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GEOTECHNICAL DESIGN MANUAL - CHAPTER 7

CHAPTER 7-SLOPE STABILITY ANALYSIS

GEO-ENVIRONMENTAL SECTION

OREGON DEPARTMENT OF TRANSPORTATION

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SUMMARY OF CHANGES

Chapter	Summary of changes made	Date revised
7	Updated All Chapter Content	3/28/2018

7 Slope Stability Analysis

7.1 General

Slope stability analysis is used for typical geotechnical design tasks, including but not limited to the following:

- Design maximum inclinations of permanent cut and fill slopes;
- Design of temporary excavations and shoring systems;
- Design stability and bearing capacity of embankments supported on weak, soft foundation soils for staged-construction;
- Design global and compound stability of retaining walls; and
- Assess forces and deformations of bridge deep foundations from effects of seismic liquefaction/lateral spread and potentially unstable slopes.

Stability analysis techniques specific to rock slopes, are described in Chapter 12. Detailed stability assessment of landslides is described in Chapter 13.

Embankments that do not support structures with side slopes of 2H:1V or flatter would typically not require a project specific slope stability evaluation. Exceptions would include embankments constructed from highly plastic soils, soft subgrade conditions (deep organics, peat, diatomaceous soils, etc.), slopes subject to inundation, or other cases where, in the designer's judgement, analysis is warranted.

For cut slopes, excavations at 2H:1V or flatter in uniform native soils would also typically not require site specific evaluation. Layered formations, over consolidated clay soils, and colluvium (landslide debris) would be notable exceptions.

7.2 Limit Equilibrium

Slope stability design should be consistent with state-of-the-practice design guidelines, including but not limited to the referenced publications in Section 7.7. Slope stability design shall be evaluated using conventional limit equilibrium methods, and analyses should be performed using a state-of-the-practice slope stability computer program such as the most current versions of Slope/W® (Geo-Slope International), Slide® (Rocscience, Inc.), and/or ReSSA® (ADAMA Engineering, Inc.).

Limit equilibrium analysis procedures calculate the soil shear strengths that are at equilibrium with the applied soil shear stresses. In other words, the strengths that, if present, would result in a slope balanced between failure and stability. The relative stability of the slope is represented by the ratio of actual shear strengths to the equilibrium shear strengths (the ratio being the factor of safety).

Limit equilibrium analyses for the purposes of slope stability generally consist of splitting the soil mass into a series of vertical slices that can be analyzed in statics. The numerical solutions to these problems are, in general, statically indeterminate as there are more unknowns than the number of available equations. To resolve this, assumption must be made to reduce the number of unknowns. Most methods make assumptions with respect to the magnitude and ratio of interslice shear and normal forces. Perhaps of more consequence, the various methods vary with respect to whether they satisfy both moment and force equilibrium or merely force equilibrium. Methods that meet both force and moment equilibrium generally are more rigorous and preferred.

Engineers completing slope stability calculations should be familiar with the assumptions and limitations of the methods selected. Further, for more complex analyses, the use of multiple methods for comparison is prudent. The following table summarizes the methods typically included in slope stability software packages.

Table 7.1 Table Slope Stability Methods, Details and Assumptions

Method	**Force Equilibrium**	**Moment Equilibrium**	**Assumptions**
Ordinary or Fellenius		X	The slip surface is circular and the forces on the sides of the slices are neglected.
Bishop's simplified		X	The slip surface is circular and the forces on the sides of the slices are horizontal (no shear)
Janbu's simplified	X		The inclinations of the interslice forces are assumed
Spencer	X	X	Interslice forces are parallel and the position of the normal force on the base of the slice is assumed
Morgenstern-Price	X	X	Interslice shear force is related to interslice normal force by $X = \lambda f(x)E$ and the position of the normal force on the base of the slice is assumed
Corps of Engineers	X		The inclinations of the interslice forces are assumed
Lowe-Karafiath	X		The inclinations of the interslice forces are assumed
Sarma	X	X	Interslice shear force is related to the available interslice shear force, interslice shear strength depends on shear strength parameters, pore water pressures, and the horizontal component

As noted in the preceding table, limit equilibrium methods vary significantly with respect to the assumptions included and the limitations therein. Although all of the methods cited are technically valid, the use of methods that satisfy both force and moment equilibrium is more rigorous and would generally be preferred. A complete analysis would include using at least two analysis methods to evaluate whether or not a particular analysis is overly biased by the method's assumptions.

