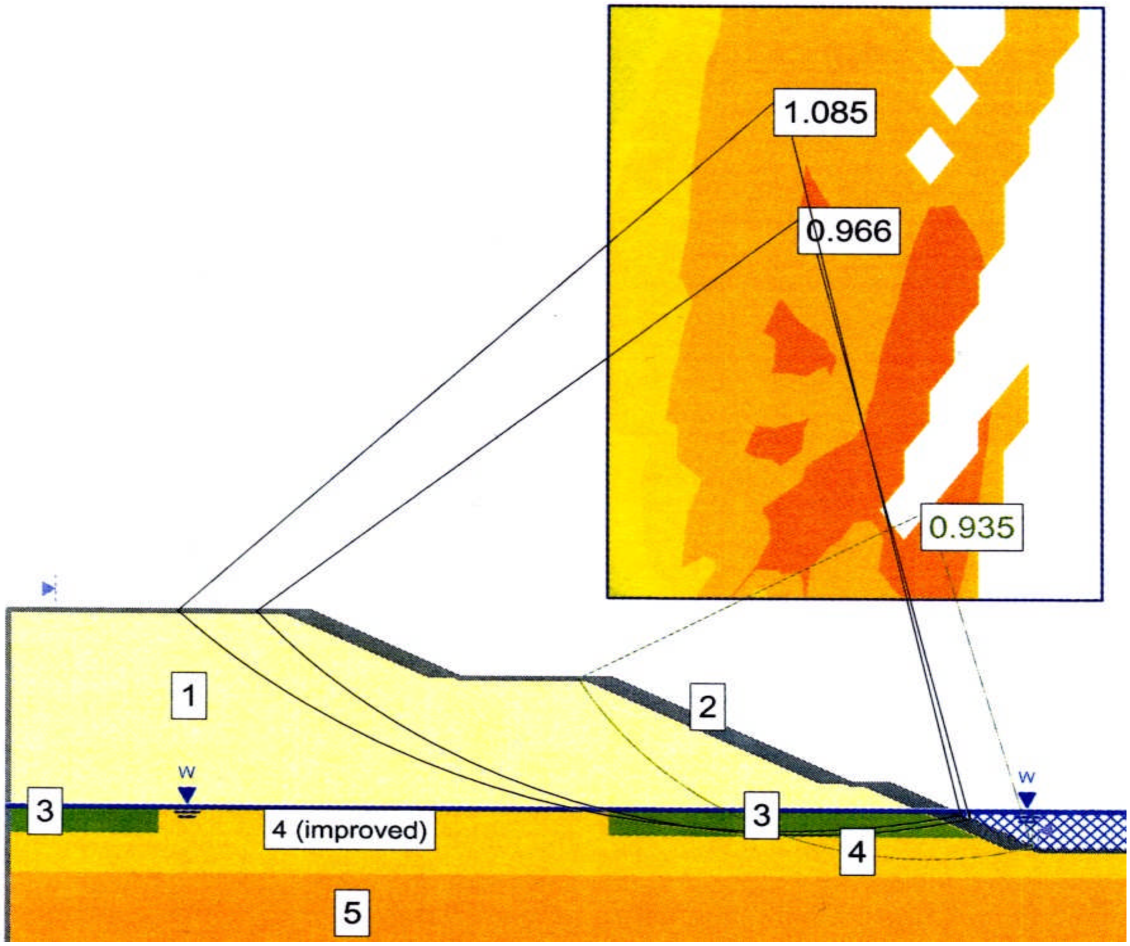
GEOTECHNICAL POLICIES AND PROCEDURES MANUAL

CHAPTER 11

ANALYSIS AND DESIGN



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1. PURPOSE

After exploration and testing have been completed, the Geotechnical Engineer must organize and analyze all existing data and provide design recommendations. The extent of the analysis depends upon the scope of the project and the soils/rock involved.

2. INTRODUCTION

Many factors must be considered during the analysis and design phase of projects. Figure 11-5 provides guidelines for types of analyses that should be performed. The references cited in the text provide suggested methods of analysis and design. Figure 11-4 provides a list of computer software programs used by the Department. In using these references and software programs, be aware that engineering technology progresses rapidly and those methods and software programs are being improved or new methods or programs introduced frequently. The Geotechnical Engineer should keep abreast of the state-of-the-art practice for appropriate and economical designs. The Geotechnical Engineer needs to consult with the Principal Geotechnical Engineer when new techniques are to be utilized.

3. ROADWAY EMBANKMENT MATERIALS

The suitability of in situ materials for use as roadway Embankment Borrow is determined by analysis of the results of reconnaissance and subsurface exploration. Embankment materials must meet the Department Standard Specifications for Road and Bridge Construction, unless alternate specifications are needed.

The subsurface materials encountered during soil explorations should be classified, and interpretations of the stratigraphy should be made. Soils should be grouped if they belong within the same stratum. If planned testing identifies dissimilar types of soils within the same stratum, additional sampling and testing may be required to better define the in situ materials and potential variabilities. On occasion, dissimilar soil types may be grouped for such reasons as borderline test results, or insufficient quantities of in situ material to economically justify separation of the material during construction. Some engineering judgment must undoubtedly be used in grouping and stratifying soil types. Conclusions should be clearly explained and justified in the Geotechnical Report. Each stratum should be analyzed to define characteristics that may affect the design.

3.1 Suitable use of Materials within Project

The Geotechnical Engineer should determine the appropriate suitability of materials to be excavated in the project. The uses of specific materials are typically based on classifications and tests, such as gradations, R-value, plasticity, moisture, rock durability, and degree of weathering. The Geotechnical Engineer should determine if materials within the proposed project excavations meet required specifications of Borrow Embankment materials. Estimates should be made of available quantities of each identified material type. The location of material types should be identified in order to provide information for project designers, construction schedule planners, and contractors regarding potential earthwork issues. Sometimes the sequencing of cross-hauls to excavate, process and deliver materials can be complex and therefore specific information can be beneficial during project development and construction. Borrow Embankment materials have R-values greater than or equal to 45. Typically roadway embankment materials must meet required specifications for Borrow Embankment materials. The Geotechnical Engineer must provide detailed and accurate information on the boring logs and in the Geotechnical Report regarding the drilling process and description of encountered soils during the subsurface investigation. This will assist the Contractor to make a reasonable interpretation of the subsurface conditions to be encountered and avoid a change of conditions when excavating. Contractors review boring logs and Geotechnical Reports to make their assessments of excavatability requirements. The Geotechnical Engineer should evaluate whether controlled blasting techniques are to be used to develop appropriate cut slopes in rock.

Rock excavation and blasting are described in greater detail in Section 6.

3.2 Limits of Usable Materials

The limits of in situ materials considered unsuitable, for use on the project, or as a foundation to support structural elements of the project, should be defined, and the effect of each material on roadway performance should be assessed. Refer to Sections 203 and 207 of the Department Standard Specifications for Road and Bridge Construction for requirements on excavation and replacement of these materials. In areas where excavation may be excessive, but the potential for problems exists if not excavated, possible stabilization methods to be considered include placement of geotextile, surcharging, mixing the soil with lime, cement, or fly ash, or a combination of these.

3.3 Corrosivity

Results of field and/or laboratory tests should be reviewed and the potential for corrosion of the various foundation and drainage system components should be assessed. The Structural/Chemical Section of the Materials Division provides the recommendation.

3.4 Drainage

The permeability and infiltration rates of cut slopes and embankment materials should be estimated based on test results or knowledge of the material characteristics. This information, along with data on the depth to groundwater, can be used in assessing the need and design of a drainage system. Subsurface drainage systems may include pavement underdrains and interceptor drains. Surface drainage systems may include retention, detention, and infiltration basins. The Hydraulics Section of the Department's Roadway Design Division designs surface drainage systems.

3.5 Earthwork Factors

Earthwork factors (shrink/swell) used in estimating cut and fill quantities are typically based on local experience. Shrink/swell is the percent decrease or increase in volume that occurs when a material is excavated from an in situ location and used to construct an embankment. In general, rock that is blasted from in place and used in an embankment will swell. The Geotechnical Engineer should determine the shrink/swell factors for the materials to be excavated, and estimate average values according to roadway excavation plans. The roadway alignment can be divided into sections to group excavated materials that are similar. These factors affect the Roadway Design Engineer's materials quantities estimate for the project (refer to Church, (1981).

The values of shrink/swell factors vary considerably depending on the method of fill construction and the level of compaction effort applied. Typically, material to be wasted from a project is placed in a disposal area by end dumping using gravity compaction. For highway construction, earth materials are placed and compacted using heavy compaction equipment as specified in the Department Standard Specifications for Road and Bridge Construction. For most hard rock, such as granite or limestone, the swell can range up to 36 percent. Conversely, due to the compaction effort, soils that are removed from in place and then placed and compacted in an embankment tend to shrink, typically averaging about 10 percent. If the Geotechnical Engineer cannot determine the shrinkage factor for excavated soil used as Borrow Embankment material, a value of 15% should be recommended.

3.6 Other Considerations

Presence of characteristics and features determined from soil explorations that affect the roadway design, include expansive soils, springs, sinkholes, rock, or soft subgrade. The effect of these characteristics on roadway performance should be assessed.

4. EMBANKMENT SETTLEMENT

The magnitude and time rate of settlement of embankments are typically estimated using one-dimensional consolidation theory and strip loading stress distributions. Several conditions that can cause settlement include loading due to new embankments, embankment widening, lowering of groundwater (including temporary dewatering), temporary fills and stockpiles. FHWA publications (Soils and Foundations Workshop, 2000; and Advanced Course on Slope Stability, 1994) provide technical guidance for estimating settlement at abutments, along embankment centerline and edge of pavement, and at locations beyond embankment toes where sensitive structures or facilities might exist. The Geotechnical Engineer may use FoSSA software, provided by the Federal Highway Administration to estimate settlements. Other software programs such as EMBANK or SIGMA/W (a finite element program) are available.

If design analyses indicate excessive settlement magnitude or time, mitigation measures should be evaluated. Mitigation measures could include: (1) surcharging, (2) removal of settlement-prone material, (3) installation of vertical drainage systems such as wick drains, sand drains, or stone columns, (4) ground improvement such as jet grouting, dynamic compaction, deep soil mixing, vibroflotation, and placing piles, (5) reducing loads by decreasing the height of embankments, substituting lightweight fill (sawdust, shredded tires, baked shale, extruded polystyrene-EPS), (6) spanning the compressible area with a pile-supported structure (bridge or viaduct), and (7) column supported embankments.

The use of surcharging is a very common mitigation measure. The principle is to over-consolidate the foundation soil to reduce postconstruction settlement. The postconstruction settlement is estimated by comparing the settlement versus time graphs for both the design embankment and for the embankment plus surcharge condition. Sometimes, surcharging does not produce the desired amount of preloading settlement within the timeframe constraints. If this is the case, then the alternative mitigation measures are evaluated.

