# GEOTECHNICAL TECHNICAL GUIDANCE MANUAL

May 2007



U.S.Department of Transportation
**Federal Highway Administration**

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# SECTION 1 INTRODUCTION

This geotechnical Technical Guidance Manual (TGM) provides technical guidance for geotechnical work performed by the Federal Lands Highway (FLH). It provides guidance for understanding and applying policies, standards and criteria in recognition of the need to manage financial and public safety risk and accomplish the missions of FHWA, FLH and partner agencies. Specific topics include geotechnical reconnaissance, site and subsurface investigation, analysis and design, reporting, PS&E involvement, construction support, performance monitoring, emergency response and consultant roles.

The guidance in this TGM supports the policies, standards and standard practices presented in Chapter 6 of the Project Development and Design Manual (PDDM). Additionally, the TGM provides guidance for activities where standards and standard practices do not exist and it provides access to and guidance for the use of new technologies. Chapter 6 of the PDDM is the source for general direction on "what" should be performed, whereas guidance herein provides recommendations and options for "how" to perform these tasks.

Like the PDDM, the TGM is intended to be used primarily as a web-based electronic reference document. Not all guidance is presented directly in the manual. When published sources present guidance that satisfies the requirements of FLH, or does so with only minor modification required, the TGM provides citations and links to those sources. If necessary, commentary on the application of these sources is provided here. This is done to keep the TGM small and more manageable, and also to allow easy and timely incorporation of new guidance as it is developed and published by FLH, FHWA and others.

Technical guidance references cited and linked in this manual are classified as either "Primary", or "Secondary". Primary sources either present preferred guidance on how to accomplish a task or, when equal guidance is available through many sources, the Primary source is most widely available. "Secondary" sources are additional documents that are often relied on for FLH work; they present guidance to augment the Primary source.

The PDDM presents work requirements through the official statement of policy and standards so it is an essential companion manual to the TGM. The TGM does not stand alone; policies and standards are repeated here only as necessary to offer guidance on their application. If discrepancy in the statement of policy or standards exists, the PDDM has precedence.

Division-level documents also exist within FLH to provide guidance on unique technical practices or procedures at FLH Divisions; where these exist they should be followed for work within that Division. Also, although the organization of each of the Divisions is similar, there are differences. For this reason, the project delivery process and how the Geotechnical Discipline works within that process is described largely at the Division level. The relationship between the TGM, PDDM and other available guidance and manuals is shown in Exhibit 1.1–A. Expanded explanation of the content, uses and status of these documents is presented in Exhibit 1.1–B.

**Exhibit 1.1–A FLH DOCUMENTS: GEOTECHNICAL GUIDANCE**

**Relationship of the TGM to other Guidance, References and Procedures**

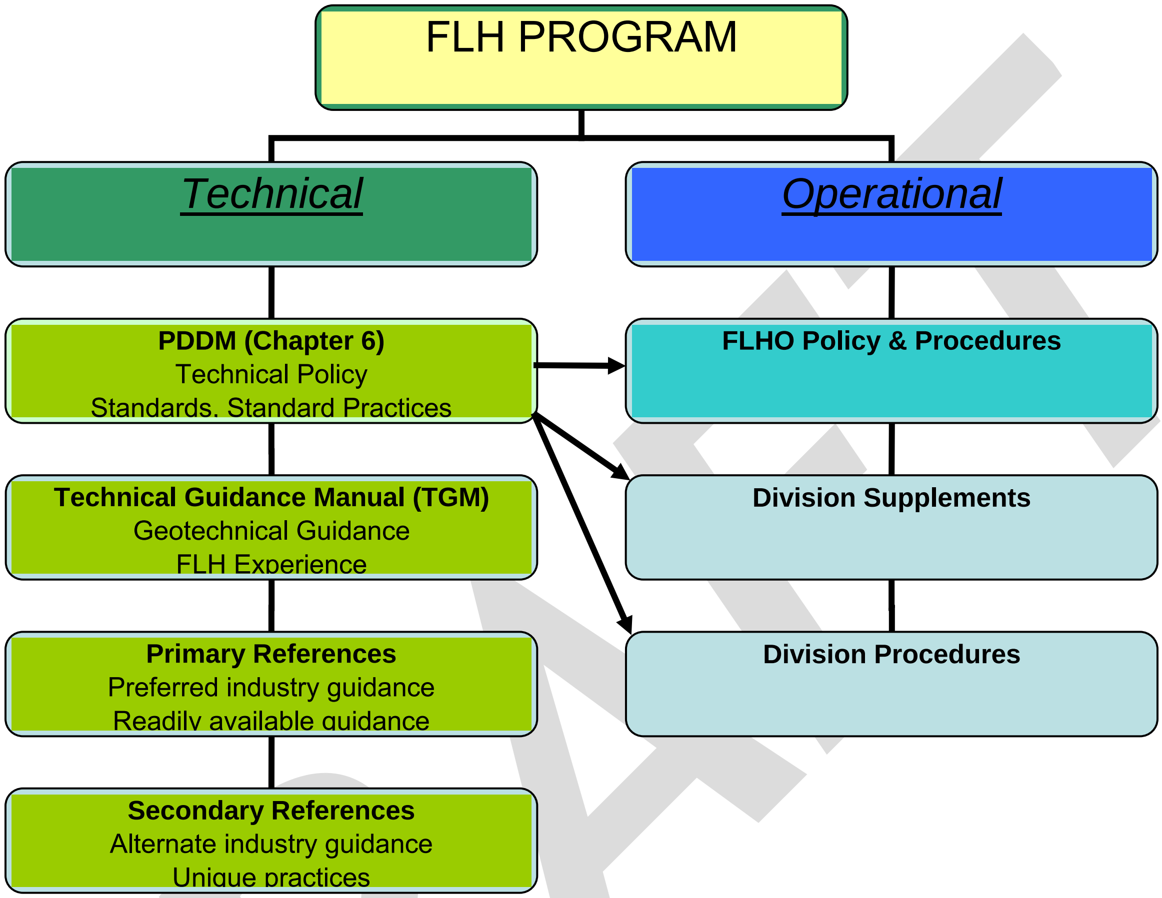


**Exhibit 1.1–B FLH DOCUMENTS – GEOTECHNICAL GUIDANCE**

| PDDM Chapter 6: Geotechnical "Portal" (High-level guidance) |
|---|
| • FLH policies guiding geotechnical work (including sources of directives) |
| • FLH standards for geotechnical work |
| • Links to institutional guidance and standards of practice in geotechnical Technical Guidance Manual (TGM) |
| • Links to recommended technical references and bibliography |
| • Links to unique FLH Division procedures |

| TGM (Technical Guidance Manual – Geotechnical) (Technical-level guidance) |
| --- |
| • FLH institutional guidance for geotechnical work |
| • FLH standards of practice for geotechnical work (describes the “what” and provides links to “how” procedures) |
| • Links to recommended technical references that are often used in FLH Geotechnical Discipline work, described in context with FLH standards of practice |
| • Bibliography references (sorted by 20 geotechnical topics). Database identifies whether references are available on internet. |
| • Links to unique FLH Division procedures |

| FLH Division Procedures - Geotechnical (Guidance that varies between FLH Divisions) |
| --- |
| • FLH institutional guidance and exceptions at Division level |
| • FLH Division operations and procedures |

