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Chapter 15
ROADWAY SLOPES AND EMBANKMENTS

15.1 GENERAL

15.1.1 Overview

Many of MDT's roadway projects require the design and construction of roadway slopes and embankments. In most cases, roadway slopes and embankments will be used to meet grade and alignment requirements in areas of changing topography. However, roadway slopes and embankments can also be used to form temporary access routes or work platforms during construction. This Chapter summarizes procedures that the project geotechnical specialist should follow when conducting geotechnical studies for roadway slopes and embankments. These procedures cover natural soil and rock slopes, as well as engineered fills.

Roadway slopes and embankments are considered separately in this Chapter, primarily because of the different geologic conditions that will occur for each:

1. Roadway Slopes. Roadway slopes are defined by the existing geology at a site. They may involve excavation of a cut slope or construction adjacent to a natural slope. Relative to the embankment fill, geologic conditions for roadway slopes will normally be more variable.

2. Roadway Embankments. Embankments involve fills constructed on natural soil. The embankment fill is either imported from off site or relocated from another portion of the project and placed on the existing ground. Contract documents specify fill placement procedures, material requirements and compaction requirements.

The different geologic conditions for roadway slopes and embankments result in different geotechnical requirements relative to field explorations and engineering design. For example:

- The primary geotechnical concern for roadway slopes is the stability of the slope. The stability assessment requires characterization of geologic layers and groundwater conditions of the existing material. Engineering design activities focus on the evaluation of short- and long-term stability for different groundwater, material strength and seismic load assumptions. If slope stability is inadequate, improvement procedures typically involve flattening slopes, drainage improvements and the use of a retaining wall or some type of ground improvement. Retaining structures range from standard cantilever walls to soil nail walls, see Chapter 17.

- The primary geotechnical design issues for embankments include bearing capacity, slope stability and long-term settlement. These design issues are often controlled by the engineering characteristics of the geologic material below the fill rather than the properties of the fill. Consequently, geotechnical explorations for the embankment focus on characterization of the existing foundation material, and engineering design evaluates how these existing materials respond to the load from the new fill. If an unacceptable response is predicted, methods of improving the soil below the fill to achieve better performance may be required before the fill is constructed. The focus of the embankment design study, however, can be on the embankment fill performance where embankment side slopes are steep in relation to the shear strength of the anticipated fill soil or where lightweight fill (e.g., pumice, geofoam) rather than mineral fill will be used for the embankment.

Thorough geotechnical analyses and design are important for both the roadway slope and embankment. Inadequate consideration of geotechnical design requirements can result in construction and operational problems. For roadway slopes, a primary design consideration is the potential failure of slopes during construction or during operations, which may represent life-safety risks. If slope movements are slow, the primarily problem could be long-term maintenance requirements to remove earth or rock as it encroaches on the roadway. In the case of embankments, slope stability and bearing capacity failures can occur during construction, causing construction slowdowns and Contractor claims. Settlements beneath the embankment following construction can result in poor ride quality of the roadway, leading to long-term maintenance requirements and premature pavement failures.

15.1.2 Responsibilities

Responsibility for the design of roadway slopes and embankments resides with the Geotechnical Section and Road Design Section. Other units are involved as necessary.

15.1.2.1 Geotechnical Section

For roadway slopes and embankments, the Geotechnical Section:

- plans and then performs the geotechnical explorations, including field investigations and laboratory testing, see Chapters 8 and 9;
- conducts geotechnical design studies to evaluate slope stability for natural and cut slopes and bearing capacity, side slope stability and settlement for embankments;
- provides construction recommendations, including subgrade preparation requirements, maximum slope angles for construction and long-term operations, and the need for ground improvement where settlements are excessive or where earthquake loading could result in embankment or slope damage;
- identifies, installs and monitors instrumentation and develops special provisions for construction as needed; and
- supports the District Construction personnel and Road Design Section if construction issues develop.

Procedures for planning and documenting the field work and design results are identified in Chapters 4 and 5.

15.1.2.2 Road Design Section

For roadway slopes and embankment, the Road Design Section:

- sets a roadway alignment and grade. Fill/cut slope ratios are generated using preset "standard" slope ratios in the Geopak software used by the Road Design Section. The standard ratios are based on the fill/cut height. Right-of-way limits are defined by these preliminary slope ratios. The Geotechnical Section reviews the preliminary slope ratios and provides recommendations for adjustments, where needed, as part of the project design process. The recommendations for adjustment are based upon the geotechnical investigations and analyses, economics, right-of-way considerations, etc. Ultimately, the Geotechnical Section and the Road Design Section work together to determine the final slope ratios considering these factors.

- prepares plans and specifications for construction with input and review by the Geotechnical Section.

- prepares earthwork estimates including estimated cut-and-fill volumes during preparation of project plans and specifications. Where practical, the earthwork design balances the estimated material quantities for proposed embankments with the estimated quantities obtained from proposed cut areas within the project limits. This "earthwork balance" attempts to minimize the use of an imported material or prevent hauling material off the project site. The Geotechnical Section is often requested to review and/or provide internal shrinkage and bulking (swell) values for project quantity estimates.

Experience at MDT has been that swell (bulking) can range from 0% to 15%, and shrink (shrinkage) can range from 10% to 30% with higher values sometimes occurring. The size of the project will affect the importance of the shrink and swell estimates. As the amount of excavated or imported materials increase, the importance of these estimates increase. Determining accurate values is difficult and is a function of numerous factors, some of which are related to the contractor's operation and, thus, are not known at the time of design. Experienced personnel within the Geotechnical Section can provide guidance on these values and the Construction Engineering Services and/or District can sometimes provide information about shrink/swell values within a specific area based upon previous localized construction experience.

15.1.2.3 Other MDT Units

Depending on project requirements, other MDT Units may also participate in the design and construction for roadway slopes and embankments. These Units include:

- Bridge Bureau, if retaining walls are needed for slope stabilization and when approach fills are being designed for bridges;

- Hydraulics Section, if the embankment or cut slope could encroach on or be inundated by flowing water;

- Environmental Services Bureau, if the embankment is being constructed near wetlands or other environmentally sensitive areas, or if the cut slope could result in permanent environmental damage; and
- Hazardous Waste Section, if the embankment is being constructed where contaminated soil or groundwater conditions are known to exist.

15.1.3 References

For further guidance on the design of roadway slopes and embankments, the project geotechnical specialist should consider the following references:

1. Roadway Slope Design.
    - *Soils and Foundations Reference Manual* - Volume I, FHWA-NHI-06-088 and Volume II, FHWA-NHI-06-089;
    - *Rockfall Hazard Classification and Mitigation System Report*, Montana Department of Transportation;
    - *Rock Slopes Reference Manual*, FHWA-HI-99-007;
    - *Rock Slope Engineering*, Hoek and Bray, The Institution of Mining and Metallurgy, 1981;
    - *Highway Maintenance and Slide Restoration Workshop*, FHWA TR-80-040;
    - *Soil Strength and Slope Stability*, J.M. Duncan and S.G. Wright, 2005;
    - *Rock Foundations*, EM 1110-1-2908, Corps of Engineers, 1994; and
    - *Landslides in Practice: Investigation, Analysis, and Remedial/Preventative Options Soils*, D. H. Cornforth, 2005.
2. Embankment Design.
    - *Soils and Foundations* Soils and Foundations Reference Manual - Volume I, FHWA-NHI-06-088 and Volume II, FHWA-NHI-06-089;
    - *Design and Construction of Stone Columns*, FWHA-RD-83/02C;
    - NCHRP Report 529 *Geofoam Applications in the Design and Construction of Highway Embankments*, Transportation Research Board;
    - *Soil Slope and Embankment Design*, FHWA NHI-01-026; and
    - *NAVFAC Soil Mechanics Design Manual*, 7.1, Department of the Navy, Naval Facilities Engineering Command.

15.2 ROADWAY SLOPES

15.2.1 General

This Section addresses existing slopes adjacent to roadways or slopes resulting from roadway excavations. Existing slopes are referred to as natural slopes, while the excavated slopes are referred to as cut slopes. The stability of either category of slope is determined by the existing geology of the area. The geology can be either soil or rock, and groundwater can exist within the slope or below the slope. These slopes are considered separate from slopes associated with embankment construction, referred to as engineered slopes. Section 15.3 discusses engineered slopes.

There has been a history of slope movement in natural and cut slopes throughout Montana. This movement has ranged from relatively small failures in over-steepened slopes along roadways to very large landslides and rockfalls. Slope failures in these natural deposits have been attributed to a number of causes, including:

- water infiltration;
- oversteepening of existing slopes from natural processes (e.g., erosion, new developments) or from new construction;
- external loads;
- utility trenching near the top of slope;
- re-activation of old landslide surfaces, often associated with bentonitic layers; and
- seismicity.

The Geotechnical Section handles the stability of natural slopes and cuts on a routine basis; however, two areas are worthy of special note because of their size and difficulty in mitigating the potential for instability:

1. Cretaceous Shale. Very large landslides have occurred in Montana within cretaceous shale (e.g., Bear Paw shale, Two Medicine shale). These slides have occurred on slopes that are 4:1 or flatter in steepness. Failures are usually related to bedding planes comprised of very low-strength bentonitic materials. Other shales or soils can also be problematic where bentonite is known to exist.

2. Mountainous Areas. The angles of slopes in these areas are often at or greater than the value required for stability. Under these conditions, landslides and rockfalls occur when combinations of weather conditions and snow loads exceed the marginal stability of these slopes. Mitigating these conditions is very difficult for various reasons, including access and cost. For further discussion on the rockfall potential, see MDT's *Rockfall Hazard Classification and Mitigation System Report*. This *Report* identifies locations were rockfall hazards are considered to be high.

In general, cut slope heights and inclinations provided in the *MDT Road Highway Manual* are based on right-of-way space limitations and need to be individually analyzed for stability.

Locations requiring detailed review are those where slopes are high and steep, where soils are fine-grained or are known to be weak or where slope failures have occurred in the past.

15.2.2 Evaluation of Slope Stability for Soil Sites

The project geotechnical specialist determines the stability of natural and cut slopes by analyzing the geologic conditions at a site, including the location of groundwater. Consider the factors in the following Sections when planning and carrying out an assessment of the stability of natural and cut slopes. Section 15.4 provides a summary of factors of safety (FS) requirements that must be satisfied when evaluating the stability of roadway slopes at soil site.