Vertical drainage systems can be used to speed the rate of settlement of a fill placed on top of a soft soil deposit. This method effectively shortens the drainage path that pore water must travel during consolidation under an applied load. The Geotechnical Engineer selects the drain spacing that results in a consolidation that meets the project design and construction requirements (i.e., settlements have occurred to an acceptable magnitude prior to construction of a settlement sensitive element, such as a bridge abutment). A wick drain consists of a plastic drainage core wrapped in a nonwoven geotextile. The drains are installed by a mandrel on a rig that drives the mandrel with continuous down pressure or vibration. The mandrel is extended to the design elevation and then retracted. An anchor plate at the bottom of the drain prevents the drain from being pulled upwards with the mandrel. After installation of the drains, a free draining sand blanket is installed on the ground surface to enable free flow of water from the drains. The fill embankment is then constructed on top of the sand blanket.

Jet grouting mixes cement with the native soil to create a higher capacity and less compressible foundation. Silt and sand soils are best suited for this type of mitigation technique since they are readily cut and mixed by the water jets. Clay soils may not break down sufficiently. Dynamic compaction and vibroflotation (including stone columns) increase the density of subsurface soils. Silt and sand soils are best suited for these types of mitigations techniques since they relieve excess pore water pressures generated by the dynamic motions. Placing piles can be used (although infrequently in practice) to distribute embankment loads to a lower, less compressible stratum.

Sometimes a proposed roadway alignment can be modified to reduce embankment heights in critical settlement-prone areas. If not, another option to reduce embankment weight is to substitute lightweight materials instead of soils in the embankment. Sawdust or wood fiberfills have been used for decades. Since the 1980s, shredded tires have also been used in embankments. Drawbacks for these lightweight materials are: (1) having the potential for combustion, and (2) occurrence of surface deflections/rebound under traffic loads. Baked shale and extruded polystyrene (EPS) do not have these drawbacks; although EPS needs to be encapsulated to prevent damage from solvent spills, such as gasoline. EPS is the lightest of the lightweight construction fill materials, with a density of about 2 lbs/cu. ft.

On some projects such as those in wetlands, much mitigation may not be permitted due to environmental constraints. Use of a bridge or viaduct may be acceptable because of having minimal environmental impacts, as well as avoiding need of support on compressible soils. Structural mitigations are typically the most expensive options.

5. SOIL CUT AND FILL SLOPE STABILITY

Short cut and fill slopes are typically evaluated using precedence, experience, and judgment. The Geotechnical Engineer needs to recognize when the height of a cut/fill slope or poor subsurface conditions warrant performing stability analyses. A quick form of analysis is the use of stability charts and graphs that solve relatively simple and common cases. The U.S. Forest Service Slope Stability manuals provide a compilation of stability charts. Also, refer to FHWA's Advanced Course on Slope Stability manual. Cuts and fills that have irregular geometry, intermediate groundwater levels, or low-strength soils may justify the use of limit equilibrium stability analyses. Most landslides should be evaluated with stability analysis.

For decades now, FHWA has endorsed XSTABL, which is a stability analysis software based on the STABL programs developed by Purdue University in the 1970s. This program has been modified into the user-friendly program PCSTABL. There are several other software packages which some have greater capabilities and output formats, such as Slope-W, UTEXAS, and PC-Slope. Refer to FHWA publications (Soil and Foundation Workshop and Advanced Course on Slope Stability).

Typical cut and fill slopes have inclination angles of 2H:1V. Steeper slopes can be used based on analyses using representative shear strength or based on local precedence. When the analyses indicate that slope angles do not have a sufficient level of stability (Factor of Safety, F.S.), mitigation measures may be necessary. The following is a list of possible mitigations for cut and fill slopes.

<u>Cut Slope Mitigation</u>

- Flatten the cut slope angle
- Lower the groundwater level using drainage methods (trench drains, interceptor drains)
- Replace some of the cut slope material with higher strength material, such as rock fill (rock inlay)
- Reinforce cuts (soil nails)

<u>Fill Slope Mitigation</u>

- Flatten the fill slope angle
- Install drainage measures (underdrain blankets, trench drains)
- Use an embankment fill material with higher shear strength properties (for example,

embankments constructed with rock fill can generally support 1.5H:1V slopes)

- Place toe counterberms
- Place rockfill shear key
- Improve the foundation materials (overexcavate and replace)
- Use staged embankment placement (allowing excess pore water pressures to dissipate in the foundation soils between each fill stage)
- Reinforce the fill slopes (RSS)

5.1 Embankments Over Liquefiable Ground

Foundation soils that are potentially liquefiable (typically saturated, loose silts and sands) can lose strength during earthquake shaking or possibly from blasting-induced vibrations. Embankments overlying soils that liquefy can experience stability problems including slumping, lateral spreading, and subsidence.

The simplified analyses methods involve comparing the cyclic strength of the soil to the cyclic stresses caused by the earthquake. If the stresses exceed the strength, the material has a factor of safety of less than 1.0 against liquefaction and is determined to be potentially liquefiable. An estimate of the cyclic strength (termed the cyclic resistance ratio or CRR) is commonly obtained from in situ tests including the Standard Penetration, Cone Penetrometer, or geophysical. An estimate of the cyclic stresses caused by an earthquake (termed the cyclic stress ratio or CSR) is obtained from a simplified formula developed by Seed and Idriss (1971). More detailed evaluations of cyclic stresses and in situ static stresses can be obtained from one-dimensional ground response programs such as SHAKE, or from two-dimensional, nonlinear, finite-difference programs such as FLAC (Itasca Consulting Group, 1995). Estimates of potential lateral spread can be obtained from empirical procedures developed by Youd, et al. (2002). Mitigation measures can include dynamic compaction, blasting and vibroflotation, placing stone columns, permeation and jet grouting, removal of the potentially liquefiable layer, or possible relocation of the project.

Refer to the 1997 FHWA, Geotechnical Engineering, Circular No. 3, “Design Guidance: Geotechnical Earthquake Engineering for Highways,” Vol. 1; and by recent NCEER workshops (see, “Liquefaction Resistance of Soils: Summary Report” from the 1996 NCEER; 1998 NCEER/NSF, “Workshops on Evaluation of Liquefaction Resistance of Soils,” Youd and Idriss, and ASCE, “Geotechnical and Geoenvironmental Engineering,” April, 2001).

5.2 Reinforced Soil Slopes

Reinforced Soil Slopes (RSS) consist of tensile reinforcements in soil backfill allowing the slope to be constructed steeper than without the reinforcement. Depending on the materials used, the slope inclinations can be constructed up to 70 degrees from the horizontal. Primary reinforcing elements provide overall stability, while secondary (shorter) reinforcing elements are used to provide near face stability. Typically, various types of slope facing such as erosion control blankets, geogrids, gabions, or shotcrete are used to prevent near surface erosion and raveling, especially for steep slopes.

All RSS must be designed for external stability such as sliding and deep seated, local bearing capacity failure, and excessive settlement from both short- and long-term conditions. Reinforcement requirements must be designed to adequately account for the internal stability of the slope. Mechanically Stabilized Earth (MSE) walls and Reinforced Soil Slopes (RSS) – Design and Construction Guidelines (FHWA-NHI-00-043) provide detailed design procedures for reinforced soil slopes. The design concepts are similar to MSE walls.

RSS are relatively easy to construct, and have a lower cost relative to MSE walls. Proper drainage is needed behind the reinforced mass to prevent development of hydrostatic pressures. Design of reinforced slopes requires that sufficient width be provided to install reinforcing elements. In road rehabilitation projects, construction of the required backfill zone could impact the travel lanes or may necessitate acquiring additional right-of-way. Reinforced soil slopes have a number of advantages including:

- Requiring less fill material and having a smaller overall footprint, which can reduce right-of-way acquisition and environmental impacts in sensitive areas.
- Often allowing, onsite materials to be used for construction.
- Assisting growth of vegetation on slope face for a more environmentally acceptable appearance.

6. ROCK CUT SLOPES

The Geotechnical Engineer should take into account the structural and strength properties of the rock to develop designs that address the constructability concerns and long-term performance of the finished cut slopes. The objective of the design process is to determine the cut slope angle for the steepest continuous slope without intermediate slope benches that addresses cut slope performance (reduced rockfall) and safety while reducing excavation quantities. Rock slope stabilization and rockfall protective measures may be required to reduce rockfall hazards, minimize environmental and right-of-way impacts, and meet other project goals. The Geotechnical Engineer should review:

- FHWA, “Rock Blasting and Overbreak Control,” NHI Course No. 13211, FHWA-HI-92-001
- FHWA, “Rock Slopes,” NHI Course No. 130235 - Module 5
- FHWA, “Rock Slopes: Design, Excavation, Stabilization,” FHWA-TS-89-045
- FHWA, “Rockfall Hazard Mitigation Methods - Participant Workbook,” FHWA SA-93-085
- FHWA, “Rockfall Hazard Rating System - Participant’s Handbook,” FHWA SA-93-057
- Pierson, L.A., Gullixson, C.F., and Chassie, R.G., “Rockfall Catchment Area Design Guide, Final Report SPR-3(032),” Report No. FHWA-OR-RD-01-04

- FHWA, “Guide Controlled Blasting Specification” (1985)

6.1 Predesign Tasks

A general knowledge of the geology of the area can be obtained by examining published geologic maps and reports along with any available aerial photographs. Slope angles and slope heights are determined using topographic maps and cross sections. If site topographic maps are not available, topographic surveys need to be performed.

The Geotechnical Engineer needs to characterize the rock in the slope and consider rock classification, degree of weathering, presence of discontinuities, and degradability. Degradability of rock, such as the weathering potential and erodibility, needs to be considered in the design because these characteristics can adversely affect the long-term stability of the slope.

Discontinuities, such as joints, foliations, shears, and faults, are important factors in the stability of rock slopes. The orientation, frequency, persistence, and shear strength of rock discontinuities are obtained from existing cuts, outcrops or rock core. The measurements of the strike and dip (dip and dip direction) of the discontinuities along with their shear strength are typically presented and evaluated on stereonets to determine if rockfall is kinematically possible. When rock properties and discontinuities need to be investigated beyond their ground surface expressions or when no outcrops are available, coring exploration is used to obtain rock cores. The cores provide information on the roughness and infilling of discontinuities. Oriented coring exploration may be used to obtain information on the dip and dip direction of discontinuities.

The shear strength of rock along the discontinuities that separate the rock mass into discrete blocks is a much more critical rock slope stability parameter than the strength of the intact rock. In simplest terms, the shear strength governs the angle at which one rock block will begin to slide over an adjoining block. The resistance to sliding is controlled by both the macro roughness (irregularities such as steps or undulations on the joint surfaces) and the micro roughness. The micro roughness is related to the texture of the rock and any movement that may have occurred between adjacent blocks that may have created slickensides or gouge. The resistance to sliding can be determined by performing shear tests in the field or laboratory, or it can be estimated in the field by observing the inclinations of pre-existing failure surfaces.

Groundwater conditions must be evaluated for the design and analysis of rock cut slopes. Groundwater pressure acting within the discontinuities can cause significant destabilization by decreasing the shear strength due to uplift and/or increasing the driving forces acting on the block. Typically, the groundwater level within a slope can be estimated by observing seepages from and around the rock slope. When groundwater conditions are unknown and groundwater is expected to influence the stability of the slope, groundwater pressures can be measured using piezometers.

