unconfined compressive strength and effective stress cohesion intercept are even larger still, often reaching values as high as 80 percent. The COV values determined in this study for these parameters are generally greater than those reported in the literature, which may be a result of procedural differences between the measurements used in this study and those used in other studies, or may be a result of the fact that the site considered in the current study is rather large.

### 6.3. Interpretation of Direct Shear Tests

The direct shear test results were used to find the effective friction angle and the effective cohesion. The plots provided by MoDOT were re-interpreted in two ways: considering cohesion and neglecting cohesion. Determining the values from the plots of the test results is an example of the uncertainty in soil properties due to the interpretation of test results. In this case, a best-fit line is drawn through the three points for each direct shear test. The best-fit line may be drawn differently by different people, and in some cases data points may appear to be erroneous and may be chosen to be neglected in drawing the line. The friction angle is found from the slope of the line and the cohesion is found from the intercept. When cohesion was not considered, the best-fit line was forced through the origin. As shown in Table 6.2, whether or not cohesion was considered did not have a substantial effect on the mean and standard deviation of the friction angle for each soil type.

Table 6.2. Summary of distribution parameters for each soil property and soil type.

| Soil Property and Soil Type | Number of Samples | Mean | Standard Deviation | *COV* (%) | Distribution | Duncan, 2000 *COV* (%) |
|---|---|---|---|---|---|---|
| Plasticity Index | | | | | | |
| CL | 110 | 14.2 | 4.7 | 33 | -- | -- |
| CH | 23 | 35.3 | 6.8 | 19 | -- | |
| ML | 53 | 6.2 | 3.6 | 58 | -- | |
| CL-ML | 13 | 6.0 | 0.9 | 15 | -- | |
| Water Content (%) | | | | | | |
| CL | 194 | 26.9 | 5.9 | 22 | -- | -- |
| CH | 39 | 29.0 | 14.3 | 49 | -- | |
| ML | 84 | 28.1 | 6.7 | 24 | -- | |
| CL-ML | 21 | 27.5 | 27.5 | 12 | -- | |
| Unconfined Compressive Strength, $q_u$, from Pocket Penetrometer (tsf) | | | | | | |
| CL | 520 | 1.09 | 1.07 | 98 | Lognormal | -- |
| CH | 76 | 1.86 | 1.09 | 59 | Lognormal | |
| ML | 195 | 1.04 | 1.10 | 106 | Lognormal | |
| CL-ML | 93 | 0.78 | 0.78 | 99 | Lognormal | |
| Undrained Shear Strength, $s_u$, from Torvane (tsf) | | | | | | |
| CL | 256 | 0.40 | 0.20 | 49 | Lognormal | -- |
| CH | 35 | 0.69 | 0.26 | 37 | Lognormal | |
| ML | 125 | 0.30 | 0.19 | 64 | Lognormal | |
| CL-ML | 28 | 0.29 | 0.18 | 61 | Lognormal | |
| Unconfined Compressive Strength, $q_u$ (tsf) | | | | | | |
| CL | 22 | 0.76 | 0.61 | 80 | Lognormal | 13-40 |
| CH | 7 | 1.23 | 0.72 | 58 | Lognormal | |
| ML | 11 | 0.59 | 0.26 | 43 | Lognormal | |
| CL-ML | 3 | 0.87 | 0.64 | 74 | Lognormal | |
| Saturated Unit Weight, $\gamma_{sat}$ (pcf) | | | | | | |
| CL | 60 | 123.1 | 4.1 | 3 | Normal | 3-7 |
| CH | 4 | 119.6 | 3.4 | 3 | Normal | |
| ML | 33 | 123.6 | 5.6 | 5 | Normal | |
| CL-ML | 5 | 126.6 | 4.2 | 3 | Normal | |
| Effective Stress Friction Angle, $\phi'$ (degrees) | | | | | | |
| CL | 39 | 26.5 | 4.9 | 19 | Normal | 2-13 |
| CH | 9 | 21.8 | 5.9 | 27 | Normal | |
| ML | 15 | 30.5 | 3.4 | 11 | Normal | |
| CL-ML | 3 | 27.2 | 4.0 | 15 | Normal | |
| Effective Stress Cohesion Intercept, *c*' (psf) | | | | | | |
| CL | 39 | 167.1 | 126.4 | 76 | Lognormal | -- |
| CH | 9 | 397.6 | 258.3 | 65 | Lognormal | |
| ML | 15 | 103.8 | 84.2 | 81 | Lognormal | |
| CL-ML | 3 | 121.7 | 75.2 | 62 | Lognormal | |
| Effective Stress Friction Angle, $\phi'$ (degrees) with Zero Cohesion | | | | | | |
| CL | 39 | 27.9 | 4.5 | 16 | Normal | 2-13 |
| CH | 9 | 25.5 | 5.0 | 19 | Normal | |
| ML | 15 | 31.4 | 2.8 | 9 | Normal | |
| CL-ML | 3 | 28.3 | 3.5 | 12 | Normal | |

### 6.4. Determination of Distribution Type

When a large number of samples were available for a particular soil property measurement and a particular soil type, the distribution pattern was easily classified as either a normal or lognormal distribution. However, in cases with only a small number of samples for a particular soil type, the distribution pattern was more difficult to determine. These cases are as follows:

- Unconfined Compressive Strength ($q_u$): 4 samples for high plasticity clay (CH), 10 samples for silt (ML), and 3 samples for silty clay (CL-ML). For these soil types, a lognormal distribution was assigned since a lognormal distribution was indicated for the 17 samples of low plasticity clay (CL).
- Saturated Unit Weight: 5 samples for silty clay (CL-ML). A normal distribution was used for this case since a normal distribution was indicated by the histograms of the other soil types.
- Effective Stress Friction Angle (with and without cohesion): 9 samples for high plasticity clay (CH) and 3 samples for silty clay (CL-ML). A normal distribution was used for both of these soil types since a normal distribution was indicated by the histograms of the CL and ML soil types (39 and 15 samples, respectively).
- Effective Cohesion: 9 samples for high plasticity clay (CH) and 3 samples for silty clay (CL-ML). A lognormal distribution was used for both of these soil types since a lognormal distribution was indicated by the histograms of the CL and ML soil types (39 and 15 samples, respectively).

For any of the cases listed above with small samples sizes, a uniform distribution could have been used to describe the data shown on the histograms. However, since normal or lognormal distributions were indicated for these properties with other soil types, the same distribution was also assigned to the soil types with small sample sizes.

### 6.5. Further Data Analysis Required

The data used in this study were from a single site, located near the Mississippi River in St. Louis, and the results are presented as a “typical” example of variability for a site. While the results presented are typical, it should be expect that results for other sites may differ from those presented because of differences in site conditions, site size, testing procedures, etc. Additional work to analyze data from other sites should therefore be undertaken to better establish collective “baseline” probabilistic parameters (i.e. for all Missouri soils by analyzing the data collectively) as well as to establish appropriate ranges for parameters at individual sites (i.e. by analyzing the data on a site by site basis).

# 7. Implementation

This chapter contains discussion of issues related to implementation of LRFD for analysis of slope stability along with a recommended plan for implementing LRFD for slope stability analysis by the Missouri Department of Transportation. The resistance factors established by matching to current ASD procedures are also compared to those established by probabilistic calibrations, and implications of these comparisons based on the variability of key properties presented in Chapter 6 are discussed.

## 7.1. Comparison of Resistance Factors from Alternative Calibrations

Figure 7.1 shows a comparison of the shear strength resistance factors established by calibrations to match ASD procedures with those established through probabilistic calibrations for a probability of failure of 0.01 (1 in 100). The computed resistance factors established through the ASD matching process ranged from a minimum of 0.67 to a maximum of 1.00 depending on current procedures; only the minimum value is shown in the figure for clarity. Recall that in both cases, the load factor to be applied to soil unit weight was assumed to be 1.0. The following sections provide discussion of several observations that can be made regarding the respective resistance factors.

### 7.1.1. Sensitivity of Resistance Factors to Variability and Uncertainty

The resistance factors established by matching to ASD procedures are independent of the COV of the input parameters. This is the critical limitation of current ASD procedures and of LRFD procedures utilizing load and resistance factors established by matching to ASD procedures. The result of this lack of dependence on the uncertainty in the parameters is that some cases are inevitably designed with excessive levels of safety (reliability), while others are designed with inadequate levels of safety.

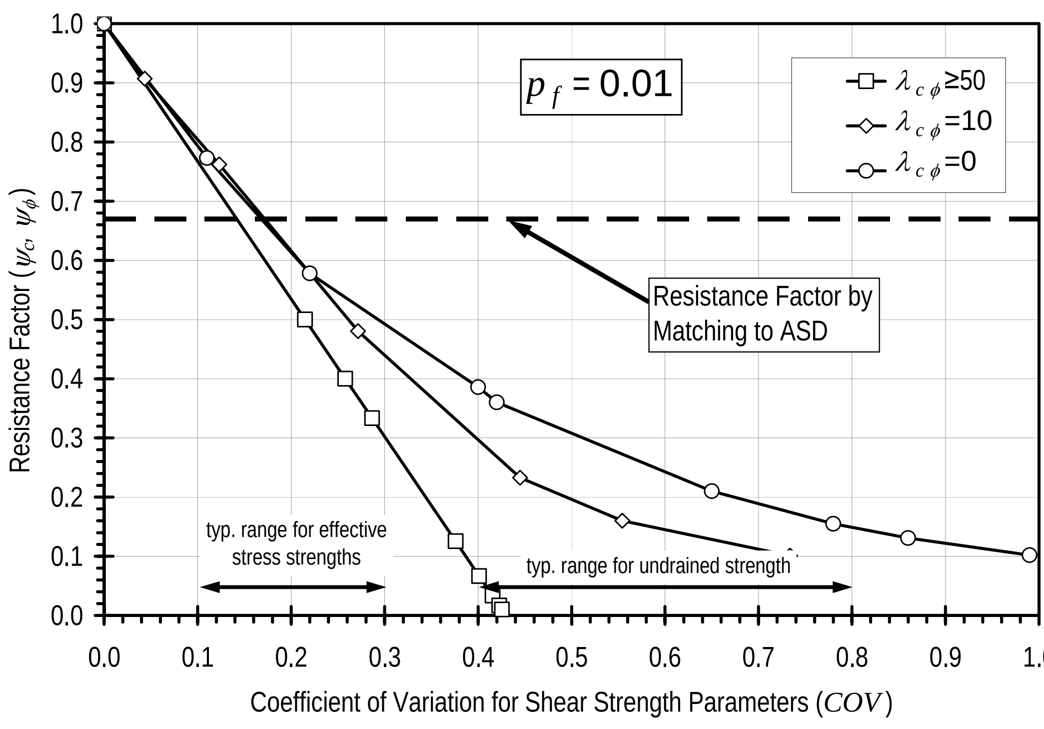



Figure 7.1 Comparison of resistance factor relations determined from probabilistic calibrations with relation established by matching to ASD for target $F$=1.5.

Conversely, the resistance factors established by probabilistic calibrations are highly sensitive to the COV of the shear strength parameters as evidenced by the steep inclination of the resistance factor relations. This sensitivity to COV provides strong motivation for adoption of LRFD analysis procedures that account for the variability of soils at a site so that consistent levels of safety can be achieved from site to site.

#### 7.1.2. Target Probability of Failure

The resistance factors established by matching to ASD procedures are not associated with a particular target probability of failure. Rather, they are associated with a generally unknown level of safety that has been developed over time based on empirical observations and adjustments. However, as a result of the independence of these procedures to COV, the level of safety is not consistent from case to case and depends heavily on the level of site investigation, quality of sampling and testing, interpretation of test results, and judgment of the analyst. Furthermore, comparisons of resistance factor relations for different target probabilities of failure in Figures 5.6, 5.8, and 5.9, all show that the appropriate resistance factors are also sensitive to the target probability of failure (as is expected). As such, even though existing procedures have been found to be "acceptable" in the sense that notable failures attributed to problems with current design procedures have not been observed, it is highly likely that this result arises from substantial levels of hidden conservatism that is likely not warranted for all sites or conditions.

#### 7.1.3. Relative Magnitude of Shear Strength Resistance Factors

Finally, the resistance factors established by probabilistic calibrations are observed to often be substantially lower than those obtained by matching to ASD procedures, particularly for higher values of COV for the shear strength parameters. A simple, but incorrect explanation for these differences could be that current procedures are simply less conservative than what is suggested by the probabilistic calibrations. However, if this were true then MoDOT would be observing extremely high failure rates (on the order of 1 in 10, or at least greater than 1 in 100) for many of its slopes. Since such high failure rates have not been observed, this simple explanation is simply not justified.

A much more plausible explanation lies in the affiliated tasks and procedures that are associated with design and analysis of earth slopes. It was previously noted in Chapter 4 that the resistance factors determined by matching to current ASD procedures were tied to specific procedures for site investigation, laboratory testing, test interpretation, and other factors because the procedures introduce some inherent conservatism into the analyses. The resistance factors determined through probabilistic calibrations are not subject to these restrictions and, in fact, the nominal unfactored parameters used for the probabilistic calibrations are really "true field" values for the parameters rather than laboratory estimates of these parameters. For example, it is widely known that laboratory measurements of the undrained shear strength (say, using unconfined compression tests) are generally substantially lower than the undrained shear strength of the in-situ soil in the field because of effects of sample disturbance, testing method, and other effects. Such effects can result in laboratory measured strengths that are frequently less than the field undrained shear strength by a factor of 2 or greater, which introduces a substantial bias into the analysis. Additional bias can also be introduced by the designer through the practice of selecting strength parameters that are actually somewhat lower than the means of the measured values. While "true field" values are in fact what is preferably used for design, the fact of the matter is that such values cannot be realistically obtained (although improvements can be made). The result is that the probabilistic calibrations are missing some presently unquantified bias that must be introduced to account for current practices for site investigation, testing, interpretation, and analysis.

Because of the bias introduced by current practices, the overall magnitudes of the resistance factors determined using the two alternative calibration procedures cannot be directly compared as presented. A more proper comparison requires that the procedural bias either be removed from the resistance factors obtained from the ASD calibrations or incorporated into the resistance factors obtained from probabilistic calibrations. Such corrections were not performed as part of this work because sufficient data are not presently available to allow the magnitude of the bias to be quantified at the present time. As a result, use of the reported resistance factors is likely to lead to excessive conservatism until they can be revised to incorporate this bias.

#### 7.1.4. Relative Merits of Resistance Factors from Alternative Calibrations

At the present time, neither the resistance factors determined by calibration to current ASD procedures nor the resistance factors established by probabilistic calibrations are completely satisfactory.

The resistance factors determined by calibration to ASD procedures suffer from lack of dependence on the uncertainty involved at a particular site, which limits the benefits which can be realized by transitioning to LRFD analysis. However, these resistance factors inherently incorporate any bias involved with current site investigation and analysis procedures, even if such bias has not been quantified. Nevertheless, the principal benefits of LRFD cannot be realized by adopting these resistance factors in the long-term. Conversely, the resistance factors established by probabilistic calibrations are more fundamentally sound and provide the potential to realize the primary benefits of LRFD – that of producing consistent and appropriate levels of safety regardless of the uncertainty present in the slope conditions. However, the fact that the bias has not yet been quantified or incorporated into the developed resistance factors requires that additional work be performed before these benefits can be realized.

Several additional impediments to implementation of the probabilistically calibrated resistance factors at this time include:

- The resistance factors established by probabilistic calibrations have yet to be tested in actual design scenarios. While it is believed that the proposed procedure is general enough to endure actual implementation (aside from the bias issue), prudent engineering dictates that some additional evaluations be performed to identify any critical practical issues with implementation. These evaluations should include consideration of sites with complex subsurface stratigraphy and with a variety of pore pressure conditions.
- Procedures for establishing appropriate values of the COV for the input parameters based on results of current site investigations have yet to be developed at this time. While some general guidance is available in the literature, there is a dire need for clear and specific procedures for establishing appropriate quantifications of uncertainty for specific sites.
- Evaluations presented above have demonstrated the insensitivity of the probabilistically calibrated resistance factors to both slope angle and slope height. However, no evaluations have been performed to evaluate sensitivity to pore water pressure conditions. While it is believed that the resistance factors will be found insensitive to the magnitude of pore water pressures, some evaluation of this issue is warranted.

None of these issues present insurmountable obstacles to implementation, but they do necessitate that additional work be performed prior to being able to take full advantage of the promise offered by LRFD. It should also be noted that these additional evaluations are also necessary for, and will be beneficial to implementation of LRFD procedures for other geotechnical applications such as foundations and earth retaining structures.

### 7.2. Recommended Process for Implementation of LRFD for Slope Stability Analysis

Based on the considerations described above, it is clear that implementation of LRFD for slope stability analysis using probabilistically calibrated resistance factors is not possible at the present time. However, the results of these calibrations clearly demonstrate the substantial benefits that can be achieved by continuing to work to get such factors implemented. Therefore, since immediate implementation of resistance factors calibrated to match current ASD procedures is possible and since such implementation serves as an important step towards eventual "complete" implementation of LRFD procedures (including probabilistically calibrated load and resistance factors), a staged implementation process is recommended for immediate implementation. This process should include the following specific actions on behalf of MoDOT:

1. Begin implementation of general LRFD procedure using load and resistance factors established by matching to current ASD procedures, as provided in Table 4.1.
2. Simultaneously engage in activities necessary to transition to load and resistance factors established through probabilistic calibrations. These activities should include, at a minimum, the following:
    a. Development of appropriate procedures for establishing COV for the respective input parameters from information currently evaluated in routine site investigations;
    b. Evaluation of bias in key parameters following current site investigation and testing procedures; and

   c. Evaluation/verification of probabilistic load and resistance factors for a series of actual project cases, especially those involving complex stratigraphy and pore pressure conditions.

3. As these activities are completed, adopt revised load and resistance factors based on probabilistic calibrations.

By adopting this process, MoDOT can immediately begin to realize some of the benefits of LRFD, can begin to become accustomed to the alternative design procedures, and over time can adapt the immediate implementation to better take advantage of the substantial benefits of adopting the LRFD approach. A reasonable time frame for completing this entire process is approximately two years.

# 8. Summary, Conclusions, and Recommendations

This chapter provides a summary of the report, followed by a series of major conclusions reached based on the results of this work. In addition, a series of recommendations are provided to facilitate implementation of the LRFD approach for analysis of earth slopes, and to effect improvements to these procedures over time.

## 8.1. Summary

This report has described activities performed to develop procedures for design of earth slopes using the Load and Resistance Factor Design (LRFD) approach. The basic procedure is similar to existing Allowable Stress Design (ASD) procedures in many respects. The major differences between current ASD procedures and the proposed LRFD procedure involves the following activities:

1. Use of “factored” values for common input parameters for slope stability analyses instead of the unfactored, or nominal input parameters currently used.
2. Use of a stability criterion corresponding to a factor of safety of 1.0 rather than a specified limit or target factor of safety as is currently used in ASD procedures.

In a general sense, the intent of the respective ASD and LRFD procedures is the same – to incorporate an appropriate level of conservatism into the design. However, the two procedures differ in how the conservatism is incorporated into the procedures. In the ASD approach, conservatism is generally incorporated through the use of a single “factor of safety” that is intended to account for uncertainties in all design parameters, although additional (often unknown) conservatism may also be incorporated in the procedures through selection of design parameters. Conversely, in the LRFD procedure, conservatism is incorporated through “load factors” and “resistance factors” that are applied to each input parameter. The result of separating the conservatism through the various input parameters is that more consistency can be achieved across a variety of sites and structures, thereby leading to more efficient and effective designs and better use of limited construction funds.

A necessary step for implementing LRFD is to “calibrate” the established load and resistance factors for the respective input parameters. Two alternative approaches, each with distinct advantages and disadvantages, can be utilized for this purpose. Both approaches were utilized in this project. The first and simplest approach is to calibrate the load and resistance factors so that designs produced using LRFD procedures are essentially identical to those produced using current ASD procedures. Results of calibrations of this type are presented in Chapter 4. The advantages of this approach include that it is relatively simple, that it requires no additional information or input beyond what is already required for ASD procedures, and that it makes use of the largely successful application of current procedures over many years. The primary disadvantage of calibrating by matching to current ASD procedures is that it limits the most significant benefits that can be realized by more elaborated implementations of LRFD. These benefits include producing *consistent* and *appropriate* levels of safety across a broad range of design conditions, regardless of the variability of a site, and providing explicit information to designers to allow them to better balance costs associated with additional investigations and costs associated with employing additional conservatism in design. Nevertheless, these calibrations serve as a necessary first step towards transitioning to preferred LRFD procedures and also serve as a baseline for evaluation of alternative calibrations.

The second approach to calibration of load and resistance factors for LRFD is to calibrate load and resistance factors so that they produce a consistent level of safety (reliability) regardless of the uncertainty in the input parameters. This approach is termed probabilistic calibration. Results of such probabilistic calibrations are presented in Chapter 5. While these calibrations are substantially more complex and labor intensive than those following the simpler method, they permit the true promise of LRFD to be realized when fully implemented. In particular, these calibrations provide crucial information to designers on the relationship between the level of uncertainty in design parameters and the appropriate level of conservatism that should be used to achieve a given level of safety (target reliability). Regardless of which calibration procedure is adopted, once the appropriate load and resistance factors are established, the general procedure for application of LRFD is essentially identical except for the need to quantify the level of uncertainty present for applications where load and resistance factors are a function of uncertainty. Results of evaluations performed for one site to facilitate quantification of uncertainty as part of this project were presented in Chapter 6.

While the results presented in this report clearly demonstrated the substantial advantages of implementing LRFD procedures with load and resistance factors determined from probabilistic calibrations, several impediments to immediate implementation of such factors currently exist as described in Chapter 7. First and foremost among these impediments is lack of quantification of bias introduced into the slope stability analysis input parameters by current site investigation, testing, and analysis procedures. As a result of these impediments, but with full realization of the notable benefits of implementing LRFD procedures, a staged implementation plan is recommended. This plan consists of first implementing LRFD using load and resistance factors established by calibration to ASD procedures while simultaneously working to address the impediments to implementation of load and resistance factors established through probabilistic calibrations. This staged process will provide for immediate realization of some of the benefits of LRFD while simultaneously allowing personnel to become accustomed to the change in procedures. Then, as the impediments to implementation of preferable load and resistance factors are addressed, revised load and resistance factors can be adopted without major procedural changes to the general design process and the full benefits of LRFD can begin to be realized.