15.2.2.1 Overview of Approach for Stability Assessment

The stability analysis of a natural slope or cut slope involves the following general approach:

1. Soil Strength Parameters. Review the soil strength parameters for each material within the slope. This will include determining the effects of the following:
   - long-term versus short-term loading (drained versus undrained) strength;
   - selection of strength parameters from test results, including total stress and effective stress; and
   - any necessary adjustments for mode of failure (e.g., triaxial compression, simple shear, triaxial extension).
2. Groundwater. Determine the groundwater level (piezometric level) for the average and worse-case conditions. Consider the potential for variations in groundwater levels, artesian effects, perched water, potential for rapid drawdown and the effects of irrigation.
3. Cross-Section. Analyze cross-sections along the slope to determine vertical and lateral limits and other details (e.g., soil density) of each soil layer.
4. External Loads. Analyze the effect of soil loads and seismic loads on slope stability.
5. Method of Analysis. Consider the pros and cons for each method of analysis, including the iterative nature of the program and the importance of parametric studies.
6. Construction. Consider the construction schedule and any safety issues that may arise during construction.

The following sections discuss additional details for each of these steps.

15.2.2.2 Method of Analysis

Although simple hand calculations and stability charts are available, computer software (e.g., GSTABL, XSTABL, SLOPE/W) for evaluating slope stability is the preferred method of analysis.

GSTABL is currently used in the Geotechnical Section. The use of computer software is the preferred approach because of the ability to:

- model soil layering and groundwater conditions within the slope. Thin, low strength soil layers can be included in the analysis. Both total and effective strength parameters can be used to evaluate short- and long-term stability as discussed in Section 15.2.2.5;
- perform parametric studies to evaluate the potential effects of uncertainties in soil properties, changes in properties that occur with load or time or fluctuations in groundwater location; and
- evaluate the effects of external loads, including traffic loads.

The Geotechnical Section also uses computer software to back-analyze previous slope failures. Results from the back-analyses can be used to understand the possible cause of failure, to help quantify soil or groundwater conditions that contributed to failure and to establish cost-effective stabilization procedures. There are, however, potential limitations associated with the back-analysis process, which must be understood when back analyses are performed. For further discussion of these limitations, review the discussion in *Soil Strength and Slope Stability*.

While the geotechnical specialist will normally use computer software to evaluate slope stability, the use of charts and hand calculations should not be completely abandoned. Slope stability charts and hand calculations provide an important independent evaluation to check the results of computer analyses.

15.2.2.3 Information for Stability Analysis

Stability analysis requires accurate information on geology and groundwater conditions at a site. Consider these requirements when determining the types, locations and depths of field explorations, as discussed in Chapter 8. The field exploration program must:

- Identify low-strength soil layers that could serve as sliding surfaces. CPT investigations can be particularly valuable for this task at some sites;
- Collect appropriate samples. Determine whether undisturbed samples are required and, if so, the type of sampler and the handling of these samples; and
- Establish the groundwater elevation. This may require installing and reading piezometers over time to determine the fluctuation in groundwater table.

During the planning phase, it may be valuable to perform preliminary stability analyses based on expected soil conditions. Information from preliminary analyses can often help determine the appropriate location and depths of explorations. Seismic refraction lines are sometimes valuable in identifying the depth to rock. If soft clay layers are possible, it may be desirable to conduct in-situ vane shear tests to obtain strength information for soft clay deposits. Chapter 8 provides a detailed discussion of other alternatives to consider during field explorations.

15.2.2.4 Loading Conditions

The loading conditions that are normally evaluated during the stability analyses include:

1. Short-Term Loading. This condition is important for cut slopes or slopes that have new loads (e.g., roadway embankments). Consider the following:

   a. Cohesive Soils. Use the undrained strength to determine the stability. In this loading state, the porewater pressures do not have time to dissipate. Consequently, strength is determined based on the state of stress that existed before the new load.

   b. Cohesionless Soils. If the soil is a relatively clean cohesionless soil, the porewater pressures could dissipate as quickly as the soil is loaded. In this case, determine the strength by the drained properties of the soil using the effective stresses in the analysis.

2. Long-Term Loading. This condition governs the behavior of many natural slopes; however, it is also appropriate for determining the long-term strength of a cut slope. In this case, porewater pressures are fully dissipated. For cohesive or cohesionless soils, effective strength parameters determine the strength of the soil for long-term conditions. Any new load is assumed to develop slowly enough that excess pore pressures do not develop. This is referred to as the drained case. For cohesionless soil, the drained case can occur during short-term loading, if porewater pressure can dissipate quickly enough.

3. Seismic Loading. In areas where the potential for seismic conditions exists, the stability of a slope under seismic loads can be an important consideration. The seismic case is similar to a short-term loading, but has an additional horizontal force representing the inertial loads from the earthquake. The additional horizontal force is defined in terms of the peak ground acceleration, as discussed in Section 15.2.4.

It is not always easy to determine whether soil will behave in a drained or undrained manner. Both the rate of loading and the permeability of the soil will determine whether the soil responds in a drained or undrained state. Because it is often very difficult to predict the rate of loading during design, the best approach is to check both the drained and undrained cases, and to use the more critical strength as the basis for design.

15.2.2.5 Selection of Strength Parameters

Stability analysis requires establishing strength and groundwater conditions for each soil layer identified during the field exploration program. Before assigning strength parameters, develop an accurate cross-section of the site. Use this cross-section to develop potential failure mechanisms and loading conditions that may control stability. The type of soil and the loading condition will determine the method used to develop strength parameters for analyses. Chapters 8 and 9 discuss the types of field and laboratory tests.

The following summarizes the recommended method for quantifying strength parameters for slope stability analyses:

1. Cohesionless Soils. For layers comprised primarily of cohesionless soils, it will often be sufficient to use empirical correlations based on SPT blowcounts or CPT tip resistances in conjunction with index and classification tests. For some projects, either direct shear tests or consolidated drained (CD) triaxial shear tests may be useful. However, it is very difficult to obtain intact samples of cohesionless soil and, therefore, these tests would normally be conducted on reconstituted samples. Any in-situ characteristics of the cohesionless soil will usually be lost during the reconstitution process, making the test results of limited value. In this case, place greater reliance on empirical correlations and the SPT blowcount or CPT tip resistance.

2. Cohesive Soils. Where cohesive soils are encountered, in-situ and laboratory strength tests are conducted to define the strength of the soil. In-situ field tests include the vane shear test and the CPT. Both methods provide an estimate of the undrained strength of the soil under the existing state of stress. If high quality soil samples are available, laboratory strength tests can be conducted to define the soil strength. Depending on the drainage and loading conditions implemented in the tests, either total stress or effective stress parameters will be obtained. For stability analyses, consolidated undrained (CU) tests with porewater pressure measurements are generally the preferred method of laboratory strength determination, because the strength parameters obtained from these tests can be used to represent different stress states and drainage conditions. Results from the CU tests include cohesion and friction angle for both total and effective stress conditions.

In most cases, the peak strength will be of interest for stability assessments. However, some stiff fissured clays and shales (e.g., cretaceous shale), exhibit significant strength reduction with shearing. This reduction typically occurs in sensitive clays with high liquidity indices. After reaching the peak strength during shear, a significant reduction in strength occurs, sometimes by a factor of 10 or more. For these types of soils, assess the stability under both the peak and the fully softened (residual) strength cases to determine the potential consequences of the soil deforming enough along the shear plane to mobilize the residual strength. Special laboratory testing may be necessary to estimate the residual strength of the soil. Remolded field vane shear tests can also be used to estimate the residual strength.

Studies have shown that the primary mode of shear during slope failures is not always consistent with the mode of failure that occurs in laboratory triaxial tests and is dependent upon the location along the potential failure plane. The strength developed along a failure surface is more closely represented by the strength from a direct simple shear (DSS) test rather than a conventional triaxial test. Furthermore, the DSS strength is lower than the strength from the CU test, unless special (CK_oU) triaxial tests are conducted. Because most laboratories are not able to conduct either DSS or CK_oU tests, conversion factors are used to adjust the more conventional CU strengths to DSS strengths. Chapter 10 provides the conversion factors for comparing triaxial compression tests to DSS.

15.2.2.6 Groundwater Conditions

Groundwater conditions often are the cause of slope failures in natural or cut slopes. A number of factors determine or influence groundwater conditions in slopes including infiltration (precipitation or irrigation), artesian pressures, perched water layers or simply the phreatic groundwater surface. The groundwater level likely will vary with time of year depending on climatic changes, heavy rainfalls, irrigation and changes in river elevation. These changes in groundwater location influence the effective stresses, which will affect the stability of a slope. Consequently, a key step in the stability analysis involves identification of both the current location of the groundwater and potential fluctuations of groundwater. Often this will require installing piezometers at the site and recording groundwater elevation changes with time. Chapter 8 provides a discussion on requirements for groundwater measurement.

Consider the following items when assigning groundwater conditions at a site during slope stability analyses:

1. For static analyses, use the maximum groundwater elevation for design. If, however, artesian or perched groundwater conditions occur, define the groundwater elevation to give the correct effective stress conditions within the limits of the analyses.

2. During seismic loading, use the long-term mean groundwater elevation. The logic for using the mean is that it is unlikely an earthquake will occur at the same time as the maximum water condition. However, if the maximum water condition may persist for several months, then the use of a higher level would be appropriate.

3. If rapid decreases in water elevation could occur (e.g., adjacent to a river), it may be necessary to examine a rapid drawdown case where the groundwater has a significant gradient away from the river. In this case, methods used for evaluating stability of slopes for earth dams would be appropriate.

15.2.2.7 Methods of Analysis

There are two general approaches when conducting stability analyses: (1) limit equilibrium analyses where only the factor of safety is determined, and (2) numerical modeling where stresses and displacement within the soil mass are obtained. The limit equilibrium approach is used in most conventional slope stability programs. Numerical modeling typically involves the use of finite element or finite difference software.