For projects of great complexity, it may be prudent to use more complex modeling such as three dimensional methods or finite difference methods such as the computer program FLAC. The complexities of such modeling make verification difficult. Complex analyses should be backed up with a parallel analysis using traditional methods to check that the results are broadly consistent with conventional analysis.

As noted in the documentation for Slope/W and most other slope stability software, a shallow face failure may be the controlling mechanism for a cohesionless slope. Frequently, that failure mechanism is precluded in reality by the localized cohesive nature of developed vegetation. Under those circumstances, the application of modest cohesion in the outer soils of the slope may be necessary.

7.2.1 Drained vs Undrained Analysis

The decision to select drained and/or undrained analyses requires knowledge of the loading regime, groundwater and seepage conditions, and soil permeability (as represented by consolidation characteristics). Frequently, short term stability analyses are said to utilize undrained strengths while long term stability analyses are said to utilize drained parameters. While broadly true, the actual issue is more complex.

An undrained analysis assumes that the analyzed load is applied faster than the excess pore water pressures resulting from the load can dissipate. Therefore, the presence of undrained conditions is heavily influenced by the nature of the soils and the drainage regime.

For saturated soils, the amount of time necessary to achieve a drained condition is governed by the following equation:

Equation 7.1

$$t_{99} = 4\frac{D^2}{c_v}$$

Where t_{99} is the time required to reach 99 percent dissipation of excess porewater pressures, D is the length of the drainage path, and c_v is the coefficient of consolidation. For sand and gravels, the value of c_v would generally be in excess of 100,000 ft^2/yr. As such, for granular soils the majority of loading conditions would be drained (rapid drawdown being a notable exception). For fine grained soils, the consolidation characteristics and drainage path length could be such that undrained conditions will exist for years after the new load is placed. However, fine grained soils with short drainage paths or in an unsaturated state could reach a drained condition during the placement of the load (as with a large, slow-progress embankment constructed on unsaturated silts).

The following table derived from FHWA (2005) summarizes the soil parameters typically used for analysis conditions.

Table 7.2 Shear Strengths, Drainage Condition, Pore Pressure and Unit Weights for Slope Stability Analysis

	Condition		
	Undrained/End of Construction	Intermediate/Multi-Stage Loading	Drained/Longterm
Analysis procedure and shear strength for free draining soils	Effective stress analysis, using c' and Φ'	Effective stress analysis, using c' and Φ'	Effective stress analysis, using c' and Φ'
Analysis procedure and shear strength for impermeable soils	Total stress analysis, using c and Φ from in situ, UU, or CU tests	Total stress analysis, using c_u from CU tests and estimate of consolidation pressure	Effective stress analysis, using c' and Φ'

	Condition		
	Undrained/End of Construction	Intermediate/Multi-Stage Loading	Drained/Longterm
Internal pore pressures	For total stress analysis, no internal pore pressure, set μ equal to zero.	For total stress analysis, no internal pore pressure, set μ equal to zero.	μ from seepage analyses
	For effective stress analysis, μ from seepage analyses	For effective stress analysis, μ from seepage analyses	
External water pressures	Include	Include	Include
Unit Weights	Total	Total	Total
Note: Multi-stage loading includes stage construction, rapid drawdown, and any other condition where a period of consolidation under one set of loads is followed by a change in load under undrained condition.			

(From Table 5-2 FHWA-NHI-05-123)

7.3 Geotechnical Design Parameters for Slope Stability Analysis

Geotechnical soil and rock design parameters are required for slope stability analysis with strength parameters developed using methodologies presented in Chapter 5 and the other referenced publications in Section 7.7. Slope stability analysis should consider the cases of short-term and long-term stability using appropriate soil strength parameters, groundwater, and piezometric levels.