Rockfall hazards and potential rock slope problems have been evaluated for many of the Department's highways through the implementation of the Rockfall Hazard Rating System (RHRS). The maintenance history, including a description of past problems and interim mitigations measures are included in the RHRS database. The Geotechnical Engineer should refer to FHWA, Rockfall Hazard Rating System - Participant's Manual, FHWA-SA-93-057, 1993.

6.2 Slope Stability Analyses

The stability of hard rock slopes is highly controlled by discontinuities (joint and joint sets) within the rock. Failures tend to occur as discrete blocks. Discontinuities form planes of weakness. Without discontinuities, rock slopes, even those composed of relatively weak rock, could stand hundreds of feet tall without potential of failure. Kinematic analysis of the discontinuities is performed to determine the most likely mode of failure. This is followed by slope stability analyses to determine the factor of safety.

6.2.1 Kinematic Analyses

A kinematic analysis is the first step in evaluating slope stability. This analysis establishes the possible failure modes of the blocks that comprise the slope. The analysis determines if the orientations (dip and dip direction) of the various discontinuities will interact with the cut slope orientation and inclination to form discrete blocks with the potential to fail without regard to any forces that may be involved. Failure modes typically fall within one of three categories: plane failure, wedge failure, or toppling. Where a rock mass is highly fractured by randomly oriented discontinuities or composed of very weak rock, the mode of failure may be circular as in a soil slope.

The analysis involves a comparison of the orientations of the dominant discontinuity sets with the orientation of the cut slope. Where discrete blocks are formed and where the failure surfaces that bound these blocks dip out of the slope at an angle steeper than the shear strength along the discontinuity, failure is kinematically possible. A stereonet is used to display the discontinuity and slope data in this analysis. For detailed discussions of stereographic analysis, refer to Hoek and Bray (1981), Hoek and Brown (1980), and Goodman (1976).

6.2.2 Stability Analyses

After the kinematic analyses have identified the most likely mode(s) of failure, the next step is to perform a stability analysis using the shear strength of discontinuities and groundwater conditions. The objective is to calculate the factor of safety of the slope or individual block being analyzed. Each of the failure modes follows.

- Plane Failure – The reference discusses general procedures, influence of groundwater and tension cracks, and reinforcement of slopes (use of rock bolts and anchors) and has several practical examples that are helpful in understanding the procedures involved.

- Wedge Failure – The reference contains wedge stability charts that can be used when the two discontinuity planes forming the wedge have frictional strength only and the slope is fully drained. These charts may be suitable for a preliminary design of highway cut slopes. Wedge failure stability analyses are more complex than plane failures. Computer programs (SWEDGE, YAWC, GOLDPIT, and Key Block Theory) may be used to perform the analyses. Appendix IV of the reference describes analytical solutions that may be used with computers and programmable calculators.
- Toppling Failure – The reference contains an analytical method that may be applied to a few special cases of toppling failures.
- Circular Failure – Slope failures in decomposed rock, closely fractured rock, or rock fills generally occur along a curved surface and are not controlled by discontinuities. These slopes may be analyzed using circular failure surfaces similar to the method used in analyzing soil slopes. Computer programs, such as XSTABL are routinely used for the analysis.

6.2.3 Factor of Safety

The minimum factor of safety (FS) to be used in stability analyses for a specific rock slope depends on factors such as:

- The degree of uncertainty in the stability analysis inputs; the most important being the amount of intact rock, shear strength and groundwater conditions
- Costs of constructing the slope to be more stable
- Costs and other consequences of the slope failure
- Whether the slope is temporary or permanent

Typical FS values range from 1.3 to 1.5; however, based on engineering judgment, values outside of this range may be appropriate, depending on the circumstances.

6.2.4 Computer Programs

Computer programs such as Rocscience and Rockpack III are available to perform rock slope stability analyses. Rockfall events can be simulated using computer programs such as CRSP (Colorado Rockfall Simulation Program), Version 4.0 and Rocfall, Version 4.0. These programs allow hundreds or even thousands of rockfall events to be quickly simulated. This number of events typically represents many years of actual rockfall. The output includes rockfall trajectories and the bounce heights and total kinetic energies at selected locations on or beyond the base of the slope. The results are useful in determining the optimum location and capacity of certain mitigation measures, such as rockfall barriers and catch fences.

6.3 Rock Slope Design

Rock slope design consists of determining (1) the orientation of the cut, (2) the steepness of the cut, and (3) the need for mitigation measures if the resulting factor of safety is deemed too low or the rockfall potential onto the facility is unacceptably high.

6.3.1 Design Standards and Policies

The Geotechnical Engineer should verify the applicable Department standards and policies to confirm the practices to be followed. The Department Standard Specifications for Road and Bridge Construction includes standard requirements for rock construction, including the following sections:

- 203.03.03, Blasting
- 203.03.04, Rock Cuts
- 203.03.05, Overbreak

Although thorough engineering analyses should be performed, it is important to note that due to uncertainties in defining the controlling conditions present within a rock mass, sound engineering judgment should be applied in the design of rock slopes. Experience is the best predictor of the effectiveness of a rock slope or rockfall remedial design. Case histories in similar rock conditions should be consulted to provide additional guidance.

6.3.2 Selecting Slope Angle

Several factors affect how steep a rock slope should be cut including the orientation and strength of the discontinuities within the slope, the anticipated method of construction, and whether additional measures will be used to enhance slope stability. Some methods of slope construction damage the rock such that the finished cut slope has an increased likelihood of long-term rockfall. Uncontrolled blasting, for example, can cause fracturing and open existing fractures tens of feet into the slope. A finished cut slope can be constructed by excavating the rock using heavy equipment ripping or production blasting techniques, or it can be augmented with controlled blasting methods. The use of controlled blasting, either presplitting (preshear) or trim (cushion) blasting, produces a cut slope with significantly less potential for rockfall.

Local experience with similar rock type should be investigated. In some cases, right-of-way limitations or other factors, such as economics, may require the design slope to be steeper than desirable. If the resulting factor of safety is determined to be too low, or the potential for rockfall is estimated to be unacceptably high during the design life, rock slope stabilization and rockfall mitigation measures should be included in the design.

6.3.3 Construction Considerations and Mitigation Measures

In addition to the natural rock discontinuities that control the stability of rock slopes, fractures caused by poor blasting techniques could increase the rockfall potential. Mitigation measures to enhance stability include installation of reinforcement, drainage, and erosion protection systems. The following is a partial list of available techniques:

- Controlled Blasting - Lightly loaded, aligned and closely spaced blast holes are used to form the final cut slope face in a manner that minimizes the affects of the intense detonation gas pressures caused by production blasting. The controlled blasting is performed either before the main production blasting is detonated (presplit blasting) or after the production blasting (cushion blasting). In presplit blasting, the row of control blast holes is detonated to form a break in the slope along the final cut slope, which serves to vent production gas pressure and keep it from penetrating and damaging the rock that will form the final cut face. In cushion blasting, the row of control blast holes is detonated last to trim off the rock outside the cut slope. The cushion blasting technique is most commonly used in weaker rock conditions or wherever the thickness of rock to be excavated is less than 15 feet. Controlled blasting is routinely used for rock cuts that are 0.75H:1V or steeper. The limiting factor is the inability to maintain proper blast hole alignments on flatter slopes.

- Rock Removal - One method to mitigate an unstable rock slope is to remove the potentially unstable rock by hand scaling, blast scaling, or excavation equipment techniques. In the construction of new rock cuts, rock scaling is generally required and treated as incidental to the payment for the type of excavation performed.
- Screening and Barrier Systems – Draped mesh system (slope screening) applies limited normal force against the rock face, and primarily serves to control the descent of falling rocks into the roadside collection area. Barrier systems can range from concrete or gabion wall barriers to proprietary systems, such as the Brugg Barrier Fences.
- Reinforcement - Structural reinforcement can be provided by rock bolts, dowels, and cable lashing. Tensioned rock bolts are used to increase the normal stress along the discontinuity where sliding is possible, thus increasing the shear strength of the discontinuity. They may also be used to anchor potentially unstable rock blocks in place. Dowels are untensioned rock bolts or shear pins used to resist lateral movement of rock blocks by their lateral capacity. Cable lashing uses tensioned cable(s) to increase the normal force against the face of an isolated block to increase sliding resistance.
- Drainage - Dewatering to reduce groundwater pressures acting within the rock slope improves slope stability. Reduced groundwater pressure within a discontinuity increases the shear strength, while lowering the groundwater height within tension cracks reduces the driving force on a rock block. Proper drainage of rock slopes could be achieved by installing drain holes (weep holes, horizontal drains) or vertical relief wells. Various measures, such as construction of surface drains and ditches minimize water infiltration and therefore prevent build up of groundwater pressures.
- Erosion Protection - Soils, decomposed rocks, highly fractured rocks, and certain types of rocks are susceptible to erosion or degradation. When hard rock, resistant to erosion, is underlain by an erodible or degradable layer, loss of support for the overlying rock may develop over time. This may create an unstable condition.

Stopping this process can be accomplished by applying shotcrete to the surface of the less resistant zones. Weep holes are installed to prevent buildup of groundwater pressures behind the shotcrete. To improve the performance of shotcrete, wire mesh or steel fibers are routinely used to reinforce the shotcrete.

- Buttresses - When an overhanging rock is large and it is impractical to remove or reinforce it, buttresses can be used to support the overhanging rock and increase its stability. Buttresses serve two functions: (1) protect or retain underlying erodible material, and (2) support the overhang.

6.3.4 Rockfall Control Design

In many rock slopes, the potential for rockfall remains even after mitigation measures are in place. It may be impractical to stabilize all potentially unstable rocks. In these situations, the likelihood of rocks reaching the road should be evaluated and appropriate control or protection measures should be recommended. The consequences and probabilities of falling rocks reaching the road or facilities should be weighed against the cost of installing control measures. Rockfall mitigation measures generally fall into two major categories: (1) measures to prevent rockfalls (scaling, rock bolts, dowels, cable lashing, etc.), and (2) measures to control the manner in which rocks fall or to absorb energies and restrict falling rocks into roads and facilities (slope mesh, fallout areas, barriers, catch fences, etc.).

Fallout area or ditch design may be performed with the aid of the detailed design charts included in Pierson, et al., “Rockfall Catchment Area Design Guide” (2001). If the slope is too complex to allow direct use of design charts, actual rock rolling tests or rockfall simulation analyses should be performed. In most cases, rolling rocks is not practical or possible, and computer simulation is the preferred method. The CRSP (Colorado Rockfall Simulation Program) program (Colorado DOT, et al., 2000), is widely used for this purpose. The computer program RocFall available from Rocscience Inc. (See Section 6.2.4 above) is another program with some additional capabilities. These programs may be used to aid in the design of fallout areas and the capacity and placement of barriers.

Scaling, the removal of loose rock from the cut slope face, is routinely used to provide an immediate reduction in the rockfall potential; however, it is considered a temporary measure. Reinforcement or external support methods including, shotcrete, dowels, rock bolts, rock anchors, cable lashing, or concrete buttresses, can provide longer-term protection, as can various measures that intercept and control rockfalls, such as fallout areas (ditches), draped mesh, catch fence, or rockfall barrier systems.