### 8.2. Conclusions

A number of significant conclusions can be drawn from the results of this project. The most notable conclusions are listed below:

1. The procedure proposed for performing slope stability analysis using LRFD concepts is feasible and provides significant potential for producing consistent levels of safety for slopes located on a broad range of sites.
2. Load and resistance factors established by matching to current ASD procedures serve as an effective preliminary step for adoption of LRFD for slope stability analysis. However, use of these load and resistance factors will limit the benefits that can be achieved through adopting LRFD.
3. Load and resistance factors established through probabilistic calibrations demonstrate that appropriate resistance factors are highly sensitive to the level of uncertainty present at a given site. As such, full realization of the benefits of LRFD requires that probabilistically calibrated load and resistance factors be eventually adopted.
4. Several impediments to implementation of probabilistically calibrated load and resistance factors exist that prevent current implementation of these factors. However, these impediments can be realistically addressed through additional research.
5. A staged implementation plan consisting of preliminary implementation of LRFD using load and resistance factors determined by calibration to existing ASD procedures followed by subsequent revision of these load and resistance factors to those determined by probabilistic calibrations is a reasonable approach to achieve the full benefits of LRFD in a relatively short period of time.

### 8.3. Recommendations

The primary recommendation from this work is to adopt a staged implementation plan to implement the proposed LRFD procedure for design of earth slopes. The plan consists of the following steps:

1. Begin implementation of general LRFD procedure using load and resistance factors established by matching to current ASD procedures, as provided in Table 4.1.
2. Simultaneously engage in activities necessary to transition to load and resistance factors established through probabilistic calibrations. These activities should include, at a minimum, the following:
    a. Development of appropriate procedures for establishing COV for the respective input parameters from information currently evaluated in routine site investigations;

   b. Evaluation of current bias in key parameters following current site investigation and testing procedures; and
   c. Evaluation/verification of probabilistic load and resistance factors for a series of actual project cases, especially those involving complex stratigraphy and pore pressure conditions.
3. As these activities are completed, adopt revised load and resistance factors based on probabilistic calibrations.

Additional recommendations for future work to expand or improve upon the proposed procedure include:

1. Develop additional LRFD procedures to incorporate pore pressures or associated design parameters into the LRFD procedural framework. This is expected to be a difficult task as many of the pore pressure conditions routinely encountered by state departments of transportation are not readily amenable to probabilistic evaluation. Nevertheless, pore pressures represent a significant source of uncertainty in many slope stability analysis cases so the problem warrants significant attention.
2. The potential for using load factors for soil unit weight other than 1.0 should be evaluated. As described in this report, the uncertainty in soil unit weight is generally small compared to the uncertainty in other input parameters. However, unit weight does introduce some uncertainty and therefore should be evaluated more closely to see if other load factors are justified. In these evaluations attention should be paid to conditions were soil unit weight may act as a resistance, in addition to a load (e.g. toe buttresses, etc).
3. LRFD calibrations should be expanded to include additional input parameters such as surcharge loads, reinforcement loads, etc. In expanding the procedure, care should be applied to ensure that developed load and resistance factors are as consistent with existing load and resistance factors for other applications (e.g. shallow foundations, retaining walls, etc.) as possible.
4. Specific guidance and procedures to facilitate establishing appropriate levels of uncertainty for a particular condition/site should be developed. Work on this important problem is underway, but significantly more attention is required if the full benefits of LRFD are to be realized. This work should also provide benefits to adoption of LRFD procedures for other geotechnical applications as well.
5. Work should be undertaken immediately to begin to quantify the bias inherent in current procedures for site investigation, testing, and analysis related to the stability of slopes. Such work is critical to future implementation of probabilistically calibrated load and resistance factors, and will be useful for implementation of LRFD for other applications as well.
6. Additional evaluations of the proposed probabilistic load and resistance factors for actual design cases should be undertaken to identify any unforeseen practical issues with implementation of these factors. Such evaluations are critical to timely implementation of the probabilistically calibrated factors and timely realization of the full benefits of LRFD for slope stability analysis.

# References

AASHTO (1994), *LRFD Highway Bridge Design Specifications*, American Association of State Highway and Transportation Officials, Washington, D.C., First Edition.

Baecher, G.B., and J.T. Christian (2003), *Reliability and Statistics in Geotechnical Engineering*, John Wiley and Sons, West Sussex, England.

Bishop, A.W., and N. Morgenstern (1960), "Stability Coefficients for Earth Slopes," *Geotechnique*, Institution of Civil Engineers, Vol. 10, No. 4, pp. 129-147.

Duncan, J.M., M. Navin, and K. Patterson (1999), *Manual for Geotechnical Engineering Reliability Calculations*, Virginia Polytechnic Institute and State University Center for Geotechnical Practice and Research, Blacksburg, Virginia.

Duncan, J.M. (2000), "Factors of Safety and Reliability in Geotechnical Engineering," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 126, No. 4, pp. 307-316.

Duncan, J.M., and S.G. Wright (2005), *Soil Strength and Slope Stability*, John Wiley and Sons, Hoboken, New Jersey.

FHWA (2001), *Load and Resistance Factor Design (LRFD) for Highway Bridge Substructures*, Reference manual and participant notebook for NHI Course No. 13068, FHWA HI-98-032.

Janbu, N. (1954), *Stability Analysis of Slopes with Dimensionless Parameters*, Harvard Soil Mechanics Series, No. 46, Harvard University, Cambridge, Massachusetts.

Paikowsky, S.G. (2004), *Load and Resistance Factor Design (LRFD) for Deep Foundations*, NCHRP Report 507, Transportation Research Board, Washington, D.C.

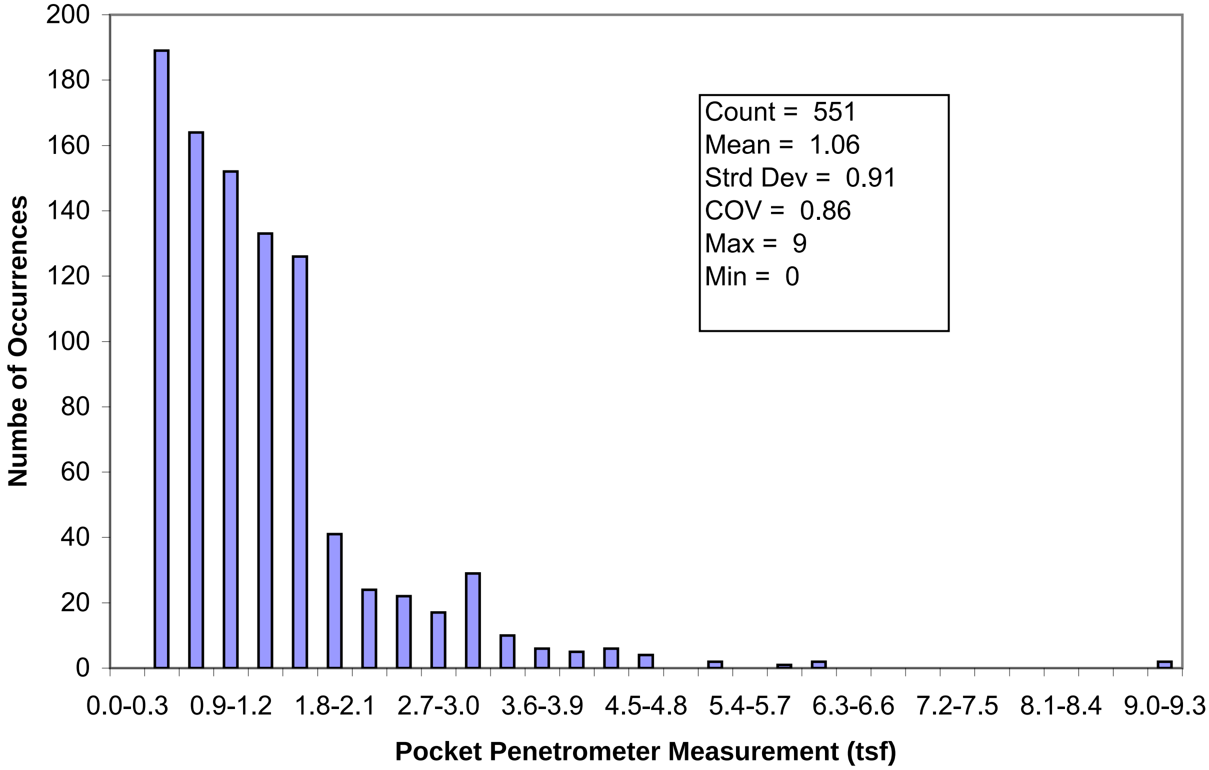
USACE (1999), *Risk-based Analysis in Geotechnical Engineering for Support of Planning Studies*, U.S. Army Corps of Engineers, ETL 1110-2-556, Washington, D.C.

Wolff, T.F. (1996), *Application of Time-based Reliability Analysis to Corps of Engineers' Geotechnical Engineering Problems*, report to St. Louis District, U.S. Army Corps of Engineers.

Wolff, T.F., A. Hassan, R. Khan, I. Ur-Rasul, and M. Miller (2004), *Geotechnical Reliability of Dam and Levee Enhancements*, U.S. Army Corps of Engineers Research and Development Center, ERDC/GSL CR-04-1.

# APPENDIX A

# HISTOGRAMS AND DISTRIBUTIONS FOR LOW PLASTICITY CLAYS



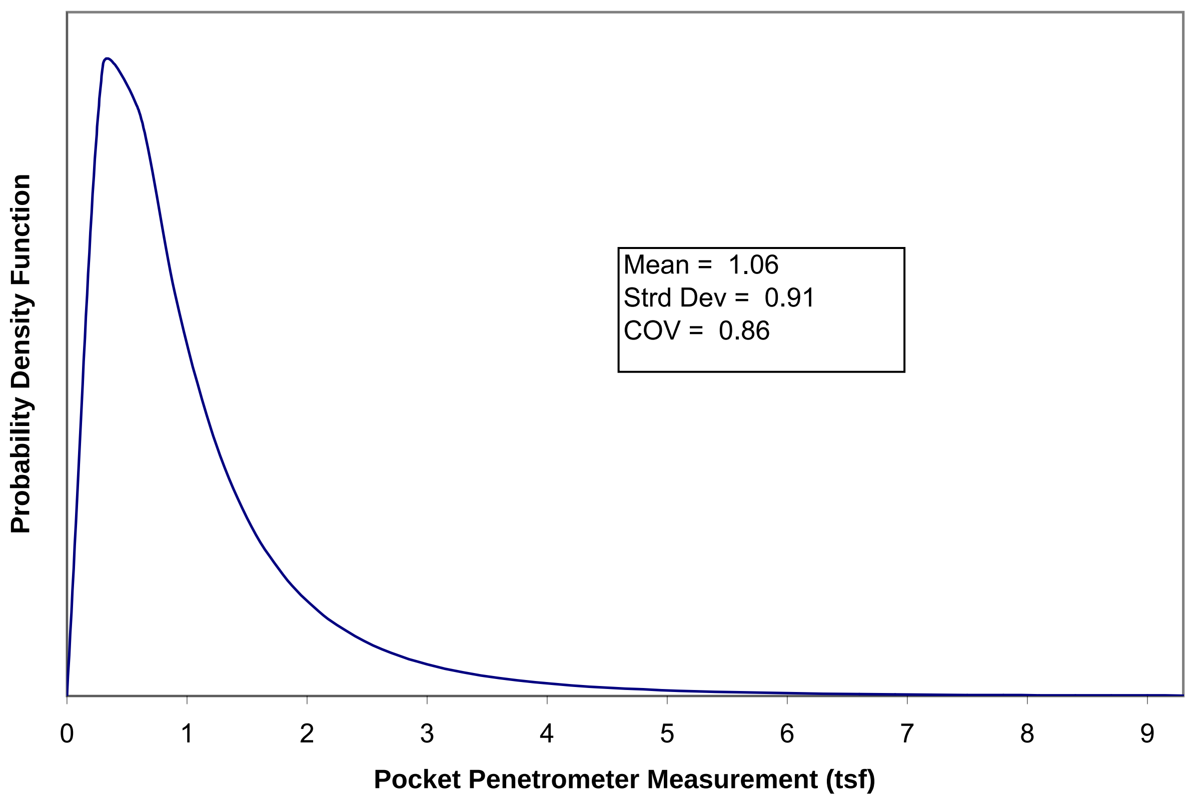
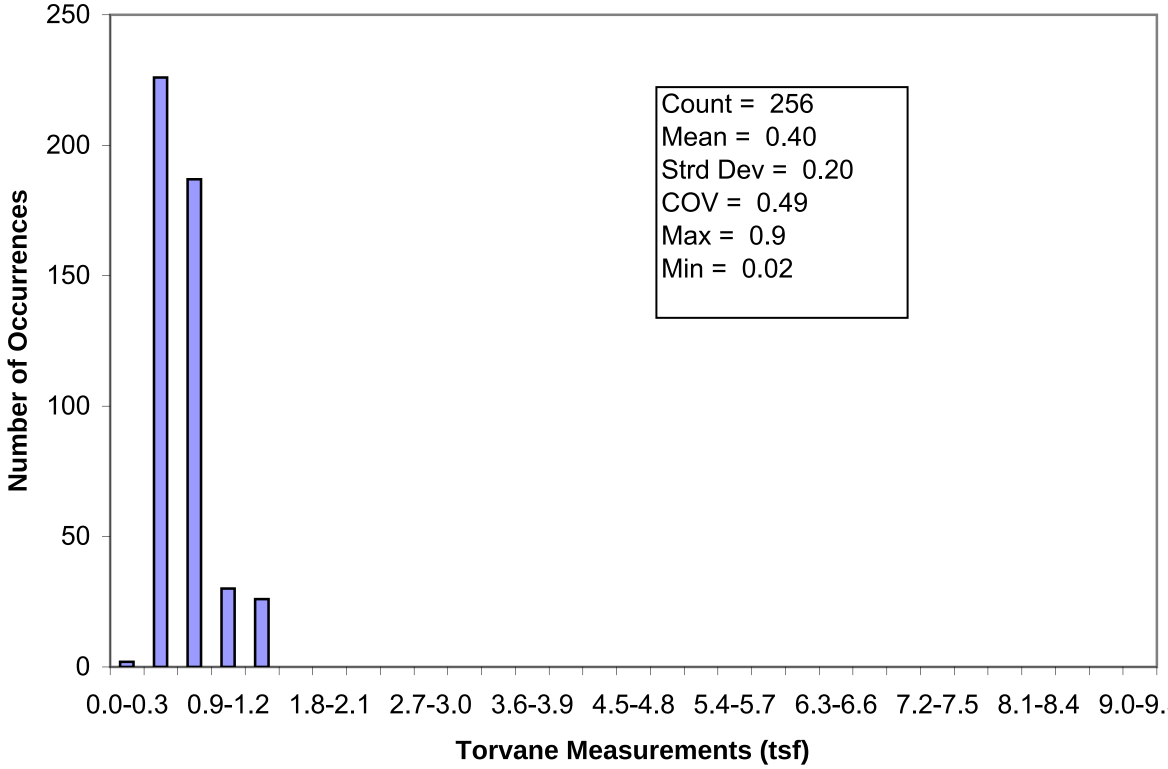




Figure A.1. Histogram and probability density function for pocket penetrometer measurements for low plasticity clay (CL).



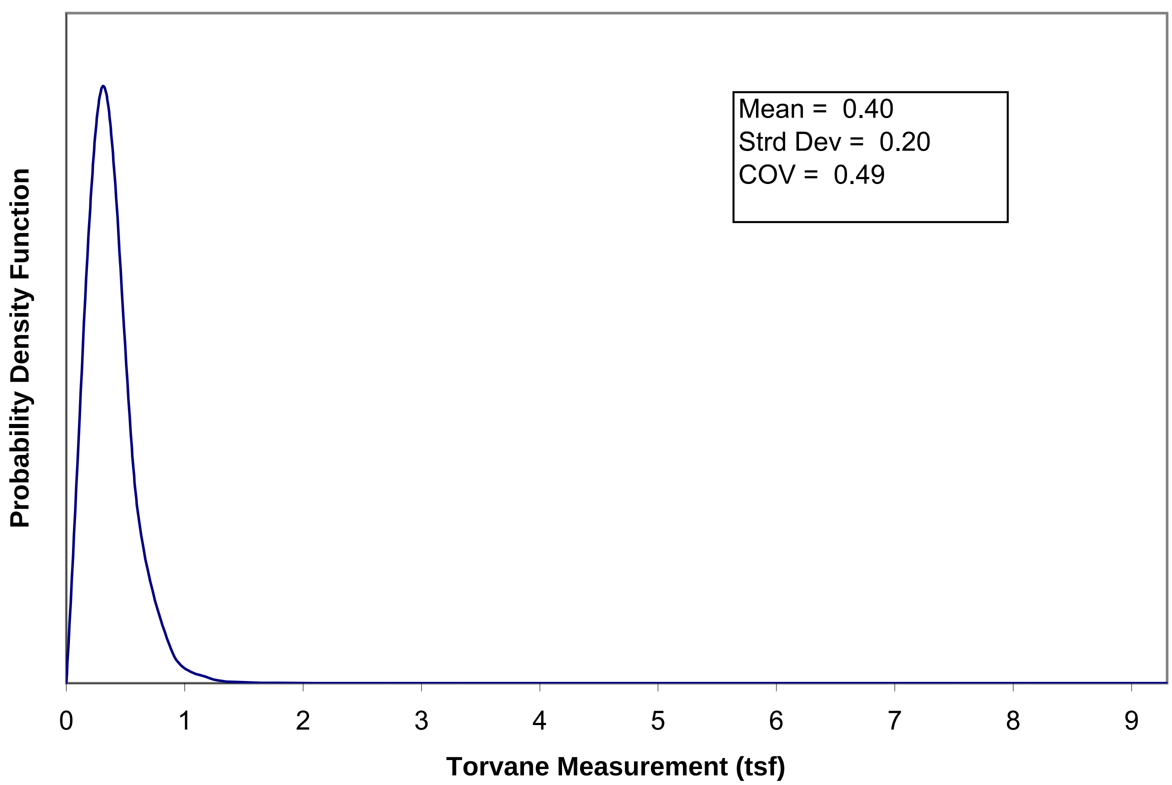
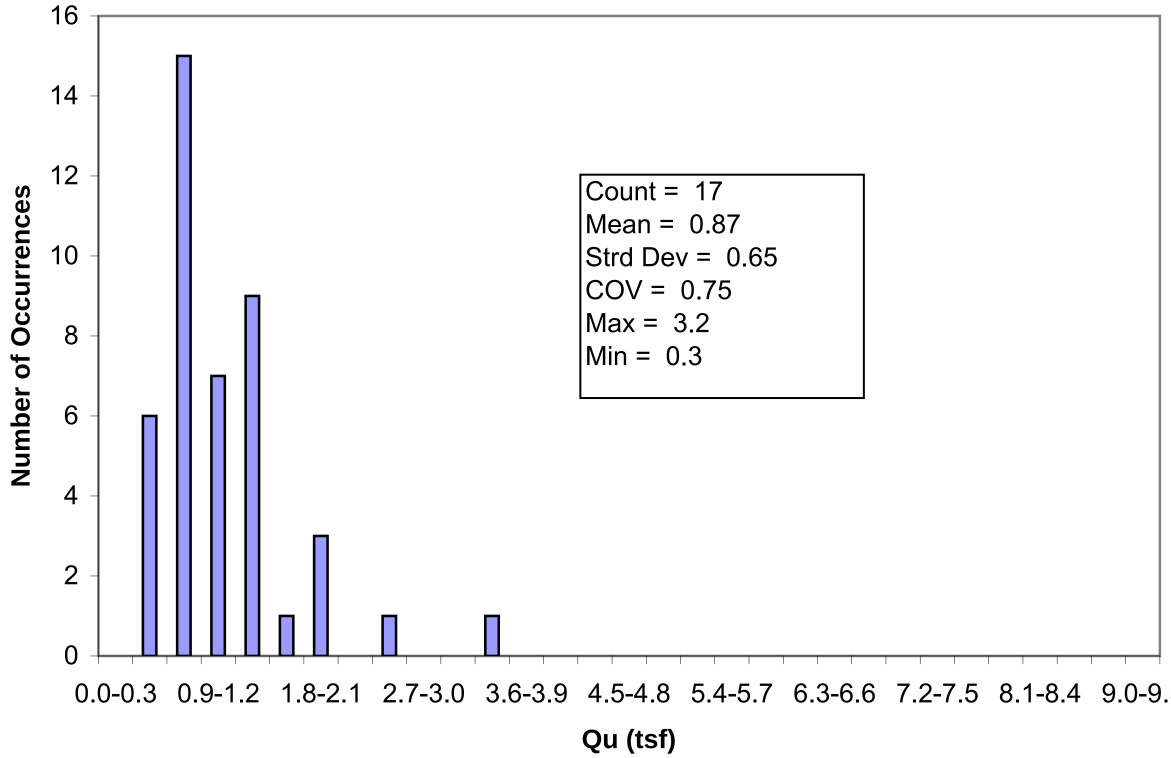




Figure A.2. Histogram and probability density function for torvane measurements for low plasticity clay (CL).