## 1.1 GEOTECHNICAL DISCIPLINE

The FLH Geotechnical Discipline in each of the three Division offices provides geotechnical engineering and engineering geology services for geotechnical related aspects of design, emergency response and construction support. The discipline is comprised of in-house and contracted geotechnical engineers, engineering geologists, and geologists collectively named ‘Geotechnical Professionals’. The FLH Headquarters office provides administrative direction and policy related assistance to the Division offices, including the Geotechnical Discipline.

Geotechnical work performed by Federal Lands Highways is unique in that much of the work deals with low volume roads on resource sensitive public lands. This is an important distinction because these areas have significant and diverse stakeholders, regulations, management goals, environmental resources, cultural resources, wildlife, and intrinsic scenic beauty. Furthermore, Federal Lands Highways is a partner with federal land management agencies and other government property managers and owners, but does not own or manage federal land, or the improvements it designs and constructs. As such, upon successful completion, another agency accepts FLH projects and agrees to maintain them. Geotechnical work and designs should:

- Be respectful of the land, client agency goals, tribal values, cultural significance of landforms and sites, wildlife, and habitat.
- Provide a safe passage for residents, through travelers, visitors, tourists, recreationists, and wildlife.
- Minimize impacts to existing features and conditions in a “light on the land” manner, meaning that improvements should blend into the setting with as little impact as possible.

- Be accomplished within budget constraints, recognizing that funding is often comparatively less for low-volume, rural public access roads than for higher volume state and municipal projects.

The combination of protecting cultural and environmental resources, accommodating public lands stakeholders and their values/regulations, and working within limited funding, means searching for geotechnical solutions that are context-sensitive and cost effective. Dealing with the variability of FLH projects, terrains, climates and client agency constraints requires flexibility and resourcefulness.

Therefore appropriate scope for geotechnical work will be different for many Federal Lands projects as compared to high volume state primary and interstate highways. This manual provides guidance in identifying and planning appropriate levels of geotechnical practice to fit the unique circumstances and addresses challenges posed by potential conflicts between stated policies, standards, and project constraints.

Geotechnical services for FLH projects should meet the geotechnical and geological standards of care and diligence that others in that profession ordinarily exercise under like circumstances. The geotechnical professional needs to perform the appropriate level of engineering and recommend solutions to "protect the safety, health and welfare of the public in the performance of their professional duties," which is a primary obligation under professional engineering licensure statutes. Professional experience and judgment will be necessary.

The state-of-the-practice of the geotechnical field involves engineering judgment to provide the most efficient and economical investigations and designs. While this chapter provides standards and direction to specific guidance, it is not intended to limit the individual Geotechnical Professional from exercising their professional judgment and experience. Dealing with the variability of FLH projects, terrains, climates and partner agency constraints requires flexibility and resourcefulness. Geotechnical work is to be conducted in accordance with accepted geotechnical standards-of-care by engineers or engineering geologists who possess adequate geotechnical training and experience.

## 1.2 GEOTECHNICAL ROLE IN PROJECT DEVELOPMENT

The Geotechnical Discipline is one of several disciplines that have a role in project development. As such, collaboration and communication is very important. Typically FLH establishes a Cross-Functional Team (CFT) for each project. The project CFT is led by the Project Manager and is supported by a member of all relevant disciplines. The Geotechnical Discipline is relevant to all projects involving the following, in addition to other unique projects:

- Structures, which are primarily walls and bridges,
- Significant earthwork, either cuts or fills,
- Geohazards, which are primarily landslides and rockfall,
- Seismic hazards, and
- Problematic soil, such as frost and swell susceptible soil.

The GD needs to communicate and collaborate with all disciplines, but most frequently finds the need to work together with the Design discipline for layouts and consequences of cuts ant fills, pavements discipline for subgrade issues, the H&H Discipline for scour considerations, and the Structures Discipline for structure design criteria and loads.

Depending on the scope of the project and the amount of structures and earthwork, and anticipated hazards or problematic soils, the extent of geotechnical involvement varies. There is a general chronology to geotechnical involvement in project delivery as follows:

- Initiate and scope the project
- Study available geotechnical data
- Perform field reconnaissance
- Perform preliminary project investigations
- Perform supplemental project investigations
- Compile and summarize data
- Perform geotechnical analyses
- Prepare geotechnical report(s)
- Provide support to PS&E development
- Provide support through award and construction

Because projects are often remotely located and travel costs are relatively high, effort should be made to plan field work in advance so as to get the most from it and minimize the need for supplemental trips for exploration. For example, sometimes preliminary investigations are done along with field reconnaissance, or they are effectively lumped in with the supplemental investigations so that there is only one period of investigation. These decisions are made on a project by project basis and depend on how early the project scope is defined and the potential for geotechnical issues to have major impact on the project. The activities fall into the categories of preliminary work, geotechnical investigation, geotechnical recommendations, geotechnical PS&E support, and geotechnical construction support, and these activities are presented in the following subsections.

### 1.2.1 PRELIMINARY WORK

For the project definition phase, the geotechnical recommendations provided will be at the conceptual/feasibility level. The investigation for this phase usually consists of a field reconnaissance by the geotechnical professional and a review of the existing records, geologic maps, and so forth. For projects that lack significant geotechnical information or are complex, some soil borings may be drilled at critical locations for development of the project definition.

A key role of the geotechnical professional in this stage of a project is to identify potential fatal flaws with the project, potential constructability issues, and geotechnical hazards such as earthquake sources and faults, liquefaction, landslides, rockfall, and soft ground.

The geotechnical professional should provide conceptual hazard avoidance or mitigation plans to address all the identified broad geotechnical issues and construction support. An assessment of the effect geotechnical issues have on construction staging and project constructability will be made. Future geotechnical design services needed in terms of time, cost, and the need for special permits to perform the geotechnical investigation are determined. This phase is generally conducted prior to the 30% plans level.

Refer to Section 3.1 for more information.