7. LANDSLIDES

Two geotechnical references for landslide investigations for transportation projects are (1) TRB Special Report 247, “Landslides: Investigation and Mitigation,” and (2) FHWA, “Advanced Course on Slope Stability”. There are many technical papers regarding numerous advancements in the state-of-the-art analyses for landslides, assisting Geotechnical

Engineers to continually update their knowledge. In critical landslide mitigation applications, it may be advisable to retain a landslide expert to provide guidance to the Department or to perform the investigation and studies.

Landslides can be improperly diagnosed because of inadequate geologic reconnaissance/interpretation and poorly conceived exploration/instrumentation programs. Responding to and investigating a landslide is likened to a forensic investigation. The Geotechnical Engineer is searching for clues and evidence, and needs to resolve all apparent conflicts and contradictions in the perceived causative explanation/model.

Mitigation plans to stop small slides can be made through a combination of precedence, experience and judgment. An example is constructing a rock inlay to replace small slumps. More complicated and/or larger landslides generally require an extensive exploration/instrumentation program, along with expert geology and geotechnical engineering. Common mitigation measures are summarized in Figure 11-2.

8. GEOTECHNICAL EARTHQUAKE ENGINEERING DESIGN

Earthquake engineering is a multidisciplinary design process involving the fields of geology, seismology, geotechnical engineering and structural engineering. Field mapping, aerial photograph interpretation, geophysical testing and other investigative procedures to delineate faults and fault zones are performed. Fault data is used to develop ground motion parameters, typically bedrock motions, at the ground surface (commonly referred to as the outcropping rock motion). This information could include maximum acceleration, maximum velocity, and duration of shaking. The motions could also be presented in the form of digitalized acceleration-time records of an earthquake. These first two tasks can be time-consuming and expensive to perform for every project. Accordingly, site-specific geologic and seismic hazard evaluations are typically only performed for critical structures. For noncritical structures, ground motion parameters are usually obtained from existing regional studies and available literature.

Geotechnical Engineers evaluate various potential ground shaking hazards involving soil and rock, including:

- Amplification or attenuation of bedrock motion through overlying soil deposits
- Liquefaction which could cause loss of bearing pressure resistance, lateral spreading and ground settlement of loose, saturated, cohesionless soil deposits
- Causing Increased lateral earth pressures on retaining structures
- Causing landslides, rockfalls, and slope and embankment instability
- Causing fault rupture

Bridge Engineers are primarily interested in the lateral forces applied to structural facilities, including bridges, viaducts, buildings, and retaining walls. Geotechnical Engineers provide soil and ground response parameters to the Bridge Engineer for calculation of the shear forces acting on the structures as a result of the earthquake shaking and other possible secondary loading effects on structures, including liquefaction-induced lateral spread and settlement.

Geotechnical earthquake engineering has developed significantly since the 1970s, and although research continues in this discipline, generally accepted design procedures have been established for many of the potential hazards. However, as with most areas of geotechnical practice, the Geotechnical Engineer needs to keep abreast of current research to maintain competence with general standards of practice.

The Department follows AASHTO guidelines for seismic design of transportation facilities. The current AASHTO guidelines are outlined in Standard Specifications for Highway Bridges. Article 3.21 of AASHTO (2002), Division 1, states that seismic design must consider the following items: (1) the relationship of the site to active faults, (2) the seismic response of the soils at the site, and (3) the dynamic response characteristics of the structure. For bridges and roadway structures, the Geotechnical Engineer is responsible for analyzing items (1) and (2), and providing the results to the Bridge Engineer who analyzes item (3). For cuts and embankments, the Geotechnical Engineer is responsible for analyzing all three items.

8.1 Seismicity

The western portion of Nevada, known as the Nevada Seismic Zone, has experienced large earthquakes in historic times, and is considered one of the more seismically active areas in the United States.

The relationship of the site to active faults is represented using peak bedrock acceleration maps. For noncritical structures, the acceleration coefficient (A) is obtained from Article 3.2 of Division IA of AASHTO, 2002. The maps of horizontal acceleration in rock, A, are based on 90 percent probability of not being exceeded in 50 years. This corresponds to an approximate 475-year return period. Article 11.3.3.8 of Department Bridge Manual (1991) specifies that the minimum expected bedrock acceleration is 0.15g.

For very large or critical structures, a site-specific seismic hazard evaluation can be performed. These studies are performed on a probabilistic or deterministic basis. A probabilistic evaluation estimates the level of ground acceleration for a given return period for all potential seismic sources. A deterministic evaluation provides an estimate of the maximum ground acceleration that would be caused by each fault source or source zone. The individual fault source or source zone that results in the largest ground acceleration at the site is commonly referred to as the Maximum Credible Earthquake or MCE.

According to section 3.4 of Division 1A of AASHTO, 2002, a bridge is determined to be in one of four Seismic Performance Categories (SPC), A through D, based on the Acceleration Coefficient (A) and the Importance Classification (IC). Refer to special seismic design requirements for the foundations and abutments of bridges in SPC B, C, and D. The Geotechnical Engineer may be required to perform investigations to identify potential hazards and obtain seismic design information related to: liquefaction, slope instability, fill settlement, and increased lateral earth pressure.

8.2 Seismic Response of Soils

For the design of bridges according to Section 3.5.1 of Division 1A of AASHTO, 2002, the seismic response of the soils is expressed by the Site Coefficient (S), which is in turn determined by the Soil Profile Type. The Geotechnical Engineer performs subsurface explorations, classifies the subsurface materials, and determines the Soil Profile Type. A site-specific response may be developed by using various computer programs (SHAKE, TARA, FLAC, etc.) depending on the complexity and importance of the structure. However, site-specific response analyses using computer programs are typically not performed, except for special cases.

8.3 Dynamic Response Characteristics of Structures

Seismic lateral forces acting on a structure are influenced by the seismic response of the soils at the site and the fundamental period of the structure. Typically, elastic seismic coefficients, as defined in Division 1A of the 1998 Commentary in AASHTO, 2002, are used to define the earthquake load to be used in the elastic analysis for seismic effects. Article 3.6 of Division 1A of AASHTO, 2002, states an alternate method that can be used is with a 5% damped, site-specific, response spectrum developed by a qualified professional. The Geotechnical Engineer may develop the site-response spectrum by using the computer program SHAKE. Earthquake time histories to be used in SHAKE should be selected to closely match the estimated ground motions for the site. The 1997 FHWA, Geotechnical Engineering, Circular No. 3, provides a summary of seismic design procedures, including selection of representative earthquake time histories.

8.4 Liquefaction

The selection of ground motion parameters for the lateral force design procedures discussed in Division 1A of AASHTO, 2002, assumes that the soil overlying bedrock is not liquefiable. If loose, saturated, cohesionless deposits are subjected to cyclic shear stresses (typically an earthquake, less commonly blasting or construction-induced vibrations), the tendency for the soil to densify will result in a temporary increase in pore water pressure. This in turn results in a decrease in effective stress and a weakening of the soil. Structures founded in liquefiable soil can lose bearing pressure resistance or skin friction, and can be subjected to increased lateral and vertical loads from lateral spreading and settlement of the liquefied deposit. Liquefaction has caused a number of bridge failures during past earthquakes. The recommended procedure to evaluate the liquefaction potential is based on the Standard Penetration Test blowcounts of soils. The liquefaction evaluation procedure is described in many standard references on geotechnical earthquake engineering, including Chapter 8 of FHWA, “Earthquake Engineering” (1997), Vol. I. An example of a liquefaction evaluation is in Example 5 of FHWA, “Earthquake Engineering” (1997), Vol. II.

8.5 Seismic Slope Stability

Earthquake shaking can result in failures of natural slopes and man-made embankments. The standard procedure for evaluating the stability of a nonliquefiable slope is the pseudostatic analysis, where a lateral force is applied to the center of gravity of a soil mass having a failure potential when performing a limit equilibrium analyses. The selection of shear strength in slope stability analyses involving seismic loadings should be based on short-term undrained shear strengths. The pseudostatic procedure does not provide an estimate of potential seismic deformations. In many instances, the stability of a slope during an earthquake may drop below a factor of safety of 1 for only a brief period of time during the transient shaking. In this case, a pseudostatic analysis would indicate an unacceptable factor of safety below 1, but the actual deformation of the slope or embankment would be minimal and the overall performance acceptable.

One method to estimate seismic deformations of nonliquefiable slopes is the Newmark Sliding Block Analysis. This method uses the yield acceleration of a slide mass and a seismic time history to estimate the permanent seismic deformation. This method, however, is not used on a routine basis. Refer to FHWA, “Earthquake Engineering,” (1997), and Kramer “Geotechnical Earthquake Engineering,” (1996), for more details on the deformation analyses.

The seismic slope stability and deformation analyses are not applicable to liquefiable materials. These analyses are based on the assumption that the shear strength of the soil remains relatively constant with deformation and strain. However, liquefiable soils in slopes can lose most of their shear strengths, develop into a flow failure condition, and displace considerable amounts. The mechanical/physical response of soil during flow failure is not conducive to engineering analysis at this time.

One method to evaluate the potential flow failure in sloping ground is to assign liquefied residual shear strengths to the soil layers with low factors of safety against liquefaction. If the limit equilibrium slope stability analyses give low factors of safety, then flow failure should be considered a possibility at the site. Ground improvement and/or project relocation are options to consider when this occurs.

8.6 Seismic Analysis of Retaining Structures

Earthquake shaking results in increased lateral earth pressures acting on retaining structures. Types of structures needing analyses may include bridge abutments, conventional cantilever retaining walls, Mechanically Stabilized Earth (MSE) walls, tieback walls, and soil nail walls. The needed analyses involve estimating the increase in lateral earth pressures exerted on the walls by earthquakes. The Mononobe-Okabe Method is generally used for walls free to yield about their bases. A modified Mononobe-Okabe is used for walls that are not free to rotate. Refer to Standard Specifications for Highway Bridges, Articles c6.4.3, c7.4.3, and c7.4.5 of AASHTO (2002), for the seismic requirements for abutments in SPC B, C, and D, respectively, in the 1998 Commentary of AASHTO, 2002, FHWA, “Earthquake

Engineering" (1997), Vol. I and Kramer, "Geotechnical Earthquake Engineering," (1996).

9. FOUNDATIONS

Foundations must have adequate capacity to support the design load combinations and satisfy the serviceability requirements established by the Bridge Engineer. Serviceability requirements establish allowable settlement and deflections. Foundation design is generally an iterative process between the Geotechnical and Bridge Engineer. These iterations mean that the Geotechnical Engineer may have to reevaluate the design many times. Therefore, it is important to document the assumptions made during the design process and the justification for design decisions.

Foundations are classified as shallow, deep, or hybrid. The most economical foundation type depends on types of subsurface soils and groundwater conditions, design loads, design scour elevations, serviceability requirements, and construction sequence. Shallow foundations consist of spread footings or mats. Deep foundations include driven piles, micropiles, and drilled shafts. Hybrid foundations are a combination of shallow and deep foundations.