Features of these two methods are summarized below:

1. <u>Limit Equilibrium Approach</u>: Most stability evaluations will involve the use of limit equilibrium methods. Various computer software packages (e.g., GSTABL, XSTABL, SLIDE, SLOPE/W) are available that allow the limit equilibrium analysis to be performed relatively quickly and with limited special training. Chapter 13 identifies some of the common software for this approach. The software packages listed provide a choice of methods for modeling and analyzing the slope stability problem. The methods differ in the assumptions used to address the interslice forces; whether, the method of analysis satisfies only force equilibrium or force and moment equilibrium. Some methods consider only force equilibrium, while others address both force and moment equilibrium. Common methods of analysis include the Ordinary Method of Slices, Bishop and Janbu's Simplified Methods, Bishop's Complete Procedure, Morgenstern and Price's Method and the Spencer Method. For a detailed discussion of these methods, see the publication *Soil Strength and Slope Stability*. The Spencer Method is often considered

the simplest method that satisfies overall moment, individual slice, horizontal force and vertical force equilibrium.

2. Numerical Modeling: Numerical modeling offers the advantage that obtained results are in the form of predicted displacements, stresses and strains within the model analyzed. This approach is useful when assessing potential consequences of a low factor of safety on a roadway or bridge. Typical software packages used for this purpose include FLAC and PLAXIS. As noted in Chapter 13, these methods require considerable skill in setting up, conducting and interpreting results from the numerical model. In recent years, improvements in the software have made this approach more efficient and less user-dependent.

15.2.2.8 Other Considerations during Slope Stability Analyses

Consider the following factors when conducting a limit equilibrium stability analysis:

1. Details of Soil Layering. One of the most critical steps in setting up the stability analysis is to develop an accurate cross-section for the site. This cross-section should identify major changes in geology and the location of groundwater. Of particular importance is the identification of soft layers within an otherwise competent soil profile. Soft layers or bedding planes with weak interfaces can serve as critical surfaces for sliding. Special laboratory tests may be required to identify the strength of these layers. Similarly, zones of excess porewater pressure from artesian conditions must be appropriately defined and included in the stability model.

2. Non-Circular versus Circular Failure Surfaces. When developing the slope stability model, most software packages allow the user to specify either a circular or a non-circular representation of the failure surface being considered in the stability analysis. Many natural and cut slopes possess layers that will serve as preferential sliding surfaces. For these locations, the method of analysis must allow sliding wedge failure mechanisms. Sites that consist of thick deposits of uniform cohesive soils are usually represented by circular failure surface. Where there is a combination of layers with different thicknesses and strengths, the stability analysis should be evaluated for both the circular and non-circular failure surfaces.

3. Vertical and Lateral Limits of Analysis. Most limit equilibrium stability software provides a two-dimensional representation of the slope. This allows the vertical extent to be specified but not the horizontal limits. In other words, the computer model represents a vertical plane through the slope (or slice) with a unit width. The user can specify the vertical limits of the search for the lowest factor of safety. These limits can be modified in most computer programs to identify deep or shallow failure surface. The inability to represent the lateral extent of the model is not usually considered a significant limitation, particularly if the width of the potential sliding surface is more than 20% of the length. In this case, the two dimensional model will generally be conservative by 10% to 15%. It is possible to define a number of vertical planes through the failure surface and then take a weighted average of the resulting factor of safety values to define a composite factory of safety. While this approach has some validity, it is generally not recommended because of inherent uncertainties in the development of resisting forces.

4. Tension Cracks. Cohesive soil sites will often develop a tension crack at the ground surface, which should be included in the stability analysis. The depth of the tension crack can be estimated from the following equation:

$$D_{crack} = 2c_d / [\gamma z \tan (45 - \phi_d/2)] \quad \text{(Equation 15.2-1)}$$

where:

D_{crack} = depth of the crack

c_d = soil cohesion

γ = soil unit weight

ϕ_d = friction angle. The subscript "d" is for the mobilized strength parameters.

The failure surface for the stability analysis should not extend beyond the crack.

5. External Loads. Include external loads or forces above the failure surface in the stability analyses if they are a permanent load (e.g., building). The frequency of occurrence determines whether temporary loads (e.g., traffic) are included in the analysis. For example, if a roadway is heavily traveled, the traffic load should be included as an exterior force. However, if the road is lightly traveled, a reduced value of the traffic load would normally be applied. AASHTO *LRFD Bridge Design Specifications* suggests using a load factor of 0.5 with live loads, which would be equivalent to using half of the live load. Typically, a live load of 250 psf (12 kPa) is used in the stability analysis to represent the traffic load.

6. Uncertainty Considerations. One of the important tasks during the stability analysis is to develop an appreciation for the effects of soil property and groundwater changes on the stability estimate. In most cases, there will be uncertainties in strength parameters and groundwater assumptions, even when the best possible field and laboratory testing methods are performed. These uncertainties result from various sources, including state of stress, sample disturbance and mode of failure. The potential effects of these uncertainties can be investigated by conducting parametric evaluations to show the effects of variations in strength parameters, groundwater location and external loading conditions. Some computer software allows statistical methods to be used to define the probability of failure for material property assumptions. For further discussion of reliability and uncertainty methods, including the use of the 3σ rule and the reliability index (β) for evaluating uncertainty, see the publication *Soil Strength and Slope Stability*.

7. Construction Schedule. Where possible, evaluate the construction schedule as part of the stability assessment, though this requirement is not as critical as for an embankment. The construction schedule provides information to help determine whether short- or long-term loading conditions occur for certain projects. Because the construction schedule is often not available, both short and long-term loading conditions may need to be evaluated for site conditions.

15.2.2.9 Documentation

Chapter 5 identifies general procedures for preparing geotechnical reports. When summarizing the results of slope stability analyses, include the following specific information regarding the slope stability analysis:

- plan drawings showing the existing topography and the locations of explorations, roadways and other external loads (include in project file);
- cross-section drawings used to set up the stability model, including primary soil or rock layering, location of groundwater and test hole logs summarizing soil or rock conditions (include in project file);
- strengths and unit weights assigned to each soil or rock layer. The narrative part of the report should explain the bases for these strengths, particularly focusing on short- and long-term loading conditions (include in project file);
- the method of analysis used to determine the slope stability. If computer methods are used, identify the computer program and the method of analysis (e.g., Spencer's procedure, Ordinary Method of Slices) and include copies of the input/output files in the appendices (include in project file);
- conclusions from the stability analyses, including a discussion of uncertainties in the analyses and recommendations on the method of mitigation, if appropriate (include in Geotechnical Report); and
- factor of safety used for design and explanation of why this factor of safety was selected (include in Geotechnical Report).

15.2.3 Evaluation of Slope Stability for Rock Sites

The stability of rock slopes is an important consideration for roadways that are located on or next to rock slopes or excavated into rock slopes. The objectives of the rock stability analysis are to determine global stability as well as the stability of individual blocks of rock. This section summarizes the modes of failure, methods of assessing stability, determination of rock strengths and acceptable factors of safety. Section 15.4 provides a summary of factor of safety (FS) requirements that must be satisfied when evaluating the stability of rock slopes.

15.2.3.1 General

To evaluate the stability of rock slopes, it is important to determine:

- the top location of rock, if covered with overburden;
- any variation or discontinuities (fracture/joint) patterns and conditions; and
- strength and groundwater conditions.

Characterization of rock location and discontinuities can be accomplished through drilling of boreholes (Section 8.3.4), seismic refraction (Section 8.3.9.6.2) and/or surface mapping of exposed rock faces. Rock strength evaluations can involve laboratory or field testing; however, often strengths are based on a combination of published information and rock mass conditions. Groundwater conditions are established by either monitoring piezometers installed in the rock or observing seeps and flows during field reconnaissance.

The *MDT Rockfall Hazard Classification and Mitigation Report* provides guidance on known rockfall locations within Montana and the proposed mitigation to stabilize the slope.

15.2.3.2 Modes of Failure

The failure of rock slopes is controlled primarily by the orientation and spacing of discontinuities (e.g., joints, bedding planes) within the rock mass, and the orientation and the angle of inclination of the slope. These parameters will determine the mode of failure. For analysis purposes, the modes of failure are divided into the following general groups:

1. <u>Sliding Failure</u>. These failures include planar, wedge and circular failures, and involve single or multiple blocks sliding on a bedding plane or a joint set striking approximately parallel to the slope strike. Wedge failures can occur in rock masses with two or more sets of discontinuities whose lines of intersection are approximately perpendicular to the strike of the slope and dip toward the plane of the slope. Global sliding failures can occur along circular slip paths, particularly in highly weathered and decomposed rock masses, highly fractured rock masses or in weak rock (e.g., shales, poorly cemented limestone).

2. <u>Toppling Failure</u>. This failure involves overturning or rotation of columns or blocks of rock about a fixed base. Closely spaced, steeply dipping discontinuity sets that dip away from the slope are necessary for toppling. Toppling failure is usually initiated by layer separation with movement in the direction of the free face or excavation.

3. <u>Sloughing Failure</u>. This failure is generally characterized by occasional rock falls or localized slumping of rocks degraded by weathering. Rock falls occur when blocks become loose and isolated by weathering and erosion. Both rock falls and localized slumping can occur.

Geometric boundaries imposed by the orientation, spacing and continuity of joints and free surfaces define the potential modes of failure. Several factors can initiate failure including erosion, groundwater, temperature and state of stress. The Corps of Engineers publication *Rock Foundations* provides a discussion of each of these factors.

15.2.3.3 Methods of Assessing Stability

The method selected for analyzing the slope depends on the potential failure mode — sliding, toppling or localized sloughing. Key considerations include location of joints, bedding planes and rock strength properties (i.e., intact rock and interface properties). The failures are identified as follows:

1. <u>Sliding Stability</u>. Sliding stability of rock slopes is usually evaluated using limit equilibrium methods. Various methods have been developed to analyze sliding stability

for planar slip surfaces, three-dimensional wedge shaped slip surfaces and circular slip surfaces. *Rock Slope Engineering* (Hoek and Bray, 1981) provides a good summary of assumptions and limitations associated with each of these methods of analysis, including importance of tension cracks on the slope for planar failures. Equations for estimating the factor of safety for planar and wedge surfaces are also provided.

2. <u>Toppling Stability</u>. Toppling occurs if two conditions apply: (1) the projected resultant force (body weight plus any additional applied force) acting on the block falls outside of the base of the block and (2) the inclination of the surface on which the block rests is less than the friction angle between the block and the surface. A combination of sliding and toppling occur when the slope angle is greater than the interface friction, and the width to height ratio (b/h) of the block is less than tan ϕ, where ϕ is the interface friction. Various types of toppling failure and limit equilibrium methods for evaluating toppling are described in *Rock Slope Engineering*.