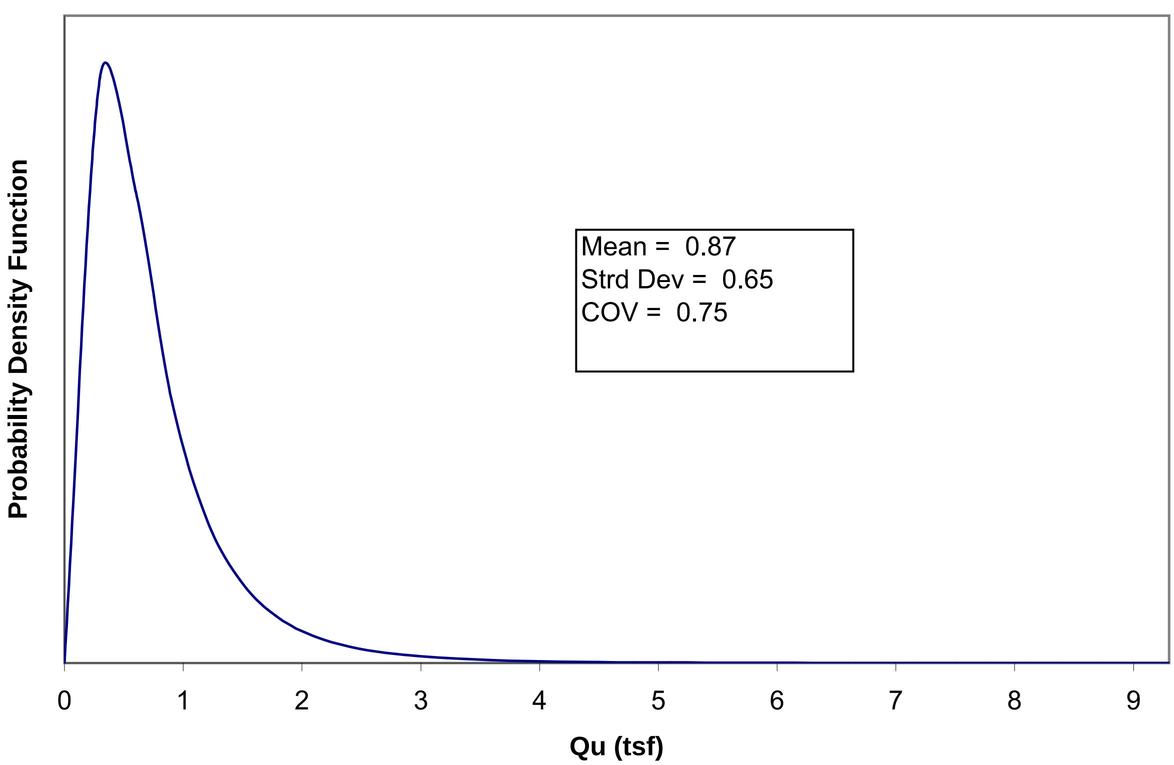
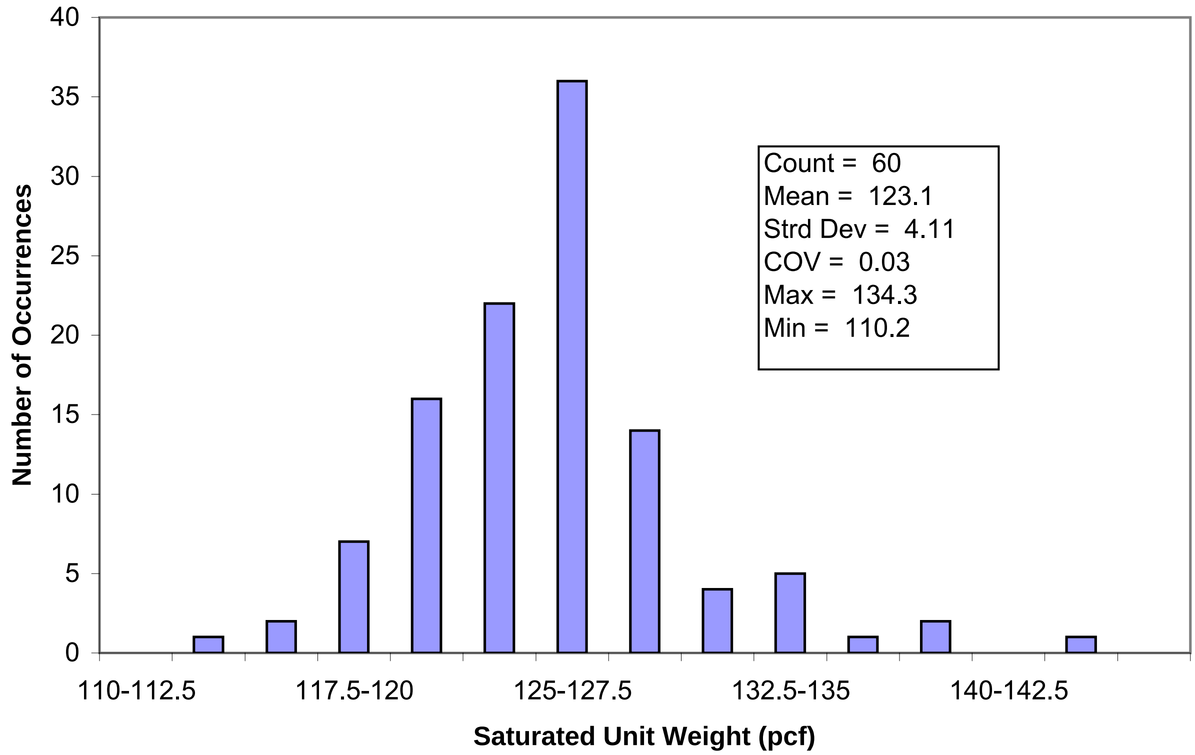




Figure A.3. Histogram and probability density function for unconfined compression tests for low plasticity clay (CL).



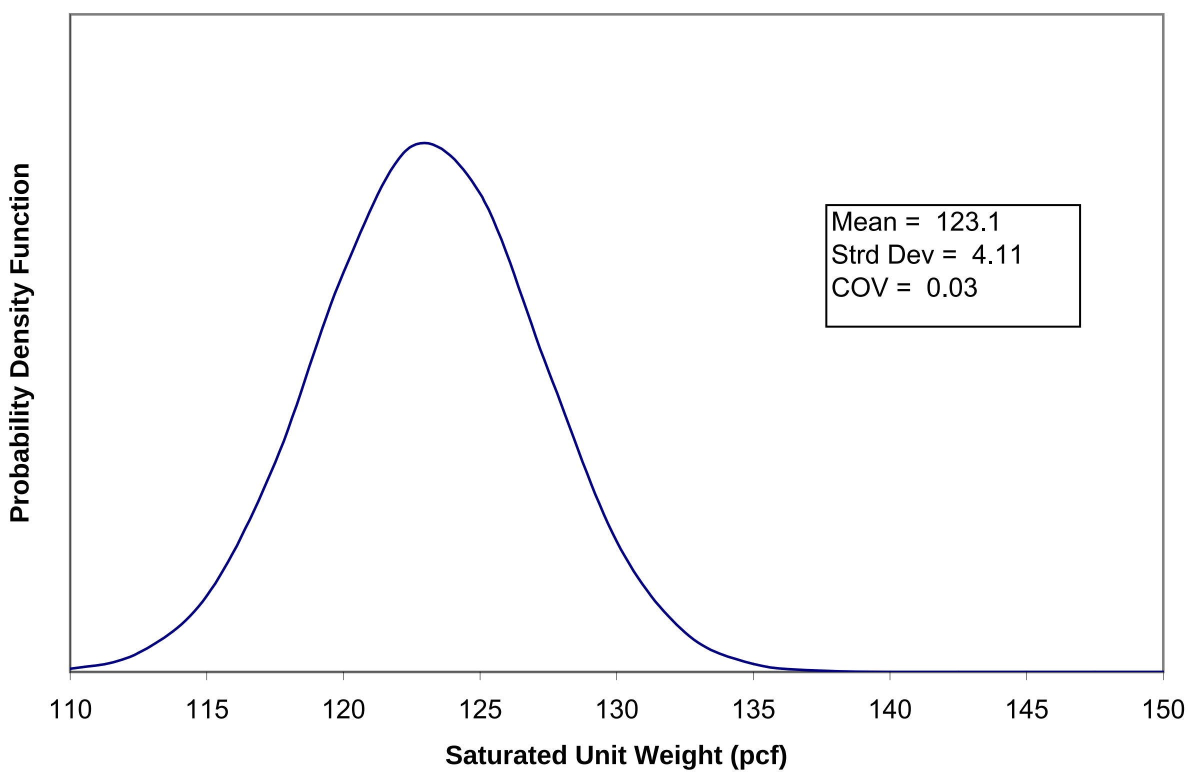
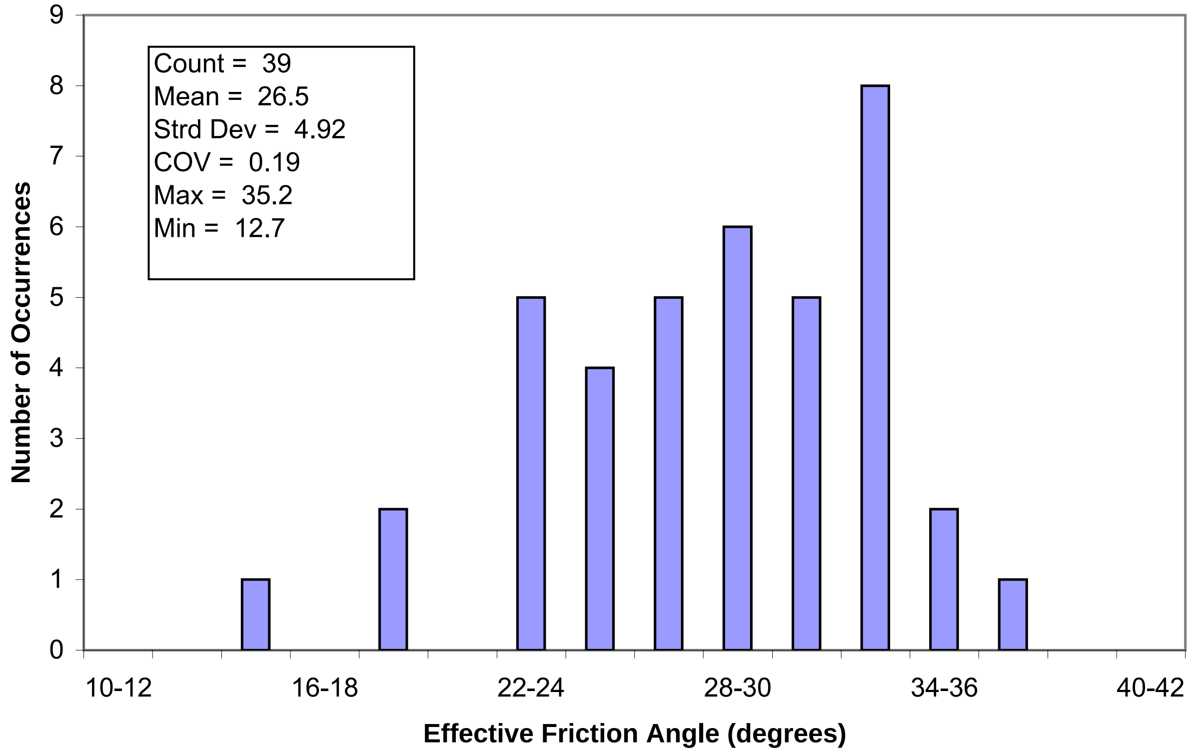




Figure A.4. Histogram and probability density function for saturated unit weight of low plasticity clay (CL).



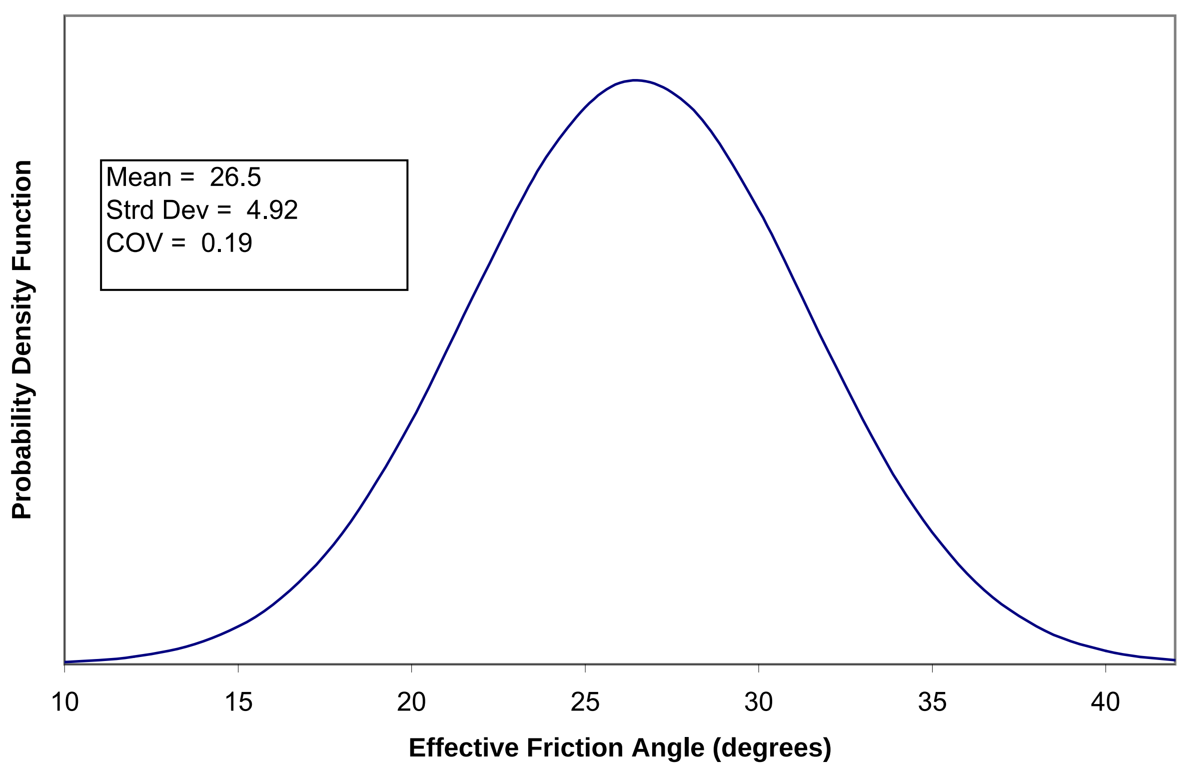
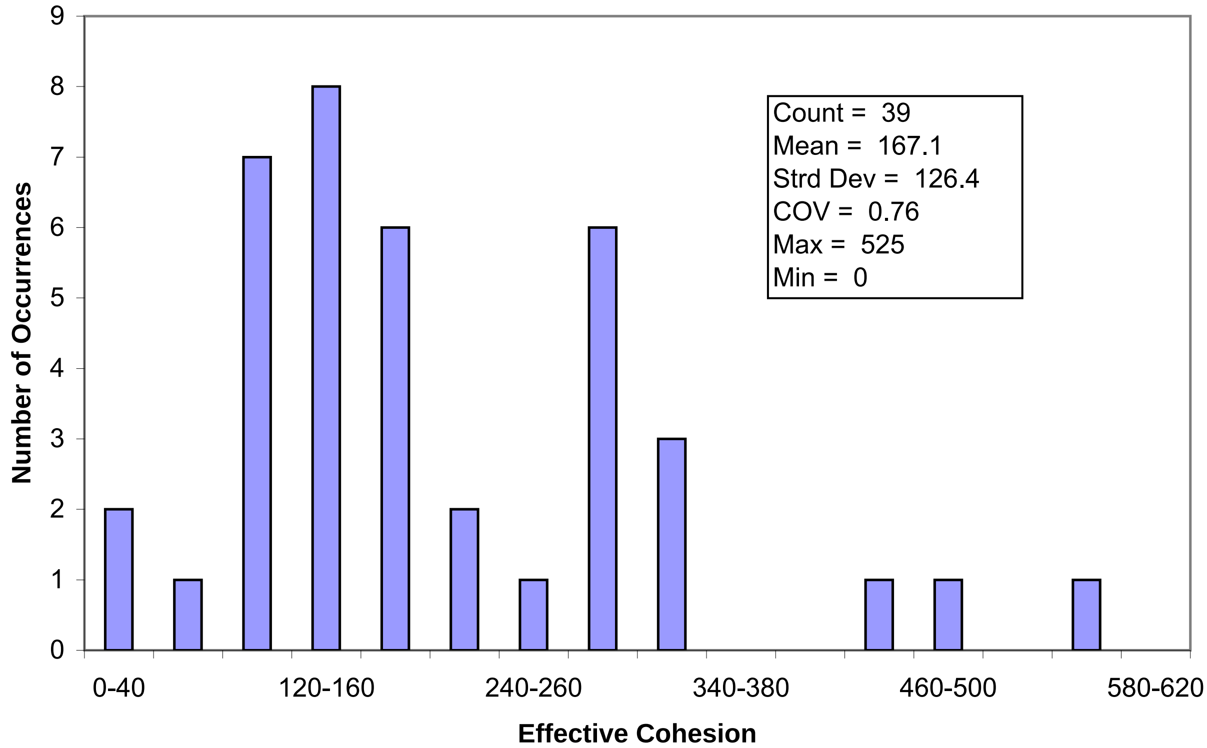




Figure A.5. Histogram and probability density function for effective friction angle of low plasticity clay (CL).



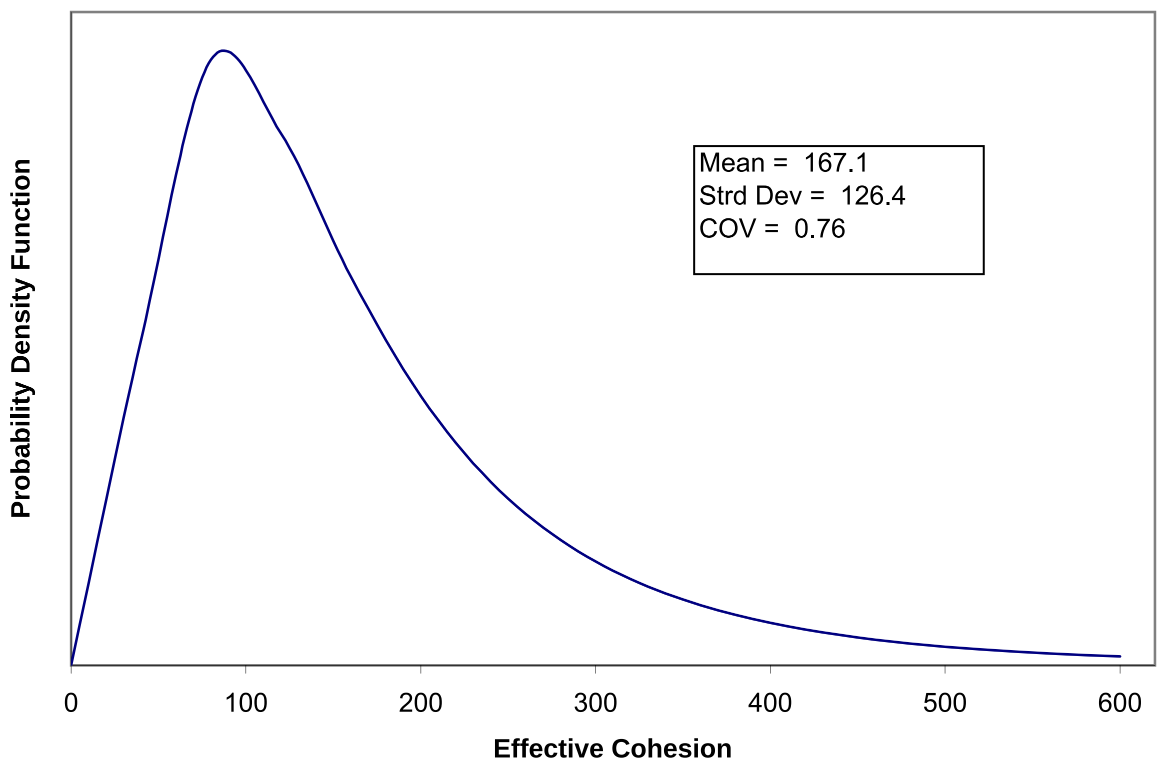
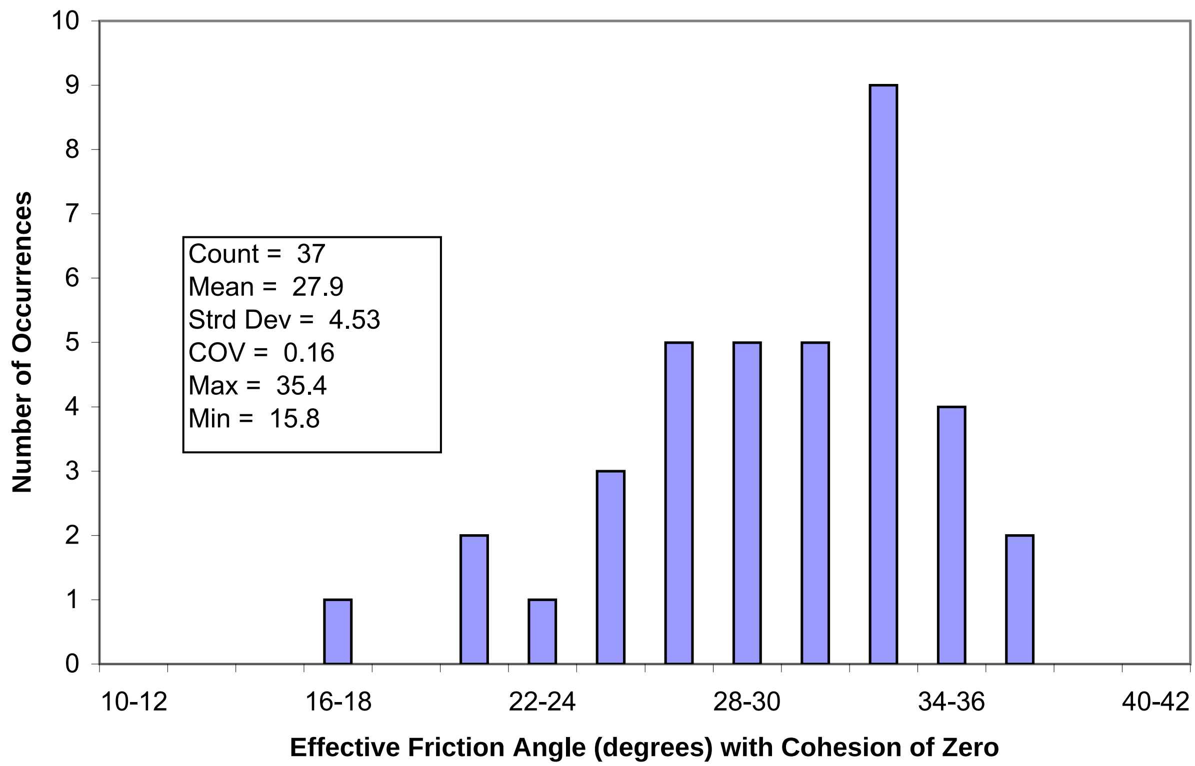




Figure A.6. Histogram and probability density function for saturated unit weight of low plasticity clay (CL).