### 1.2.2 GEOTECHNICAL INVESTIGATION

This activity includes earthwork, bridge foundations, retaining wall, landslide, and material source investigations. The required activity tasks include: 1) develop site investigation plan; 2) obtain necessary drilling permits; 3) coordinate with partner agency; 4) schedule investigation equipment (procure contract?); 5) conduct site investigation (drilling and geophysics); 6) prepare field boring logs; 7) arrange for necessary laboratory testing; and 8) evaluate constructability issues.

It is in this phase that roadway design refines and defines the project's alignment, sets profiles and grade, and identifies specific project elements to be addressed. The degree of geotechnical investigation is scoped to determine geologic and subsurface conditions, and ultimately soil and rock parameters for design of applicable geotechnical items. An initial evaluation of proposed construction is made to confirm feasibility and to identify potential constructability issues.

Field investigations are generally conducted prior to the 70% plan-in-hand package delivery, but may extend beyond that due to bridge TS&L developments, for example. It is not uncommon for more than one field investigation to occur, depending on design developments.

Refer to Section 3.2 for more information.

### 1.2.3 GEOTECHNICAL RECOMMENDATIONS

This activity consists of conducting geotechnical analyses and preparing a report with final geotechnical recommendations for earthwork, bridge foundations, retaining walls, geohazards, and material sources. The required activity tasks include: 1) summary of soil/rock testing results; 2) prepare final boring logs; 3) prepare plan and profile of selected subsurface investigations; 4) conduct geotechnical analysis and design; and 5) prepare conclusions and recommendations, and 6) prepare and distribute geotechnical report.

Once the preliminary project elements and alignments for the project are established, the geotechnical professional should perform analyses of various technical issues and construction options. For example, assessments of feasible cut and fill slopes to enable roadway design to establish the right-of-way needs for the project. Where walls may be needed, using approximate wall locations and heights from the roadway designer, an assessment of feasible wall types is performed by the geotechnical professional, primarily to establish right-of-way and easement needs. Additional design considerations are described in Section 4.

Conceptual and/or more detailed preliminary bridge foundation design, for example, Type, Size, & Location (TS&L), if required, may be conducted during this phase (if it was not conducted during the project definition phase) to evaluate bridge alternatives and develop a more accurate estimate of cost.

Before the end of this phase, the geotechnical data necessary to allow future completion of the PS&E level design work is gathered (final geometric data, test hole data, etc.). This work is generally drafted and completed prior to the 70% PS&E package delivery, though it may lag due to design changes, environmental clearance issues, bridge TS&L delivery, etc.

Refer to Section 5.1 for more information on documentation and reporting.

### 1.2.4 GEOTECHNICAL PS&E

This activity consists of conducting geotechnical reviews of plans, specifications and estimates (PS&E). This work is generally accomplished between the 30% and 70% PS&E process and includes the Plans-In-Hand task at the 95% level. The required activity tasks include: 1) reviews of plans, specifications and estimates to validate and incorporate geotechnical recommendations; 2) writing special contract requirements (SCR's) for geotechnical design elements that are not addressed by the standard specifications; 3) attendance at onsite or offsite plan review meetings; and 4) associated internal and external correspondence.

It is in this phase that final design of all geotechnical project features is accomplished. Recommendations for these designs, as well as special provisions and plan details to incorporate the geotechnical design recommendations in the PS&E, are provided in the geotechnical reports and memorandums prepared by the geotechnical professional. Detailed recommendations for the staging and constructability of the project geotechnical features are also provided. Plans and specifications are reviewed to confirm that the designs are consistent with geotechnical data and recommendations. Input may be requested on pay items associated with geotechnical items for development of Engineer's Estimates.

Refer to Section 5.2 for more information on PS&E.

### 1.2.5 CONSTRUCTION SUPPORT

This activity consists of evaluating geotechnical issues raised during the bid process, evaluating contractor submittals and shop drawings, participating in Prebid and Preconstruction meetings where significant geotechnical items require clarification, advising the FLH construction project manager and inspectors on complex designs and requirements, troubleshooting geotechnical difficulties and resolving potential "changed conditions" issues, providing design changes and monitoring/evaluating instrumentation. The amount of assistance for any project depends on the relative complexity and requests from the FLH construction project manager.

Refer to Section 5.3 for more information on construction support.

## 1.3 USE OF THE TGM

The TGM is intended for Geotechnical Discipline use by FLH staff and contractors. Though the TGM may be of value to those practicing in related disciplines, when this level of geotechnical guidance is sought, a Geotechnical Professional should be assigned to the project and should be responsible for interpreting the guidance herein and for conveying geotechnical recommendations.

This manual is intended to be used primarily in two ways. First, it is to be accessed through topical subsections of PDDM Chapter 6 for guidance on applying policy and standards, for when exceptions to standards might be warranted, and methods for completing non-standardized work. Second, the TGM is the source of the highest-level “how to” guidance and should be used to educate or reacquaint the Geotechnical Professional with preferred methods of conducting FLH geotechnical work, and presentation formats. Though the guidance here is not required, it is preferred because it leads to consistency, facilitates understanding by our customers and efficient transfer and review of projects within the Discipline.

The TGM provides three other valuable resources. Appendix A is a list of primary, secondary and other cited sources (tertiary) of technical guidance that are cited in the PDDM and throughout the TGM. The TGM Bibliography is a source of geotechnical reference documents and, where possible, a link to them. It is presented in a topical and alphabetic format in Appendix B. Appendix C provides an incomplete listing of federal laws and regulations. Although not complete, this listing can provide background on the laws and regulations driving our activities and those of our partners.

It is the responsibility of all FLH Geotechnical Professionals and consultants to become familiar with the materials presented in this manual and in PDDM Chapter 6, and apply them appropriately while performing Geotechnical Discipline work. Any questions involving interpretation of or exception to the content of this manual are to be referred to the Geotechnical Discipline Leader or Division Geotechnical Team Leaders. This is a live manual, it belongs to the Geotechnical Discipline and it is the Discipline’s responsibility to assure its accuracy and usefulness. It is anticipated that revisions may be appropriate quarterly and that a new version, fully incorporating the revisions, would be posted annually. Suggested revisions should be sent to the Geotechnical Discipline Leader, with courtesy copy also sent to the Geotechnical Team Leader from the requesting Division. Suggested revisions will be agreed upon and incorporated in a procedure agreed upon by the Geotechnical Discipline.

