## 7.1   Overview

Slope stability analysis is used in a wide variety of geotechnical engineering problems, including, but not limited to, the following:

- Determination of stable cut and fill slopes
- Assessment of overall stability of retaining walls, including global and compound stability (includes permanent systems and temporary shoring systems)
- Assessment of overall stability of shallow and deep foundations for structures located on slopes or over potentially unstable soils, including the determination of lateral forces applied to foundations and walls due to potentially unstable slopes
- Stability assessment of landslides (mechanisms of failure, and determination of design properties through back-analysis), and design of mitigation techniques to improve stability
- Evaluation of instability due to liquefaction

Types of slope stability analyses include rotational slope failure, translational failure, irregular surfaces of sliding, and infinite slope failure. Stability analysis techniques specific to rock slopes, other than highly fractured rock masses that can in effect be treated as soil, are described in Chapter 12. Detailed stability assessment of landslides is described in Chapter 13.

## 7.2   Development of Design Parameters and Other Input Data for Slope Stability Analysis

The input data needed for slope stability analysis is described in Chapter 2 for site investigation considerations, Chapters 9 and 10 for fills and cuts, and Chapter 13 for landslides. Chapter 5 provides requirements for the assessment of design property input parameters.

Detailed assessment of soil and rock stratigraphy is critical to the proper assessment of slope stability, and is in itself a direct input parameter for slope stability analysis. It is important to define any thin weak layers present, the presence of slickensides, etc., as these fine details of the stratigraphy could control the stability of the slope in question. Knowledge of the geologic nature of the <u>strata</u> present at the site and knowledge of past performance of such <u>strata</u> may also be critical factors in the assessment of slope stability. <u>See</u> Chapter 5 <u>for additional requirements and discussion regarding the determination and characterization of geologic strata and the determination of ESU's for design purposes.</u>

Whether long-term or short-term stability is in view, and which will control the stability of the slope, will affect the selection of soil and rock shear strength parameters used as input in the analysis. For short-term stability analysis, undrained shear strength parameters should be obtained. For long-term stability analysis, drained shear strength parameters should be obtained. For assessing the stability of landslides, residual shear strength parameters will be needed, since the soil has in such has typically deformed

enough to reach a residual value. For highly overconsolidated clays, such as the Seattle clays (e.g., Lawton Formation), if the slope is relatively free to deform after the cut is made or is otherwise unloaded, <u>even if a structure such as a wall is placed to retain the slope after that deformation has already occurred,</u> residual shear strength parameters should be obtained and used for the stability analysis. See Chapter 5 for <u>requirements</u> on the development of shear strength parameters.

Detailed assessment of the groundwater regime within and beneath the slope/landslide mass is also critical. Detailed piezometric data at multiple locations and depths within and below the slope will likely be needed, depending on the geologic complexity of the stratigraphy and groundwater conditions. Potential seepage at the face of the slope must be assessed and addressed. In some cases, detailed flow net analysis may be needed. If seepage does exit at the slope face, the potential for soil piping should also be assessed as a slope stability failure mechanism, especially in highly erodable silts and sands. If groundwater varies seasonally, long-term monitoring of the groundwater levels in the soil should be conducted. If groundwater levels tend to be responsive to significant rainfall events, the long-term groundwater monitoring should be continuous, and on-site rainfall data collection should also be considered.

## 7.3   Design Requirements

Limit equilibrium methods shall be used to assess slope stability. The Modified Bishop, simplified Janbu, Spencer, or other widely accepted slope stability analysis methods should be used for rotational, translational and irregular surface failure mechanisms. <u>Each limit equilibrium method varies with regard to assumptions used and how stability is determined. Therefore, a minimum of two limit equilibrium methods should be used and compared to one another to ensure that the the level of safety in the slope is accurately assessed.</u> In cases where the stability failure mechanisms anticipated are not well modeled by limit equilibrium techniques, or if deformation analysis of the slope is required, more sophisticated analysis techniques (e.g., finite difference methods such as is used by the computer program FLAC) may be used in addition to the limit equilibrium methodologies. Since these more sophisticated methods are quite sensitive to the quality of the input data and the details of the model setup, including the selection of constitutive models used to represent the material properties and behavior, limit equilibrium methods should also be used in such cases<u>, and input parameters should be measured or assessed from back-analysis techniques whenever possible.</u> If the differences in the results are significant, <u>the reasons for the differences shall be assessed with consideration to</u> any available field observations to assess the correctness of the design model used. <u>If the reasons for the differences cannot be assessed, and if the FLAC model provides a less conservative result than the limit equilibrium based methods, the limit equilibrium based methods shall govern the design.</u>

If the potential slope failure mechanism is anticipated to be relatively shallow and parallel to the slope face, with or without seepage affects, an infinite slope analysis should be conducted. Typically, slope heights of 15 to 20 feet or more are required to have this type of failure mechanism. For infinite slopes consisting of cohesionless soils that are either above the water table or that are fully submerged, the factor of safety for slope stability is determined as follows:

$$FS = \frac{Tan\phi}{Tan\beta} \qquad\qquad (7\text{-}1)$$

Where:
$\phi$    =    the angle of internal friction for the soil
$\beta$    =    the slope angle relative to the horizontal

For infinite slopes that have seepage at the slope face, the factor of safety for slope stability is determined as follows:

$$FS = \left(\frac{\gamma_b}{\gamma s}\right) \frac{Tan\phi}{Tan\beta} \qquad\qquad (7\text{-}2)$$

Where:
$\gamma b$    =    the buoyant unit weight of the soil
$\gamma s$    =    the saturated unit weight of the soil

Considering that the buoyant unit weight is roughly one-half of the saturated unit weight, seepage on the slope face can reduce the factor of safety by a factor of two, a condition which should obviously be avoided through some type of drainage if at all possible; otherwise much flatter slopes will be needed. When using the infinite slope method, if the FS is near or below 1.0 to 1.15, severe erosion or shallow slumping is likely. Vegetation on the slope can help to reduce this problem, as the vegetation roots add cohesion to the surficial soil, improving stability. Note that conducting an infinite slope analysis does not preclude the need to check for deeper slope failure mechanisms, such as would be assessed by the Modified Bishop or similar methods listed above.