9.1 Service Load vs. Load and Resistance Factor Design

Foundations are designed based on Service Load Design (SLD) or Load and Resistance Factor Design (LRFD) using the procedures outlined in the AASHTO references. The Bridge Engineer typically determines the design method. When using SLD, the Geotechnical Engineer uses actual or unfactored loads for the design provided by the Bridge Engineer. Recommended safety factors, load and resistance factors, and load combinations are outlined in AASHTO references.

9.2 Foundation Feasibility

The Geotechnical Engineer should consider several items when evaluating potential foundation systems. Cost is always a consideration. Typically, cost does not impact the choice between shallow or deep foundation systems, because in most cases, the site is either suitable for shallow foundations or it is not. Cost becomes more of a factor when comparing different types of deep foundations.

Depth to suitable bearing material is the first factor to consider when choosing between shallow or deep foundations. If suitable material is at a reasonable depth, the Geotechnical Engineer should consider potential impacts of scour, groundwater, and construction sequence on shallow foundations. Scour may preclude the use of a shallow foundation if the scour level is lower than the suitable bearing material. Groundwater impacts bearing capacity and constructability. Construction sequence could impact the bearing capacity and settlements of shallow foundations. The Geotechnical Engineer should consider local practice for founding structures when choosing a foundation system. Sometimes, experience precludes a foundation type for reasons that are not readily evident.

Deep foundations can be classified as driven or drilled systems. A drilled system is typically better suited to cases where significant penetration into hard or dense material is required either to develop uplift loads or to get below the design scour depth. Driven systems are generally better suited where subsurface conditions would make drilling problematic. Conditions that make drilling difficult include encountering boulders, uniform-graded gravel, voids, and artesian groundwater. The availability of raw materials, cost of labor, and regional geology generally favor one type of deep foundation over others. It is often useful to compare different deep foundations on the basis of cost per ton of design load or per foot of installed length. There is generally enough information in recent Department bid tabulations to compare foundations on this basis.

The type of foundation for all supports of a structure needs to be compatible with each other in order to minimize differential settlement between supports. Other considerations include maintaining the simplicity of structural analysis under dynamic loads, and potential structure widening in the future. If there appears to be a sound reason to use a combination of different foundation types, the Geotechnical Engineer should seek the input of the Bridge Engineer. Situations are evaluated on a case-by-case basis by the Bridge Engineer.

9.3 Spread Footings

Geotechnical Engineering, Circular No. 6, “Shallow Foundations,” 2002, References FHWA RD-89-185, FHWA HI-88-009, AASHTO Standard Specifications, and NHI course manual for Shallow Foundations, NHI No. 132037 (Module 7) present design methods and commentary on bearing capacity and settlement of shallow foundations. The design of shallow foundations on soils is generally controlled by allowable settlement criteria, not by shear failure. The Geotechnical Engineer should focus efforts on settlement evaluation, rather than bearing capacity for most sites. The design scour depth could make excavating to construct shallow foundations unfeasible. Groundwater depth could influence constructability as well. Groundwater flow into footing excavation can loosen potential bearing material and make forming and pouring a footing difficult. Spread footings for any structure must have a minimum of 2 feet embedment depth from bottom of the footing to the finished grade.

9.4 Foundations on Rock

AASHTO Standard Specifications, FHWA Circular No. 6, and NHI Course No. 132037 (Module 7), provide a summary of methods to calculate bearing capacity of competent as well as jointed rock. More in depth discussions are available in the References (Wylie and Canadian). The Geotechnical Engineer should determine the influence that dominant joint sets in the rock have on foundation performance.

9.5 Deep Foundations

Before planning subsurface explorations, the Geotechnical Engineer should perform preliminary calculations based on available information to estimate the depth of the required foundation elements. Minimum exploration depths are outlined in AASHTO references. In no case should deep foundation tip elevations be deeper than the subsurface explorations.

9.5.1 Axial Capacity

Both compression and uplift axial capacities should be calculated for deep foundations. References FHWA-HI-97-014 and AASHTO Standard Specifications outline design procedures for driven piles. References FHWA-IF-99-025 and AASHTO Standard Specifications for Highway Bridges describe the design procedures for drilled shafts. The Geotechnical Engineer should consider subsurface conditions and construction sequence to evaluate the potential for downdrag loads on piles. In general, downdrag is a concern whenever the ground moves downward 0.1 to 0.25 inches relative to the pile. Downdrag can occur due to settlement, shrink/swell, or liquefaction.

9.5.2 Lateral Capacity

References FHWA-HI-97-014 and FHWA-IP-84-11 are helpful references for lateral load design procedures. Generally, the Geotechnical Engineer provides soil parameters to the Bridge Engineer so that LPILE program or other approved programs can be used for the lateral analysis of the foundation. The Geotechnical Engineer should consider construction methods and sequence when developing LPILE parameters. Assumed conditions should be provided to the Bridge Engineer with the parameters.

9.5.3 Seismic Analyses

Seismic analysis should be performed to evaluate both axial and lateral loading conditions during and after a seismic event. The greatest influence on axial capacity is the temporary loss of skin friction during soil liquefaction and the increased downdrag force from post-liquefaction settlement. Liquefaction can also cause lateral spreading of sloping ground, which in turn increases the lateral forces acting on the pile and reduces available soil resistance to overlying inertial forces. The seismic evaluation and design of soil-pile interaction is an area of active research.

9.5.4 Liquefaction Potential and Mitigation

Liquefaction potential should be evaluated per the references cited in this Chapter. Generally, if liquefiable soils are present, some means of mitigation is required to protect structure foundations. Typically, the Department policy does not require liquefaction mitigation for approach embankments. The Geotechnical Engineer should clearly explain to the design team the need of any ground improvement mitigations and any consequences if the mitigations are not implemented. For example, if liquefaction mitigation is proposed for an area around deep foundations but not under approach fills; complete reconstruction of approach fills may be required following a seismic event.

9.5.5 Scour Considerations

Typically, the Hydraulics Engineer determines the depth of scour with input from the Geotechnical Engineer. The following items are typically required to complete the scour analyses: (1) boring logs, (2) grain size analyses to characterize river bed materials, and (3) a description of the geomorphology of the site (i.e., floodplain stream, crossing of a delta). If scour has a major impact on the foundation design, the Hydraulics Engineer should be notified to consider designing some type of scour protection or revetment around deep foundations. References FHWA, Hydraulic Engineering Circular (HEC), HEC 18, HEC 20, and HEC 23, are helpful references. In some cases, it is more economical to design deep foundations for the scour case.

9.5.6 Design Phase Load Testing

The decision of whether or not to conduct foundation load tests during the design phase is based on economics and the degree of uncertainty acceptable for the design. Design phase load tests should also be considered whenever loads are high and there is no redundancy in the foundation system. For medium to large projects, the cost of conducting load tests during the design phase may be offset by savings in construction. Site-specific load tests allow the Geotechnical Engineer to use lower factors of safety for design, which results in lower construction costs. There are several types of load tests, including static, dynamic, Statnamic, and Osterberg load cell tests.

9.6 Driven Piles

References FHWA-HI-97-014 and AASHTO Standard Specifications for Highway Bridges outline design procedures for driven piles. Dynamic pile driving analysis using a Pile Driving Analyzer (PDA) can be performed during design and/or construction phase. The objective of using the PDA during the design phase is to develop site-specific engineering properties in order to develop a more cost effective design with less conservative assumptions and lower factors of safety. However, the use of the PDA testing in the design phase can be more costly than during construction phase because the contractor must mobilize pile driving equipment for a relatively small amount of work. During construction, the PDA is used to confirm design assumptions.

Soil setup occurs when piles are driven into saturated clays and loose to medium dense sands. Positive pore water pressures generated during pile installation result in low capacities at the end of driving. As excess pore water pressures dissipate, effective stress and pile capacity increase. PDA testing can be used to quantify setup if piles are instrumented during initial driving and on restrike. The Geotechnical Engineer can use wave equation analyses to develop driving criteria for end of driving conditions. Using criteria for end of driving conditions eliminates the need for restrikes during production.

9.7 Drilled Shafts

References "Drilled Shafts: Construction Procedures and Design Methods," FHWA-IF-99-025 and AASHTO Standard Specifications for Highway Bridges outline design procedures for drilled shafts. FHWA provides more complete narrative and construction considerations, but design methodologies are similar. Design of drilled shafts varies from driven piles because the construction processes are different. Drilling results in lower horizontal effective stress than displacement type piles. In addition, drilling fluids and incomplete base cleaning contribute to lower unit skin friction and end bearing than driven piles. The primary advantage drilled shafts have over driven piles is their large size and resultant large capacity.

Load testing can be performed during design and/or construction of drilled shafts. However, this is seldom done for Department projects. The objective of testing during the design phase is to develop a more cost effective design by using lower factors of safety. There are several methods for load testing drilled shafts. In general, static load tests are not feasible for drilled shafts due to the need of a high reaction force frame system.

9.8 Auger Cast Piles (ACP)

Design of Auger Cast Piles (ACP) is similar to drilled shafts. ACP typically has higher unit skin friction than drilled shafts. This is due to the fact that grout is injected under pressure for ACP, and sidewall asperities have a larger influence for ACP than drilled shafts because ACP has smaller diameters than drilled shafts. Local experience or load testing is generally required to choose parameters for design.

9.9 Micropiles

Micropiles are small diameter drilled piles. Reference FHWA-SA-97-070 summarizes the design methodology. Micropiles are typically used where site restrictions prohibit the use of large foundation construction equipment. Micropiles are installed by specialized drills typically used to install tiebacks. Due to their size, Micropiles have lower capacity than large deep foundation elements. Production rates are typically lower for Micropile rigs than for other types of deep foundation construction. These factors contribute to the relatively high cost of Micropiles.

10. RETAINING WALL SELECTION AND DESIGN

There are a variety of wall types and a number of factors that control wall type selection. The Geotechnical Engineer should have an understanding of the applications of each wall type, exploration and design requirements, construction methods, and relative costs. The FHWA, Geotechnical Engineering, Circular No. 2, "Earth Retaining Systems," (SA-96-038), provides an overview of wall types with general information pertaining to selection criteria, and design and analysis procedures.

Wall types can be classified into fill wall and cut wall applications. Examples of fill walls include standard cantilever walls, modular gravity walls (gabions, bin walls, and crib walls), and Mechanically Stabilized Earth (MSE) walls. Cut walls include soil nail walls, cantilever soldier pile walls, and ground anchored walls (other than nail walls). Some wall types require a unique design for both internal and external stability. Other walls have standardized or proprietary designs for internal stability with external stability analyzed by the Geotechnical Engineer. Geotechnical Engineers should be able to develop their own designs as well as evaluate and review standardized and proprietary wall designs.

Factor of safety recommendations for Service Load Design (SLD) of gravity and semi-gravity walls (standard cantilever, modular gravity, and MSE) are provided in Figure 11-3. Refer to AASHTO, “Standard Specifications for Highway Bridges,” 17th ed., for Load Factor Design (LFD) and other loading criteria. Performance factors for Load Resistance Factor Design (LRFD) are discussed in the AASHTO, LRFD Bridge Design Specifications, 2nd ed. Section 12.2 of the Department Bridge Manual (1991) states that seismic design of retaining walls must use the LFD Method, and that all other design cases should use SLD. In general, AASHTO recommended factors of safety should be used for wall designs. However, engineering judgment may allow using lower factors of safety when wall loadings are well understood, and wall costs are high. One example is a landslide stabilization wall where using AASHTO recommended factors of safety can be quite expensive. Depending on the level of understanding of slide conditions, this may merit the selection of a lower safety factor.