7.3.1 Soil Distribution and Cross Section

The goal of geologic research, site reconnaissance, subsurface explorations, laboratory testing, and site monitoring is to allow for the development of a model of the subsurface conditions that are likely to influence the project. For the purpose of slope stability evaluation, the overall site model will ultimately need to be summarized into a series of two dimensional cross sections. The development of stability cross sections often requires significant levels professional judgement. The review and involvement of multiple professionals in this portion of a project is always advantageous.

It should be noted that the stability cross section and the geologic cross section may be different. The stability cross section is intended to show zones of material that will exhibit similar broad responses to stress and strain. Fine detail and differentiation that may be appropriate for a geologic cross section may be overly complex for stability modeling. Conversely, it may be advantageous to split geologic formations into layering or zones that will behave differently under stress and strain but may be of one geologic origin.

Stability cross sections should be based on careful review of all of the data available to the designer. Further, the layout of the cross section should be iterative as the analysis progresses. The sensitivity of the model to assumed material transitions should be evaluated as sometimes moving a material boundary as little as six inches can have a significant impact on the analysis results. As the analysis develops, the designer should be looking for critical features and parameters that control the results. For example, if the critical failure surfaces tend to converge on a single location, the topography and shear strength in that location should be evaluated for conformity to the overall model.

With respect to tilted or sloping layering, a variety of approaches are available for analysis of this condition. One approach would be to create a series of layers that roughly follow the mapped layering with interstitial weak layers. That approach implies a level of knowledge with respect to layering that may not exist. Alternatively, anisotropic strength values can be input for a single layer. The advantage of using anisotropic strength is that the sensitivity of the analysis to the angle of the anisotropy and the directional strength ratio can be easily evaluated and can be varied in a statisitical analysis.

7.3.2 Pore Pressure, Seepage, and Groundwater

Most slope failures are the result of reductions in effective stress associated with groundwater, inundation, and/or seepage. Detailed assessment of the pore water pressure and seepage regime within and beneath the slope is therefore critical in stability modeling.

Long term, multi season or year monitoring of piezometric data at multiple locations within the site represent the best opportunity for developing an accurate and representative model for pore pressures and seepage forces within the analyzed slope. Such data should be supplemented with field observations of seeps, springs, surface inundation, as well as published data with respect to groundwater.

In reality, many small projects are of too limited a duration to allow for long term monitoring. When such data is not available, a conservative assumption with respect to the groundwater regime should be made, based on subsurface explorations, surface observations, and knowledge of local geology and hydrogeology.

Depending on the complexity of the seepage and groundwater regime, a detailed analysis either by hand with flow nets or using software such as Seep/W may be required. Such cases would include rapid drawdown (discussed in a subsequent subsection) and drains installed for landslide remediation.

7.3.2.1 Submerged Slopes

Fully submerged slopes are relatively straightforward to analyze and are generally addressed using buoyant weights and total stress analysis. A completely submerged slope would be unlikely to be included in an ODOT project.

Much more common would be analysis of partially submerged slopes. Examples of partially submerged slopes would include roadway embankments that toe out in bodies of waters such as lakes and rivers or dam and pond side slopes for stormwater facilities. The choices for analyzing the submerged portion are to use total weights with applied surface water forces or buoyant weights with seepage forces. Merely using buoyant weights for the portions below the water surface, or defining a water surface in space ignores the buttressing effect of the free water. The standing water should be modeled as a normal force derived from hydrostatic pressures.

7.3.2.2 Rapid Drawdown

Rapid drawdown is a case where the water level adjacent to the slope lowers at a rate faster than the hydrostatic pressures can equilibrate through seepage. This condition can occur on the slopes adjacent to a reservoir, river, or canal following a long period of rainfall accumulation, planned lowering of water through control structures, or failure of water impoundment structure. Rapid drawdown is most prevalent in clayey slopes in which the excess pore water pressures do not dissipate as the water recedes, thereby keeping the overall shear strength low.

As with any stability case, an analysis using effective stress techniques coupled with in-depth knowledge of porewater and seepage pressures can be completed. Development of such a model for a rapid drawdown cased would require a seepage analysis of a draining slope. Although hand calculation methods exist, it would generally be preferable to use computerized models such as Seep/W, with the results ported to Slope/W for the stability calculations.