Division Supplements are linked from this chapter when there unique guidance appropriate for specific Divisions. Division Supplements also contain guidance on processes, and quality control and assurance. Division Supplements are linked through text boxes, as follows:

| |
|---|
| *Refer to [EFLHD – CFLHD – WFLHD] Division supplements for more information.* |

# SECTION 2 GUIDANCE AND REFERENCES

## 2.1 POLICIES FOR THE GEOTECHNICAL DISCIPLINE

Policies are defined in the PDDM as being guiding principles that are followed without exception. Thus, they are quite general and serve the purpose of defining a philosophy, rather than defining specifically what to do. Policies presented in this chapter are interpretations of agency directives and objectives based on legislation and federal regulations pertaining to FLH and FHWA. The policies are not interpreted elsewhere in a way that is specifically relevant to the Geotechnical Professional; the sources of the policies are identified and the basis of interpretations explained here. These policies are the guiding principles that are to be followed at all times in the conduct of geotechnical work for Federal Lands Highway.

Geotechnical engineering for FLH can be very challenging; projects are located from the Atlantic to the Pacific and from the tropics to the arctic. The natural settings and geotechnical issues vary tremendously; however, an equal challenge comes from the variety of projects and stakeholders. Some projects are four lane divided highways and bridges, but much of the work deals with low volume roads on resource sensitive public lands. These areas have significant and diverse stakeholders, regulations, management goals, environmental resources, cultural resources, wildlife, and intrinsic scenic beauty. Furthermore, FLH is a partner with federal land management agencies and other government property managers and owners, but does not own or manage federal land, or the improvements it designs and constructs. As such, upon successful completion, another agency accepts FLH projects and agrees to maintain them. With this in mind, geotechnical work should embrace the following key FLH project delivery objectives:

1. Be respectful of the land, client agency goals, tribal values, cultural significance of landforms and sites, wildlife, and habitat;

2. Provide a safe passage for residents, through travelers, visitors, tourists, recreationists, and wildlife;

3. Minimize impacts to existing features and conditions in a “light on the land” manner; blend improvements into the setting with as little impact as possible: and

4. Complete work within budget constraints, recognizing that funding is often comparatively less for low-volume, rural public access roads than for higher volume state and municipal projects.

The combination of protecting cultural and environmental resources, accommodating public lands stakeholders and their values/regulations, and working within limited funding, means searching for geotechnical solutions that are both context-sensitive and cost effective. Dealing with the variability of FLH projects, terrains, climates and client agency constraints requires flexibility and resourcefulness.

Sometimes the appropriate scope for geotechnical work will be different for FLH projects than it is for high volume state primary and interstate highways. This chapter provides guidance in identifying and planning appropriate levels of geotechnical practice to fit the unique circumstances and addresses challenges posed by potential conflicts between stated goals and available funding. The highest-level guidance is in the form of policy, which is followed without exception. Seven Geotechnical Policies provide this high level guidance for the Geotechnical Professional:

1. Support the mission, vision and program management objectives of FLH and FHWA
2. Meet the technical scope requirements defined by the PDDM
3. Advance the state of practice by seeking and implementing new technology
4. Demonstrate environmental stewardship in investigations and designs
5. Demonstrate financial, cultural and natural resource stewardship
6. Conduct work safely and seek safety improvement solutions
7. Achieve quality through established quality assurance and oversight procedures

The guiding principles represented by these policies often guide in somewhat different directions. The policies are similar to guy lines supporting a tower. If the tower is centered none of the guy lines will be overloaded and broken; if the work is centered, none of the policies will be broken. When policies pull in different directions the FLH Geotechnical Professional should strive to keep their work and recommendations centered.

The following paragraphs present each policy, the source documents through which it was identified, and an interpretation to further explain the guidance it provides. Commentary is used to provide additional discussion and examples.

| | | |
|---|---|---|
| 1. | CFR Title 23 | Code of Federal Regulations Highways Title 23 |
| 2. | FAPG Transmittal 16 | Federal Aid Policy Guide Transmittal 16 NS 23CFR635 (1996) |
| 3. | FLH Business Plan | *FLH Business Plan* |
| 4. | FLH Safety Memo 2004 | *FLH Safety Philosophy* Memorandum (2004) |
| 5. | FLH Transmittal 12 | Federal Lands Highway Manual, Chapter 3, Section C, Subsection 2, Transmittal 12 (1983) |
| 6. | FLH Transmittal 18 | Federal Lands Highway Manual, Chapter 1, Section A, Subsection 1, Transmittal 18 (1983) |
| 7. | FLH Transmittal 21 | Federal Lands Highway Manual, Chapter 1, Section A, Subsection 2, Transmittal 21 (1983) |

## POLICY 1: SUPPORT THE MISSION, VISION AND PROGRAM MANAGEMENT OBJECTIVES OF FLH AND FHWA.

**FLH Geotechnical Discipline policy is to support the mission, vision and program management policies of FLH and FHWA.** This policy is clearly the penultimate policy; the other six policies guide practice to meet Geotechnical Policy 1.

This geotechnical policy is based on the following: CFR Title 23, FLH Transmittal 12, FLH Transmittal 18, FLH Transmittal 21, FLH Business Plan, and FLH Safety Memo 2004. These agency policies and objectives define the application of the federal legislation and regulations that authorize agency activities.

Commentary: *These linked documents direct the Geotechnical Professional to perform work that is consistent with prevailing laws and regulations, executive orders, DOT orders, FHWA regulations and administrative rules, FLH goals, policies, and applicable standards of practice. General roles and responsibilities of FLH Divisions are described in FLH Transmittal 21, specifically including the Geotechnical Discipline. Divisions are to follow policies and regulations and be consistent with national standards, specifications and manuals. FLH Transmittal 12 states that standards be followed in general. CFR Title 23 Part 625.4(d) (I) specifies that AASHTO Standard Specifications for Highway Bridges be followed. CFR Title 23 specifies material codes and requirements in Parts 625.41(d) (I) and 635, with reference to the AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing. FLH Transmittal 12 provides an exception process for use when these standards are inappropriate. CFR Title 23 states that designs shall conform to the particular needs of each locality and be conducive to safety, durability and economy of maintenance. An important goal is to provide the highest practical and feasible level of safety for people and property and to reduce highway hazards. Transmittal 12 further states that partner agencies be informed of relative risks and consequences when waiving standard practices. FLH safety memo (2004) describes the philosophy of balancing technical standards, environmental stewardship and partner agency requirements. The Geotechnical Discipline management will seek clarification within FHWA when confronted with situations that are not defined herein.*

## POLICY 2: MEET THE TECHNICAL SCOPE REQUIREMENTS DEFINED BY THE PDDM

FLH Geotechnical Discipline policy is to meet the technical scope defined by the standards and guidance presented here in the PDDM regarding project development activities, including investigation, analysis, reporting, PS&E development, construction support, ERFO support and other agency needs. This policy defines that geotechnical project development and emergency response/repair work is guided by the contents of PDDM Chapter 6 and the companion Geotechnical TGM.