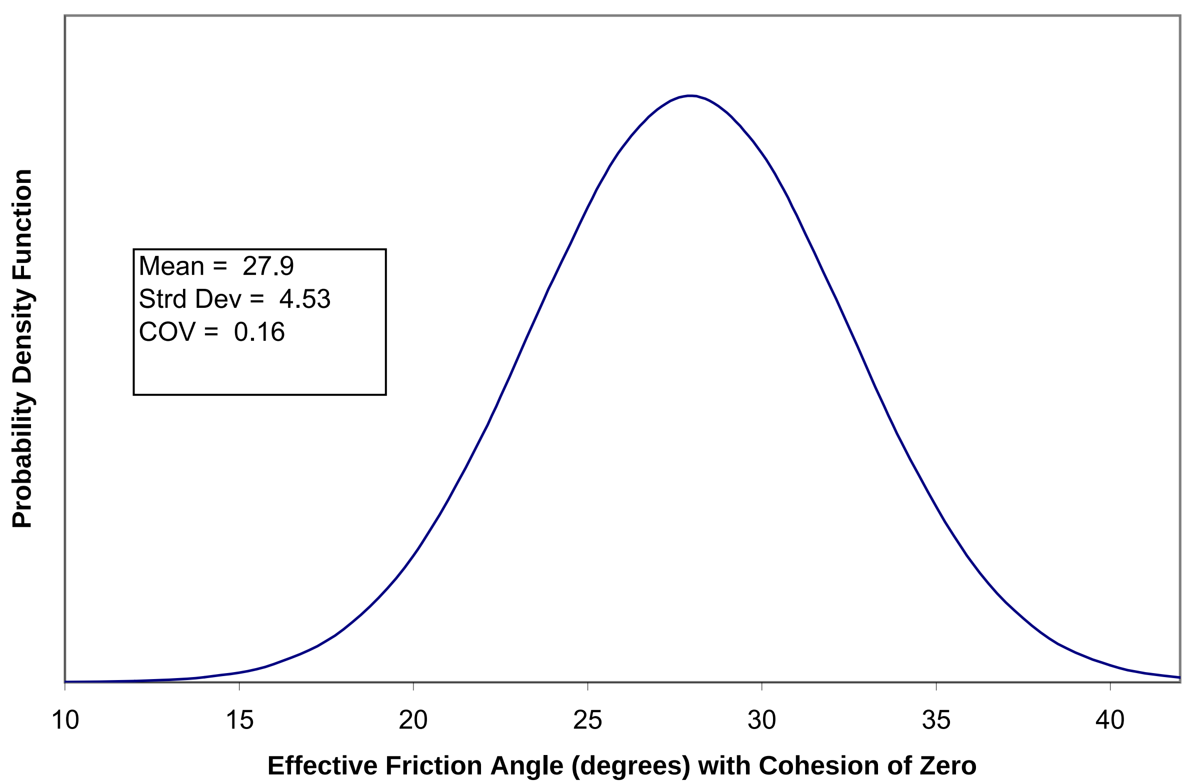
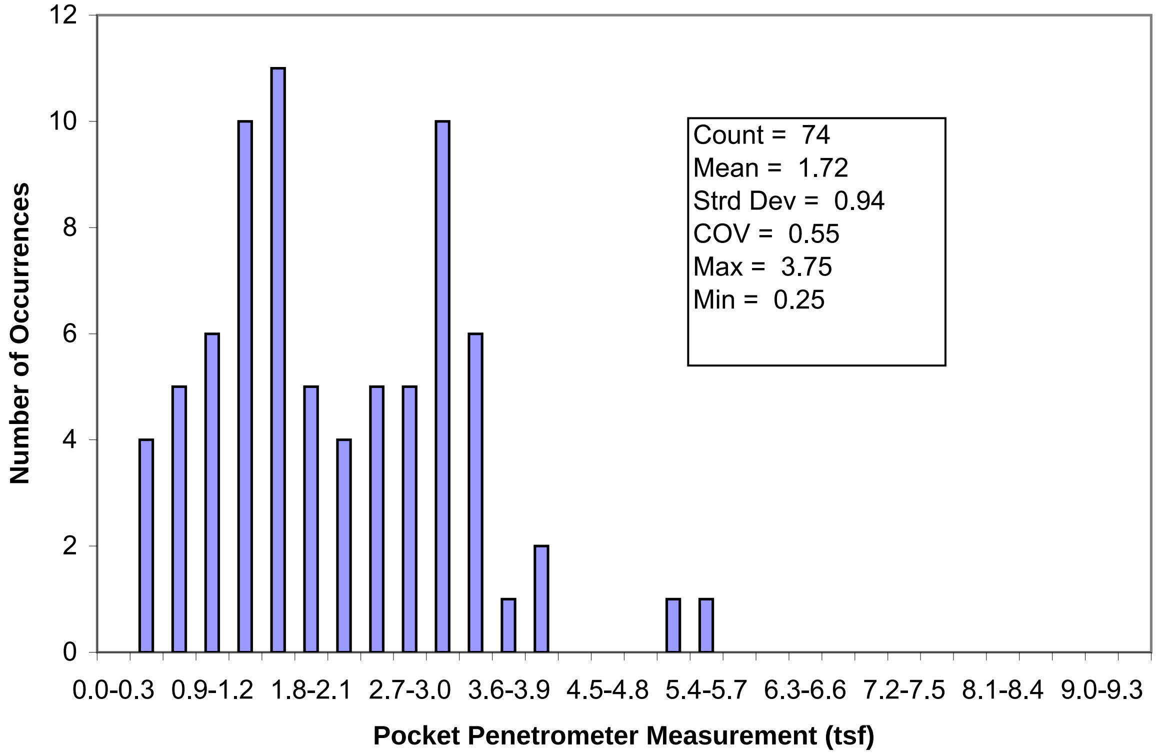




Figure A.7. Histogram and probability density function for effective friction angle, with zero cohesion, of low plasticity clay (CL).

# APPENDIX B

## HISTOGRAMS AND DISTRIBUTIONS FOR HIGH PLASTICITY CLAYS



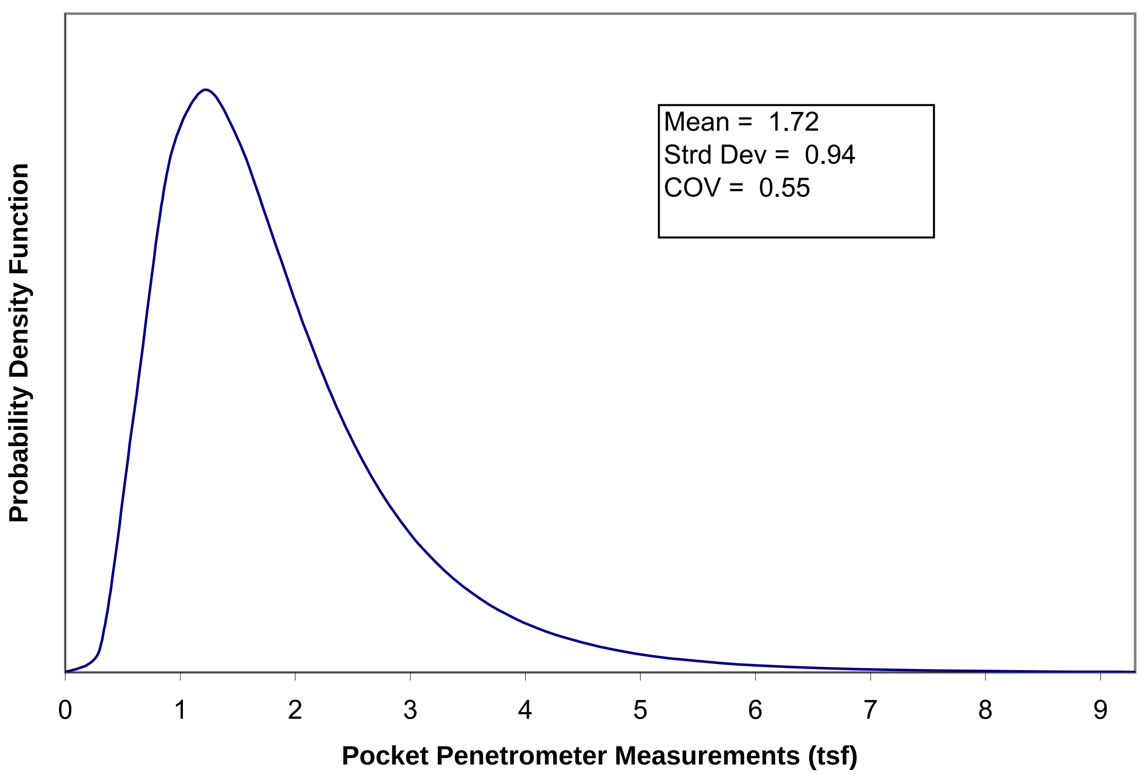
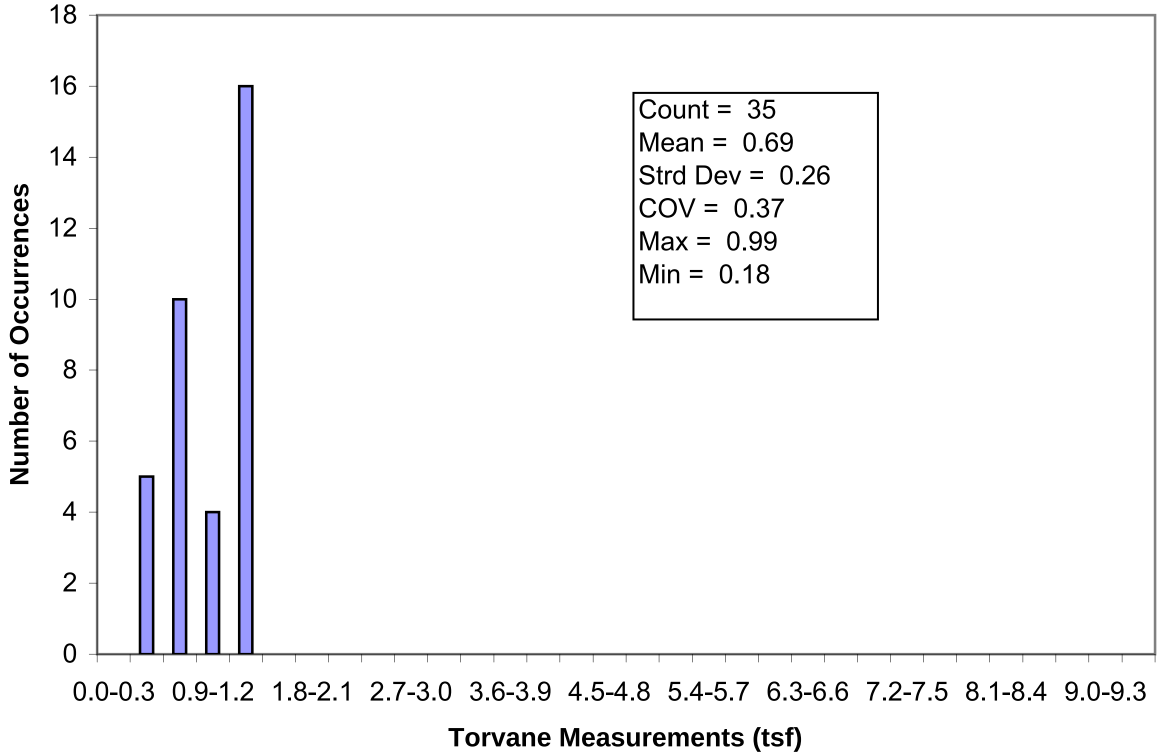




Figure B.1. Histogram and probability density function for pocket penetrometer measurements for high plasticity clay (CH).



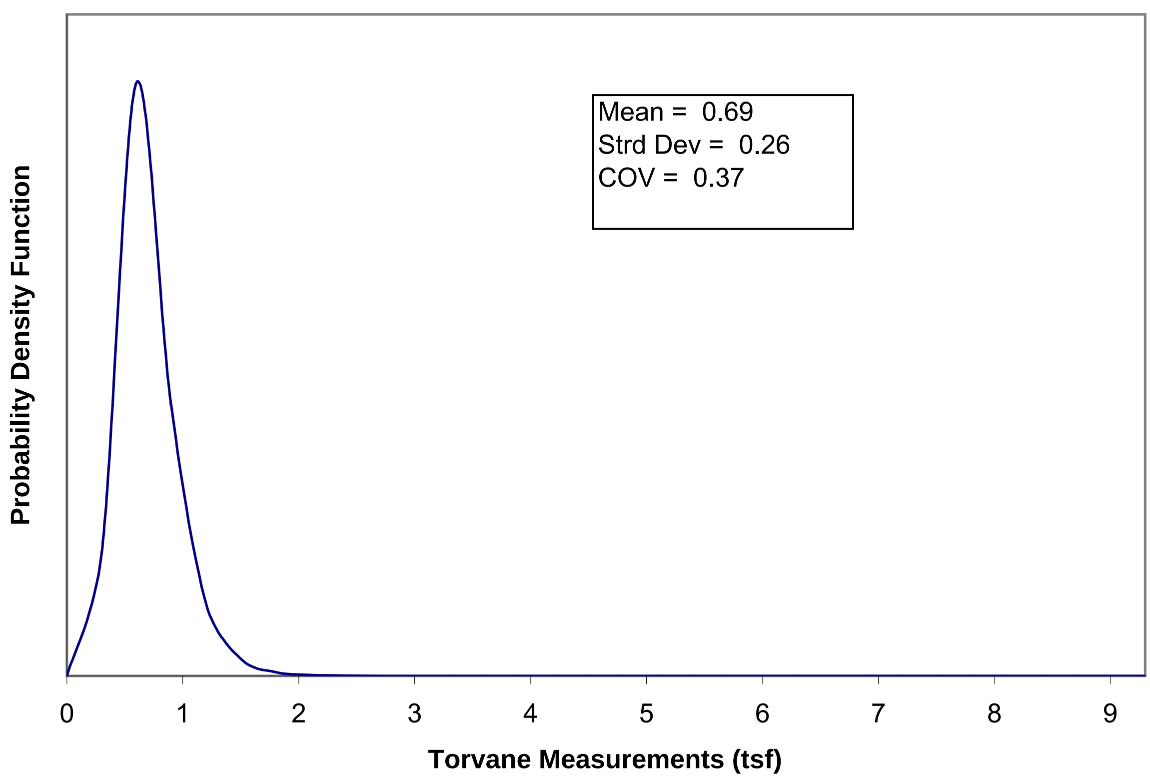
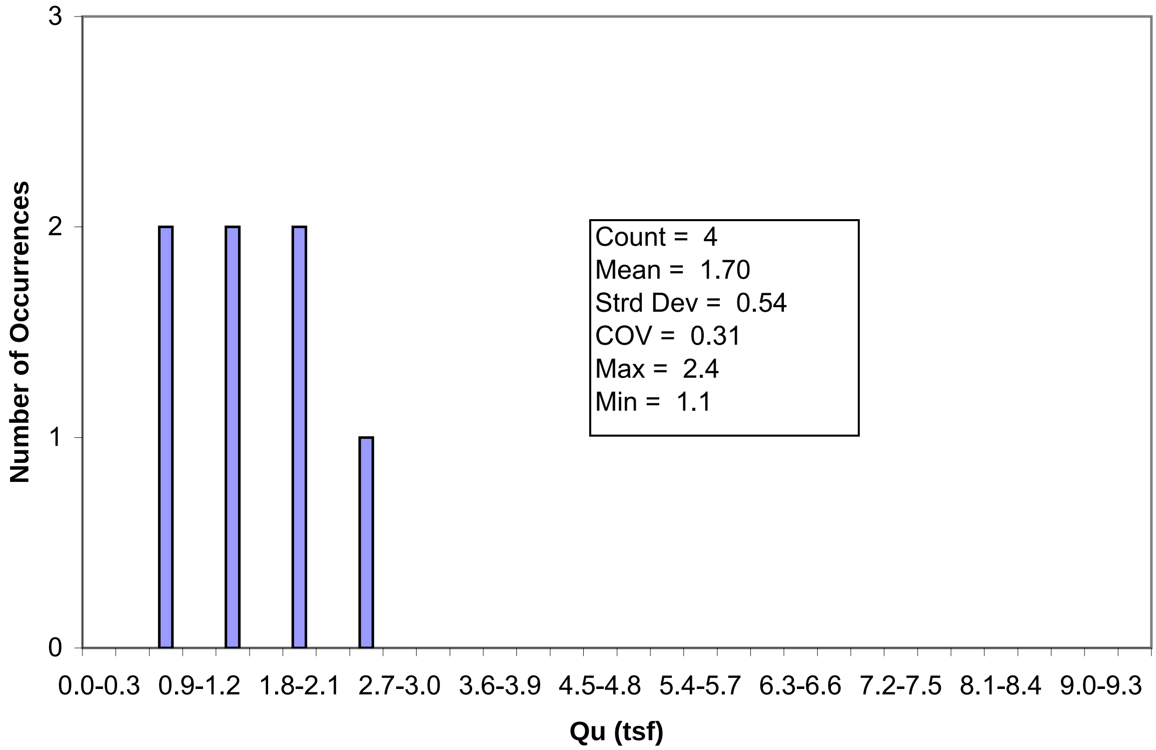




Figure B.2. Histogram and probability density function for torvane measurements for high plasticity clay (CH).



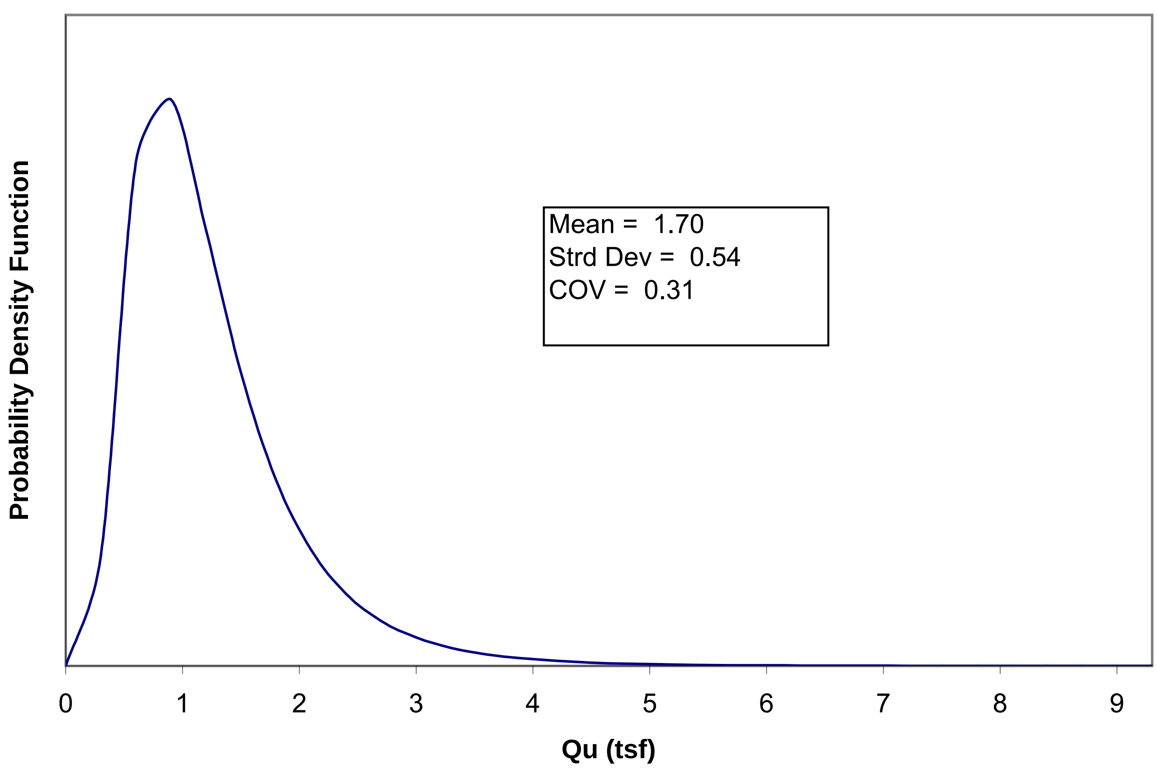




Figure B.3. Histogram and probability density function for unconfined compression tests for high plasticity clay (CH).

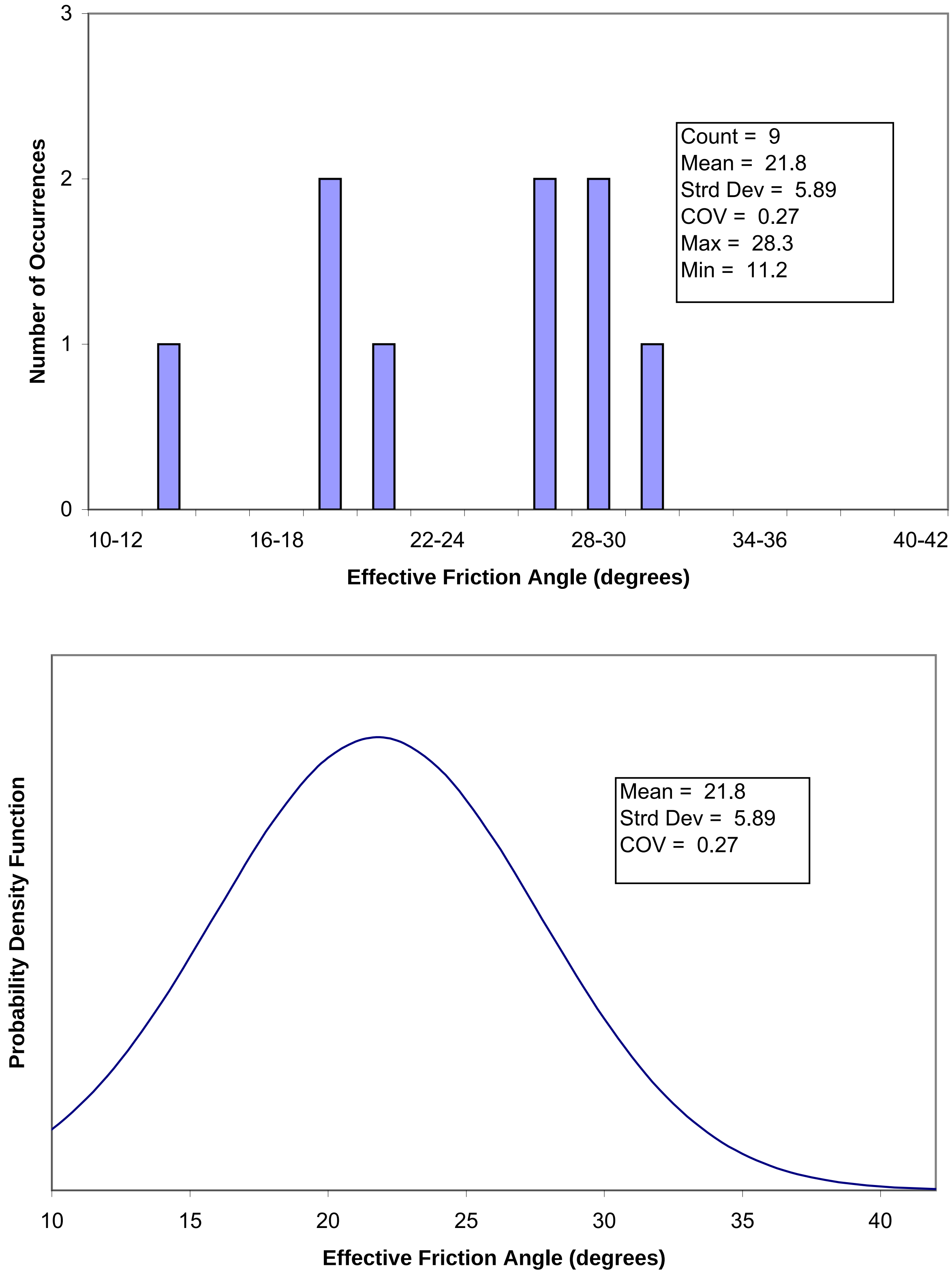
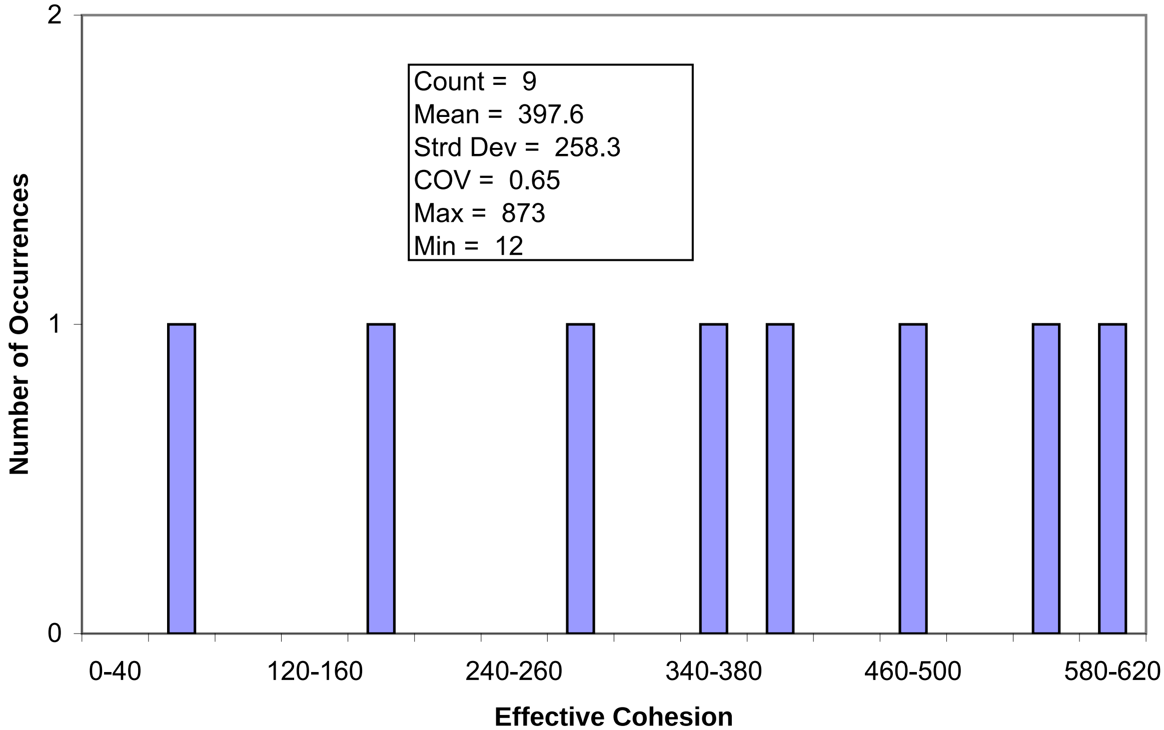



Figure B.4. Histogram and probability density function for effective friction angle of high plasticity clay (CH).