10.1 Standard Cantilever Walls

A concrete cantilever wall is constructed of cast-in-place reinforced concrete, consisting of a vertical stem and footing slab base connected to form the shape of an inverted T. After curing, the back of the wall is backfilled with free-draining, granular backfill. The backfill weight on the heel of the footing slab enables the structure to function as a gravity wall. General texts, such as NAVFAC, Canadian Geotechnical Manual, Terzaghi & Peck, and Peck, Hanson, & Thornburn, and AASHTO (17th ed.) provide guidelines and design charts for analysis of static conditions for standard walls. Seismic induced lateral earth pressures should be determined using the Mononobe-Okobe analysis. The FHWA, Geotechnical Engineering, Circular No. 3, provides guidance on this analysis approach. AASHTO (17th ed.) recommends using a seismic coefficient equal to one-half the acceleration coefficient (k_h = 0.5A) in the Mononobe-Okobe analysis for unrestrained walls. For nonyielding walls, the seismic lateral earth pressure can be approximated by using a seismic coefficient of (k_h = 1.5A) in the analysis. It should be noted that the procedure described in this reference provides the combined static and seismic loadings. It is recommended this combined loading be applied at the midheight of the wall. Section 12.2 of the Department Bridge Manual (1991) states that seismic design of retaining walls must be used in the Load Factor Design (LFD). The LFD method uses different load factors for the static and seismic forces. The Geotechnical Engineer should work closely with the Bridge Engineer to determine the appropriate earth pressure loadings for seismic conditions so that the design is performed in accordance with current Department procedures.

The Department Standard Plans include drawings of many different cantilever walls for various geometric and ground conditions. These drawings include information such as assumed surcharge forces, back slope angles, foundation capacities, seismic accelerations, and Backfill soil properties. The Geotechnical Engineer is responsible for providing all soil parameters needed to design walls that are not covered in the Department Standard Plans, as well as determining the suitability of use of the walls covered in the Department Standard Plans for the project. The Bridge Engineer performs all stability analyses.

10.2 Modular Gravity Walls

Modular gravity walls use interlocking soil or rock-filled concrete, timber, or steel modules that resist earth pressures by acting as a gravity wall. Examples include gabion walls, bin walls, concrete block walls, and crib walls. These wall types commonly use proprietary materials.

Earth pressures for modular gravity walls are determined using the same procedures as for standard cantilever walls. Because many of these wall types are proprietary, it is recommended that Geotechnical Engineers use manufacturers' literature for design, and check them with generic methods. Manufacturers include Maccaferri and Hilfiker (gabions), Criblock (crib walls), Contech and Double-Wal (bin walls). Seismic design of these walls is base on the Mononobe-Okobe analysis. These walls are relatively easy to construct, and have a relatively low cost. Modular gravity walls are likely to deform more than concrete cantilever walls, so the tolerable settlements of upslope structures should be considered. Department procedures recommend against use of rockery walls due to a lack of design guidance from FHWA and AASHTO. They can be used only under special circumstances (i.e., aesthetics) with approval from the Department Bridge Engineer.

10.3 Mechanically Stabilized Earth (MSE) Walls

Mechanically Stabilized Earth (MSE) walls consist of tensile reinforcements in soil backfill, with facing elements that are vertical or near vertical. The reinforced mass functions as a gravity wall. The Department has specific procedures and requirements for the design of MSE walls. The policy memorandum (dated September 27, 2002) is presented in Figure 11-1. AASHTO (17th ed.) describes the state of the practice design procedure for MSE walls. A recent, comprehensive reference on MSE walls is FHWA's manual on "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Design and Construction Guidelines" (FHWA-NHI-00-043).

MSE walls are commonly used for medium to large wall and grade separation projects because they are often less expensive than concrete cantilever walls. Most MSE wall applications use proprietary systems where the internal design is performed by the wall vendor. The Department Research Division maintains names of approved MSE wall vendors and systems in the Qualified Products List (QPL).

The Geotechnical Engineer is responsible for performing the external stability analysis in the design phase of the project, and providing the required reinforcement lengths to the Bridge Division to be included in the Construction Plans. The vendor performs the internal and external stability analyses and evaluates the adequacy of reinforcement lengths shown in the Construction Plans. The vendor submits calculations and shop drawings showing the actual reinforcement lengths to be used on the project based on the longer of needed reinforcement lengths for external or internal stability analyses. The Geotechnical Engineer reviews the submitted calculations and shop drawings for approval.

10.4 Soil Nail Walls

Soil nails are closely spaced, passive reinforcements used to strengthen existing ground. They consist of steel bars grouted into the soil connected to a temporary or permanent shotcrete facing. They are constructed in a top down manner and are used to support an excavation face.

Soil nail walls are an economical alternative to ground anchored walls when installed in the appropriate soil conditions. The following are some of the items that should be taken into account when considering use of soil nail walls:

- A 6-foot high excavation face must stand unsupported for at least 48 hours. This requires some cohesion or cementation of the subject soils.
- Drilling into cohesionless materials requires the use of temporary casing during drilling. This has a significant impact on construction costs.
- Excavations in soft clays are unsuitable for soil nails due to the low frictional resistance of the materials.
- The excavation face should be dry or dewatered to permit stability of the vertical excavation.
- Proper corrosion protection is extremely important for long-term performance of a soil nail wall. The Department uses a double protection system for all soil nail walls. Typically NDOT does not use epoxy coating as a corrosion protection measure.
- Soil nail walls are not recommended where the ground could deform, such as landslides.

The FHWA, Manual for Design and Construction Monitoring of Soil Nail Walls (FHWA-SA-96-069) is recommended for design of soil nail walls. Analysis programs such as GoldNail (available from FHWA) and Snailz (available from http://www.dot.ca.gov/hq/esc/geotech) should be used for the design. In addition to static and seismic conditions of the completed wall configuration, the stability for each stage in the construction sequence of a soil nail wall should be evaluated. The controlling condition is often a construction case. The FHWA Soil Nailing Field Inspectors Manual (FHWA-SA-93-068) provides practical information to understand construction procedures.

10.5 Cantilevered Soldier Pile and Sheet Pile Walls

These walls consist of vertical elements that derive lateral resistance from embedment into soil below the exposed face, and support the retained soil with facing elements or the piles themselves. These walls are often used for temporary excavations to limit upslope deformations during construction. Permanent applications include short walls that are part of a taller wall section utilizing ground anchors, and where an unsupported excavation is not desired. Wall heights typically are limited to a maximum of 15 feet unless they are also supported by ground anchors. These walls do not work well when embedded in deep soft soils, where the passive resistance on the front of the wall is low. AASHTO Standard Specifications for Highway Bridges contains the relevant design charts to determine the wall loadings. Bridge Engineers determine the appropriate sizes of the structural elements based on the applied loads.

10.6 Ground Anchor Wall Systems

Ground anchored wall systems consist of ground anchors (cement-grouted, prestressed steel tendons installed in soil or rock) connected to wall elements consisting of soldier piles or concrete bearing pads. They are usually constructed in a top down manner and are most often used to support an excavation face. They can also be used for landslide stabilization.

Ground anchored walls are more expensive than most traditional walls due to the need of uncommon construction equipment and skills. However, they are well suited where deformations of adjacent structures are of concern. Ground anchored walls are applicable to a wider range of materials than soil nail walls. The following are some of the items that should be taken into account when considering use of ground-anchored walls:

- Underground easements should be obtained to protect the anchors throughout their functional life.
- Proper corrosion protection is essential to achieve the design life of the structure. The Department uses a double protection system for all ground anchored walls. Typically the Department does not use epoxy coating as a corrosion protection measure.
- The upper level of anchors should be located and oriented below the zone normally used for buried utilities and guardrail posts.
- Acceptance of ground anchors should be based on proof tests of each anchor. Sometimes preproduction tests and long-term monitoring may also be required.
- Anchors bonded in clays may have long-term creep problems. Anchors in clays should be creep tested.
- The contract documents should require the contractor to determine the anchor bond length necessary to resist the applied anchor force.

Refer to the FHWA, Geotechnical Engineering, Circular No. 4, “Ground Anchors and

Anchored Systems", (FHWA-IF-99-015), and the Post Tensioning Institute (PTI) publication, "Recommendations for Prestressed Rock and Soil Anchors", 1996. In addition to static and seismic conditions of the completed wall configuration, the stability for each stage in the construction sequence of a ground anchored wall should be evaluated. The controlling condition is often a construction case.

11. DEWATERING

Design of dewatering systems is typically the Contractor's responsibility. The Geotechnical Engineer should have a basic understanding of the advantages and disadvantages of different dewatering systems. The Geotechnical Engineer should be able to perform calculations to establish requirements of dewatering systems. The references (Powers, and COE) provide a background and narrative for dewatering system design.

12. FIGURES

11-1: Policies and Procedures Memorandum

STATE OF NEVADA

DEPARTMENT OF TRANSPORTATION

M E M O R A N D U M

September 27, 2000

To: See List Below

From: Parviz Noori, Assistant Chief Materials Engineer – Geotechnical PN

Subject: Policies and Procedures Memorandum No. C028-2000-01
MSE Walls
NDOT's and Wall Supplier's Responsibilities
Contract Plans and Shop Drawings
Standard Specifications and Special Provisions

The purpose of this memo is to help clarify responsibilities of the contractor and each NDOT Division concerning cost, analysis, details, and required information in regard to contract documents.

I. Responsibilities:

A. The Roadway Design Division will provide (to the Geotechnical Section and the Bridge Division) alignment and profile of the wall, and cross sections at 8 meter intervals for the length of the wall. Cross sections will include elevations of top of the wall, existing (original) ground intersecting the plane of the wall, proposed ground at the exposed face at the base of the wall, bottom of the slope in front of the base (slope supporting the wall) of the wall (if applicable), and top of the slope above (slope being retained by the wall) the wall (if applicable). In addition, exact inclination angles of slopes above (retained slope) and/or below (supporting the wall) the wall (if applicable) will be provided by the Roadway Design Division. Any ditch information behind the top of the wall should also be shown. This information may be provided in tabular form or by placing these elevations on the aforementioned cross sections. Topographical information for the existing ground condition and completed condition will be provided to a distance of at least three times the wall height in front and behind the wall.

B. NDOT is responsible for deep seated (global, rotational) external stability. The Geotechnical Section will conduct global stability analyses and provide design recommendations for wall stability.

C. NDOT is responsible for external stability. The Geotechnical Section will conduct external stability analyses with respect to sliding, overturning, and bearing pressure failures.