3. <u>Sloughing Stability</u>. This type of stability includes landslides, talus slope failures and debris flows. They are evaluated by conducting hand or computer analyses, similar to methods used to evaluate the stability of soil slopes. Often these slopes are at their angle of repose. Small changes in groundwater conditions or loading can trigger a slope instability. Characterization of the strength properties of these materials is particularly difficult because of the wide range of material types.

Chapter 13 identifies software that can be used to perform the analyses. Specific software can handle water pressures, surcharges and seismic loading for planar and wedge analyses. Software is also available for evaluating rock toppling. These programs include the statistics of rockfalls, energy, velocity and bounce height. Some of the same computer software used for the evaluation of slope stability in soils are also used for these analyses.

15.2.3.4 Strength and Groundwater Determination

Determination of the appropriate strength to use in the rock slope stability analysis is more complicated for rock than for soil, because of the effects of the rock mass properties. For example, a rock block can have an unconfined compressive strength of 20,000 psi (14 MPa) or more and still be marginally stable if an adverse discontinuity daylights on the slope face. In this case, the interface resistance would control stability and not the intact rock strength. The project geotechnical specialist should use the references provided in Section 15.1.3 for additional information.

Groundwater that occurs along a discontinuity or within a slope mass will reduce the stability of the rock slope, either by introducing an uplift pressure at the discontinuity or by lowering the effective stresses in broken rock. Changes in groundwater can be either permanent or seasonal and, therefore, both the location and the fluctuation of the phreatic surface should be established as part of the field exploration program.

15.2.3.5 Documentation

The documentation required for a stability analysis of a rock slope is generally similar to the requirements described for a soil slope; see Section 15.2.2.9. In addition to the soil slope requirements, provide the following additional information:

- detailed discussions of the procedures used to locate potential discontinuities that could serve as failure planes (in project file);
- results from field reconnaissance and borehole explorations used to develop three-dimensional drawings (stereonet representations) showing postulated location of discontinuities (in project file);
- expected location of groundwater (in Geotechnical Report);
- detailed discussions of the methods used to establish the strength parameters that characterize the discontinuity and the rock mass (in Geotechnical Report);
- results of stability analyses, including the methods used and assumptions made (in Geotechnical Report); and
- uncertainties in the results (in Geotechnical Report).

15.2.4 Seismic Analyses

It is important to identify the potential for slope failures during seismic loading. Use the maps provided by AASHTO for determining the level of ground motion; see Chapter 19. The following identifies the two methods that normally are used by the Geotechnical Section for making the seismic stability assessment. If the subsurface soils are susceptible to liquefaction, the potential for a flow failure or lateral spread must be considered.

15.2.4.1 Pseudo-Static Analyses

For most seismic slope stability studies, use the pseudo-static method. This approach involves using the limit equilibrium computer programs as discussed in Section 15.2.2.7 for performing static stability analyses. The only difference is that a horizontal force is introduced to represent the inertial loading from the earthquake. The inertial force is based on the product of a horizontal seismic coefficient and the mass of soil. Most computer programs internally compute the resulting horizontal force for a user-supplied seismic coefficient.

15.2.4.1.1 Seismic Coefficient

The seismic coefficient used in the pseudo-static stability analyses is usually 50% of the peak ground acceleration (PGA) from the AASHTO map after the PGA has been adjusted for local site effects. The 50% reduction factor is based on results from analytical studies that show for this reduction, deformations of the slope will be limited to a few inches to a foot (50 mm to 300 mm).

While the 50% reduction is commonly used, publications can be found showing the reduction factor can range from 0.33 to 1.0. The lower reduction factor implies more deformation is acceptable. If a factor of 1.0 is used, and the resulting factor of safety is 1.0 or greater, then no deformation is predicted.

Many slope stability programs also allow a vertical seismic coefficient to be included in the analyses, along with the horizontal seismic coefficient. However, standard practice is to not include the vertical coefficient during the pseudo-static analysis. The rationale for this is that the vertical component occurs in both directions (i.e., up and down) and the net effect is that the vertical acceleration does not contribute to instability.

15.2.4.1.2 Selection of Strength Parameters

For all but clean, well-draining cohesionless soils, use total stress (undrained) strength parameters in the seismic stability analyses. The reason for using total stress parameters is that the loading is relatively rapid, with cycles of peak load occurring in a fraction of a second and the total duration of loading being less than 30 seconds for most earthquakes. For these loading rates, the permeability of cohesive soils and cohesionless soils with some percentage of fines (i.e., 15% or more) is too low to allow drainage of excess porewater pressures during seismic loading.

The following guidance is for strength determination in soils:

1. Undrained Cohesive Soils. The undrained strength of cohesive soils is rate dependent, and the first cycle of rapid loading will result in a strength that exceeds the static undrained strength — up to 40%. With each additional cycle of load, the dynamic strength decreases. Results of laboratory tests have shown that for all but the largest magnitude earthquakes, the static strength approximates the combined effects of rate of loading and number of cycles of load. However, for large earthquakes with magnitudes greater than 7.0, the undrained strength is often multiplied by a factor of 0.9 to account for the increased number of loading cycles.

2. Liquefiable Soils. The selection of appropriate properties for cohesionless soils is more critical, particularly if the cohesionless soil is loose and located below the groundwater table. In this case, liquefaction can occur; see Chapter 19. If the soil liquefies, use the residual strength of the soil in the stability analysis. Methods identified in the papers “SPT-Based Analysis of Pore Pressure Generation and Undrained Residual Strength” (Seed and Harder, 1990); “Liquefied Strength Ratio from Liquefaction Flow Failure Case Histories” (Olson and Stark, 2002); and "SPT- and CPT-Based Relationships for the Residual Shear Strength of Liquefied Soils” (Idriss and Boulanger, 2007) can be used to estimate the residual strength of liquefied soils.

3. Non-Liquefiable Cohesionless Soils. If the soil is granular and not liquefiable, the drained friction angle of the soil would usually be used in the pseudo-static stability analysis. Normal practice is to include some component of cohesion with the drained friction angle, particularly if the soil is located above the groundwater table. The amount of cohesion can range from 50 psf to 200 psf (2 kPA to 10 kPA), depending on the amount of fines within the soil.

When conducting pseudo-static stability analyses for rock slopes, the same properties as used for the static stability analyses will usually be acceptable. The exception would be if a thick cohesive in-fill material occurs along a discontinuity. In this case, use the undrained strength of the infill material to characterize the strength.

15.2.4.1.3 Factor of Safety

Results from pseudo-static analyses are presented in terms of a factor of safety. The seismic performance of the slope is usually considered acceptable if the factor of safety is greater than 1.1 to 1.2. However, if a 50% reduction factor is used in defining the seismic coefficient, some permanent deformation should be expected. As long as liquefaction does not develop, the deformations will usually be less than a few inches to a foot (50 mm to 300 mm).

In some cases, a displacement of a few inches to a foot (50 mm to 300 mm) will not be acceptable (e.g., if a critical utility is located within the slope or if the movement affects a bridge abutment or railroad line). Likewise, if the failure surface is in highly sensitive soil and the movement could result in a residual strength developing, it may not be desirable to have this amount of movement. In these cases, it may be necessary to use retaining structures or ground improvement methods to improve the strength of the soil, thereby, reducing deformations to a tolerable level.

15.2.4.2 Displacement Analyses

Deformation (displacement) analyses can be conducted in addition to a pseudo-static limit equilibrium seismic stability evaluation. There are two approaches normally used to perform the displacement analysis — the simplified Newmark sliding block charts or more rigorous numerical analyses.

The Newmark charts relate the estimated permanent movement to the acceleration ratio (i.e., yield acceleration to the peak ground acceleration). The yield acceleration is the seismic coefficient that gives a factor of safety of 1.0 in the pseudo-static stability analysis. Various researchers have developed charts correlating permanent displacement to acceleration ratio, including “Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations” (Makdisi and Seed, 1978), “Rationalizing the Seismic Coefficient Method” (Hynes and Franklin, 1984), "Earthquake-Induced Displacements of Solid Waste Landfills" (Bray and Rathje, 1998) and “Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California” (SCEC, 2002). The method given in Appendix A of Section 11 of AASHTO *LRFD Specifications* is generally believed to be too conservative, particularly at low acceleration ratios.

Numerical methods for conducting displacement analyses can be performed using computer programs (e.g., PLAXIS, FLAC); see Chapter 13. The results of the PLAXIS and FLAC analyses include estimates of stresses, strains and permanent displacements within the slope. These programs can also be used to estimate the effects of liquefaction on slope performance. They are particularly useful if structural elements (e.g., retaining walls) are being evaluated. However, as noted in Chapter 13, the program user must have considerable skills and reliable input data to obtain meaningful results.

15.2.4.3 Documentation

Written documentation for the seismic stability analyses should be the same as that used for the static stability analyses, with the following additional requirements included in the Geotechnical Report:

- Clearly define the level of ground acceleration and the site amplification/deamplification and identity the basis of this determination (e.g., AASHTO maps, site-specific studies).
- Identify and discuss the strength selected for the stability analysis. If residual strengths are being used to represent a liquefied condition, provide the basis for the residual strength determination. Also, provide the approach, assumptions and results of the liquefaction analysis.
- Present the estimated factor of safety for the conditions and cross-section analyzed. If deformations are estimated, describe the method of determining deformation.
- Summarize possible uncertainties in the analysis. If mitigation methods are recommended, provide a discussion of the alternatives and the preferred approach.

15.2.5 <u>Special Studies for Landslides/Slope Failure</u>

Landslides and slope failures present a special set of considerations that the Geotechnical Section must address on occasion. Where landslides or slope failures occur, there is often a need to identify quickly the likely cause of the failure and to develop short- or long-term methods of mitigating the failure. If the failure is blocking or could block an interstate or heavily traveled roadway, emergency response could be required. The following discussion summarizes some of the considerations during the back analysis of landslides and slope failures.

15.2.5.1 Field Explorations

An appropriate landslide investigation process cannot be defined by a rigid set of procedures. The field investigation and the determination of necessary stabilization methods vary based on site conditions. When investigating landslides, the geotechnical specialist should consider the following:

1. <u>Area</u>. The area of an investigation is controlled by the size of the project and the extent of the topographic and geologic features encompassed in the landslide activity. The area studied must be considerably larger than that comprising the suspected active or known movement. This may be accomplished by a simple field reconnaissance to in-depth topographic surveys.
2. <u>Depth</u>. The depth of the investigation should extend deep enough to identify underlying formations that are likely to remain stable. The investigation should identify materials that have not been subject to past movement, but could be involved in future movements. During field investigations, the boring depths vary depending upon the data obtained on-site. The project geotechnical specialist will typically be responsible for determining the appropriate borehole depths.