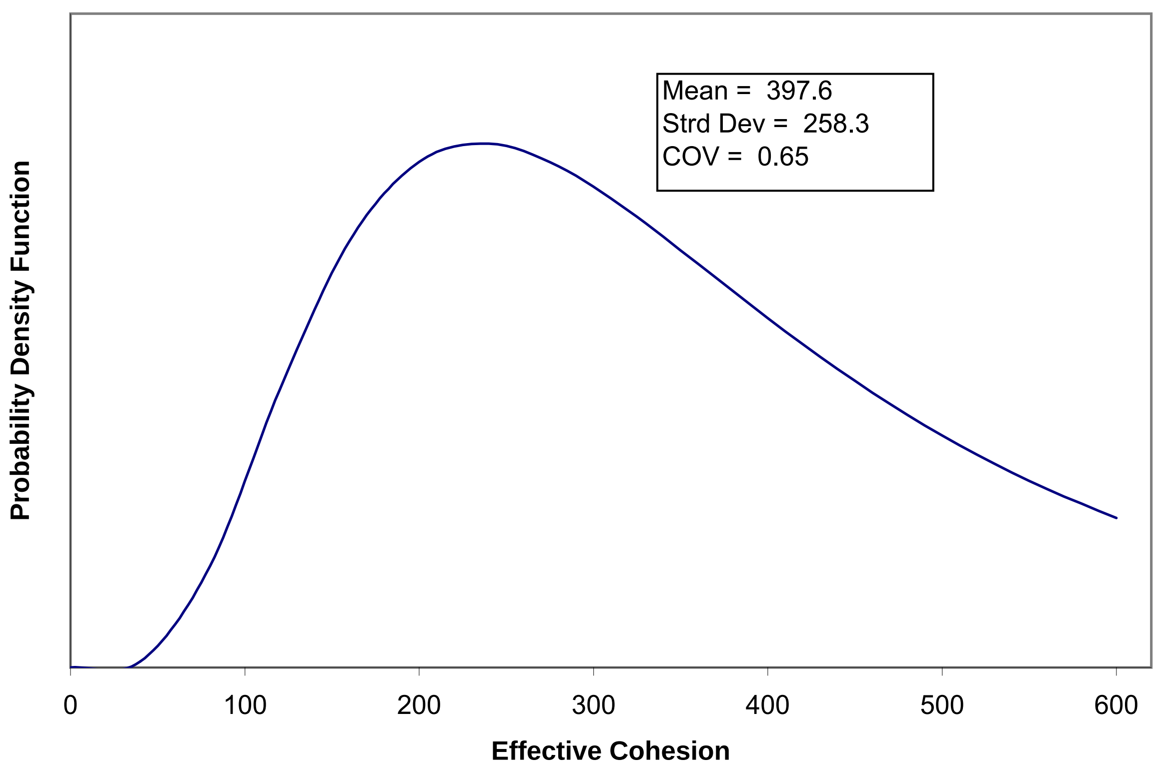
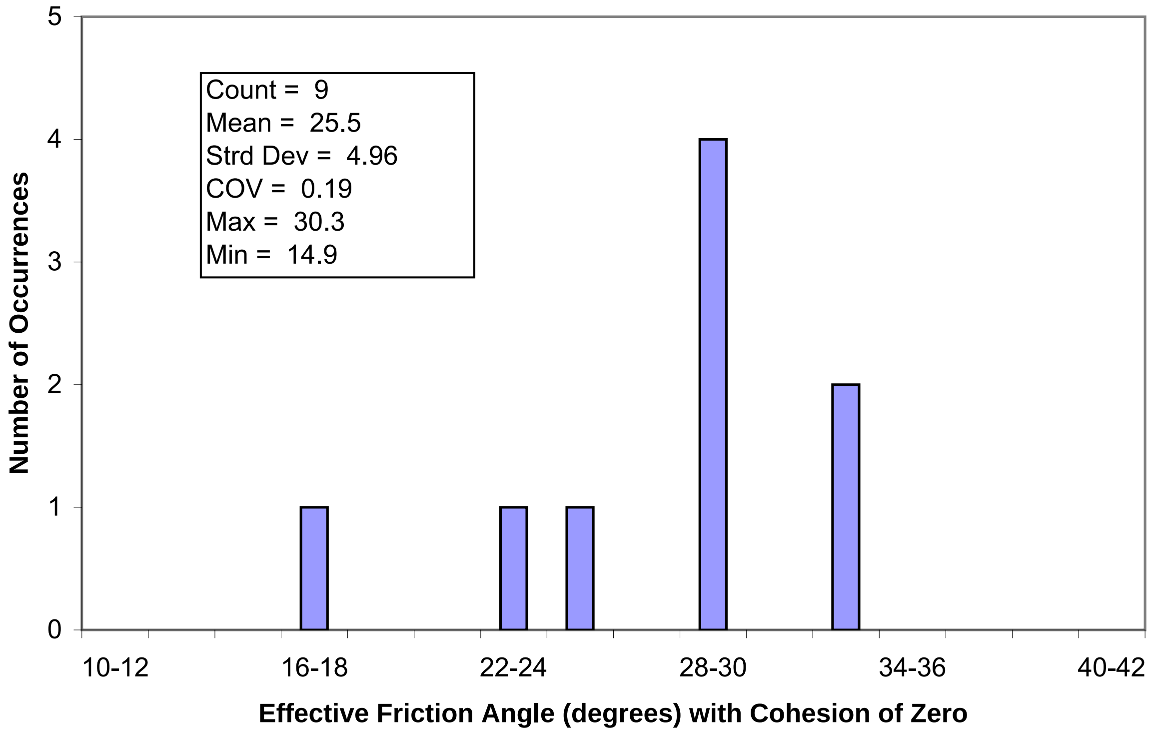




Figure B.5. Histogram and probability density function for effective cohesion of high plasticity clay (CH).



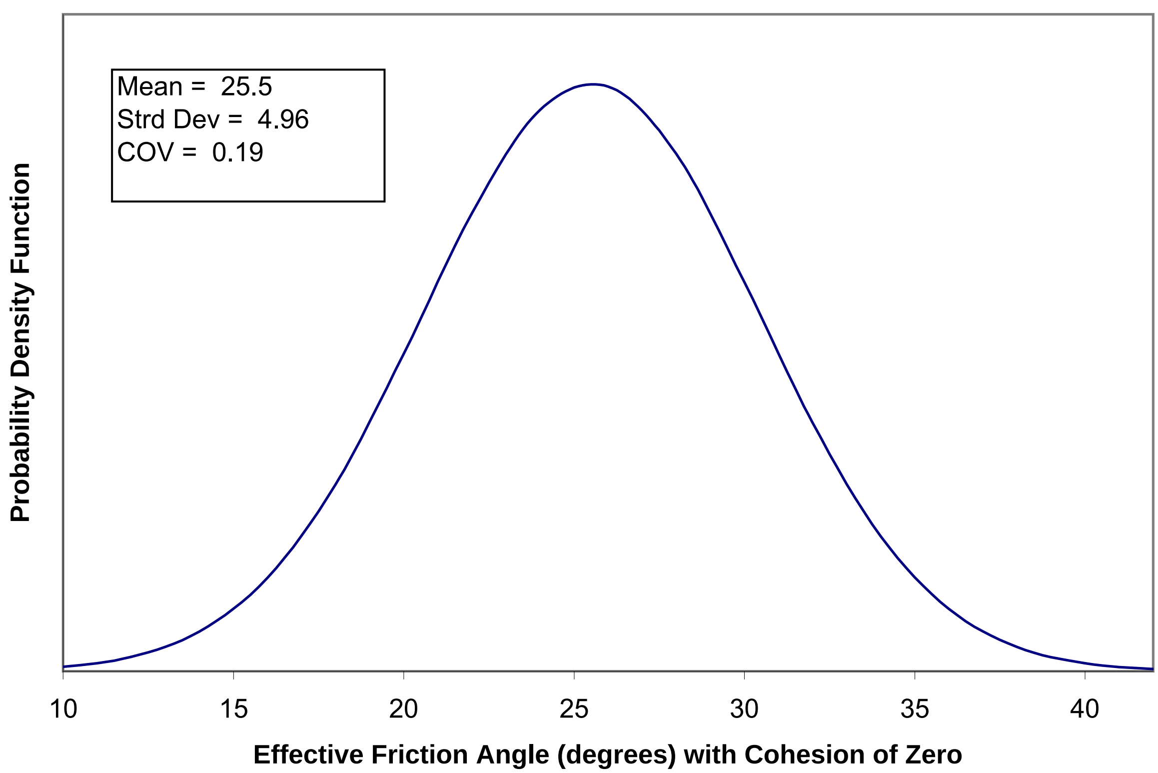
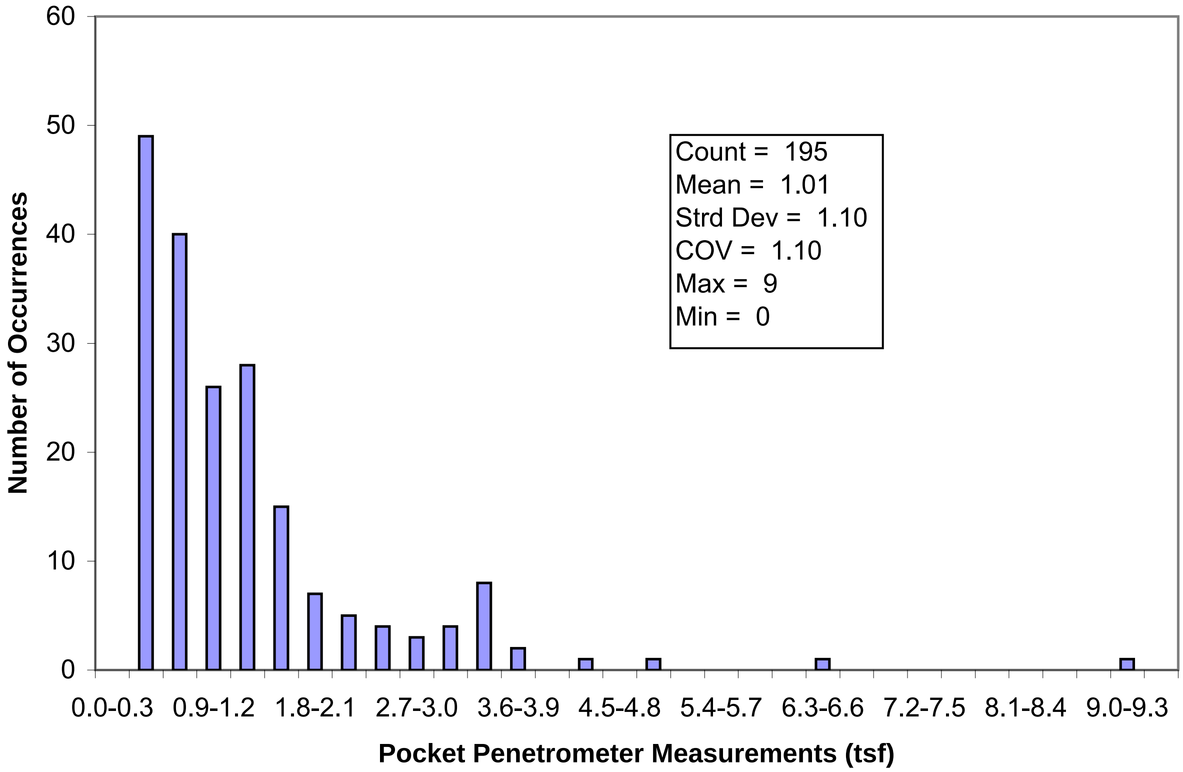




Figure B.6. Histogram and probability density function for effective friction angle of high plasticity clay (CH).

# APPENDIX C

# HISTOGRAMS AND DISTRIBUTIONS FOR SILTS



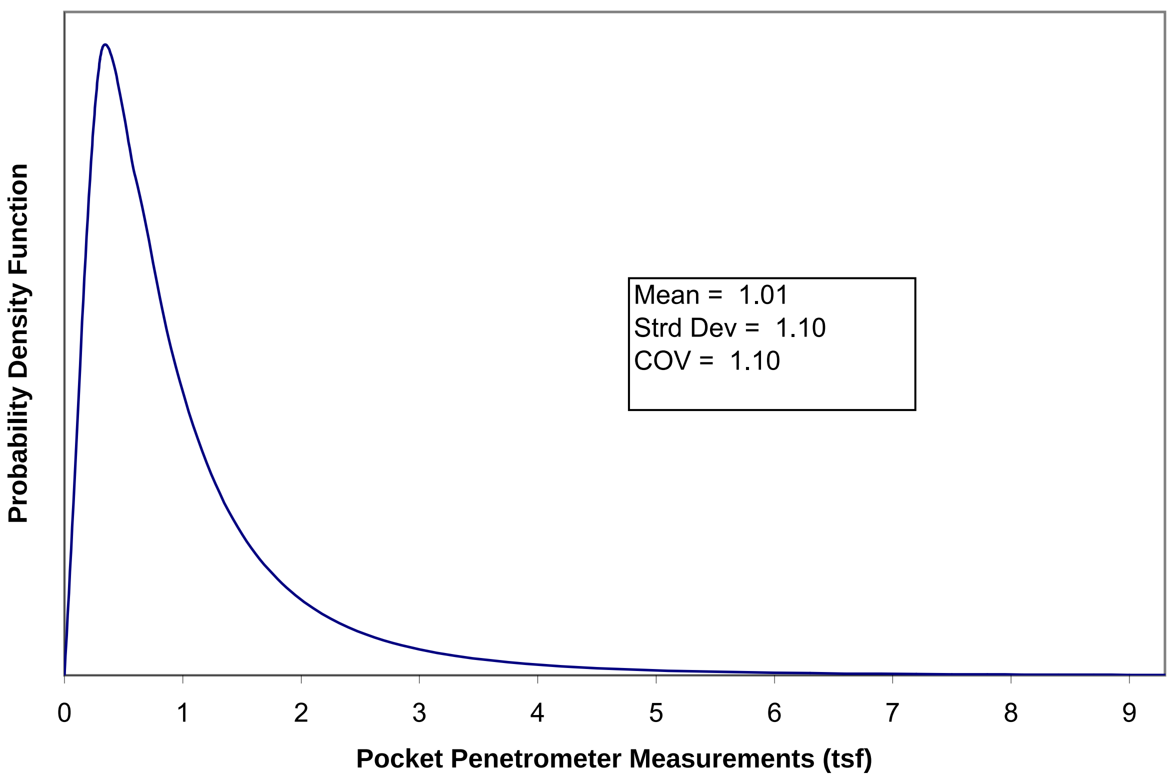
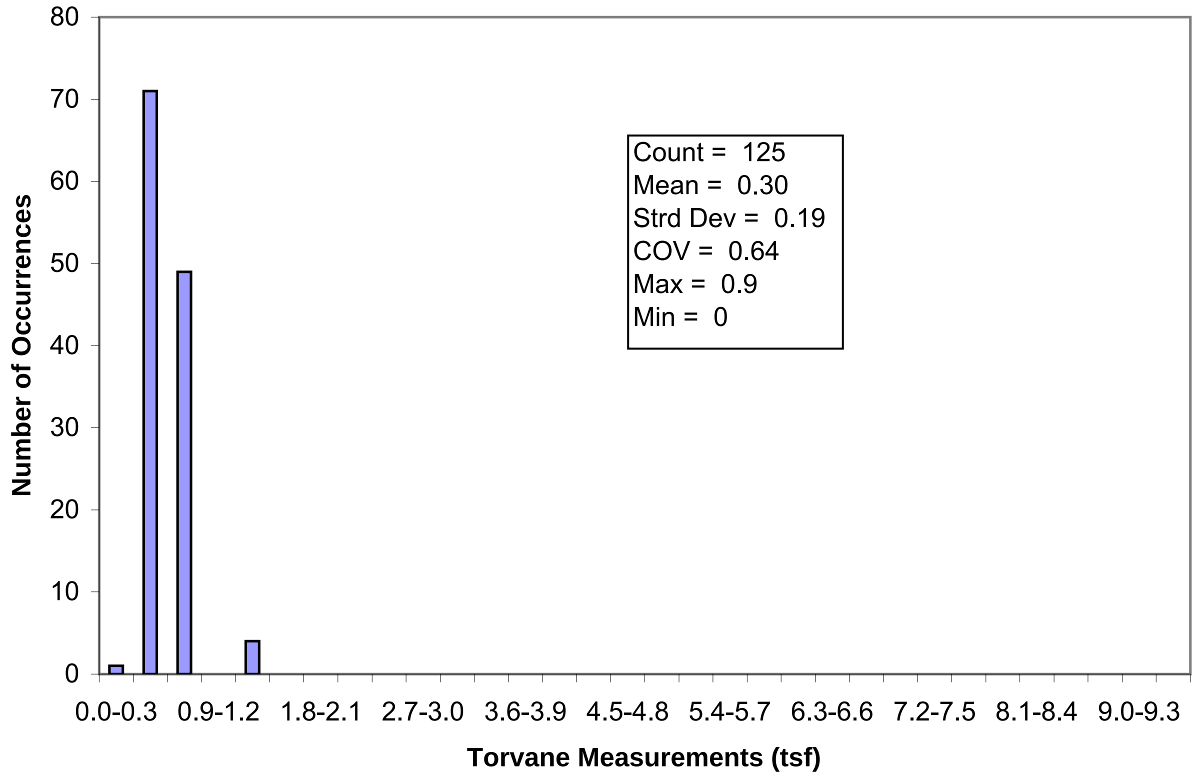




Figure C.1. Histogram and probability density function for pocket penetrometer measurements for silt (ML).



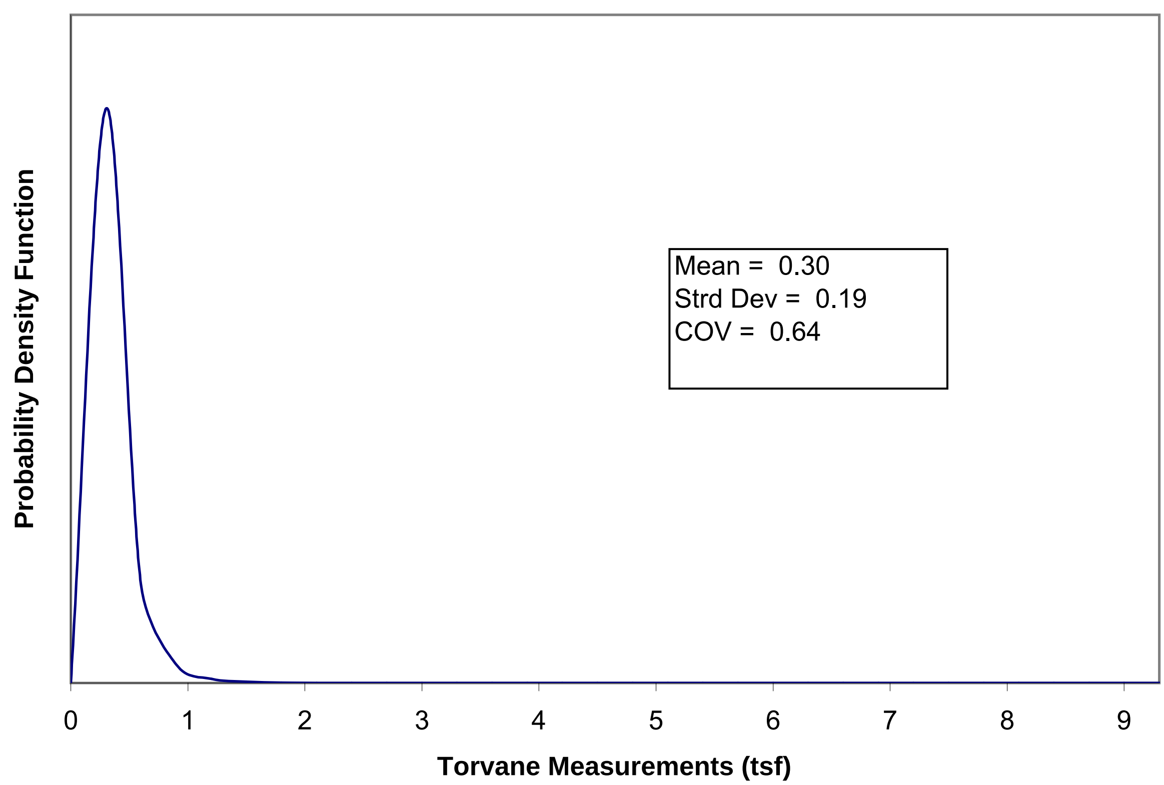
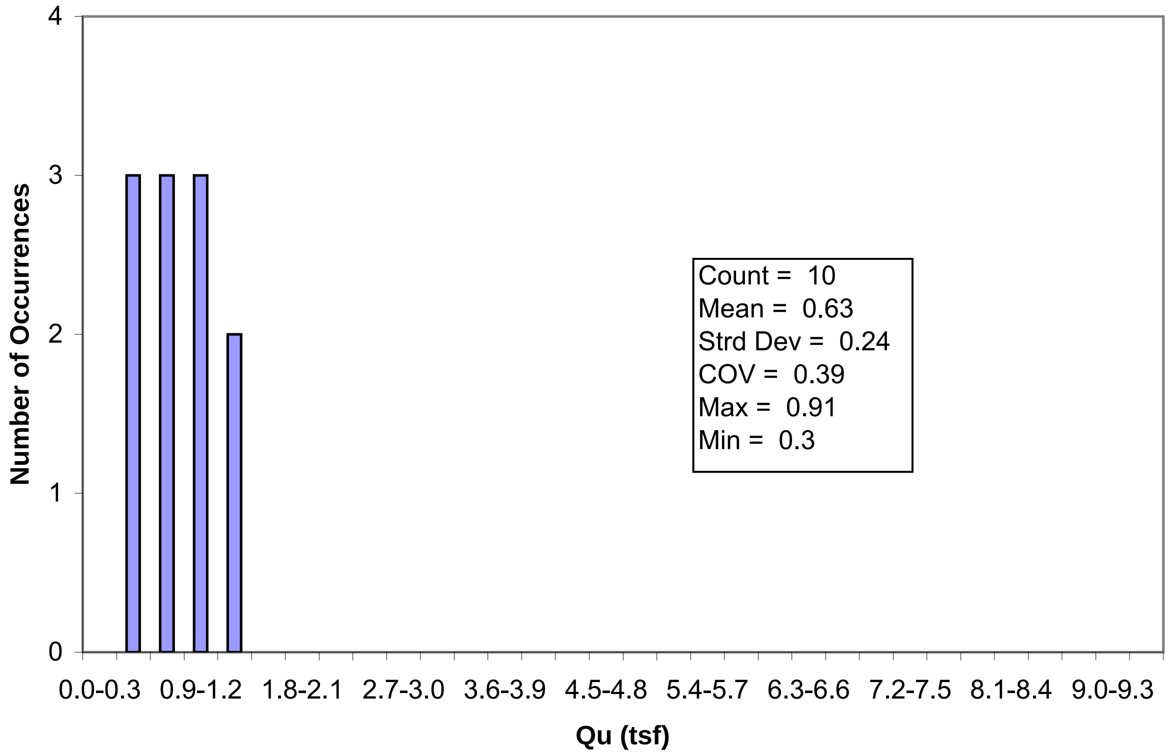




Figure C.2. Histogram and probability density function for torvane measurements for silt (ML).