D. The Geotechnical Section will design the wall with respect to external stability. Publication No. FHWA-SA-96-071 (Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, Reprinted September 1998) and/or the latest edition of the AASHTO Standard Specifications for Highway Bridges will be used to design and determine the minimum reinforcement lengths of the wall. Typical information and details to be provided by the Geotechnical Section to the Bridge Division would include the following:

1. 300 mm wide and 150 mm thick unreinforced concrete leveling pad.
2. Exact embedment depth to top of the leveling pad for the entire length of the wall.
3. Minimum 1200 mm wide bench in front of walls placed (located) on top of slopes.
4. No steeper than 1V:2H slopes in front of or on top of walls.
5. Strength properties of soils supporting the wall (foundation soils), MSE backfill, and retained backfill.
6. Magnitude of anticipated total and differential settlement.
7. Recommended waiting period, prior to construction of barrier rails, copings, concrete anchor slabs, and roadway surface.
8. Minimum required reinforcement lengths for the entire length of the wall.
9. Surcharges.

E. The wall supplier, based on information provided in contract documents, will check the external stability with respect to sliding, overturning, and bearing pressure to confirm NDOT's proposed minimum reinforcement lengths. NDOT will determine the need for any changes indicated by the contractor's external stability analysis. All costs associated with changes to the wall due to external stability shall be the responsibility of NDOT.

F. The wall supplier is responsible for internal stability. All costs associated with modifications to the overall wall geometry due to internal stability design shall be responsibility of the wall supplier.

G. The Bridge Division will estimate the quantities and prepare the contract plans.

H. It is NDOT policy not to allow placement of spread footings on embankment retained by MSE walls.

I. Piles Within MSE Walls: Piles must be placed prior to the construction of the wall. Downdrag forces need to be analyzed and friction protection material such as "Yellowjacket" sleeves or an approved equal if needed must be specified in the contract documents.

J. The soil reinforcement length and/or layout must be modified when piles are located within the wall. The wall supplier must design the reinforcement for pile locations and submit all calculations for review and approval. Bar mat systems may be cut provided at least two longitudinal bars remain connected to a transverse bar. Strap systems may be skewed up to 20 degrees from a line perpendicular to the wall face. Otherwise, bridging systems must be used.

II. Contract Plans:

The Bridge Division will prepare the contract plans. Typical details to be included in the contract plans are the following:

A. General Notes:

1. Design Specifications: AASHTO Standard Specifications for Highway Bridges, (date) with interim revisions through (date), and/or Publication No. FHWA-SA-96-071 (Reprinted September 1998).
2. Construction Specifications: NDOT Standard Specifications for Road and Bridge Construction (date) except as noted below and in the Special Provisions for the project.
3. Loading: Live load surcharge pressure equal to 610 mm of earth (When applicable).
    - Seismic acceleration = (peak ground acceleration).
    - Use one-half of peak ground acceleration for design.
    - Additional lateral load from bridge piles = (value) (When applicable).
4. Concrete: All concrete shall be class AA (or Class A) Modified (Major) concrete with F_c' = ____ MPa at 28 days.
5. Reinforcing Steel: All reinforcing steel shall be ASTM A615 grade 410 or A706.
6. Size of Wall Panel: Area of the MSE wall panel face shall not exceed (normally 2.8) square meters.
7. Soil Properties: MSE wall is designed based on the following soil properties:
    - Mechanically stabilized earth fill:
    - Minimum internal angle of friction = __, minimum cohesion = __,
    - and minimum unit weight = _______.
    - Fills not meeting the above specified soil parameters should not be approved.
    - Foundation soils:
    - Internal angle of friction = ______, cohesion = _____, and unit weight = _______.
    - Random fills (retained fills):
    - Internal angle of friction = ______, cohesion = _____, and unit weight = _______.
8. External Design Parameters:
    - Sliding friction factor = _________.
    - Allowable bearing pressure of foundation soils beneath wall = ______.
    - Ultimate bearing pressure of foundation soils beneath wall = ______.
9. Method of design: Specify if "SIMPLIFIED METHOD" or "MEYERHOF METHOD" is used for the design.

B. Plan View:

1. Alignment of the wall relative to the main or major alignment with necessary stations and offsets to exposed face of wall.
2. Alignment (if applicable) of top of the slope on top of the wall (every 8 meters and at every break point) with necessary stations and offsets to exposed face of wall (this information may be provided in a table).
3. Alignment (if applicable) of top of the slope in front of the wall (every 8 meters and at every break point) with necessary stations and offsets to exposed face of wall (this information may be provided in a table).
4. Alignment (if applicable) of bottom of the slope in front of the wall (every 8 meters and at every break point) with necessary stations and offsets to exposed face of wall (this information may be provided in a table).

C. Elevation View:

1. Elevation at the bottom of the leveling pad, the bottom of the pad should be level (and stepped if necessary).
2. Line showing the finished grade at exposed face of the wall (finished grade must not be stepped).
3. Elevation at top of the wall every 8 meters and at every break point (this information may be provided in a table).

Elevation at top of slope on top of the wall every 8 meters and at every break point (this information may be provided in a table).

4. Elevation at top of slope in front of the wall (elevation of the bench in front of the wall every 8 meters and at every break point, this information may be provided in a table).
5. Elevation at bottom of slope in front of the wall every 8 meters and at every break point (this information may be provided in a table).

D. Cross sections:

1. Minimum soil reinforcement lengths for the entire length of the wall (this information may be provided in a table).
2. Width and thickness of leveling pad.
3. Minimum soil cover on top of the leveling pad.
4. Width of the bench in front of the wall.
5. Inclination of the slope in front of the wall (this information may be provided in a table).
6. Inclination of the slope on top of the wall (this information may be provided in a table).
7. Barrier rail, coping, and concrete anchor slab on top of the wall.
8. Existing (original) ground.
9. Limits of excavation and granular backfill.
10. Limit of over-excavation (if applicable).
11. Limit of MSE Backfill (reinforced soil mass).
12. Limit of Borrow and/or Select Borrow (if applicable).

13. Limit of Drain Rock (if applicable).
14. Limit of Geotextile (if applicable).

E. Quantities:

1. Area of the wall face.
2. Volume of MSE Backfill.
3. Volume of any Structural Excavation including any over-excavation.
4. Volume of any Granular Backfill, Borrow, Select Borrow, or Drain Rock.
5. Area of any Geotextile.
6. Volume of concrete and reinforcing steel for barrier rails and concrete anchor slabs.
7. No direct payment for concrete leveling pad and reinforced concrete coping.

F. Special Details:

1. Aesthetic treatments.
2. Coping details.
3. Connection of MSE walls to wingwalls of bridges or other retaining walls (if applicable).
4. Details for drainage or other obstructions (if applicable).

III. Special Provisions:

The Geotechnical Section will be responsible for including the following in the Special Provisions:

1. Required soil specifications such as soundness and electrochemical properties for the backfill materials.
2. Required methods of testing.
3. Any required over excavation.
4. Statements that NDOT is responsible for external stability design and Wall Supplier is responsible for checking external stability design.
5. Statements that NDOT is responsible for any cost due to required changes for the external stability design.
6. Statements that Wall Supplier (Contractor) is responsible for cost increases due to internal stability requirements.
7. Statements that Wall Supplier (Contractor) must submit 7 copies of the shop drawings and calculations.
8. Statements that shop drawings and calculations must be stamped and signed by an engineer who is a registered Professional Civil Engineer licensed to practice in the State of Nevada.
9. Statements that allow NDOT 28 working days for review and approval of shop drawings and calculations and no additional working days will be provided for submittals returned for corrections.

The Bridge division will be responsible for providing the List of approved Wall Suppliers.

IV. Shop Drawings and Calculations:

The Geotechnical Section will be responsible for checking and approving the following:

1. Calculations for external stability (at every 8 meters).

2. Calculations for required soil reinforcement lengths with respect to external stability.
3. Calculations for required soil reinforcement width, thickness, and lengths with respect to internal stability.
4. Soil reinforcement lengths shown on the drawings.
5. Applied bearing pressures shown on the drawings.
6. MSE Backfill materials specifications.
7. Construction methods and procedures shown on the drawings regarding geotechnical issues.
8. Corrosion protection specifications shown on the drawings.

For Consultant designed projects, the Consultant will perform the above review and approval. The Geotechnical Section will perform a cursory review after the Consultant has completed their review.

The Bridge Division will be responsible for checking and approving the following:

1. Wall alignment and elevations.
2. Panel design and details.
3. Coping details.
4. For Consultant designed projects, the Consultant will perform the above review and approval. The Bridge Division will perform a cursory review after the Consultant has completed their review.

V. Review & Approval Procedures for Shop Drawings and Calculations:

1. The Resident Engineer will send seven copies of the contractor's submittal to the Bridge Division.
2. The Bridge Division will send six copies of the submittal to the Geotechnical Section.
3. The Geotechnical Section will contact the Resident Engineer to see if it is acceptable to contact the Wall Supplier directly.
4. The Geotechnical Section will have twelve working days from the date of receiving the submittal to review the shop drawings and calculations.
5. Shop drawings will have two stamps, one each from the Bridge Division and Geotechnical Section. Design calculations will have one stamp from the Geotechnical Section. For Consultant design projects, shop drawings will have three stamps (one Consultant (approval), one Geotechnical (reviewed) and one Bridge (reviewed)) and calculations will have two stamps (one Consultant (approval) and one Geotechnical (reviewed)). Prior to stamping the shop drawings and calculations, the Geotechnical Section will coordinate with the Bridge Division on the joint review. If the shop drawings and/or calculations need to be returned for corrections, one memo will be written to the Resident Engineer by the Bridge Division incorporating all needed corrections.
6. The Geotechnical Section will send five stamped copies of the calculations and shop drawings with corrections (if any) to the Bridge Division.
7. The Bridge Division will respond to the Resident Engineer regarding approval of the shop drawings and calculations within sixteen working

days from the date of receiving the submittal from the Resident Engineer.

If you have any questions or comments regarding this matter, please call me at 888-7786.

PN:pn

11-2: List of Landslide Mitigation Methods

Slide Mitigation	Mitigation Options
Avoid problem	• Relocate facility • Completely or partially remove unstable materials • Install bridge
Reduce driving forces	• Change line or grade • Drain surface • Drain subsurface • Reduce weight
Increase resisting forces, apply external force	• Use buttress and counterweight fills; toe berms • Use structural systems • Install anchors
Increase internal strength	• Drain subsurface • Use reinforced backfill • Install in situ reinforcement • Use biotechnical stabilization • Treat chemically • Use electro-osmosis • Treat thermally
Note: Refer to TRB Special Report 247 (Table 17-1) for further information.	

11-3: AASHTO Factors of Safety

(from Standard Specifications for Highway Bridges, 17th Ed.)