3. Data Collection. Field data collection may involve a variety of activities ranging from relatively simple reconnaissance studies to sophisticated, specialized instrumentation installations. The project geotechnical specialist determines what data should be collected. The type of data collected determines the appropriate testing equipment required. Data requirements and testing equipment are described in Section 8.3. Instrumentation options are discussed in Chapter 11.

15.2.5.2 Stability Evaluations

Once the landslide or slope failure has been described by one of the above procedures, the project geotechnical specialist will normally perform a back analysis to define soil strength parameters existing at the time of failure. The factor of safety is assumed to be 1.0 and soil properties are varied in the slope stability program until FS = 1.0. While this concept is relatively simple to apply, assumptions regarding groundwater conditions, failure geometry, and the spatial variation of soil properties within the landslide or slide failure zone will have a significant affect on the back-calculated properties. Other issues include the degree of drainage during failure and the potential for progressive failures or changing strength parameters with displacement. See discussions in *Soil Strength and Slope Stability* on the uncertainties in this approach.

Results of the back analysis can be used to develop different stabilization concepts. These concepts can range from simply removing the slide mass to the use of retaining structures or groundwater control. Section 15.2.6 provides further discussion on methods that can be used for stabilization. Benefits of a stabilization procedure can be evaluated based on the change in the factor of safety relative to the existing condition. Effects of different assumptions on strength and groundwater should be included in the assessment to account for uncertainties in the back analysis.

Further guidance on investigating landslides and slope failures can be found in the following references:

- TRB Special Report 247, *Landslides: Investigation and Mitigation*; and
- *Landslides in Practice; Investigation, Analysis and Remedial/Preventive Options in Soils*, Derek H. Cornforth.

15.2.5.3 Construction Support

The Geotechnical Section may be requested to provide oversight during repair of the landslide or slope failure, particularly if an emergency condition is identified. This support can range from documenting work done by subcontractors to conducting analyses to help decide the type of repairs that should be implemented. Often this will require close communications with the Contractor who is performing the repair. General requirements for this activity include:

- having a clear understanding of responsibilities and expectations during the work;
- providing good documentation of decisions that are made, including photographic documentation; and

- using other resources in the Geotechnical Section or specialty consultants to help confirm approaches.

15.2.6 Slope Stabilization Methods

If the results of the stability analyses indicate that the roadway slope does not meet the minimum factor of safety requirements or displacement limits, then it may be necessary to use slope stabilization methods to improve the slope performance. This Section provides a summary of stabilization methods to consider for natural or cut slopes in soil and rock.

15.2.6.1 Stabilization of Soil Slopes

Soil stabilization methods include slope regrading and groundwater control, ground improvement and structural systems. Stabilization methods for embankment fills, including embankment slopes, are discussed in Section 15.3.

15.2.6.1.1 Regrading and Groundwater Control

The simplest approach for improving the stability of a soil slope under gravity or seismic loading is to change the slope angle (flatten slope) until the stability requirements are met. This approach can often be used for cut slopes, unless right-of-way constraints or economic considerations preclude regrading. In some locations, the stability of a natural slope can be improved by constructing a stability berm at the toe of the slope. The stability berm functions by providing additional horizontal resistance to the driving force. The stability berm approach is normally more suitable for use with embankment fills, but may be used for natural slopes.

Groundwater can be a significant contribution to slope instability in some locations, primarily from reduction in effective stresses. The following are various methods of controlling groundwater:

1. Horizontal Drains. Horizontal drains involve drilling into a slope and installing a slotted pipe to reduce groundwater elevations. However, if the horizontal drain clogs or freezes, water pressures can build up in the soil resulting in instabilities later. Maintenance programs can be implemented to regularly clean horizontal drains in some geologic formations to avoid this problem.

2. Surface Water. Surface water can be directed away from slopes using pipe systems. Use this approach for natural slopes where water infiltration can change groundwater elevations. For newly exposed cut slopes, control of surface water helps minimize surface erosion problems and limit infiltration of surface water into tension cracks.

3. Trench Drains. Trench drains are excavated perpendicular to slope and filled with gravel. Either a geotextile or graded filter material is used to keep the trench from eventually clogging with fine-grained soil. Typically, MDT does not install a slotted pipe in the trench. Spacing and depth of the trenches are determined on a case-by-case basis depending on soil and groundwater conditions.

MDT does not normally allow benching the surface of slopes for surface water control purposes. The Department's experience indicates that benching often leads to additional maintenance requirements, erosion problems and additional surface water infiltration. This type of benching is different than benching required in *MDT's Standard Specifications* when a new embankment fill is constructed against an existing slope. In this case, benching is used to increase the shear resistance between the new and existing slope.

15.2.6.1.2 Ground Improvement

Various types of ground improvement can be considered. Typically, these methods involve replacing existing material within the slide plane with a stronger soil or cemented soil. Potential ground improvement methods include the following:

1. <u>Vibro-Densification</u>. This method involves densification of cohesionless soils with a vibrating probe. Spacing of densification points typically range from 5 ft to 10 ft (1.5 m to 3 m), depending on the density of the existing material. The depth of densification can be as much as 50 ft (15 m) or more. The method is most suitable in cohesionless soils that have less than 15% fines.

2. <u>Stone Columns</u>. This method involves placing columns of gravel or crushed rock in the ground at 5 ft to 10 ft (1.5 m to 3 m) spacing. The columns range in diameter from 24 in to 36 in (600 mm to 900 mm); the depth can be greater than 50 ft (15 m). Typically, the gravel or stone column is densified as it is constructed. Increased strength results from the densified column and, in the case of cohesionless soils, densification of soil between the columns occur because of increased horizontal stresses.

3. <u>Jet Grouting and Cement-Deep-Soil Mixing</u>. This ground improvement method involves mixing cement with the native soil, creating soil with an unconfined compressive strength of several hundred psi (MPa) or more. The coverage and amount of cement mixed with the soil will depend on the type of soil and the improvement goals. These methods can be used to depths of 100 ft (30 m) or more. The improved area can consist of columns, walls or cells. While this approach is perhaps the most versatile of the available ground improvement methods, it is usually the most expensive.

4. <u>Dynamic Compaction</u>. This method involves dropping a large, 10 ton to 20 ton (9 metric ton to 18 metric ton) weight from a height up to 50 ft (15 m) above the ground surface. The method works best in cohesionless soils and rock rubble located above the groundwater table. Depths of densification can extend to about 30 ft (9 m). The primary advantage of this approach is the relatively low unit cost for the improvement. One of the disadvantages is the ground vibration that occurs during each drop of the weight.

5. <u>Other Methods</u>. There are other ground improvement methods used that can effectively improve the soil, depending on the particular situation. One method includes compaction grouting, where a column of grout is formed by injecting grout at a high pressure. This method is often considered where headroom is limited (e.g., beneath a bridge abutment that needs to be stabilized).

Selecting an appropriate ground improvement method depends on the evaluation of several factors including the types of soil at the site, the depth of the critical failure zone, access requirements and design objectives. It is often best to contact a ground improvement specialty Contractor to determine the most feasible method for a project.

15.2.6.1.3 Structural Systems

The stability of cut slopes or natural ground can be enhanced by using various types of structural systems. Examples include the following:

1. Retaining Wall. In many cut-slope locations, retaining walls can be used to improve stability. The most common types of walls for cut locations are soldier pile cantilever, anchored or soil nail walls. Chapter 17 provides further discussion on retaining structures.

2. Micropiles. The intent of the micropile is to provide additional shear resistance across the slide plane through the pullout and bending capacity of the micropile. The spacing of micropiles is often very close (e.g., under 5 ft (1.5 m)). The depth can be 100 ft (30 m) or more. The stabilization design concept will often involve micropiles battered in two directions tied to a concrete pile cap to produce some frame action.

3. Tieback Anchors. A common method of stabilizing existing slopes is to use tieback anchors. The tieback anchor consists of anchor strands or bars that are grouted into the soil or rock at some distance from the face of the slope. The strand or bar is pretensioned against a retaining wall reaction block at the face of the slope. The grouted zone within the slope can be 40 ft (12 m) or more in length and is located behind the critical failure surface. Depending on the soil or rock conditions, the anchors can be located from 10 ft to 30 ft (3 m to 9 m) apart. The capacity of each anchor can be 100 kips (70 MPa) or more.

These structural stabilization systems are often more expensive than methods involving ground improvement, regrading and groundwater control. However, several of these methods can often be performance tested to develop confidence in the capacity of the stabilization method.

15.2.6.2 Stabilization in Rock Slopes

Stabilization methods for rock slopes depend on the type of failure mode identified during the field reconnaissance and through stability evaluations. The size of the feature requiring stabilization often is another important consideration when selecting the most cost-effective stabilization method. In many situations, the preferred approach for stabilizing an unstable rock block or mass is to force a controlled failure of the block or mass.

The following options are available:

1. Tieback Anchors. Tieback anchors are commonly used to stabilize rock slopes. The tieback is similar to that described for soil slopes. Reaction would normally be developed by tensioning the tieback strand against the rock or a concrete pad cast at the face of the rock.

2. Rock Bolts. This stabilization system is similar to the tieback anchor, except that the rock bolts are typically not pretensioned. Rather, the rock bolt is grouted in a borehole. Its primary objective is to tie the rock mass together through the tension and shear capacity of the grout and a high-strength reinforcing element.

3. Draped Rock Nets and Fences. Where small rock falls occur or the primary mode of failure is sloughing, a common method of stabilizing slopes is either to use high capacity rock fences or drape nets. In many cases, the intent of a barrier is to slow the rate of rockfall or catch the rock before it reaches the roadway.

The Oregon DOT manual *Rock Slope Guidelines* provides guidelines for rock slope stabilization. The *ODOT Manual* covers ditch designs and rock fall mitigation.

15.2.7 Erosion Control

Erosion control measures should be considered during the design of roadway cut slopes. Surface erosion and subsurface piping are common in clean sand, nonplastic silt and dispersive clays. Loess is particularly susceptible. However, all cut slopes should be designed with adequate drainage. Temporary and permanent erosion control facilities should be used to limit erosion and piping as much as practical.