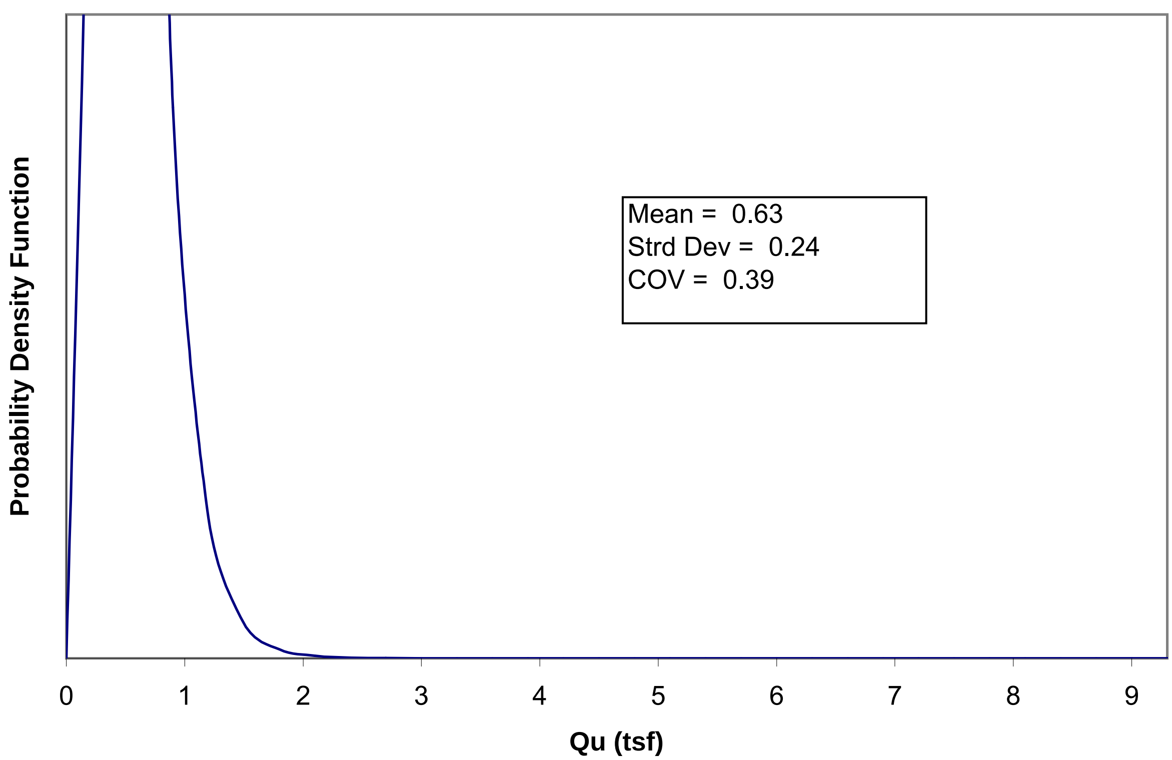
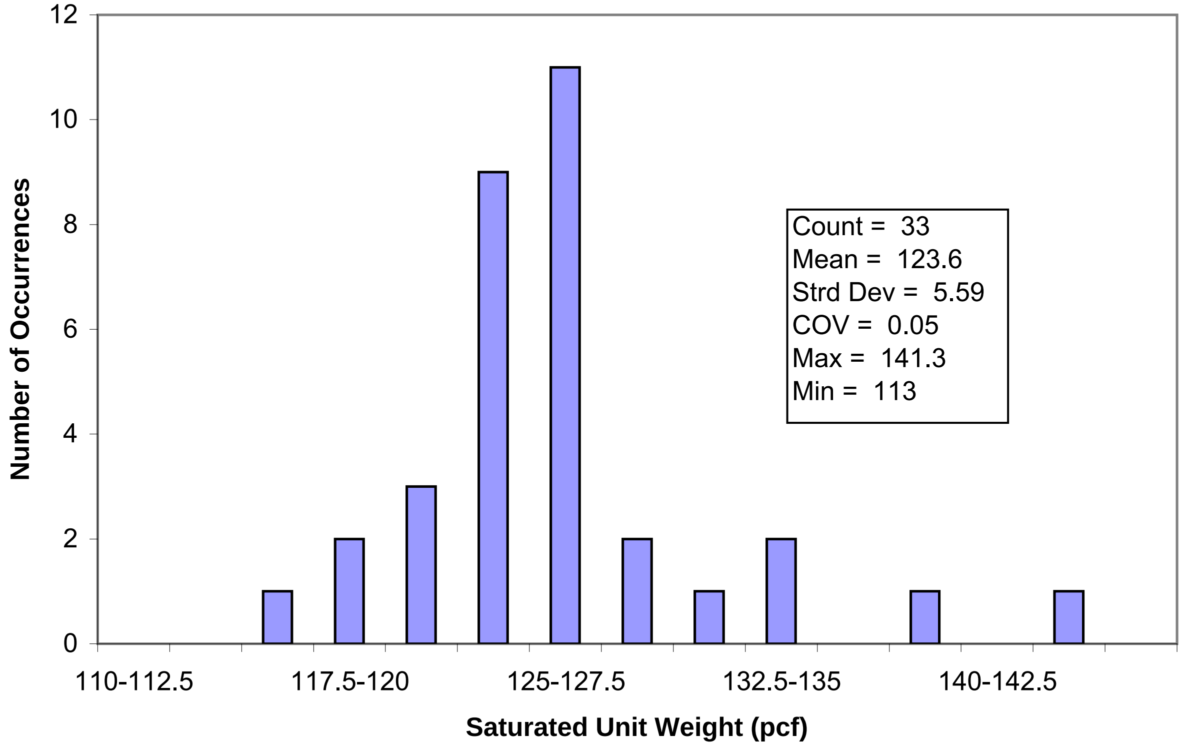




Figure C.3. Histogram and probability density function for unconfined compression tests for silt (ML).



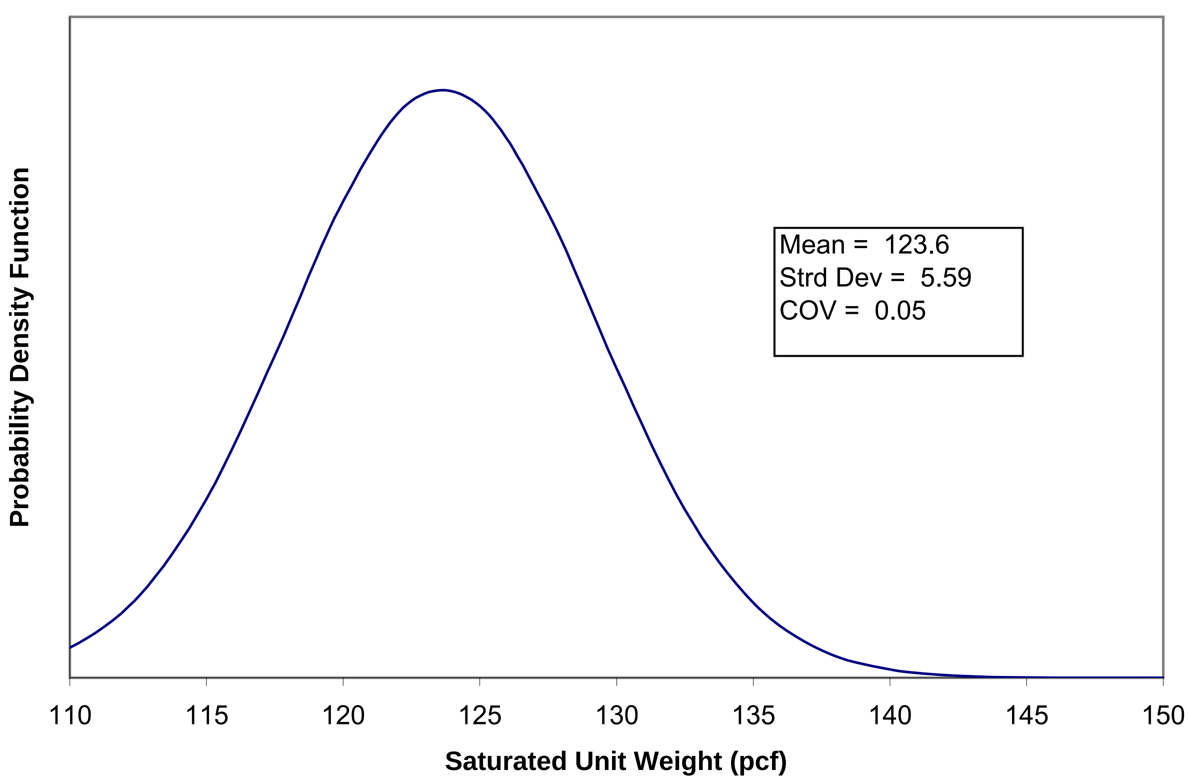
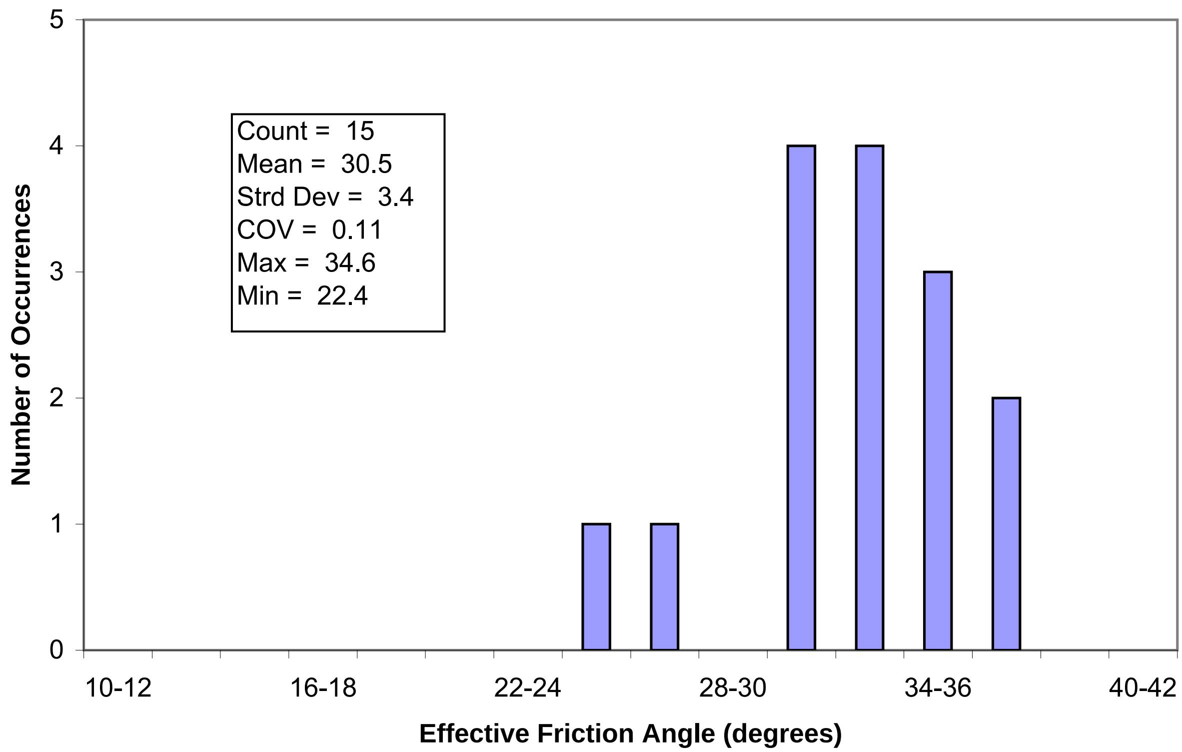




Figure C.4. Histogram and probability density function for saturated unit weight of silt (ML).



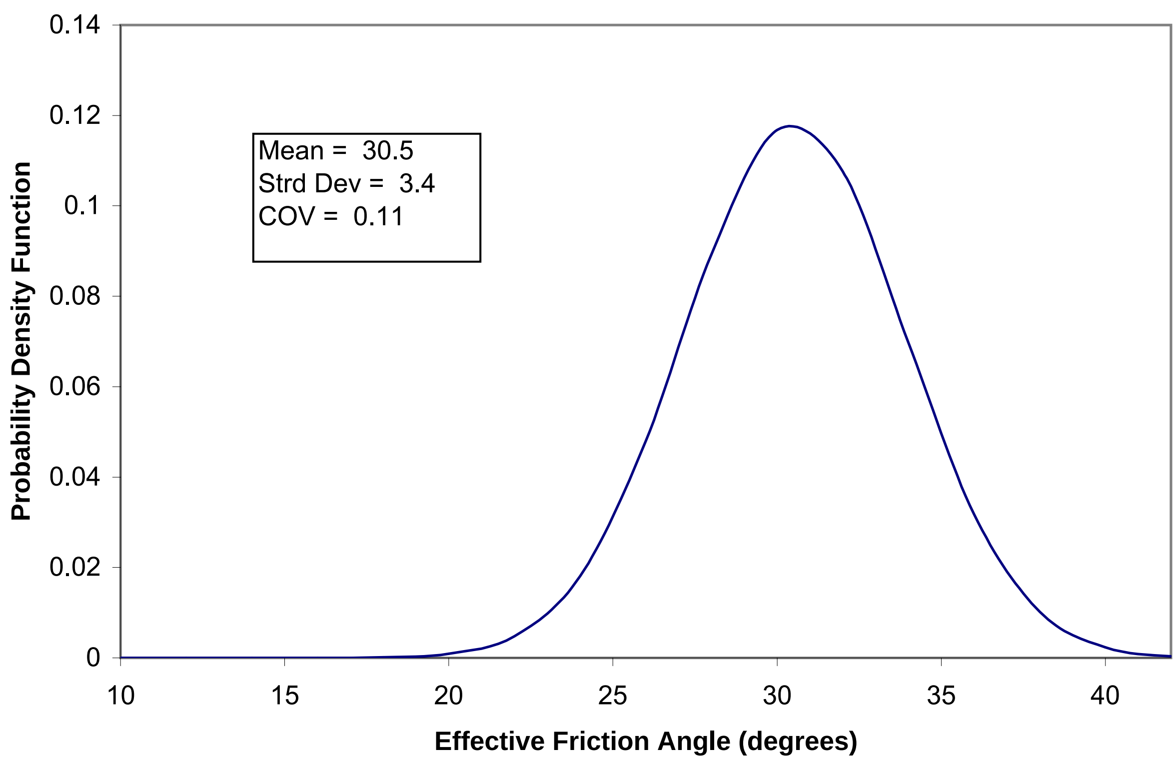
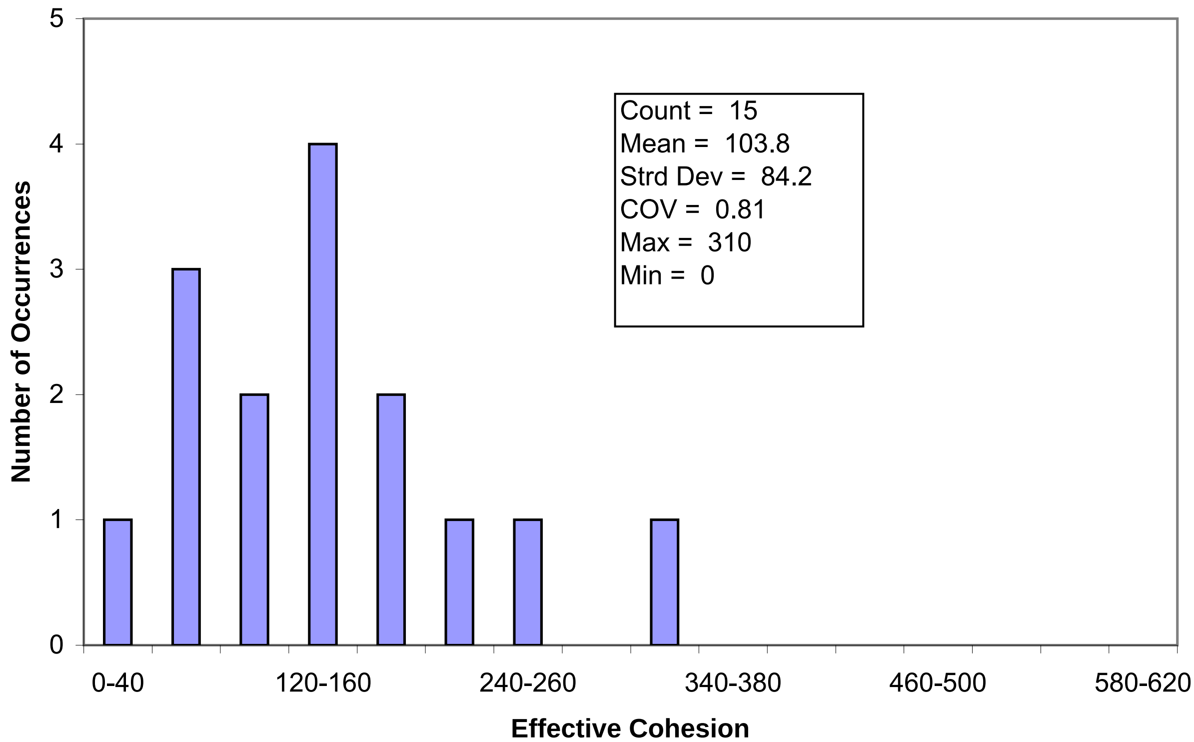




Figure C.5. Histogram and probability density function for effective friction angle of silt (ML).