This geotechnical policy is based on FLH Transmittal 21, giving the authority to DFPA (Headquarters) for issuing policy, and the approval and formal transmittal of this version of the PDDM by DFPA (Headquarters).

Commentary: *The guiding principle is to perform geotechnical work according to geotechnical policies, standards and guidance that address Geotechnical Policy 1. These investigations and analyses appropriately address geotechnical issues, project constraints, hazards, risks and uncertainties, commensurate with the opportunities, constraints, values, and context-sensitivity associated with public lands facilities and the FLH roadway and structures design standards.*

## POLICY 3: ADVANCE THE STATE OF PRACTICE BY SEEKING AND IMPLEMENTING NEW TECHNOLOGY

FLH Geotechnical Discipline policy is to evaluate, promote and implement new technology and to continually update geotechnical investigation, analysis, design, construction and monitoring capabilities.

This geotechnical policy conveys a guiding principle for utilizing advances in technology. It is based on FLH Transmittal 18, FLH Transmittal 21, and FLH Business Plan.

*Commentary: FLH Transmittal 18 addresses the use of new technology. FLH Transmittal 21 includes the development and implementation of new technology by conducting tests/experiments, demonstrating new applications, and recommending procedures. The FLH Business Plan addresses the importance of new technology to ensure best use of limited funds. The Business Plan includes the evaluation and reporting on new technologies, as well as promoting and implementing them. Report on the use of new technology to inform other Geotechnical Professionals of the relative success achieved and potential applications.*

## POLICY 4: DEMONSTRATE ENVIRONMENTAL STEWARDSHIP IN INVESTIGATIONS AND DESIGNS

FLH Geotechnical Discipline policy is to perform geotechnical investigations and develop design recommendations that minimize environmental impacts while meeting other project objectives.

This geotechnical policy conveys a guiding principle for environmental responsibility based on the following: National Environmental Policy Act of 1969 (42 USC 4321: PL 91 90), FHWA Implementing regulations: (23 CFR 771.109(c)(2)), and the FHWA Environmental Policy Statement (FHWA 1990, 1994), all of which are described in PDDM Section 3.3.

*Commentary: The FLH Business Plan defines "Environmental Stewardship" as one of the three "Vital Few" agency goals. The FLH Business Plan promotes context-sensitive designs and solutions. The FLH Geotechnical Discipline is responsible for helping FLH achieve these goals. PDDM Chapter 3 provides further environmental guidance.*

## POLICY 5: DEMONSTRATE FINANCIAL, CULTURAL AND NATURAL RESOURCE STEWARDSHIP

FLH Geotechnical Discipline policy is to coordinate and manage geotechnical work within multi-disciplinary and multi-agency project teams and within jointly established schedules, budgets, and project criteria and constraints.

This geotechnical policy conveys a key guiding principle for planning and managing geotechnical work, personnel and resources, including workforce and natural resources. It is based on FLH Transmittal 21, FLH Transmittal 12 and the FLH Business Plan.

*Commentary: Usually more than one option exists to achieve road width and grade requirements throughout the length of the project. Evaluations of design options include the assessment of risk and consequences as well as cost, consistent with FLH Transmittal 12.*

*The FLH Business Plan includes the objective of providing stewardship and oversight on construction projects, which also implies a continuum for geotechnical project involvement through design into construction.*

## POLICY 6: CONDUCT WORK SAFELY AND SEEK SAFETY IMPROVEMENT SOLUTIONS

FLH Geotechnical Discipline policy is to conduct work in a manner that is safe for workers and the public, and to seek solutions that improve safety and minimize roadside hazards on federal and tribal lands. Appropriate safety applications are to be incorporated while respecting the associated resource impacts and historic and cultural values.

This geotechnical policy conveys a guiding principle to protect the general public, FHWA personnel and contractors, and public and private property. This policy applies to work conducted as part of geotechnical services from investigation through construction, as well as the safety of the completed project with respect to geotechnical issues. It is based on FLH Transmittal 12, FLH Transmittal 18, CFR Title 23, FLH Business Plan, and FLH Safety Memo 2004. FLH safety philosophy is presented in PDDM Section 8.1.1.

*Commentary: Transmittal 18 stresses that safety precautions be used. This requirement is further stressed in the FLH Business Plan. The FLH Business Plan defines "Safety" as one of the three "Vital Few" agency goals. Standards of practice are available for traffic control when working in the roadway and safety criteria/methods when working in excavations or other holes, pits or trenches. Safety guidelines also are defined for working around heavy machinery and construction equipment, and for providing training and other means to provide safe working conditions.*

*Transmittal 12 addresses risk factors and advises that partner agencies should be informed of the relative risks and consequences. A process exists for addressing the mitigation of significant risks. Transmittal 12 directs that when the risk to the traveling public or roadway is judged to be intolerable and the issue cannot be reconciled, the Direct Federal Program Administrator should be consulted. The FLH Business Plan states that roadside hazards be minimized and that roads be improved to design standards. Partner agencies may have standards and requirements that could limit the implementation of safety features. The FLH Safety Memorandum (2004) describes the philosophy of balancing technical standards, environmental stewardship and partner agency requirements.*

## POLICY 7: ACHIEVE QUALITY THROUGH ESTABLISHED QUALITY ASSURANCE AND OVERSIGHT PROCEDURES

FLH Geotechnical Discipline policy is to strive for quality through established quality control and quality assurance (QA/QC) procedures and through oversight of geotechnical work performed through contract with others.

This geotechnical policy conveys guiding principles for performing QA/QC and managing outsourced work. It is based on FAPG Transmittal 16, FLH Transmittal 21, and the FLH Business Plan.

*Commentary: QA/QC procedures are based on mandates in the FLH Business Plan. Maintain a quality control and assurance program and apply to all project work. The organization performing the investigation and analysis is responsible for the technical adequacy of their design and activities. The Geotechnical Discipline may need to retain geotechnical consultants if their workload exceeds in-house availability and if special expertise is required that does not exist in-house. Geotechnical consultants shall also follow established QA/QC. Unless specific contract arrangements are made to the contrary, the Geotechnical Discipline does not provide QC or QA for the work of consultants, or for other FLH disciplines.*

### 2.1.1 FLH (DIRECT FEDERAL) POLICIES AND DIRECTIVES

The geotechnical policies are based on legislation and federal directives governing FLH projects and procedures. In addition, the policies reflect key principles contained in existing Federal Lands Highway and partner agency directives. Federal directives and legislation that influence FLH geotechnical policies and standards are listed in Exhibit 2.1–A. Many of the sources are available on websites.