Rapid drawdown is discussed in detail in the US Army Corps of Engineers Slope Stability Manual (2003). That document summarizes the original 1970 Corps of Engineers method that requires two sets of stability analyses. The first analysis is based on the conditions present just before the drawdown. That analysis determines the consolidation effective stresses to which the soils have been subjected. The effective stress results from the first analysis are used in the second analysis to estimate the undrained shear strengths that would exist during rapid drawdown. The reported factor of safety for the analysis is derived from the second analysis. This method is generally overly conservative and is not in broad use today.

Also described in the Corps of Engineers manual and incorporated into Slope/w is the methodology presented by Duncan, Wright, and Wong in 1970. That methodology incorporates a three stage analysis.

As with the original Corps of Engineers method, the first stage involves the analysis of the embankment before drawdown to develop undrained shear strengths. The method adds a second stage after drawdown. The drained and undrained strengths along the slip surface resulting from the second stage are compared, and the lower of the two are used in the third stage. The third stage involves the stability analysis using the computed shear strength and the final drawdown water level. The computed factor of safety from the third stage is used to represent the overall analysis results.

7.3.3 Shear Strength

Shear strength is perhaps the most important parameter to determine when completing slope stability analysis. The most common model for shear strength is the Mohr-Coulomb strength envelope. In the most-simple terms, Mohr-Coulomb strength is modeled with the following equation:

Equation 7.2

$$s = c + \sigma \tan\emptyset$$

Where s is the shear strength, c is the cohesion intercept, σ is the normal stress on the failure plane at failure, and Ø is the angle of internal friction. The form of the Mohr-Coulomb strength envelope implies that the strength relationship is linear with respect to stress on the shear plane. In reality, the envelope is typically curved, with higher levels of curvature at lower confining pressures. If detailed strength data exists, the curved strength envelope can be directly entered into slope stability software and would be preferential to a straight line fit.

The development of shear strength values for use in stability modeling is a complex subject. Professionals completing stability modeling should be knowledgeable with respect to the limitations and risks associated with strength types. Detailed information on this subject is included in a number of the references, in particular Duncan (2015).

Ideally, shear strength values used in analysis would be derived from laboratory testing completed on appropriate samples collected at the site. However, appropriate samples for testing may not be available for all projects. Further, for new construction, the actual material to be placed in the embankments may not be known and the designer may have to rely on available specifications to assume the range of material that may be used in constructing embankments.

Shear strength values can be obtained from a variety of sources including assumed values based on knowledge of the regional geology to data resulting from complex testing of soil samples. The level of detail used in obtaining shear strengths should be consistent with the complexity and risk associated with the analyzed slope.

Typically, the shear strength of fine grained soils is determined based upon laboratory testing or back calculation analysis. Empirical correlations between field data and shear strength are limited to granular cohesionless soils. As noted in Peck, Hanson, and Thornburn, “The correlation (between strength and spt values) for clays can be regarded as no more than a crude approximation, but that for sands is often reliable enough to permit the use of N-values in foundation design.” The same would be true for stability evaluation.

Prior to conducting laboratory testing, the nature of the analysis (drained, undrained, or both) must be established based on a knowledge of the loading regime and a behavior of the soil under load. Development of undrained and drained strength parameters including a brief discussion of the test methods available is presented below. Detailed information is included in a number of the references, in particular Duncan (2015).

7.3.3.1 Drained Strength

Laboratory tests available to assist in developing drained strength parameters include Consolidated Drained (CD) triaxial test and direct shear test. Sample disturbance is an issue with undrained tests, particularly with samples that are not consolidated prior to testing.

Consolidated-Drained (CD) Triaxial Test

For Consolidated Drained tests, the sample is consolidated prior to application of deviator stresses. The deviator stresses are applied slowly to allow pore pressures built up by the shearing to dissipate. The test is strain-controlled and the rate of axial deformation is kept constant. Depending on the nature of the sample, CD tests can take a long time to complete in order to allow for the dissipation of excess porewater pressures.

Direct Shear Test

The Direct Shear test is the simplest form of shear test. The sample is placed in a metal shear box and is confined by a vertical stress. A horizontal force is applied to half of the sample and the sample fails by shearing along a defined plane. The test can be either be stress-controlled or strain-controlled. Typically the test is operated slowly enough to measure consolidated-drained conditions.