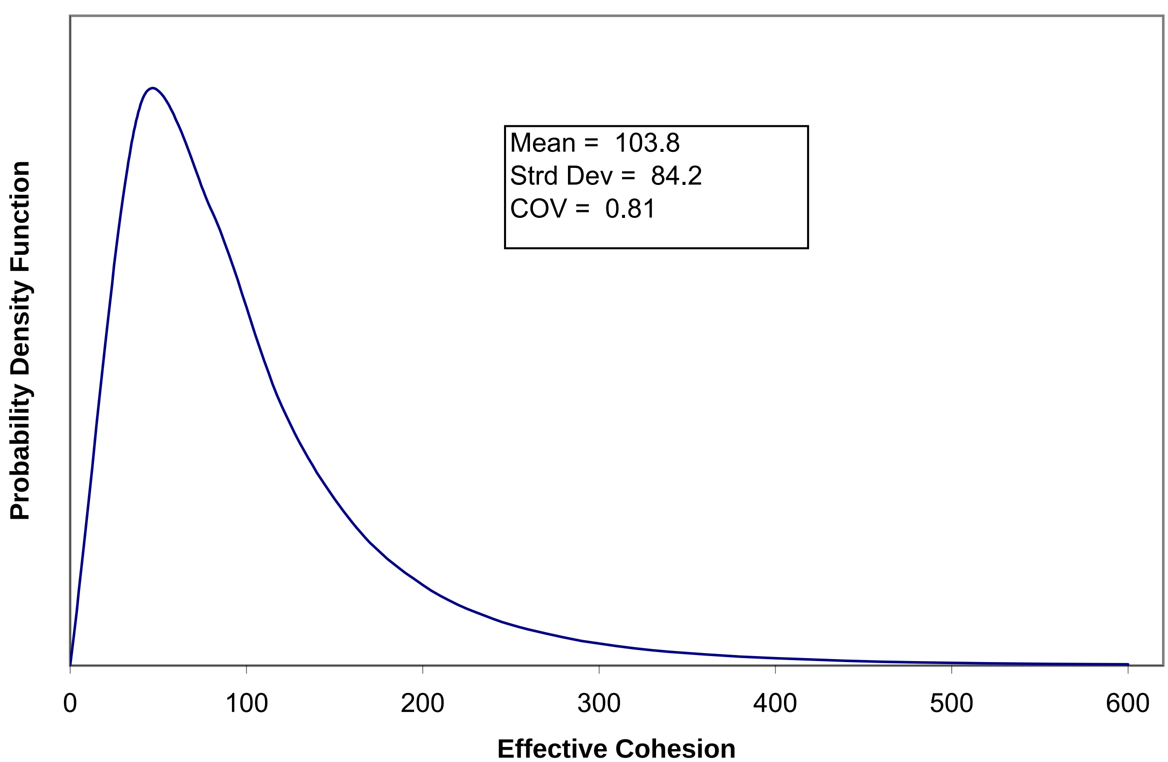
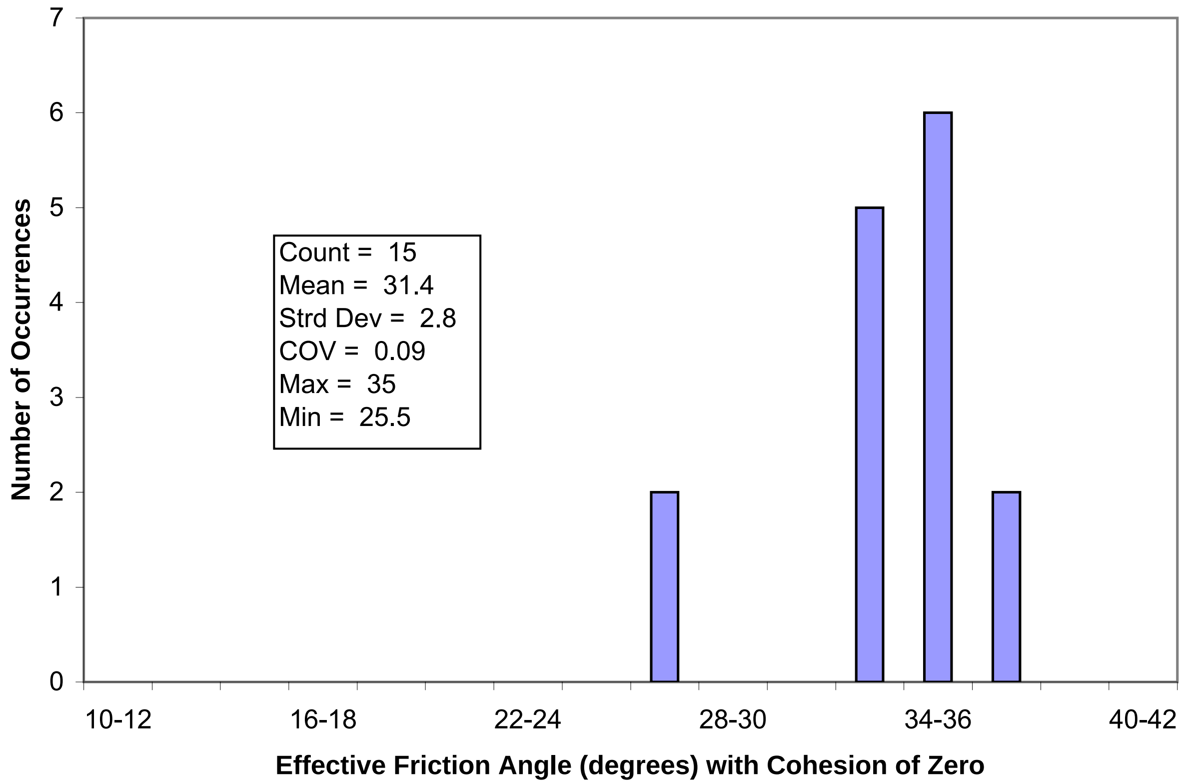




Figure C.6. Histogram and probability density function for saturated unit weight of silt (ML).



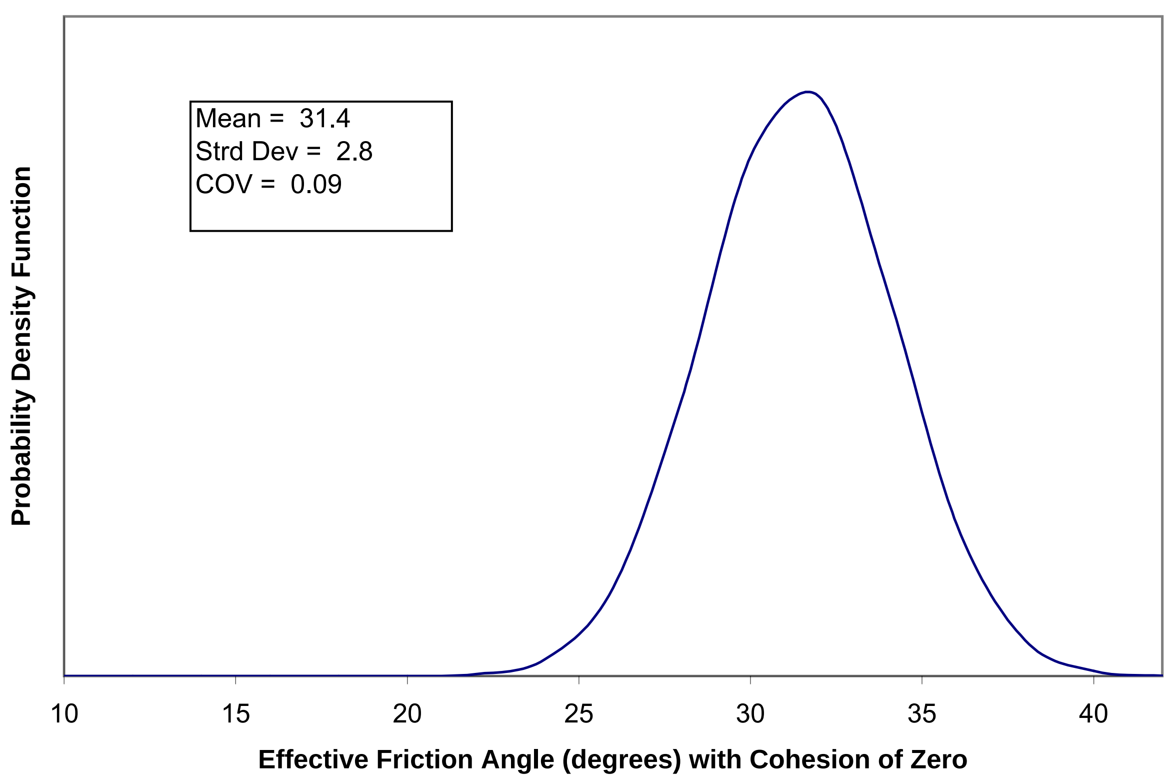
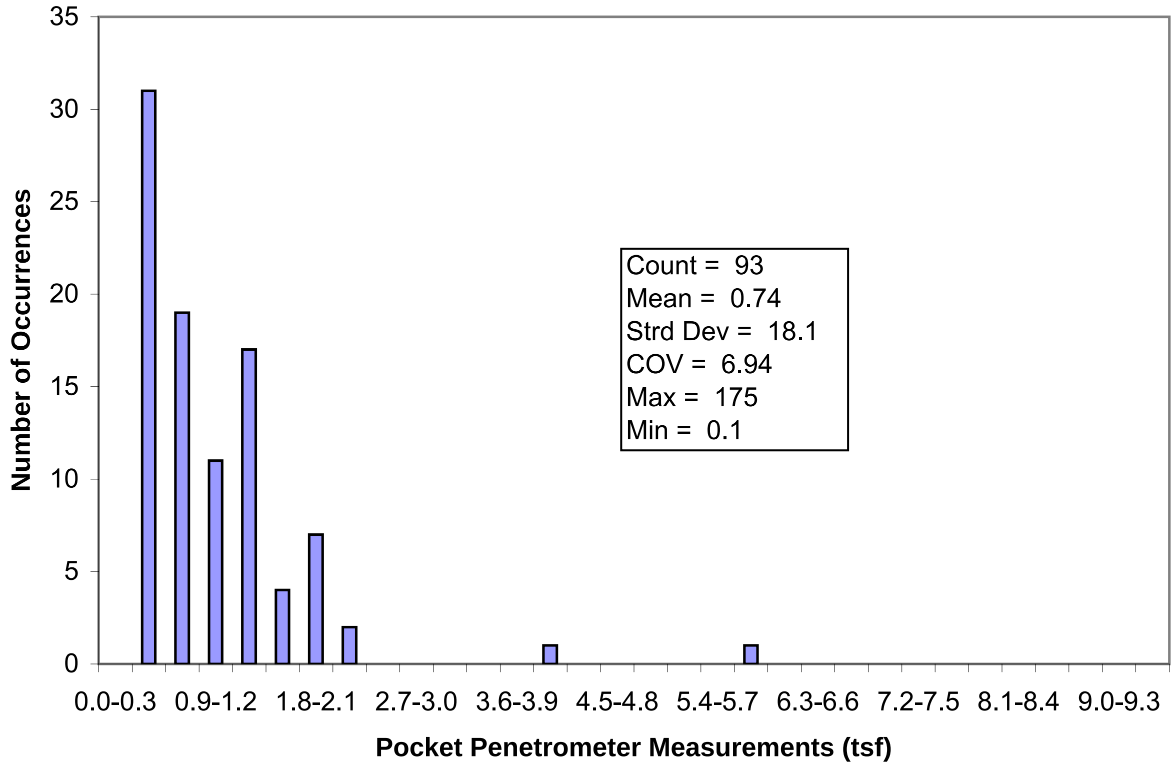




Figure C.7. Histogram and probability density function for effective friction angle, with zero cohesion, of silt (ML).

# APPENDIX D

# HISTOGRAMS AND DISTRIBUTIONS FOR SILTY CLAYS



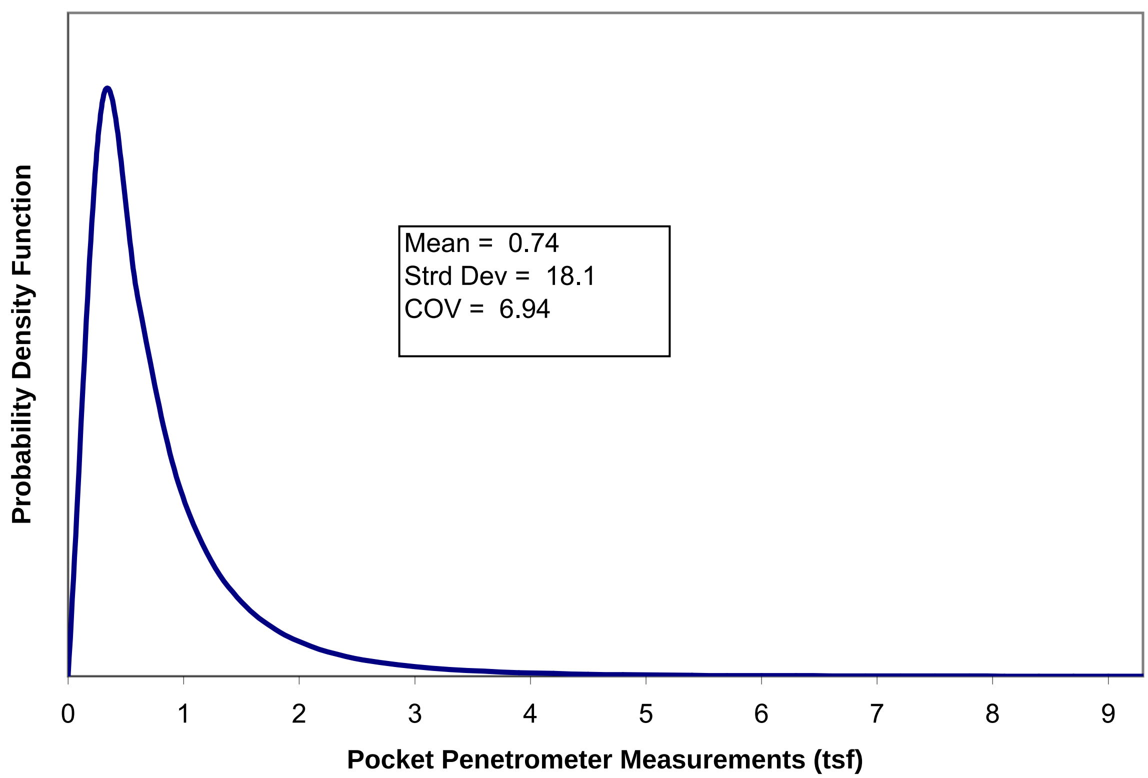
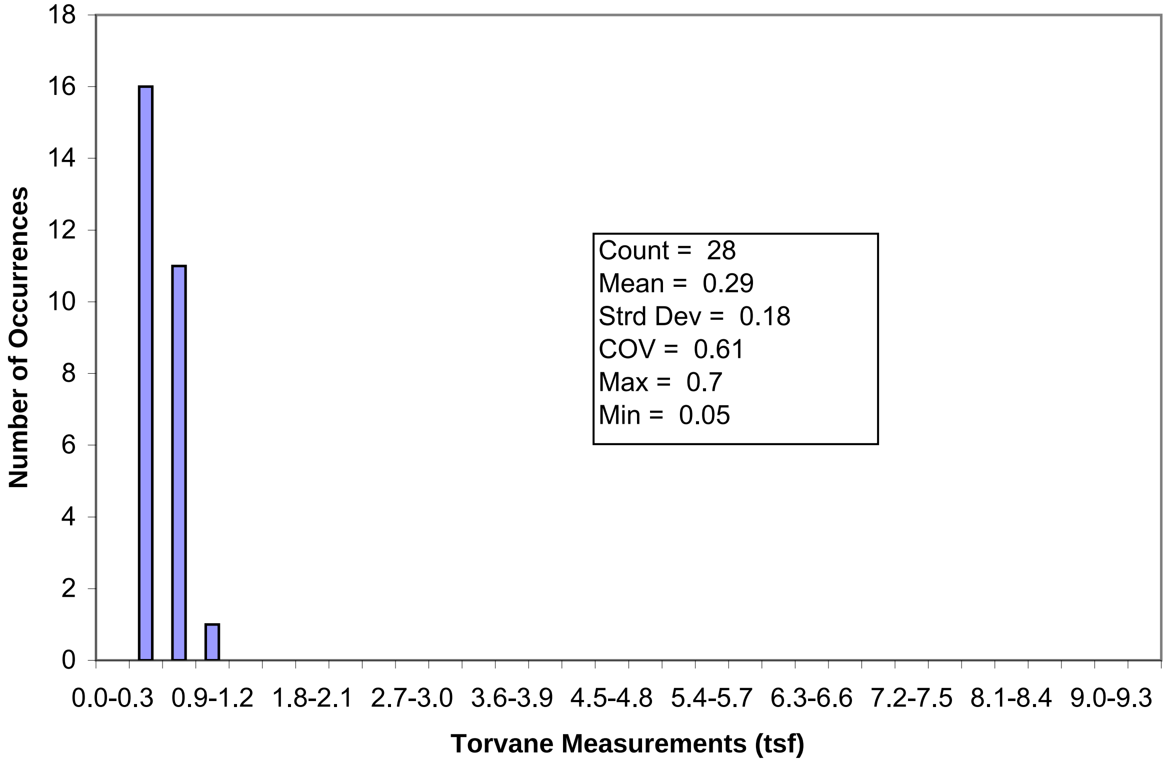




Figure D.1. Histogram and probability density function for pocket penetrometer measurements for silty clay (CL-ML).



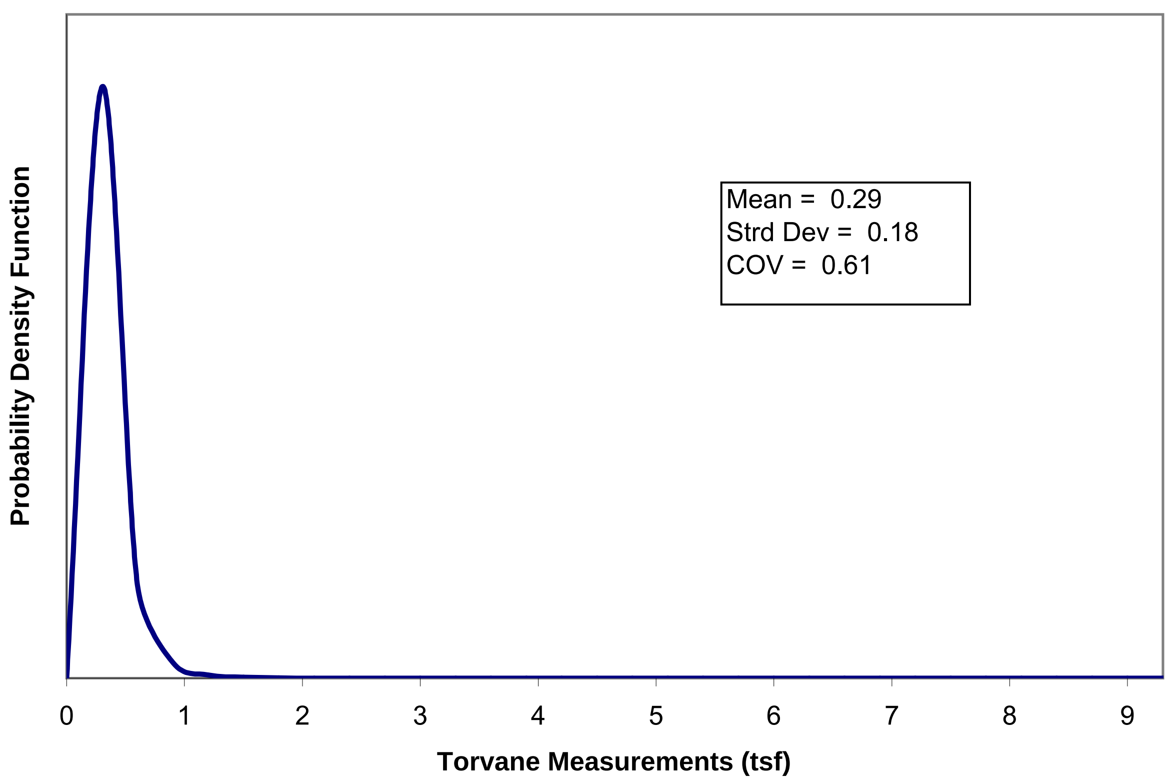
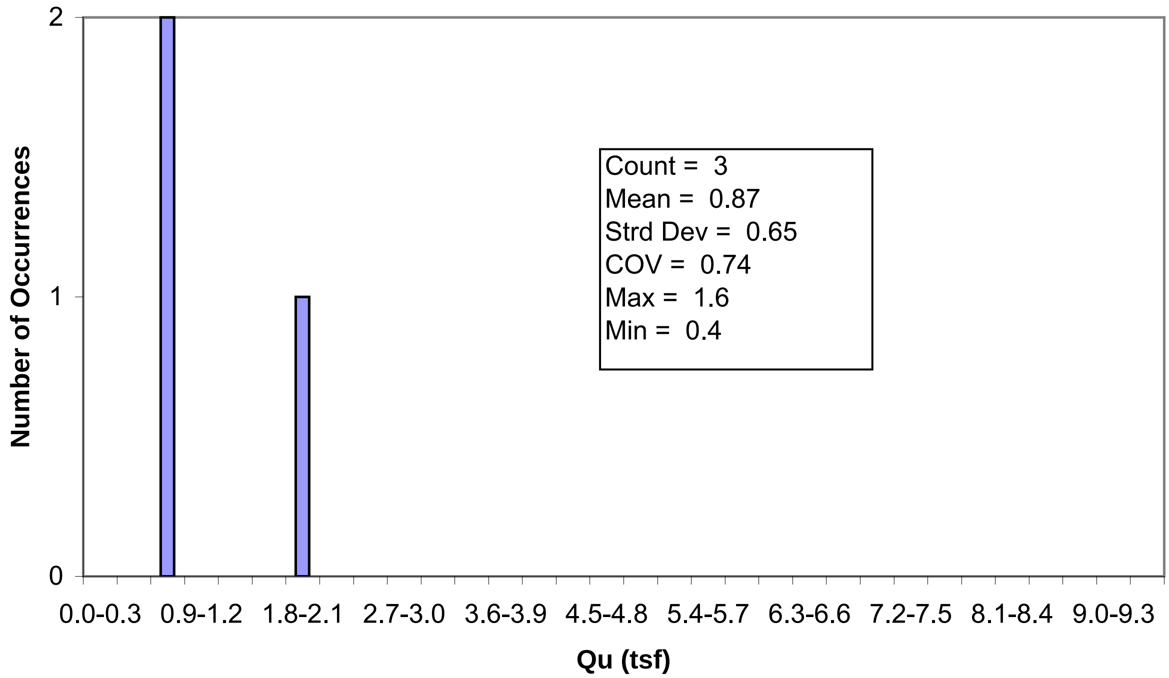




Figure D.2. Histogram and probability density function for torvane measurements for silty clay (CL-ML).