The geotechnical investigation and design of FLH projects is significantly influenced by "policies" and "policy-driven standards;" however, the practices of geotechnical engineering and geology deal with unknown subsurface conditions and subjective scientific relationships that require judgment. Therefore, "guidance" is necessary to identify potential means of accomplishing geotechnical work satisfactory to Federal Lands Highway, which requires responsible and professional judgment and discretion. Guidance is provided in PDDM Chapter 6 and the companion Geotechnical TGM, with reference to important technical requirements and criteria as well as manuals that provide further or more detailed technical guidance.

The policies described here are guiding principles that are to be followed in the conduct of geotechnical work for Federal Lands Highway. The sources of key directives associated with each policy are listed herein. The list of cited laws and directives is not necessarily an exhaustive list, but provides general direction. Commentaries are provided that describe the linkage between Federal/agency directives and the FLH geotechnical policies.

**Exhibit 2.1–A POLICY SOURCE REFERENCE LINKS**

| Policy Source | Reference Link |
|---|---|
| AASHTO Bridge Manual (HB-17) | AASHTO HB-17 |
| AASHTO Standard Specifications for Transportation Materials | AASHTO Stds HM-25-M |
| CFLH Action Plan (2004) | |

| Policy Source | Reference Link |
| --- | --- |
| Code of Federal Regulations – Highways (CFR Title 23) | http://www.access.gpo.gov/cgi-bin/cfrassemble.cgi?title=200523 http://ecfr.gpoaccess.gov/cgi/t/text/text-idx?sid=6851fee6134207fdab7e7697ef8d5bcb&c=ecfr&tpl=/ecfrbrowse/Title23/23cfrv1_02.tpl |
| ERFO Disaster Assistance Manual FHWA-FLH-04-007 (2004) | http://www.fhwa.dot.gov/flh/erfo.htm http://www.cflhd.gov/projects/erfo.cfm |
| Federal Aid Policy Guide Transmittal 16 NS 23CFR635 (1996) | http://www.access.gpo.gov/cgi-bin/cfrassemble.cgi?title=200523 |
| FHWA Checklist FHWA-ED-88-053 (revised 2003) | http://www.fhwa.dot.gov/engineering/geotech/pubs/reviewguide/checklist.cfm |
| FHWA-DF-88-003, Chapter 3 (1996) | http://www.efl.fhwa.dot.gov/design/manual/pddmch03.pdf |
| FHWA-DF-88-003, Chapter 6 (1996) | http://207.86.126.200///design/manual/pddmch06.pdf |
| FHWA GT-15 Differing Site Conditions (1996] | http://www.fhwa.dot.gov/engineering/geotech/library_listing.cfm |
| FHWA Office of Bridge Technology reference manuals | http://www.fhwa.dot.gov/engineering/geotech/index.cfm |
| FLH Business Plan | http://flhnet.fhwa.dot.gov/business_plan/Business%20Plan%20Jan3003%20Final.pdf |
| FLH Safety Philosophy Memorandum (2004) | |
| FLH Transmittal 12 (1983) | |
| FLH Transmittal 18 (1983) | |
| FLH Transmittal 21 (1983) | |
| MUTCD | |
| National Highway Institute manuals | http://www.nhi.fhwa.dot.gov/ |
| OSHA Section 29 | |
| USFS Bridge Foundation Investigation Manual | |
| USFS FSM-7170 (2000) | http://www.fs.fEditionus/im/directives/fsh/7109.52/7109.52,1.txt |

FLH Transmittal 12 (Direct Federal 3-C-2, 1983) addresses risk factors and exceptions to minimum engineering standards. It is Direct Federal's role to determine the appropriate engineering standards by which to design and construct highway projects and to judiciously use these standards. Exceptions must be carefully evaluated in relationship to risks and consequences and shall be documented. Partner agencies will be informed of the relative risks and consequences when waiving engineering standards. When the risk to the traveling public or roadway is judged to be intolerable and the issue cannot be reconciled, the Direct Federal Program Administrator shall be consulted

Federal Aid Policy Guide Transmittal 16 (NS 23CFR635; 1996) and FHWA GT-15 Differing Site Conditions (1996) address "Differing Site Conditions" and recommend that a clause be incorporated in highway projects. Guidelines are described in Geotechnical Engineering Guideline GT-15.

FLH Transmittal 18 (Direct Federal 1-A-1, 1983) addresses the program direction of Direct Federal. Authorities are cited regarding engineering, construction, new technology, and utilization of other federal agencies in administering its program.

FLH Transmittal 21 (Direct Federal 1-A-2, 1983) outlines the general roles and responsibilities of the Office of Direct Federal Programs and Direct Federal Division offices. Policy is to place the responsibility at the lowest appropriate level. Divisions establish and implement goals consistent with agency policies. This policy addresses interagency project and design agreements. Divisions conduct any and all activities required for the timely completion of all projects undertaken within the Division. Divisions practice uniformity and follow national design standards, specifications, and manuals. Geotechnical practice is specifically mentioned. This policy states that reviews should be performed to ensure compliance with policy and regulations. Divisions are advised to carry out programs and projects within budget and provide controls for internal operation. Transmittal 21 directs that skill needs be identified and training be developed for optimum Direct Federal benefits. Federal Lands Highway Divisions are encouraged to develop and implement new technology by conducting tests/experiments, demonstrating new applications, and recommending procedures.

CFR Title 23 "Highways" Part 625 designates standards, policies and specifications that are acceptable to FHWA. Designs shall conform to the particular needs of each locality and be conducive to safety, durability, and economy of maintenance. An important goal is to provide the highest practical and feasible level of safety for people and property and to reduce highway hazards. Provides for a design exception process. Part 625.4(d)(I) specifies that AASHTO Standard Specifications for Highway Bridges be followed. Material codes/requirements (including sampling and testing) are addressed in Part 635 and 625.4(d)(I).

Federal Lands Highway "Business Plan 2003 to 2007" addresses FLH mandates and goals. The FHWA "Vital Few" consist of: 1) Safety, 2) Environmental Stewardship, and 3) Congestion Mitigation. The purpose of a single Federal Lands Highway Program is to: 1) ensure effective and efficient funding and administration for a coordinated program of public roads and bridges, 2) provide needed access and transportation for Native Americans, and 3) protect and enhance our Nation's natural resources. Items of particular importance to the Geotechnical Discipline include:

- Conduct engineering and related studies to ensure best use of limited funds
- Promote context-sensitive designs/solutions
- Provide stewardship and oversight on construction projects

- Minimize roadside hazards
- Improve roads to design standards
- Provide support for natural disasters through ERFO
- Minimize impacts on Federal lands when securing materials to construct transportation projects
- Evaluate and report on new technologies. Promote and implement.
- Deliver projects within the program on budget and on schedule
- Outsourcing project design and development as necessary to meet the program delivery schedule

Eastern Federal Lands Highway Division Safety Philosophy Memorandum, December 2004 describes philosophies to optimize safety and context-sensitive requirements, recognizing that these two goals can sometimes be in conflict.