Analysis Condition	Minimum Factor of Safety (FS)
o Sliding (static) o Sliding (seismic)	1.5 1.125
o Overturning (static) o Overturning (seismic)	2.0 for footings on soil 1.5 for footings on rock 1.5 for footings on soil 1.125 for footings on rock
o Bearing Capacity (static) o Bearing Capacity (seismic)	3.0 (refer to shallow foundation section) 1.5 (refer to shallow foundation section)

11-4: Computer Programs

Vertical Static Pile Capacity		
S PILE	FHWA-SA-92-044	Ultimate vertical static pile capacity
PILEPMT	Texas A & M University	Ultimate vertical Load vs. depth, and Load vs. Settlement (using pressure meter data)
NEW NEG	Texas A & M University	Analysis of Piles subjected to negative skin friction
SHAFT 5.0	Ensoft, Inc., www.ensoftinc.com	A program for the study of drilled shafts under axial loads
TZ PILE	Ensoft, Inc., www.ensoftinc.com	Computes the load-settlement relationships of vertically-loaded piles using t-z cures and q-w curves
APILE	APILE Plus 3.0 for Windows, Ensoft, Inc., www.ensoftinc.com	Computes the axial capacity of driven piles as a function of depth
Group	Group 4.0 for Windows, Ensoft, Inc., www.ensoftinc.com	Pile group design program that calculates the distribution of loads to piles in a symmetrical group
FB-Pier	Florida Pier Program (FDOT, FHWA) (University of Florida)	Finite element analysis for deep foundations
Driven Piles		
WEAP	Gobel, G.G. & Rausche, Frank WEAP 87, Wave Equation Analysis of Pile Foundations, Volumes I-V, FHWA, 1987	Pile group design program that calculates the distribution of loads to piles in a symmetrical group
DRIVEN	FHWA-SA-98-074	Finite element analysis for deep foundations

11-4: Computer Programs, contd.

Lateral Loads on Piles		
COM624P	COM624P, Laterally Loaded Pile Analysis Program, Version 2.0, FHWA-SA-91-048, 1993, http://www.fhwa.dot.gov/bridge/software.htm	Computes deflections and stresses for laterally loaded piles and drilled shafts
LPILE	LPILE Plus 4 for Windows	Computes deflections and stresses for laterally loaded piles and drilled shafts under lateral loads
Spread Footings		
CBEAR	CBEAR Users Manual, FHWA-SA-94-034, 1996, http://www.fhwa.dot.gov/bridge/software.htm	Computes ultimate bearing capacity of spread or continuous footings on layered soil profiles
Sheet Piles		
CWALSHT	Dawkins, William P., Users Guide: Computer Program For Design and Analysis of Sheet Pile Walls by Classical Methods, Waterways Experiment Station, 1991	Design and analysis of either anchored or cantilevered sheet pile retaining walls. Moments, shear, and deflection are shown graphically
Shoring	Civil Tech, CT-SHORING, WINDOWS 3.X, 95, NT	Excavation supporting system design and analysis

11-4: Computer Programs, contd.

Slope Stability		
PCSTABL	PC-STABL5M Users Manual, FHWA, 1990; PC-STABL6 Users Manual, FHWA, 1990	Calculates factor of safety against rotational, irregular, or sliding wedge failure by simplified Bishop or Janbu, or Spencer method of slices. Version 6 is used for embankments w/reinforcement by simplified Bishop method
XSTABL	Interactive Software Designs, Inc., XSTABL. An Integrated Slope Stability Analysis Program for Personal Computers Reference Manual	Program performs a two dimensional limit equilibrium analysis to compute the factor of safety for a layered slope using the modified Bishop or Janbu methods
SLOPE-W	GEO-SLOPE International, Ltd. 1400, 633 6th Avenue SW Calgary, Alberta, Canada T2P 2Y5 http://www.geo-slope.com	Program that uses limit equilibrium theory to compute the factor of safety of earth and rock slopes
UTEXAS2	University of Texas	A slope stability program, which calculates the safety factor for either a prescribed shear surface or searches for the critical shear surface
Embankment Settlement		
EMBANK	EMBANK Users Manual, FHWA-SA-92-045, 1993	Calculates compression settlement due embankment loads

11-4: Computer Programs, contd.

Soil Nailing		
GoldNail	Golder Associates, GoldNail. A stability analysis computer program for soil nail wall design, Reference Manual Version 3.11	The program is a slip-surface, limiting-equilibrium, slope-stability model based on satisfying overall limiting equilibrium (translational and rotational) of individual free bodies defined by circular slip surfaces. GoldNail can analyze slopes with and without soil nail reinforcement or structural facing
Seismic		
Pro Shake	EduPro Civil Systems, Inc. 5141 189th Avenue NE Sammamish, WA 98074	The program is used to perform one-dimensional, equivalent linear ground response analyses
Shake2000	Ameritech Engineering Corvallis, OR http://www.shake2000.com	Used for seismic analysis of soil deposits and earth structures and to provide a first approximation of the dynamic response of a site

11-4: Computer Programs, contd.

Rock Slopes		
Rock Database Manage-mint System	NDOT, Geotechnical Section	Contains the Rockfall Hazard Rating System information on rated slopes
Rocfall	Rocscience, Toronto, Ontario, Canada	Rockfall simulation software that provides trajectory and energy predictions for rockfalls passing up to three analysis points
CRSP	Colorado Rockfall Simulation Program, available from Colorado DOT	Rockfall simulation software that provides trajectory and energy predictions for rockfalls passing a single analysis point
RockPack III	C.F. Watts & Assoc., Radford, VA	Rock slope stability software for the kinematic and slope stability analysis of structural discontinuity data

11-4: Computer Programs, contd.

MSE and Steepened Slopes		
MSEW	1.0 ADAMA Engineering, Inc., Mechanically Stabilized Earth Walls Software, Version 1.0	The program can be applied to walls reinforced with geogrids, geotextiles, wire mesh, or metal strips. It allows for reduction factors associated with polymeric reinforcement or for corrosion of metallic reinforcement
RSS	Reinforced Slope Stability A Microcomputer Program User's Manual, FHWA-SA-96-039, 1997, www.fhwa.dot.gov/bridge/software.htm	A computer program for the design and analysis of reinforced soil slopes (RSS Reinforced Slope Stability). This program analyzes and designs soil slopes strengthened with horizontal reinforcement, as well as analyzing unreinforced soil slopes. The analysis is performed using a two-dimensional limit equilibrium method
ReSSA	NHI (FHWA)	Upgrade of RSS Program
Other Programs		
Math CAD	Mathsoft Engineering and Education 101 Main Street Cambridge, MA 02142	Alternative to Excel that accepts and displays natural mathematical notation

NOTE:

Many additional programs that perform similar tasks can be obtained from the private sector. The programs listed are continually updated or revised.

11-5: Guidelines for Geotechnical Engineering Analysis

Soil Classification			**Embankment and Cut Slopes**		**Structure Foundations (Bridges & Retaining Walls)**		**Retaining Walls (Conventional, Crib, & Reinforced Soil)**	
Unified (USCS)	AASHTO (Approx)	Soil Type	Slope Stability* Analysis	Embankment Foundation Settlement Analysis	Bearing Capacity Analysis	Settlement Analysis	Lateral Earth Pressure	Stability Analysis
GW GP GM GC SW SP SM SC	A-1-a A-1-a A-1-b A-2-6 A-2-7 A-1-b A-3 A-2-4 A-2-5 A-2-6 A-2-7	GRAVEL well graded GRAVEL poorly graded GRAVEL silty GRAVEL clayey SAND well graded SAND poorly graded SAND silty SAND clayey	Stability analysis generally not required if cut or fill slope is 1½H:1V or flatter, and water table in cut slope is drawn down by underdrains. Erosion of slopes may be a problem for SW or SM soils.	Settlement analysis generally not required except possibly for SC soils.	Required for spread footings, pile, or drilled shaft foundations. Spread footings generally adequate except possibly for SC soils.	Analysis generally not needed except for SC soils or for large, heavy structures. Empirical correlations with SPT values usually used to estimate settlement.	GW, SP, SW, & SP soils generally suitable for backfill behind or in retaining or reinforced soil walls. GM, GC, SM, & SC soils generally suitable if have less than 15% fines. Lateral Earth pressure analysis required using soil angle of internal friction.	All walls should be designed to provide minimum F.S.=2 against overturning, & minimum F.S.=1.5 against sliding along base. External slope stability considerations same as previously given for cut slopes & embankments.
ML	A-4	SILT inorganic SILT Sandy	Stability analysis is required unless nonplastic. Erosion of slopes may be a problem.	Settlement analysis required unless non-plastic.	Analysis required. Spread footings generally adequate.	Analysis required. Can use SPT values if nonplastic.	These soils are not recommended for use directly behind or in retaining or rein-forced soil walls.	
CL	A-6 Lean Clay	CLAY inorganic	Required.	Required.	Analysis required. Deep foundations generally required unless soil has been preloaded.	Analysis required. Lab consolidation test data needed to estimate settlement amount & time.		
OL	A-4	SILT organic	Required.	Required.				

NOTES: These are general guidelines – Detailed slope stability analysis may not be required where past experience exists in area with similar soils and similar slope angles. This table is based on FHWA Geotechnical Checklist and Guidelines (FHWA-ED-88-053).

11-5: Guidelines for Geotechnical Engineering Analysis, contd.

Soil Classification			**Embankment and Cut Slopes**		**Structure Foundations (Bridges & Retaining Walls)**		**Retaining Walls (Conventional, Crib, & Reinforced Soil)**	
Unified (USCS)	AASHTO (Approx)	Soil Type	Slope Stability* Analysis	Embankment Foundation Settlement Analysis	Bearing Capacity Analysis	Settlement Analysis	Lateral Earth Pressure	Stability Analysis
MH	A-5	SILT inorganic	Stability analysis required. Erosion of slopes may be a problem.	Required.	Analysis required. Deep foundations generally required, unless soil has been preloaded.	Analysis required. Lab consolidation test data needed to estimate settlement amount and time.	These soils are not recommended for use directly behind or in retaining walls.	All walls should be designed to provide minimum F.S.=2 against overturning, & minimum F.S.=1.5 against sliding along base. External slope stability considerations same as previously given for cut slopes & embankments.
CH	A-7	CLAY inorganic “fat clays”	Required.	Required.				
OH	A-7	CLAY organic	Required.	Required.				
PT	--	PEAT muck	Required.	Required. Long-term settlement can be significant.	Deep foundation required, unless peat excavated and replaced.	Highly compressible. Not suitable for foundation support.		
Rock			Fills: Analysis not required for slopes 1-1½H:1V or less. Cuts: Analysis required, but depends on spacing, orientation, and strength of discontinuities, and durability of the rock.	Not required.	Analysis required for spread footings or drilled shafts – usually empirical, related to RQD (Rock Quality Designation).	Analysis only required where rock is badly weathered or closely fractured (low RQD value). May require special testing, such as pressure meter.	Lateral earth pressure analysis required using rock backfill angle of internal friction.	

REMARKS:
Soils – Temporary groundwater control may be needed for foundation excavations in GW through SM soils.
Backfill specifications for reinforced soil walls using metal reinforcement should meet the following requirements to insure use of noncorrosive backfill:

1. pH range = 5-10
2. Resistivity3,000 ohm-cm
3. Chlorides 200 ppm
4. Sulfates 1,000 ppm

Rock – Durability of shale (siltstone, claystone, mudstone) to be used in fills should be checked. Nondurable shales should be embanked as soils, i.e., placed in maximum 12” loose lifts & compacted with heavy sheepsfoot or grid rollers.

NOTES: These are general guidelines–Detailed slope stability analysis may not be required where past experience exists in area with similar soils or rock, and gives required slope angles. This table is based on FHWA Geotechnical Checklist and Guidelines (FHWA-ED-88-053).

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