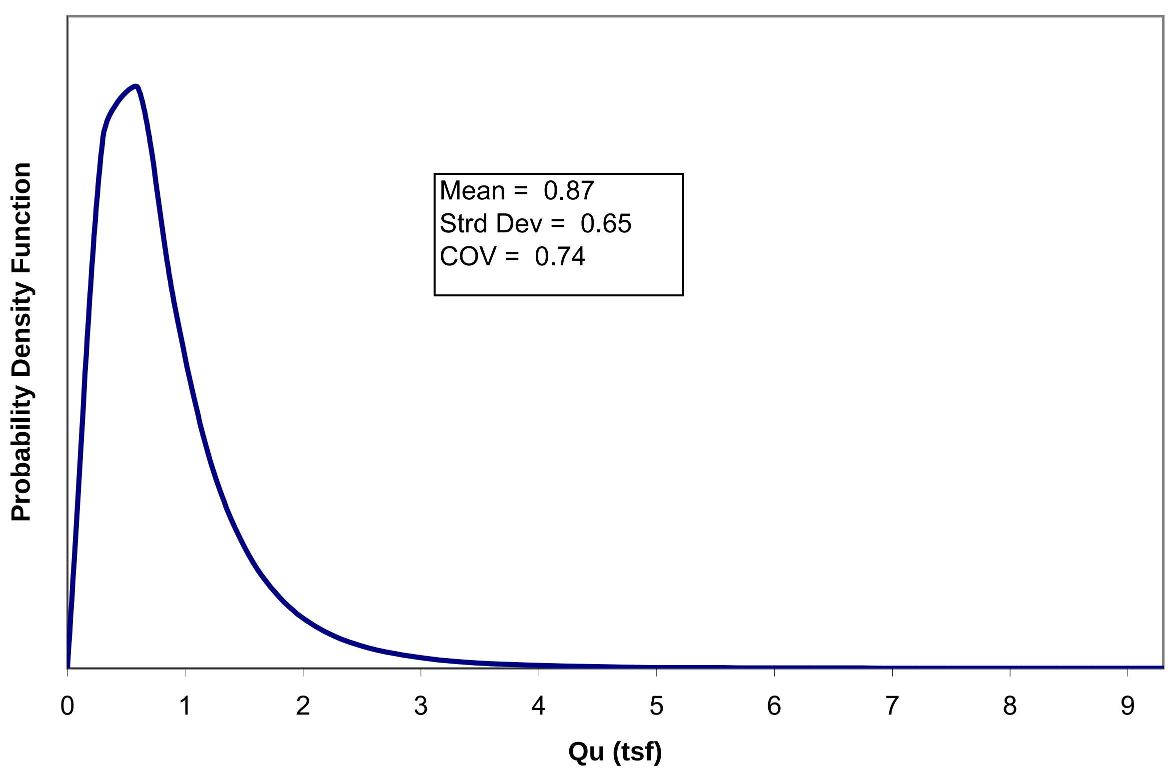
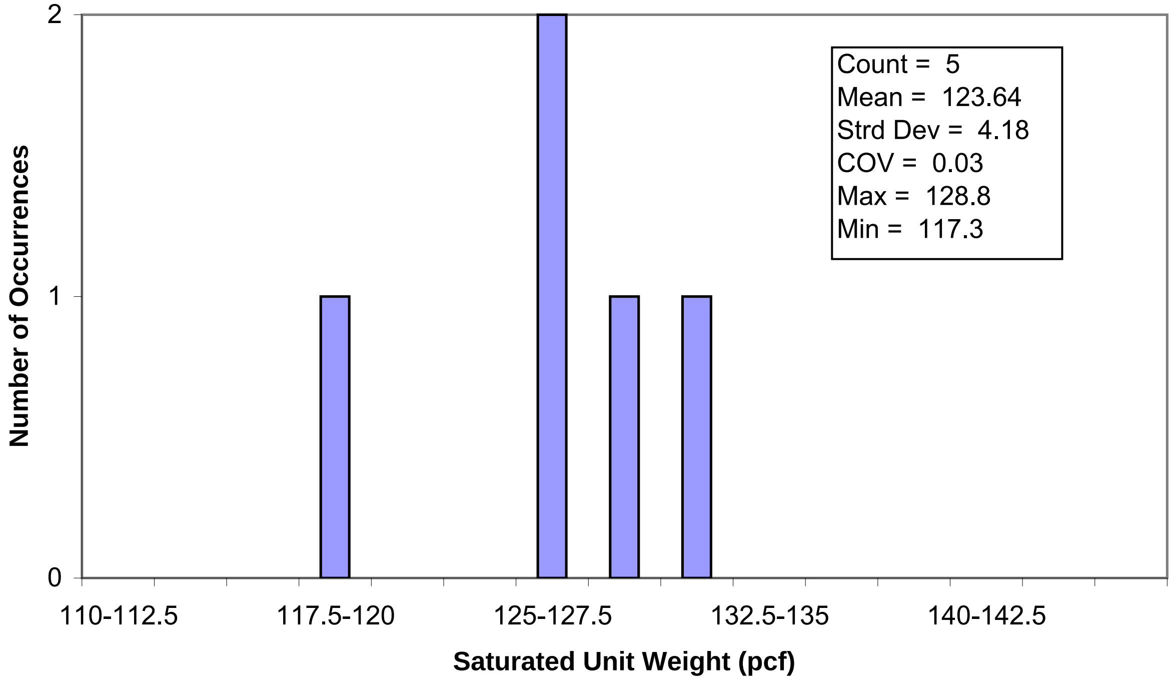




Figure D.3. Histogram and probability density function for unconfined compression tests for silty clay (CL-ML).



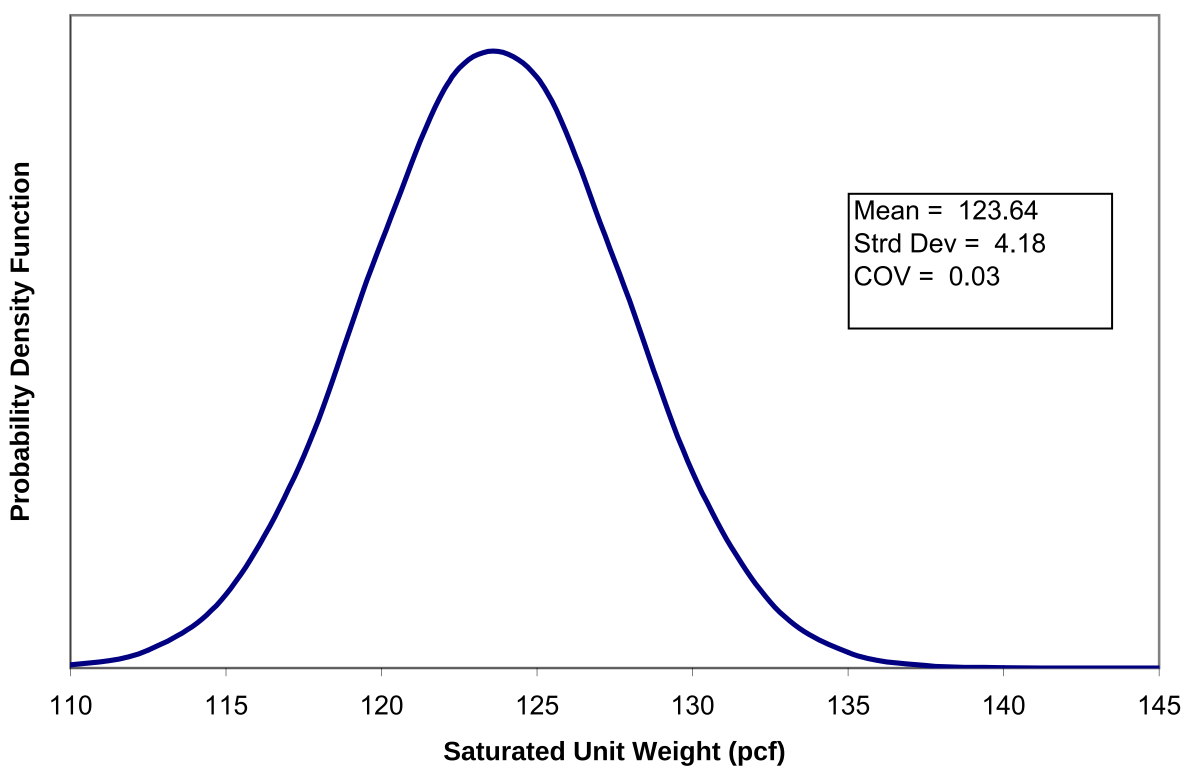
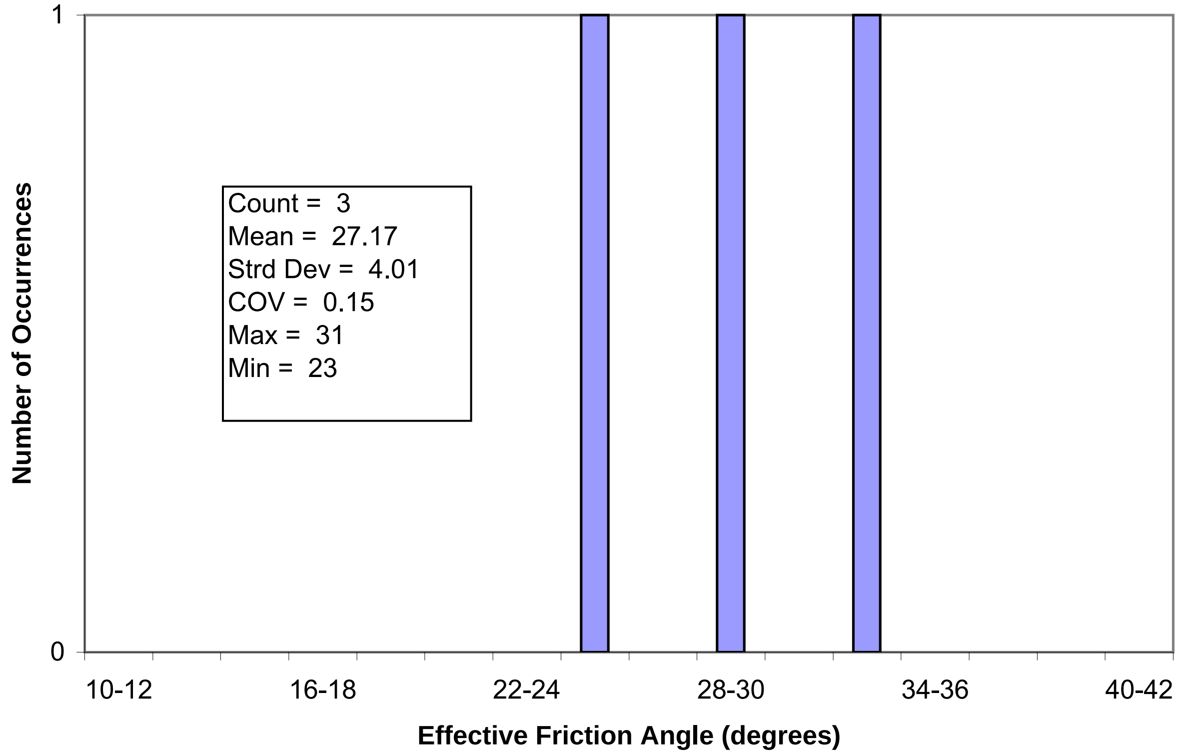




Figure D.4. Histogram and probability density function for saturated unit weight of silty clay (CL-ML).



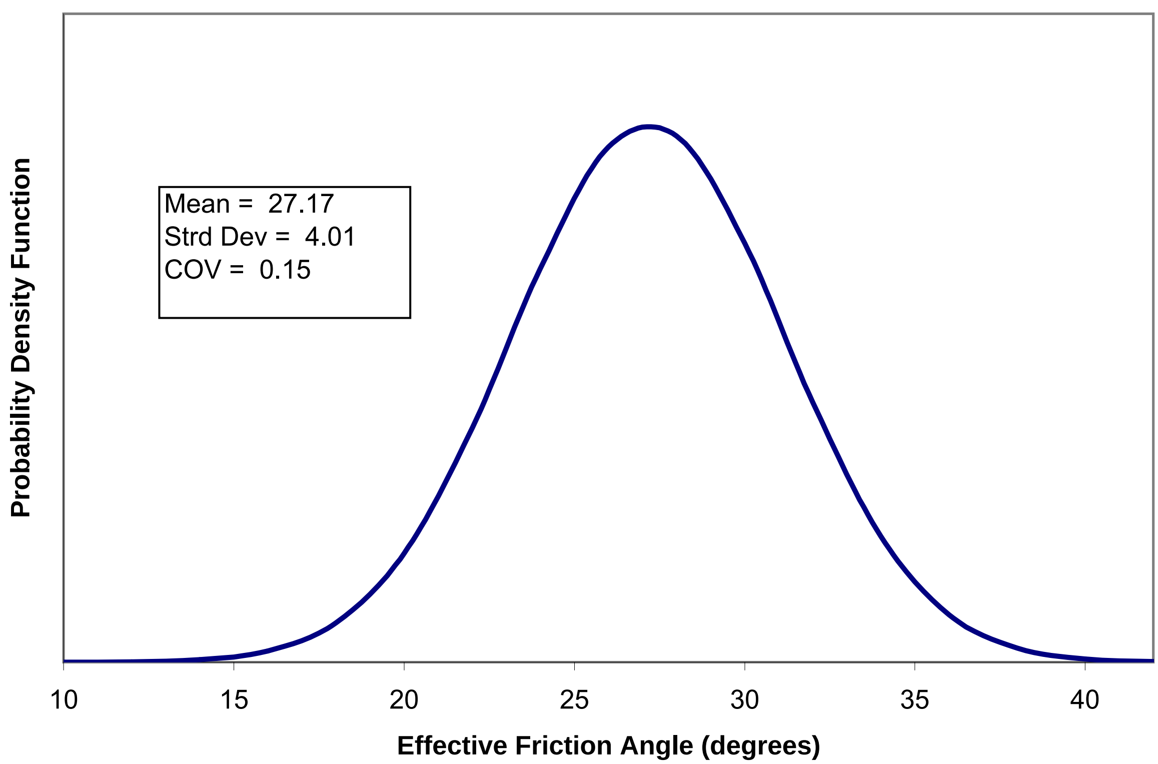
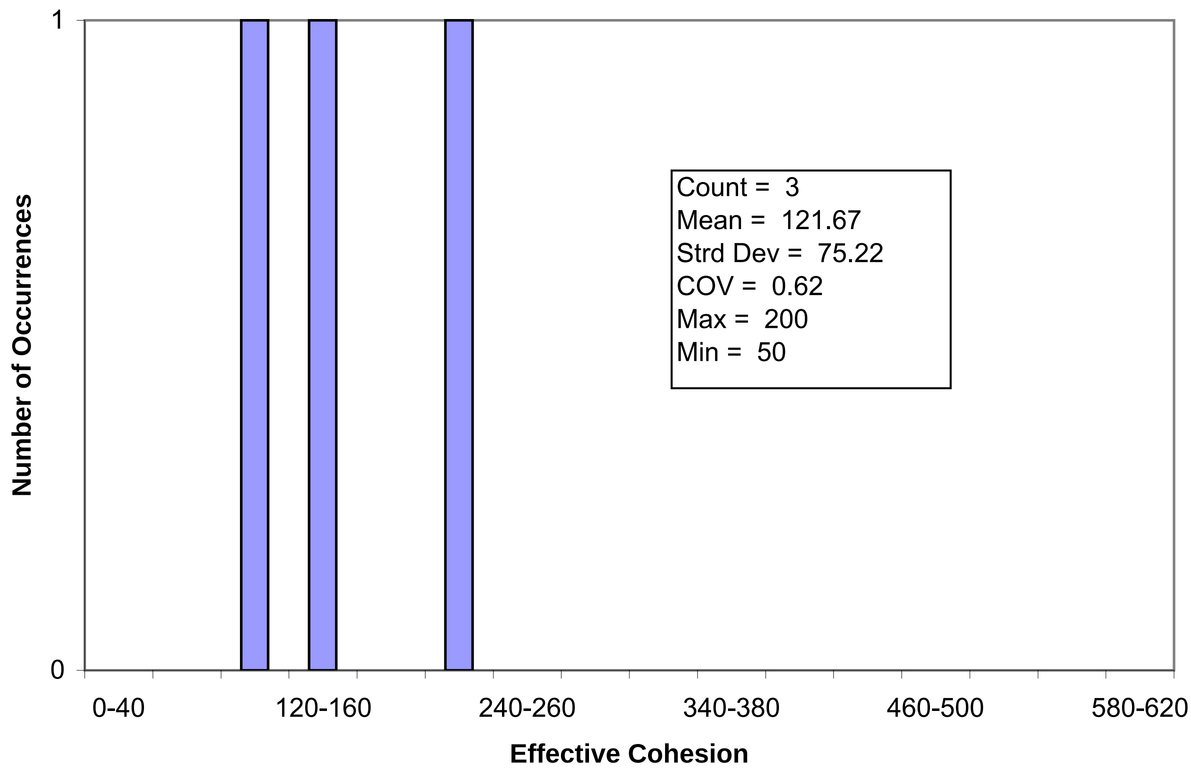




Figure D.5. Histogram and probability density function for effective friction angle for silty clay (CL-ML).



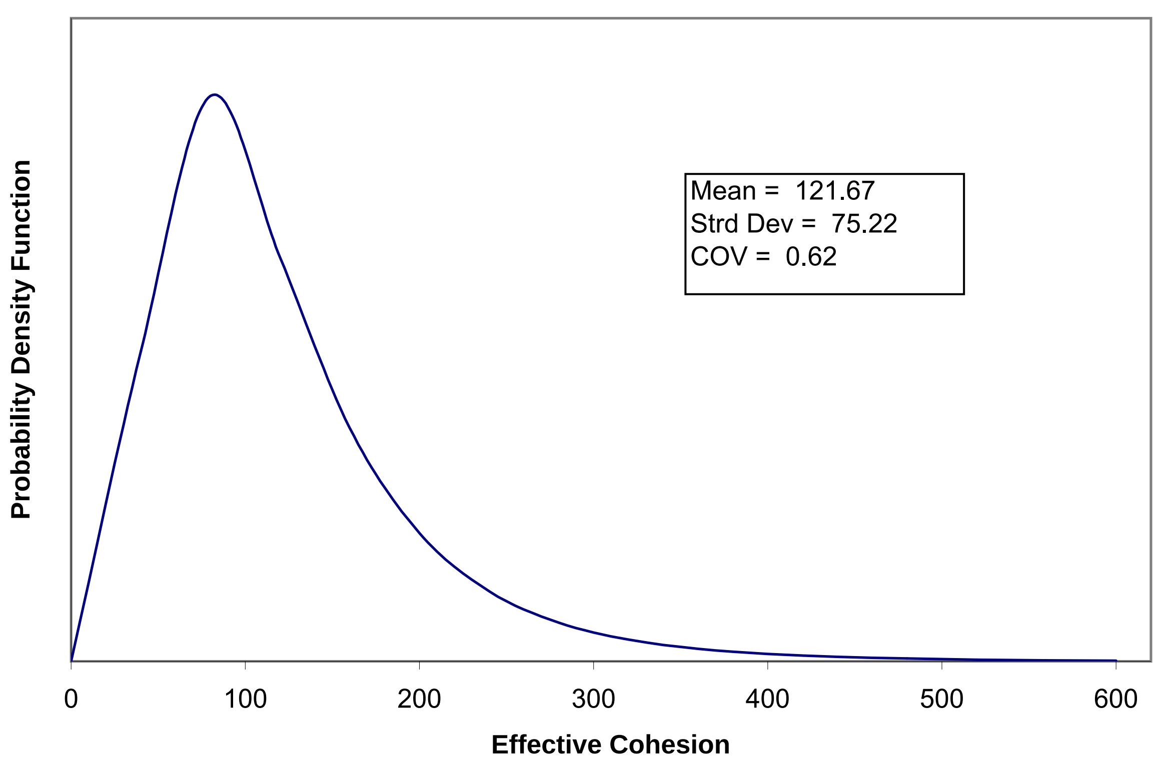
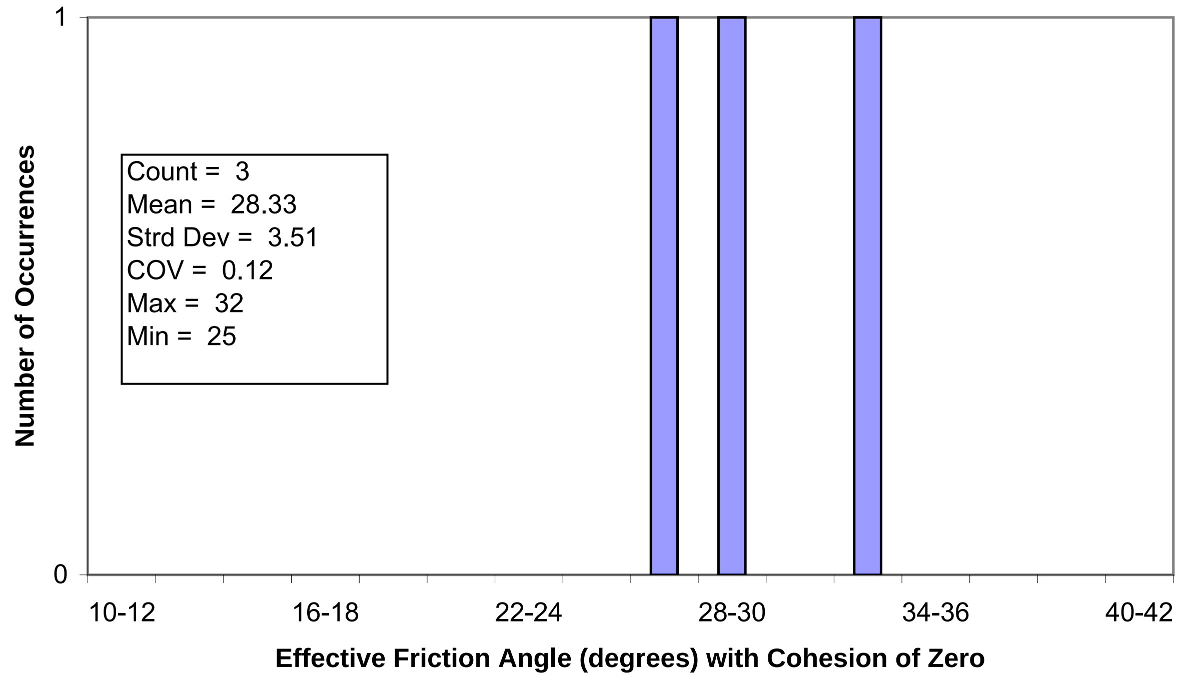




Figure D.6. Histogram and probability density function for effective cohesion of silty clay (CL-ML).



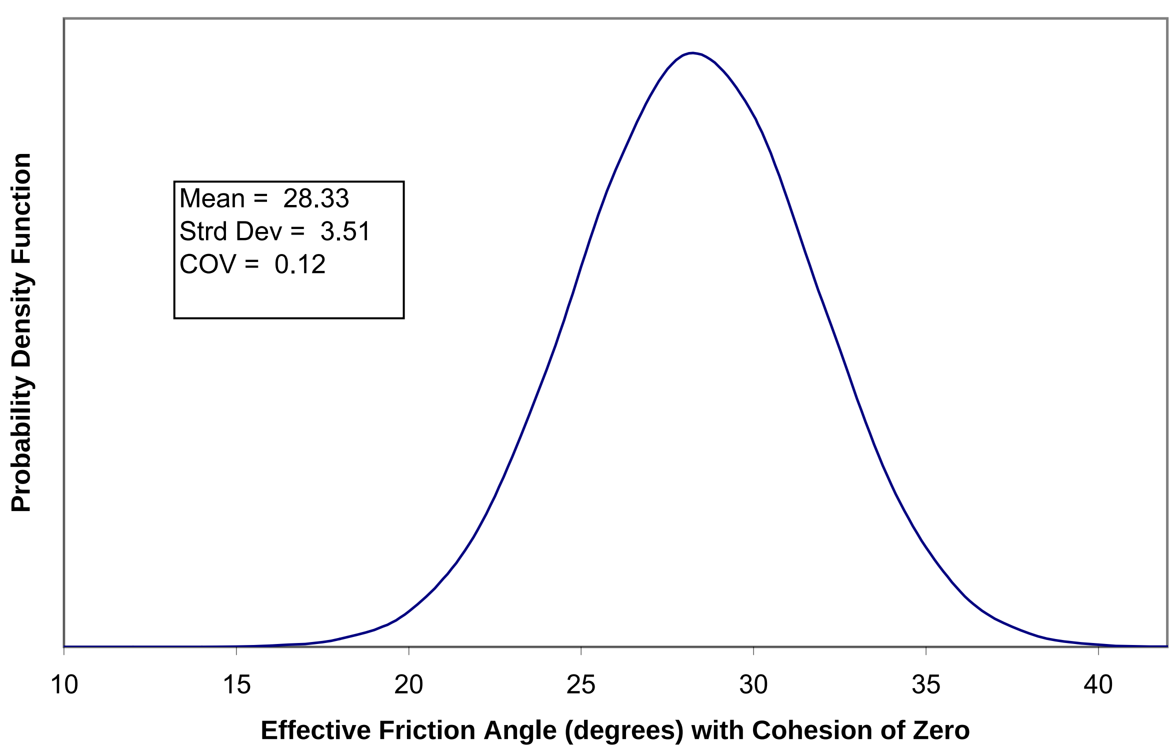




Figure D.7. Histogram and probability density function for effective friction angle, with zero cohesion, of silty clay (CL-ML).