FHWA *Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications*, FHWA-ED-88-053 (revised 2003) was developed to aid engineers in the preparation and review of geotechnical investigations and reports, as well as to check for consistency with PS&E design/construction documents. This document includes guidelines for planning the number, locations and depths of exploration holes, sampling and testing, and types of geotechnical analyses.

Eastern Federal Lands Highway Division Final Report by the Geotechnical Data Team (June 1998) addressed formats and contents of geotechnical reports, in part to reduce the potential for Differing Site Conditions claims.

Central Federal Lands Highway Division Action Plan (2004) describes the focus on "cradle to grave" project management. This plan states a commitment to deliver the projects assigned to the Federal Lands Highway Division within the SAFETEA act (2003), and recognizes the need to outsource tasks to consultants to accomplish this objective. The action plan describes 9 initiatives that include safety of transportation on Federal and Tribal lands, streamlining the environmental process, project management accountability, and workforce planning.

Chapter 6, FHWA-DF-88-003 (FLH PDDM 1996) provides guidance on geotechnical tasks and procedures based on institutional experience with partner agency requirements for roadways.

Central Federal Lands Highway Division website includes a link to a document describing Earthwork Representation: Grading Summaries and Mass Haul Diagrams (October 2004).

Western Federal Lands Highway Division Field Materials Manual (FHWA-FL-91-002, March 1994) describes standardized field test procedures.

Chapter 3, FHWA-DF-88-003 (FLH PDDM 1996) includes a description of environmental stewardship that is expected on Federal projects.

Central Federal Lands Division Scheduling Activity Codes include four main activities for geotechnical tasks during project development, designated G1 through G4.

AASHTO Standard Specifications for Highway Bridges provides standards for design of bridge and wall foundations and earth retaining systems.

AASHTO, *Standard Specifications for Methods of Sampling and Testing* provides standards for performing various soil and rock tests.

US Department of Transportation, FHWA, Office of Bridge Technology. Identifies Federal Highway publications related to geotechnical investigations, design and construction.

National Highway Institute. Provides geotechnical manuals and training related to transportation project investigation, design and construction.

Federal Highway Act of 1956. This act, also known as the National Interstate and Defense Highways Act, authorized the building of highways throughout the nation, including an interstate highway system. The movement behind the construction of a transcontinental superhighway started in the 1930s when President Franklin D. Roosevelt expressed interest in the construction of a network of toll superhighways that would provide more jobs for people in need of work during the Great Depression. The resulting legislation was the Federal-Aid Highway Act of 1938, which directed the chief of the Bureau of Public Roads (BPR) to study the feasibility of a six-route toll network. The Federal-Aid Highway Act of 1944 funded highway improvements and established major new ground by authorizing and designating the construction of 40,000 miles of a "National System of Interstate Highways." The Federal-Aid Highway Act of 1956 expanded the interstate system to 41,000 miles.

Emergency Relief for Federally Owned Roads (ERFO) Disaster Assistance Manual (2004) describes the program intent, authority, stewardship, policies, process, roles and responsibilities, and applicable FLH forms.

Safe, Accountable, Flexible and Efficient Transportation Equity Act of 2003 (SAFETEA) presents a program of projects that include those to be administered and delivered by Federal Lands Highway Divisions. This Act added four more partner agencies (BLM, BOR, COE and Military Traffic Management Command).

## 2.2 RISK MANAGEMENT

*Risk is inherent in geotechnical work and FLH projects, and it comes in several forms. Risk is incurred with respect to cost when, for example, decisions are made regarding the scope of a geotechnical investigation. A greater investigation scope generally means fewer unknowns are carried into construction, thereby reducing the risk of construction cost escalation. Risk is incurred with respect to serviceability when designs are advanced that do not fully address all possible modes of failure. For example, a slump repair along a road that crosses a much larger, but more slowly moving landslide. Risk is incurred with respect to safety when geotechnical recommendations are incorporated into critical structures such as bridges, walls, and rock slopes. The Geotechnical Discipline's responsibility lies in identifying risks incurred through* *geotechnical issues, informing project team members and partners of these risks, and assisting in evaluating whether the risks are tolerable.*

*Risks are more tolerable when they are low relative to the potential benefit of the action incurring the risk. Risk assessment is the process of assessing the probability of adverse consequences associated with activities, recommendations or designs, and for geotechnical matters it is a Geotechnical Discipline responsibility. For most FLH projects the risk assessment is not a complicated quantitative assessment, but rather a simplified practical assessment based on experience, engineering judgment and historical standard of practice on previous partner agency projects. The evaluation of potential benefit is not solely a Geotechnical Discipline responsibility as it is a interdisciplinary process requiring involvement of the Project Manager and other disciplines that have knowledge of other project aspects and different perspectives on the value of a potential benefit. The responsibility of the Geotechnical Discipline is to inform and educate the Project Manager, and other team members and stakeholders, as appropriate, of risk based on geotechnical issues and to participate in evaluation of the tolerability of that risk.*

The geotechnical policies presented in the previous section help assure that projects have a tolerable level of risk associated with them because they prescribe seeking safety, quality, and following the standards and guidance in the PDDM and TGM. In fact, on most projects, where standards and standard practices are used, risk assessment and evaluation is often implicit and does not require further attention. For this reason, standards and standard practices are used wherever possible. Standards and standard practices are introduced in PDDM Section *2.3* and presented throughout the rest of Chapter 6.

## 2.3 STANDARDS AND STANDARD PRACTICE

FLH follows engineering standard of practice, accounting for requirements and guidance provided by AASHTO, FLH engineering standards and partner agencies.

Some FLH projects are Forest Highway projects that are on state highways and some are major urban highways (e.g. GW Parkway next to Washington, DC). These are higher volume roads that are typically built to AASHTO roadway design standards. The guideline minimum boring, sampling and testing criteria and geotechnical report review checklists set forth in FHWA-ED-88-053 typically will apply.

On all FLH projects, the engineering design of structures (bridges, culverts, walls, tunnels and ancillary structures) should be in accordance with AASHTO HB-17.