Translational (block) or noncircular searches are generally more appropriate for modeling thin weak layers or suspected planes of weakness, and for modeling stability of long natural slopes or of geologic strata with pronounced shear strength anisotropy (e.g., due to layered/bedded macrostructure or pre-existing fracture patterns). If there is a disparately strong unit either below or above a thin weak unit, the user must ensure that the modeled failure plane lies within the suspected weak unit so that the most critical failure surface is modeled as accurately as possible. Circular searches for these types of conditions should generally be avoided as they do not generally model the most critical failure surface.

For very simplified cases, design charts to assess slope stability are available. Examples of simplified design charts are provided in NAVFAC DM-7 (US Department of Defense, 2005). These charts are for a c-φ soil, and apply only to relatively uniform soil conditions within and below the cut slope. They do not apply to fills over relatively soft ground, as well as to cuts in primarily cohesive soils. Since these charts are for a c-φ soil, a small cohesion will be needed to perform the calculation. If these charts are to be used, it is recommended that a cohesion of 50 to 100 psf be used in combination with the soil friction angle obtained from SPT correlation for relatively clean sands and gravels. For silty to very silty sands and gravels, the cohesion could be increased to 100 to 200 psf, but with the friction angle from SPT correlation (see Chapter 5) reduced by 2 to 3 degrees, if it is not feasible to obtain undisturbed soil samples suitable for laboratory testing to measure the soil shear strength directly. This should be considered general guidance, and good engineering judgment should be applied when selecting soil parameters for this type of an analysis. Simplified design charts shall only be used for final design of non-critical slopes that are approximately 10 feet

in height or less and that are consistent with the simplified assumptions used by the design chart. Simplified design charts may be used as applicable for larger slopes for preliminary design.

The detailed guidance for slope stability analysis provided by Abramson, et al. (1996) should be used.

For additional design requirements for temporary slopes, including application of the applicable WAC's, see Sections 15.7 and 9.5.5.

## 7.4   Resistance Factors and Safety Factors for Slope Stability Analysis

For overall stability analysis of walls and structure foundations, design shall be consistent with Chapters 6, 8 and 15 and the AASHTO LRFD Bridge Design Specifications. For slopes adjacent to but not directly supporting structures, a maximum resistance factor of 0.75 should be used. For foundations on slopes that support structures such as bridges and retaining walls, a maximum resistance factor of 0.65 should be used. <u>This reduced resistance factor also applies if the slope is not directly supporting the structure, but if slope failure occurred, it could impact and damage the structure.</u> Exceptions to this could include minor walls that have a minimal impact on the stability of the existing slope, in which the 0.75 resistance factor may be used. Since these resistance factors are combined with a load factor of 1.0 (overall stability is assessed as a service limit state only), these resistance factors of 0.75 and 0.65 are equivalent to a safety factor of 1.3 and 1.5, respectively.

For general slope stability analysis of permanent cuts, fills, and landslide repairs, a minimum safety factor of 1.25 should be used. Larger safety factors should be used if there is significant uncertainty in the analysis input parameters. The Monte Carlo simulation features now available in some slope stability computer programs may be used for this purpose, from which a probability of failure can be determined, provided a coefficient of variation for each of the input parameters can be ascertained. For considerations regarding the statistical characterization of input parameters, see Allen, et al. (2005). For minimum safety factors and resistance factors for temporary cuts, see Section 15.7.

For seismic analysis, if seismic analysis is conducted (see Chapter 6 for policies on this issue), a maximum resistance factor of 0.9 should be used for slopes involving or adjacent to walls and structure foundations. This is equivalent to a safety factor of 1.1. For other slopes (cuts, fills, and landslide repairs), a minimum safety factor of 1.05 shall be used.

| Conditions | Probability of Failure, Pf |
|---|---|
| Unacceptable in most cases | > 0.1 |
| Temporary structures with no potential life loss and low repair cost | 0.1 |
| Slope of riverbank at docks, no alternative docks, pier shutdown threatens operations | 0.01 to 0.02 |
| Low consequences of failure, repairs when time permits, repair cost less than cost to go to lower Pf | 0.01 |
| Existing large cut on interstate highway | 0.01 to 0.02 |
| New large cut (i.e., to be constructed) on interstate highway | 0.01 or less |
| Acceptable in most cases except if lives may be lost | 0.001 |
| Acceptable for all slopes | 0.0001 |
| Unnecessarily low | 0.00001 |

**Slope Stability – Probability of Failure (Adapted From Santamarina, et al., 1992)**
*Table 7-1*

## 7.5   References

Abramson, L., Boyce, G., Lee, T., and Sharma, S., 1996, *Slope Stability and Stabilization Methods*, Wiley, ISBN 0471106224.

Allen, T., Nowak, A., and Bathurst, R., 2005, Calibration to Determine Load and Resistance Factors for Geotechnical and Structural Design. TRB Circular E-C079, 83 pp.

Santamarina, J. C., Altschaeffl, A. G., and Chameau, J. L., 1992, "Reliability of Slopes: Incorporating Qualitative Information," Transportation Research Board, TRR 1343, Washington, D.C., pp. 1-5.

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