In order to evaluate larger strains associated with residual shear, torsional Direct Shear tests have been developed.

7.3.3.2 Undrained Strength

Laboratory tests available to assist in developing undrained strength parameters include Unconsolidated Undrained (UU) and Consolidated Undrained (CU) triaxial tests, unconfined compression tests, and direct simple shear tests. Sample disturbance is an issue with undrained tests, particularly with samples that are not consolidated prior to testing.

Unconsolidated–Undrained (UU) Triaxial Test

For an Unconsolidated Undrained (UU) test, no drainage is allowed during the application of confining pressure or shearing stress. Since there is no need to allow for drainage, the shearing stresses are applied relatively quickly. It is important that the moisture content of the soils during testing be consistent with field conditions.

The test results are presented in terms of total stresses and the results are used in total stress analyses. This test represents the ideal condition with respect to undrained loading in the field and if completed on a truly undisturbed sample. However, sample disturbance and lack of consolidation after sampling results in significant disturbance impacts.

Consolidated–Undrained (CU) Triaxial Test

Consolidated-Undrained test samples are allowed to drain during the consolidation stage. During this stage, the confining pressure is applied and the specimen is allowed to fully consolidate. Unlike the UU test, the sample is saturated, usually using back-pressure methods. When the consolidation stage is completed, the drainage system is closed to prevent further drainage. Typically, the porewater pressures are monitored and recorded during the shearing stage. Unlike the UU test, the shearing stresses are applied slowly in order to allow for the equalization of porewater pressures throughout the sample.

The results from CU tests are used for cases where sample disturbance is suspected or likely, for analysis of rapid drawdown, and in the analysis of staged embankment construction. And although a generally inferior test in terms of applicability of the results, CU tests are more common than CD tests owing to the significant lower time needed to complete the test. For dealing with disturbed samples, the SHANSEP and recompression approaches are available to minimize the effects of disturbance on the test results.

Unconfined Compression Test

The Unconfined Compression test is one of the fastest and least expensive methods of measuring shear strength. The method is only applicable to cohesive samples taken from undisturbed (thin wall) sampling. The sample is typically trimmed into a cylinder after extrusion. The ratio of length to diameter is between 2 and 2.5. The sample is tested in compression without confinement. The unconfined compressive strength (q_u) is the lesser of the maximum stress attained, or the stress at 15% axial strain. After testing, the sample is oven dried to determine its water content. The test is operated at speeds consistent with the assumption of drained loading. Porewater pressures are unknown during the test meaning that the effective strength cannot be determined. As such, the unconfined compressive strength is a total stress value, applicable to undrained analyses.

Direct Simple Shear Test

The Simple Shear test was developed in order to evaluate stress and strain within samples. The test creates a relatively homogeneous state of shear stress throughout the specimen, allowing for the evaluation of the stress path and sample deformation. This approach more closely models field conditions than the Direct Shear test but it less representative than triaxial testing.

Field Vane Shear Test

The field vane shear test is very effective in measuring the undrained strength of soft clays at somewhat shallow depths. The test consists of pushing a four bladed vane into the bottom of a borehole, test pit, or hand excavation. The device measures the torque needed to rotate the vane which is correlated to shear stresses on the resulting sheared cylinder of soil.

7.3.3.3 Strength Correlations and Assumed Values

Correlative approaches to developing undrained strengths include field vane shear, cpt, pressuremeter, dilatometer and SPT. Extensive correlations exist with respect to atterberg limit indices. As previously noted, estimating the strengths of cohesive soils from field measurements is generally discouraged. Exceptions would include fully softened and undrained strengths that may be reasonable to assume from atterburg limits tests and similar index tests.

AASHTO Section 10.4.6.2.4 contains direction with respect to estimating drained strengths of granular soils from SPT blowcounts.

7.3.3.4 Overconsolidated Clays

Strength loss in stiff clays exposed in excavations is well documented in literature and has been the cause of a number of the larger landslides impacting roadways in the Pacific Northwest. When subjected to deformation or changes in stress, the shear strength of stiff clays can drop from the initial undisturbed strength to values approaching the fully softened shear strength. This also occurs in embankments constructed from plastic clays, although the strength loss is generally limited to the outer 10 feet of the embankment face.