The amount of erosion that occurs along a slope is a factor of the soil type, rainfall intensity, slope angle, length of slope and vegetative cover. The first two factors cannot be controlled, but the last three factors can.

The Best Management Practices (BMPs) for temporary and permanent erosion and stormwater control as discussed in the *MDT Construction Administrative Manual* provide guidance for erosion control of natural and cut slopes. Contract documents should specify construction practices that limit the extent and duration of exposed soil. This direction could include limiting earthwork during the wet season and requiring that exposed slopes be covered, particularly for highly erodible soils.

A common practice by MDT includes placing topsoil on top of riprap to help establish growth of vegetation. The project geotechnical specialist should evaluate this procedure with respect to the potential of topsoil placement impeding drainage of the soil below the riprap and affecting the stability of the slope. Where the drainage of the soils below the riprap is critical to slope stability, the project geotechnical specialist should recommend against this topsoil placement and document this recommendation in the Geotechnical Report. These determinations are usually evaluated and recommended on a case-by-case basis.

15.3 EMBANKMENTS

15.3.1 General

Embankment design is treated as a separate category primarily because the roadway embankment is usually constructed of engineered fill imported from other locations. The engineered fill is normally compacted as it is placed. Compaction of the fill is monitored to confirm that it is constructed in accordance with the *MDT Standard Specifications.*

Many of the methods described previously for cut or natural slopes are also applicable to the evaluation of embankments slopes. However, in addition to issues related to side slope stability, the settlement of the embankment must also be considered. This settlement usually results from new loads on the soil beneath the embankment. These new loads can result in a number of construction and long-term performance issues for the embankment, including bearing failure during placement of the fill, side slope instability during and after construction and long-term settlement of the fill under the new imposed loads. If liquefiable materials are present beneath the embankment during seismic loading, the embankment fill could also undergo side slope instability and bearing failure. There is also a potential for seismic-induced side slope instability for steep slopes and for seismic-induced settlements.

15.3.2 Embankment Design Considerations

Design requirements for a highway embankment will depend on a number of factors, including the height and width of the embankment, the type of soil supporting the embankment and the location of the groundwater table. The following discussions summarize some of the issues that should be considered during design. Section 15.4 provides a summary of factor of safety (FS) requirements that must be satisfied when evaluating the stability of roadway embankments.

15.3.2.1 Loading Cases

The following loading cases are normally addressed during embankment design:

1. End-of-Construction Loading. This loading case occurs as the embankment is constructed. The primary design issue is whether the existing foundation soil can support the new embankment loads without undergoing bearing failures, side slope instability or excessive settlement. These conditions are most critical when soft cohesive soils make up the subgrade. The end-of-construction evaluations may determine that the rate of construction needs to be controlled to prevent construction failures or that ground improvement methods must be used. This is especially important because the rate of construction is not known at the time of design

2. Long-Term Operational Loading. This loading case occurs after the embankment has been constructed to the final grade and excess porewater pressures have dissipated. The long-term stability of embankment slopes should be analyzed especially in fine-grained soils. In the event the foundation soils are cohesive and not heavily over-consolidated, settlement will be a design concern. Consolidation settlement and secondary compression can continue for many years and, depending on the thickness of the fine-grained soils and the amount of loading, can be several feet (meters) or more.

Significant settlement can result in distress to the pavement at the top of the embankment, as well as bumps and dips in the pavement at cut/fill transitions, and at the transition to bridges.

3. Seismic Loading. This loading case is very infrequent; on the average, once in 1000 years based on AASHTO *LRFD Bridge Design Specifications*. The primary geotechnical concern during the design earthquake is the potential for liquefaction in foundation soils beneath the embankment. Liquefaction could lead to bearing failures, side slope failures and post-liquefaction settlement. The potential for side slope failure without liquefaction is also a design consideration. The duration of loading during a seismic event is usually short; however, porewater pressures can redistribute after an earthquake leading to liquefaction-related failures that occur several minutes after the end of the earthquake.

15.3.2.2 Site Characterization

Chapter 8 provides guidelines for characterizing the embankment foundation materials at a site. These guidelines include the approximate spacing of borings. The depth of the borings during the exploration program will typically extend to twice the bottom width of the embankment. However, depending upon the foundation soil conditions, the required boring depth could be significantly deeper or shallower than this rough approximation. It is also important to determine the groundwater table elevation.

The focus of the field investigation will depend on the type of soil encountered:

1. Cohesive Soils. If soils are predominately cohesive, then the primary design issues will be bearing capacity and side slope stability during construction and long-term settlement. These design issues will usually require collecting undisturbed soil samples for laboratory strength and consolidation testing. It may also be desirable to collect in-situ vane shear strength data and conduct CPT soundings. The vane shear test can provide valuable in-situ strength data, particularly in soft clays. CPT soundings can be used to identify locations for sampling and the occurrence of cohesionless layers that could increase the rate of consolidation. It will usually be necessary to perform laboratory triaxial compression tests (e.g., UU, CU) to determine undrained strengths, total stress parameters and effective stress parameters. Consolidation tests can be conducted to define the pre-consolidation pressure, the compressibility index and the coefficient of consolidation. High-quality undisturbed samples are required from the site for both triaxial strength and consolidation testing. Chapter 9 provides further information about these tests.

2. Cohesionless Soils. Cohesionless soils are usually less of a geotechnical design concern for static loading, as most cohesionless soils will exhibit good bearing capacity and low compressibility. Settlements will generally be small and will occur rapidly during placement of the embankment fill. If the cohesionless soil is located in a seismically active area and below the groundwater table, then liquefaction will be a concern. In this case, it is necessary to have accurate blowcounts from the SPT or cone end resistance from the CPT. Grain-size distribution data are also needed.

The site characterization process becomes more complicated when the site consists of layers of cohesive and cohesionless soils. In this case, various potential failure mechanisms will need to be evaluated.

15.3.2.3 Settlement Design Criteria

The amount of total and differential settlement that can be tolerated during and following embankment construction should be evaluated. Design criteria for side slope and for bearing stability also must be met.

The project geotechnical specialist will determine the applicable design criteria on a project-by-project basis considering several factors when determining the allowable settlement. These factors involve both roadway maintenance and safety issues resulting from the amount and rate at which the settlement occurs. For example:

1. Roadways. The normal goal is to minimize the amount of settlement that occurs after the pavement surface has been placed. When roadway differential settlement exceeds a few inches (50 mm to 100 mm) over a 100 ft (30 m) distance, noticeable bumps and dips in the pavement surface occur.

2. Bridges. At bridge abutments, the settlement results in a bump at the bridge abutment or a slope change if an approach slab is used. The target maximum amount of settlement during operation should generally be small (i.e., less than an inch (25 mm)).

3. Miscellaneous. Settlements from embankments constructed next to buildings, railroads and utilities should be limited to a fraction of an inch (millimeters).

As the design criteria for settlement become more stringent (i.e., consequences of excessive settlement become more serious), the methods used to quantify the amount and rate of settlement need to be more detailed.

The ability to quantify both the magnitude and rate of settlement will depend on the thoroughness of the field investigations, the quality of laboratory testing, the size of the embankment and type and consistency of the foundation soils. As the height and width of the embankment increases, the potential for settlement also increases because of the stress change that occurs in the foundation soil. The amount of settlement also increases as the thickness and the compressibility of the foundation soil increases.

Generally, procedures used to estimate the magnitude of settlement are much better than the methods used to estimate the rate of settlement. For cases where there is uncertainty in the settlement estimate, it may be desirable to construct and monitor test fills. Information from test-fill monitoring can be used to develop better estimates of the soil compressibility and the rate at which settlement will occur.

15.3.2.4 Side Slope Stability and Bearing Capacity

Evaluate side slope stability and bearing capacity of foundation soils for both short-term and long-term loading conditions. The short-term loading condition occurs during construction, and can limit the rate of construction for fine-grained soils. For these soils, the rate of construction is such that pore pressures in fined-grained soils do not always dissipate quickly enough to support the increased loads, potentially resulting in bearing and side slope failure during placement of the embankment fill. Significant settlement could also occur as the embankment is constructed, and these settlements may have to be considered when defining pay amounts. The issues for cohesive soils for construction are not normally design concerns for embankments constructed on cohesionless soil. These soils are able to dissipate excess porewater pressures and the amount of settlement is normally small. Note that the effects of seismic loading during construction are usually not considered. The rationale for not considering seismic loading during construction and its potential consequences (e.g., liquefaction, slope instability) is that the likelihood of the earthquake occurring during construction is relatively low and the potential consequences in terms of safety and repair are usually acceptable.

15.3.2.4.1 Bearing Failure

Bearing failures can occur for embankments located over soft cohesive foundation soils, if the embankment fill exceeds the bearing strength of the soil. The consequence of the bearing failure is squeezing the underlying soft cohesive soil laterally. Avoid this by placing the fill slowly enough that excess porewater pressures from the embankment load dissipate. Countermeasures for handling this mechanism are discussed in Section 15.3.3.

Determine the allowable bearing capacity for the embankment fill using normal bearing capacity equations and the undrained strength in the analyses. Bearing capacity factors should be modified following methods given in *AASHTO LRFD Bridge Specifications* if the thickness of the cohesive layer is thin relative to the extent of the loading. Typically, the factor of safety for the bearing should be greater than 2.0 for the construction loading condition. A lower factor of safety may be acceptable if the strength gain that will occur with consolidation of the underlying subgrade material will support the FS ≥ 2 criteria.

15.3.2.4.2 Side Slope Stability

Another common problem encountered when constructing embankments over soft foundation materials is side slope failure where the side slope fails through the subgrade material when the height of the fill reaches a certain level. Evaluate the potential for side slope failures in the same manner as used to evaluate stability of roadway cut slopes. Typically, slope stability programs (e.g., GSTABL) are used to evaluate stability. Conduct initial analyses using the undrained soil strength occurring before placement of the fill. See Section 15.2.2 for further discussion of methods used to conduct slope stability analyses.

The acceptable factor of safety from the slope stability analysis for a given condition should normally be greater than those values specified in Section 15.4. If the factor of safety is less than these values, then either the height of fill placement must be reduced until the factor of safety criterion is met, or countermeasures as discussed in Section 15.3.3 must be implemented before the start of embankment construction.