7.3.3.5 Cohesion

A parametric and/or statistical review of slope stability analyses results in the knowledge that even small values of cohesion can have a significant impact on the results. As such, the use of cohesion in slope stability modeling requires great care. Cohesion is not a property that should be assumed but rather, the cohesion intercept should be based on either significant and appropriate laboratory testing or a rigorous back calculation analysis based on a well-defined slide including measured movements at depth. An exception would be the assumption of modest levels of cohesion for the outer one to three feet of vegetated slopes, used to model the impact of vegetation in resisting shallow failures. Use of such a model should be applied cautiously.

Actual, rather than apparent, cohesion is present in soils due to cementation and/or electrostatic forces in clays associated with overconsolidation. Apparent cohesion is generally caused by suction or capillary forces in unsaturated soils.

Cohesion associated with cementation can potentially be relied on in design although cemented soils subject to changes in seepage, saturation, and stress may be subject to reductions in cementation and cohesion.

Whether or not the cohesion derives from cementation or overconsolidation, soils subjected to past movement, for which the shear planes are fully developed, are governed by residual shear and generally do not exhibit cohesion.

Therefore, the use of cohesion in the analysis of slope stability for highway cuts, embankments, and retaining walls should be limited to sites where laboratory tests document the cohesion and where the presence of cementation and/or overconsolidation is consistent with the geology. Additionally, the geology needs to clearly document that past landslide movement associated with development of residual shear strengths can be categorically ruled out. Finally, for overconsolidated clays, the soils should not be fractured or displaced, subject to changes in stress, saturation, or seepage. The inclusion of a cohesion value for the stability of colluvium or ancient landslide debris is always inappropriate.

7.3.4 Seismic Loads

Stability analysis under seismic loads is discussed in detail in Chapter 6.

7.4 Reliability and Resistance and Safety Factors for Slope Stability Analysis

For overall stability analysis of walls and structure foundations, design shall be consistent with Chapter 6, Chapter 8 and Chapter 15 and the *AASHTO LRFD Bridge Design Specifications*. This section contains minimum factors of safety for use in ODOT projects. Whether or not higher factors of safety should be applied will need to be based on the Engineer's judgement with respect to the level of information available in completing the analysis and the risk that actual conditions will vary from those assumed.

The following table summarizes minimum factors of safety (and maximum LRFD resistance factors) for projects where sufficient information exists to adequately define soil profile, slope geometry, soil shear strength and porewater pressure in the slope stability model:

Table 7.3 Minimum Required Factors of Safety and Maximum Required Resistance Factors for Slope Stability Analyses

Geotechnical Item Under Consideration	Minimum Factor of Safety	Maximum LRFD Resistance Factor	Notes
Slopes that Support Structures	1.5	0.65	AASHTO LRFD Bridge Design Specifications. Load Factor of 1.
Slopes adjacent to, but not supporting, structures	1.3	0.75	AASHTO LRFD Bridge Design Specifications. Load Factor of 1.
Embankment Side Slopes	1.25		
Cut Slopes	1.25		
Landslide Remediation	1.25		

For cases where parameters are assumed, and/or information is unknown, higher safety factors than presented would be applicable. Generally, the use of factors of safety in excess of 1.5 is unnecessary.

7.5 Specialized Analyses

This section presents specifics with respect to a variety of analysis cases that are common in highway geotechnical work.

7.5.1 Back Calculation

Back Calculation allows for the development of soil strength based on the configuration of an existing slope failure. The information provided from this type of analysis is invaluable and allows for the development of in situ parameters that are difficult to obtain through exploration and testing. The methodology relies on the concept that a slope failure at initiation and/or at rest is in equilibrium (a factor of safety of 1.0). Working backward from a known factor of safety allows for the slope to be modeled at the time that it failed.

The premise of back calculation is that a moving, or imminently moving, landslide is at a factor of safety of 1.0. In other words, as long as the material is moving but not accelerating, the forces that restrain movement must be in balance with the forces creating movement (equilibrium). A slope that hasn't failed is difficult to evaluate with respect to back calculation.