15.3.2.5 Embankment Settlement

The magnitude and rate of embankment settlement are important long-term (operational) considerations, particularly where thick deposits of cohesive soil occur. Conduct settlement analyses to determine if the amount of settlement after construction is within the project criteria. If the settlement appears to be excessive, then measures may be required to improve the soil or to force settlement to occur during construction. This section summarizes methods of analysis for cohesive soil sites and cohesionless sites under long-term loading conditions.

15.3.2.5.1 Immediate Settlement of Cohesive Soil

Elastic or immediate settlement results from distortion of cohesive soil and immediate volume change in unsaturated soil. This type of settlement is usually relatively small in magnitude and occurs concurrent with the loading. Therefore, it is often ignored in design.

If an estimate of immediate settlement is required, use the procedures presented in NAVFAC *Soil Mechanics Design Manual 7.1* to estimate the immediate settlement. The procedures in DM 7.1 depend on the width of the loaded area, the Poisson's ratio of the soil, the undrained modulus of the soil, an influence factor and the amount of stress increase in the soil. Equations and charts are available for estimating each of the required terms.

15.3.2.5.2 Consolidation Settlement

Consolidation settlements are estimated based on the variability in the thickness and void ratio of the cohesive soil layer, the pre-consolidation pressure and the change in pressure. Equations for estimating the consolidation settlements are found in the reference manuals listed in Section 15.1.3. These calculations are usually conducted manually or by using either a MathCAD or a spreadsheet approach. The software program FOSSA is also used, especially when highly variable subsurface conditions are present.

The confidence of the settlement estimate will usually depend on the ability to accurately estimate the pre-consolidation pressure and the quality of the compression index obtained from the laboratory test, which is directly related to the quality of the test sample. Poor quality consolidation test data can often be identified by inspecting the void ratio versus log pressure curve.

15.3.2.5.3 Secondary Compression

Secondary settlement occurs after excess porewater pressures associated with consolidation settlement have dissipated. This settlement continues through the life of the structure, though the normal assumption is that secondary settlement decreases according to the logarithm of time. Methods for estimating the amount of secondary settlement are given in the design manuals. For many soils, contributions from secondary settlement will be small. However, in soft soils and particularly in soils with a high peat or organic content, the amount of secondary settlement can be large and difficult to predict.

15.3.2.5.4 Rate of Settlement

The rate that the settlement occurs depends on the coefficient of consolidation for the soil, the thickness of the clay layer and the time following loading. If the clay deposit has a significant number of sand interlayers, the rate of settlement increases significantly because of the reduced drainage path length. The rate of settlement is proportional to the distance to a drainage surface squared (i.e., H^2); therefore, reducing the drainage thickness (H) by a factor of two results in a four-fold increase in the settlement rate.

Determination of the average drainage path length is an important component of the field exploration. Generally, experience has been that the rate of settlement is faster than is estimated from calculations. A part of this faster rate can be attributed to the existence of thin drainage layers. The CPT method offers a particularly valuable method of identifying the location of potential drainage layers, because of its near-continuous sampling capability.

15.3.2.5.5 Settlement Analysis for Cohesionless Soil Sites

Design requirements for cohesionless soil sites are usually fairly limited at most embankment sites, except as discussed in Section 15.3.2.6 under seismic loading. The limited requirements for cohesionless soil sites results from the rapid drainage that occurs when these soils are loaded. Even when cohesive layers cap the cohesionless layer, excess porewater pressures are normally able to redistribute.

Settlement calculations can be made for cohesionless soil sites using the elastic theory. Equations for conducting this type of analyses are found in the NHI *Soils and Foundations Reference Manual*. This approach is appropriate for a range of cohesionless soil types. Use corrected blowcounts from the SPT to develop the settlement estimates. Similar methods are available for estimating settlement in cohesionless soils from the results of CPT soundings.

15.3.2.6 Seismic Performance of Embankment

A design seismic event can cause significant damage to an embankment, depending on the level of ground shaking and the type of foundation soil. The most serious damage is normally associated with liquefaction of the foundation material. Consequences of liquefaction potentially include post-earthquake settlement, side slope instabilities and bearing failures. Use the procedures outlined in Chapter 19 to make these assessments.

Seismic loading has a low likelihood of occurrence for the AASHTO design earthquake (i.e., 7% probability of occurrence in 75 years or on the average 1 event every 1000 years). Because of this low likelihood of occurrence, it may be more cost-effective to accept the possibility of earthquake-related embankment failures as long as significant life-safety or economic risks are not involved. Most embankment damage can be repaired relatively quickly, limiting the closure of the highway if a closure is required. The exceptions to this occur at bridges where it is normally necessary to protect the structure because of the expense and long repair period. Common practice in seismically active areas is to mitigate any potential for approach-fill failure within 50 ft to 100 ft (15 m to 30 m) of the bridge abutment to protect the abutment.

15.3.2.7 Documentation

When summarizing the results of embankment stability and settlement analyses, include the following information regarding the analysis:

- plan drawings showing the existing topography and the locations of explorations, roadways and other external loads (project file only);
- cross-section drawings that were used to set up the stability and settlement models, including primary soil or rock layering, location of groundwater and test hole logs summarizing soil or rock conditions (project file only);
- strengths, consolidation parameters (p_c, C_c and c_v), void ratios and unit weights assigned to each soil or rock layer. The narrative part of the report should explain the bases for these parameters, particularly focusing on short- and long-term loading conditions (project file only);
- parameters used for seismic analyses, including residual strengths and details for the liquefaction analyses (project file only);
- the method of analysis used in the determination of slope stability and settlement. If computer methods are used for slope stability evaluations, identify the computer program and the method of analysis (e.g., Spencer's procedure, Ordinary Method of Slices) and include copies of the input/output files (Geotechnical Report Appendices); and
- conclusions from the stability and settlement analyses, including a discussion of uncertainties in the analyses and recommendations on the method of mitigation, if appropriate (Geotechnical Report).

15.3.3 <u>Embankment Stabilization</u>

If the geotechnical design determines that the embankment will not meet the design criteria during construction, long-term operations or seismic loading, stabilization measures can be used to meet the design criteria. These countermeasures range from flattening slope, staged construction, preloading and surcharging to the use of lightweight fills. The objective of the stabilization measure is usually to limit settlement; however, these measures might also be used to avoid side slope and bearing failures during construction or during a design seismic event.

15.3.3.1 Types of Stabilization Measures

The discussions in Section 15.2 for roadway slopes provide typical measures for slope stability. These same measures are applicable to an embankment slope, though the wall types would change from top-down walls to typical fill walls (e.g., semi-gravity cantilever, MSE, gabion). The following measures are primarily related to the control of settlements and bearing stability within the foundation soil underlying the embankment:

1. Flatten Slopes. Where right-of-way limits allow, slopes can be flattened to increase the factor of safety of side slope stability. Methods of analysis described previously can be used to make the stability assessment. Consider both short- and long-term loading conditions during the stability analyses. Other factors to consider when evaluating this option include possible encroachment on environmentally sensitive areas and the extra material costs associated with the flattened slope.

2. Stability Berms. MDT has successfully used stability berms at the toe of embankments to improve the stability of embankment slopes where foundation soils are soft. The berm design process involves varying the height and length of the berm in a slope stability analyses until short- and long-term factor of safety requirements are met. Material used in the berm can often be lower quality fill as the primary function of the berm is to provide an increase in soil load. Important considerations when planning the use of stability berms include right-of-way restrictions, locations of wetlands or other environmentally sensitive areas, nearby utilities or structures that could be affected by the settlements induced by loads from the berm, and extra cost of materials required to construct the stability berm.

3. Removal and Replacement. Another simple stabilization measure is to remove and replace the foundation soil, if it is determined to be too soft or too compressible for the embankment. This approach is normally limited to the upper 5 ft (1.5 m) of soil below existing grade. The soil must also be above the water table, or expensive dewatering would be required. The soil that is replaced is generally not suitable for reuse, except for landscape purposes. The imported soil should be a granular fill.

4. Geogrids and Geotextile Base Reinforcement. Geogrids and heavy geotextiles can be placed between the embankment fill and the foundation soil to provide base reinforcement. The intent of the geogrid or geotextiles is to redistribute loads, resulting in a higher bearing capacity of the foundation soil. Types of geogrid and geotextiles used for base reinforcement are discussed in Chapter 20. Design methods are summarized in the NHI course *Geosynthetic Design and Construction Guidelines.* Note that this approach is used only where the bearing capacity of the foundation soil will be exceeded, and the rate of required embankment loading is such that construction cannot wait for dissipation of pore pressures to occur. If the intent of the geotextile is to provide a separation layer between the foundation and the embankment fill, select a separation geotextile rather than a heavy geotextile used for strength.

5. Preloading and Surcharging. One of the most effective methods for controlling the magnitude and rate of settlement in cohesive soils is to use preloads and surcharges. Preloading refers to the placement of the embankment fill early enough during construction that most of the settlement has occurred by the time the roadway is opened. Surcharging involves an extra thickness of embankment fill for some period of time, and then removing the extra fill. This method is effective in speeding up the preloading process and can be used to remove secondary compression in some soils.

6. Prefabricated Vertical (PV or Wick) Drains. If computations determine that settlements in foundation soils are excessive after preloading and surcharging or that construction schedules cannot be met, one option is to install prefabricated vertical (PV) drains. The PV drain is a corrugated plastic drainage strip wrapped in a geotextile. The drainage

strip is installed in the soil at a spacing that ranges from 3 ft to 7 ft (1 m to 2.1 m) to depths of over 150 ft (45 m), although most drain depths do not usually exceed 75 ft (22.5 m). A sand or gravel blanket is constructed at the ground surface to drain water extruded from the drains. The PV drain reduces the drainage path length, thereby increasing the rate of settlement. Typically, PV drains are used in combination with surcharging. For design guidance, see FHWA RD-89, *Prefabricated Vertical Drains*.

7. Column-Supported Embankments. FHWA has sponsored research on the use of column-supported embankments for mitigating settlement of approach fills located on soft foundation soils. This approach involves the installation of stone columns on a 7 ft to 10 ft (2.1 m to 3 m) grid. The stone columns extend to a bearing layer below the ground surface. Above the stone column grid, a geotextile wrapped mattress is constructed. The mattress typically ranges from 3 ft to 5 ft (1 m to 1.5 m) in thickness. The embankment is constructed above the mattress. Loads from the embankment transfer through the mattress to the columns, and the columns transfer the forces to the underlying bearing layer. Timber piles have been used in place of the stone column in Europe and Canada.