It is important to note that a number of combinations of soil strength distribution and pore water pressure regime can result in the same 1.0 back calculated factor of safety. As such, any one solution might not be the "true" condition at failure. However, research has indicated that if the ranges of values and conditions used are broadly reasonable, then the model can be used to develop remedial schemes with the post-remediation factors of safety also being broadly reasonable. For this reason, back calculation is extremely valuable in developing landslide remedial schemes. This is particularly true to the extent that back calculated conditions can be verified through field and laboratory data.

One item that is crucial to have as accurate as possible in back calculation is the location of the shear plane. Ideally, this is evaluated through direct monitoring of inclinometers. For landslides that are not currently moving, detailed explorations may identify past slip zones through the identification of soft soils, slickensides, or material layering.

The strength and porewater pressure values developed in the back calculation analysis may require modification for analysis of the post repair or modification case. For example, the landslide may disrupt drainage patterns and seepage in ways that should be accounted for in the subsequent model. Frequently, in order to make the initiation of a landslide work in back calculation, a cohesion intercept must be applied. Care should be taken to only apply cohesion to materials where the existence can be reasonably surmised including apparent cohesion. However, in evaluating the stability of a repaired or modified slope, the continued use of cohesion should be associated with careful review, as discussed in Section 7.3.3.5.

7.5.2 Tension Cracks

When modeling existing or impending landslides, open cracks may be observed in the field. These cracks represent a significant modification to the section and are important to include in the model. Further, with cohesive soils, the slide forces may document the existence of tensile stresses in the upper portions of the slope. Since soils generally can't develop tensile forces, it is necessary to insert a crack in the analysis, negating the tension.

7.5.3 Embankments on Weak Foundations

Embankments on weak foundation soils frequently require detailed analyses. Frequently, the project design must be modified to address situations where the undrained strengths of the foundation soils are insufficient to support the proposed embankment slopes. Techniques to address this situation include placing the embankment slowly to allow excess porewater pressures to dissipate and the installation of vertical drains to shorten drainage times. Alternately, the embankment slopes can be flattened or reinforced.

One concern to address is the strain compatibility between the compacted embankment and the underlying structure. For weak foundation conditions, the compacted embankment will typically be much stiffer than the underlying soils. This issue is particularly acute with respect to cohesive soils used in constructing the embankment. In completing the short-term, undrained analysis for such a situation, the embankment should be modeled with a tension crack.

7.5.4 Analysis of Colluvium

Colluvium and ancient landslides are present throughout Oregon and can be identified by subsurface explorations or interpretation of surface topography. Although the landslide may have occurred more than 10,000 years ago, it is feasible that soils within the mass of the landslide will continue to behave as though the slide was more recent. In particular, with respect to shear strengths approaching residual shear levels. Even though no evidence of past shear planes or slickensides may be evident through the exploration program, colluvium is generally assumed to contain such features and they should be assumed to be present in any subsequent stability assessments.

7.6 Results of Computerized Analysis

As with all engineering analysis, thorough review and QA is necessary to assure that the project details have been accurately captured and to verify that the software executed without error.

1. It is important to have the slope stability analysis completed without errors. Analyses that document the presence of errors should be modified and rerun until those errors are eliminated. It is not reasonable to assume that the source of error is known and the consequences of those errors is understood.
2. Assuming that the software executed without error, the next step would be to verify that the factors of safety are reasonable. Factors of safety of less than 1.0, including negative values, are indicative of the need to make adjustments in soil strengths, project geometry, etc.
3. Next, the designer should review the most critical failure surfaces. All failure surfaces at or near the minimum, or critical, surface should reflect conditions that are physically possible. Sharp angles in failure planes, particularly near the toe, may unreasonably influence the factor of safety. This is especially problematic for defined block analyses.

For presentation, the analysis cross sections should be presented to scale, with or without an expanded scale in the vertical dimension, as needed for clarity. Typically, the family of failure surfaced developed by the software would be plotted with a heavier line representing the critical failure surface. The presentation of the model should clearly identify the soil regions and the soil parameters applied to each region.

7.7 References

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APENDIX

LIST OF TABLES

LIST OF EQUATIONS