8. Lightweight Fills. Lightweight fills are generally used for two conditions: (1) the reduction of the driving forces contributing to instability and (2) reduction of potential settlement resulting from consolidation of compressible foundation soils. Lightweight fill may be appropriate on projects where the construction schedule does not allow the use of staged construction, where existing utilities or adjacent structures are present that cannot tolerate the magnitude of settlement induced by placement of typical fill, and where post-construction settlements may be excessive under conventional fills. Lightweight fill can consist of a variety of materials including extruded polystyrene blocks (e.g., EPS, geofoam), lightweight aggregates (e.g., rhyolite, expanded shale, blast furnace slag, fly ash), wood fiber, shredded rubber tires and other materials. Lightweight fills frequently have very high costs or other disadvantages.

15.3.3.2 Procedures for Selecting Stabilization Measures

Many stabilization measures are expensive and can have a significant effect on construction staging and overall schedules. When evaluating stabilization measures, perform detailed evaluations considering the following:

1. Project Schedule. The construction schedule is one of the most significant considerations. For example, if the schedule is not critical, then extended periods of preloading and surcharging are possible. As the schedule becomes more critical, the need for PV drains increases. It is often difficult to determine the anticipated schedule during design, and an attempt should be made to determine if the project will require one or two construction seasons. Preloading and surcharging are more practical if the waiting period can occur during winter months.

2. Construction Access and Constraints. Some stabilization measures require easy access and large working areas. This would potentially preclude use in some locations (e.g., at the bottom of a gully). Stabilization measure methods that involve large vibrations (e.g., deep dynamic compaction) would not be suitable near a residential area.

3. Types and Depth of Soil. The type of soil will have a significant effect on the selection of the stabilization measure. Vibro-densification and deep dynamic compaction methods are suitable for clean granular soils, whereas stone columns and jet grouting can be used in soils with high fines content. Jet grouting and soil mixing procedures can be used to depths of 100 ft (30 m) or more, but at significantly higher unit costs.

4. Cost. Methods will range greatly in cost – from roughly $10/yd^3 ($15/m^3) of improved soil to over $300/yd^3 ($400/m^3).

The preferred approach for selecting the stabilization measure is to identify possible methods based on state-of-the-art research on ground improvement. Websites are helpful in collecting details about the different methods. After obtaining background information, it is often desirable to contact and meet with potential Contractors to determine other issues and factors that could control the success of the method. For stabilization measures that involve specialty methods, specifications have to be very clear about experience using the specific method. It is also critical to contact references to confirm that the Contractor has experience using the method.

15.3.4 Embankment Construction Design Support

A number of issues related to embankment construction may need to be addressed by the Geotechnical Section during design. These issues range from providing guidance for compaction and moisture control to assessing problematic soil conditions. See Chapter 22 for additional discussion of support provided by the Geotechnical Section during construction.

15.3.4.1 Compaction and Moisture Control

Clean granular materials are normally preferred for constructing embankments. These materials are relatively easy to place and engineering performance is relatively well established. However, for many MDT projects, materials with appreciable fines are used out of necessity for embankment construction. This is particularly the case in eastern Montana and common in other areas where the use of granular material for embankment construction would be cost prohibitive. In addition to presenting constructibility problems because of their weather sensitivity, the engineering performance of fine-grained materials, particularly relative to settlement and side-slope stability, is more sensitive to the placement method. The project geotechnical specialist may need to perform special stability and settlement evaluations on projects in which fine-grained materials are likely to be used. On some projects, these evaluations are necessary to establish requirements for compaction and moisture control during construction.

Normally, the Contractor is responsible for supplying the borrow material. The borrow source could be either other portions of the project alignment or a borrow pit in the area. The project geotechnical specialist should discuss material requirements and potential borrow sources with District personnel early in the project.

If a borrow source has been identified, then the project geotechnical specialist must determine if any special laboratory testing is required to establish design parameters for the borrow material. Generally, the properties of granular sources can be estimated from empirical relationships and from experience with these materials. However, if the borrow source is likely to be fine-grained or contain an appreciable fines content (e.g., more than 30%), it may be necessary to conduct laboratory strength tests on representative samples of the material if side-slope stability issues occur. Laboratory triaxial tests on remolded compacted samples may be necessary to support side-slope stability evaluations of embankment slopes constructed of fine-grained soil at relatively steep side slopes or when the foundation soils are soft or weak.

Laboratory tests should be conducted at the likely density to be used for construction. In most cases, the embankment material will be compacted at either optimum moisture content or a few percent dry of optimum. Both short- and long-term loading conditions should be considered. Where compaction and moisture control are expected to be difficult to meet in the field because of weather conditions or material type, consider the possibility of property variation during laboratory testing and during stability analyses.

For projects using fine-grained soil as embankment fill, the project geotechnical specialist needs to provide input on earthwork specifications regarding material types and compaction requirements that are consistent with the design. Guidance should also be provided on variations in material types, particularly property variability, that could require a re-evaluation of side slope stability.

15.3.4.2 Expansive Soils

Another construction consideration involves certain problematic soil conditions (e.g., expansive soils). These soils are encountered in areas where Cretaceous shales (e.g., Bearpaw, Niobrara shales) occur. The engineering performance of these soils can lead to significant long-term maintenance requirements if not considered appropriate. As such, the project geotechnical specialist may need to conduct additional specialized testing, including compaction, strength, compressibility and shrink/swell to adequately evaluate the response of these problematic soils to construction and loading. Chapter 9 provides additional descriptions of the laboratory testing methods.

For special or problematic soils, the project geotechnical specialist should provide guidance on compaction control based on the laboratory testing results. Expansive soils that have been placed and compacted in a dry state absorb considerable moisture during a wet season and, if highly compacted, these soils swell considerably. Expansive soils must be treated or protected so that the moisture content and density (after compaction) will not change significantly throughout the lifts.

15.3.5 <u>Erosion Control</u>

Surface erosion control is also an important part of embankment construction, particularly at locations where sand, nonplastic silt and dispersive clays are used as embankment fill materials. These materials can be very erodible, as discussed in Section 15.2.7 for cut slopes along the roadway.

Considerations for erosion control on embankment slopes are summarized in the MDT *Construction Administrative Manual* under Best Management Practices (BMPs) for temporary and permanent erosion and stormwater control. For embankment slopes, consider limiting earthwork during the wet season, limiting the extent and duration of exposed soil and requiring that exposed slopes be covered, particularly for highly erodible soils.

A common practice by MDT includes placing topsoil on top of riprap to help establish growth of vegetation. To avoid future maintenance problems from embankment slope failures, the project geotechnical specialist should evaluate this procedure on a case-by-case basis. Where the drainage of the soils below the riprap is critical to slope stability, the project geotechnical specialist should recommend against this topsoil placement and document this recommendation in the Geotechnical Report.

15.4 FACTOR OF SAFETY SELECTION

Results from limit equilibrium stability analyses are normally given in terms of a factor of safety (FS) against instability for the specified load conditions. In these analyses, the FS defines the ratio of forces resisting stability, which is defined by the strength of the soil versus the loads causing instability. The loads include not only those from gravity but also from external sources (e.g., earthquakes, traffic, water forces). A slope is considered theoretically stable if the FS is greater than 1.0. However, because of uncertainties in the method of strength determination, groundwater location and method of analysis, a margin greater than 1.0 will be required for most designs.

15.4.1 Factors Affecting Factor of Safety Selection

The design FS will depend on a number of factors. For example, the quality and extent of the exploration program will influence the FS selection. Other considerations include:

- consequence of failure (e.g., the acceptable FS might differ for a rural road versus a heavily traveled Interstate or if a major bridge or railroad is affected);
- encroachment of private right-of-ways and environmental impacts;
- type and size of failure (e.g., small local failures may be easier handled as routine maintenance, whereas a large failure may be hard to repair and take an extended period of time); and
- cost to achieve an acceptable factor of safety.

15.4.2 MDT's Requirements for FS at Soil Sites

The selected FS will depend upon specific site conditions. For normal conditions, MDT requires that slopes meet the following FS values:

- End of Construction: FS ≥ 1.3 to 1.5 (at structures)
- Long-term: FS ≥ 1.3
- Cut Slopes in Clay : FS > 1.5
- Sudden Drawdown: FS > 1.2 to 1.3
- Seismic: FS > 1.1 to 1.2

If these FS values cannot be met, stabilization methods can be considered to improve the FS value or a lower factor of safety might be accepted with the Geotechnical Engineer's approval. Possible stabilization methods are discussed in Section 15.2.6 and Section 15.3.3. Before accepting a lower FS value, the costs of stabilization versus the cost of accepting additional risk to public safety, economic disruption or repair costs should be evaluated.

If the consequence of slope instability could be significant (e.g., a major bridge being damaged), an FS ≥ 1.5 or higher may be desirable, depending on the specific conditions involved with the instability. The decision to use a higher FS value for design could have significant economic consequences, if stabilization will be required to meet the target FS. Therefore, discuss the cost of stabilization versus risk with the Project Manager and MDT Management to determine if the additional costs are warranted.

15.4.3 MDT's Requirements for FS at Rock Sites

The factor of safety for rock slopes is usually approached on a site-specific basis. For major rock slopes where the consequences of failure are severe, the minimum required calculated factor of safety should usually ranged from 1.5 to 2.0. For minor slopes or temporary construction slopes where failure would not cause a hazard to individuals or major loss of property, the minimum required factor of safety is 1.3.

For a rock slope to be judged safe with respect to failure, the minimum factor of safety for all potential failure planes must be equal to or greater than the minimum required value. As with the above discussion for soil sites, if these FS values cannot be met, stabilization methods can be considered to improve the FS value or a lower factor of safety might be accepted with the Geotechnical Engineer's approval. Possible stabilization methods are discussed in Section 15.2.6. Before accepting a lower FS value, the costs of stabilization versus the cost of accepting additional risk to public safety, economic disruption or repair costs should be evaluated.

Also, similar to the soil site, if the consequence of rock slope instability could be significant (e.g., a major bridge being damaged), an FS $\geq$ 2.0 may be desirable depending on the specific conditions involved with the instability. The decision to use a higher FS value for design could have significant economic consequences if stabilization will be required to meet the target FS. Therefore, discuss the cost of stabilization versus risk with the Project Manager and MDT Management to determine if the additional costs are warranted.