Most FLH projects are lower volume roads (NPS, USFS Forest Highway, USFS recreation roads, US Fish and Wildlife Service, Forest Highway State and County roads, BLM, and BIA). Very limited geotechnical design standards exist specifically for low volume roads. Examples are included in: TRB Compendium 1, TRB Compendium 2, TRB Compendium 3, TRB Compendium 4, TRB Compendium 5, TRB Compendium 6, TRB Compendium 7, TRB Compendium 8, TRB Compendium 9, TRB Compendium 10, TRB Compendium 11, TRB Compendium 12, TRB Compendium 13, TRB Compendium 16, and TRB Synthesis 4. On many of these low volume road projects, application of design standards for high volume roads such as AASHTO and level of site investigation set forth in FHWA-ED-88-053 may be impractical, cost-prohibitive and/or not in accord with client agency design standards or acceptable level of risk deemed suitable on that specific project.

Justification should be made when designing to less than AASHTO standards. This should be done by the geotechnical designer working hand-in-hand with the roadway designer and owner agency to arrive at a level of risk acceptable for that project. The risk assessment need not be a complicated statistical assessment, but rather a simplified practical assessment based on experience and engineering judgment and historical standard of practice on previous client agency projects. Examples include:

- Rockfall- design catchment area on low volume road to lower retention level, such as 60 percent, versus 95 percent for Interstate highway where little risk can be tolerated

- Cut slope landslide correction using FS = less than 1.25, where flattening slope to achieve 1.25 would extend a significant distance up the mountainside.

The high cost component of geotechnical site investigations is drilling. FLH philosophy on site investigations is to maximize the use of geologic interpretation and non-invasive methods in order to minimize the amount of drilling required, yet still achieve a level of knowledge commensurate with good engineering practice for similar locations/applications. An example would be a geologic inspection of existing rock cuts with good exposure; therefore widening into the cut 20 feet would not require drilling. Test pits might be an acceptable and economical exploration method; however, such invasive methods might not be acceptable to partner agencies in sensitive areas. A recommendation of site investigation less than that set forth by AASHTO HB-17 or FHWA-ED-88-053 should be derived using engineering judgment and it should be discussed with the project manager, along with the basis and associated benefits, risks and consequences.

*Standards are defined in PDDM Chapter 1 as a fixed reference to guide the approach (standard practice) and content (standard) of FLH work. Geotechnical standards and standard practices address investigation, sampling, testing, analysis, reporting, design details and special contract requirements. Standards are based on many things, including successful past precedent on FLH projects and they help achieve the following goals:*

- *Risk Management*
- *Quality*
- *Efficiency*

*Standards have been established where it has been found that a single approach or product works well in most cases. Standards have a history of use where quality has been demonstrated through successful completion and performance of projects. Standards tend to reduce time during design development and review, reduce bid prices because of familiarity developed within the construction industry, and reduce FLH oversight needs during construction. Project delivery and construction are team endeavors and standards improve efficiency because team members gain greater understanding of what to expect and how to work with what is delivered. Standards also acknowledge an understanding and acceptance of a certain, consistent level of risk.*

*Standards are not always appropriate in the Geotechnical Discipline. Over standardization can lead to inefficient designs, insensitivity to the context of individual projects, and lack of innovation. Given the wide variety of FLH projects, project constraints, and stakeholder interests, considerable flexibility is needed. This PDDM chapter presents a hierarchy of policy, standards, and guidance to allow flexibility when needed and to also keep the geotechnical practice as standard as possible so that the goals of risk management, quality, and efficiency are realized.*

*For example, the subsection on "Structure Foundations" (in section "6.3.3 Geotechnical Analysis and Design") includes the standard to design structure foundations in accordance with the current edition of the AASHTO Standard Specifications for Design of Highway Bridges (*AASHTO HB-17*). This is a widely accepted standard in the industry and it should be used whenever possible. Note, however, that designing in accordance with* AASHTO HB-17 *is not a policy and there are occasions where in order to satisfy a centered approach to the policies in Section 2.1, the* AASHTO HB-17 *standard should not be followed.*

*Another example would be with respect to investigation. Most FLH projects are low volume roads (NPS, USFS Forest Highway, USFS recreation roads, US Fish and Wildlife Service, Forest Highway State and County roads, BLM, and BIA). Very limited geotechnical design guidance exists specifically for low volume roads. One example is the TRB Compendiums 1 through 16 prepared in 1979 (see Appendix A). On many of these low volume road projects, application of investigation standards for high volume roads such as set forth in* FHWA-ED-88-053 *may be impractical or insufficient and not in accord with Geotechnical Policies, or an acceptable level of contractual risk deemed suitable on that specific project.*

When the Geotechnical Professional determines that variance is necessary in lieu of existing geotechnical standards, this determination is shared with the Project Manager for concurrence. The Geotechnical Professional writes to the Project Manager to explain the justification for the variance and how the issues of risk management, quality, and efficiency are addressed. Significant variances are first discussed with the Geotechnical Discipline Leader and/or Division Geotechnical Team Leaders for technical endorsement, and may require endorsement of FLH management.

## 2.4 TECHNICAL GUIDANCE

This manual provides guidance for where standards do not exist and for when it is appropriate to deviate from an existing standard. The TGM presents institutional experience in the form of practices that have worked well in the past on FLH projects and commentary on guidance published elsewhere. The TGM presents "how to" discussion, but does not simply reproduce most of the technical guidance that has been previously published. Rather, the TGM uses extensive links to technical references to direct the reader to additional published and on-line sources of technical guidance.

## 2.5 TECHNICAL REFERENCES

Technical references cited and linked in this manual are classified as either "Primary", or "Secondary". Primary sources either present preferred guidance on how to accomplish a task or, when equal guidance is available through many sources, the Primary source is most widely available. "Secondary" sources are additional documents that are often relied on for FLH work; they present guidance to augment the Primary source. These technical guidance references do not constitute standards unless they are specifically identified as such in the topical subsections of PDDM Chapter 6, where standards are presented. Primary and secondary sources are listed alphabetically in Appendix A and are referenced in the topical sections of this manual.

Tertiary-level references are additional references that are needed less often but are of particular value for certain specific needs. They are listed in the appendices, which also contains the Primary and Secondary sources, and presents all in alphabetical (Appendix A) and topical (Appendix B) listings.

## 2.6 STATE DOT REFERENCES

Geotechnical practice commonly includes regional bias related to regional geology, climate, resource availability, etc. State DOTs have often developed practices based on these regional factors and such experience and practice may be reflected in their published guidelines. On occasion, it is necessary to interface with the state DOT or to design according to their standards as a stakeholder and possibly a maintaining agency for the finished project. Published state DOT geotechnical guidance is listed in Exhibit 2.6–A. Unless specific project criteria direct otherwise, where state DOT guidance differs from FLH guidance presented here and in the TGM, FLH guidance has precedence.

**Exhibit 2.6–A STATE DOT GEOTECHNICAL GUIDANCE**

| State | Geotechnical Guidance |
|---|---|
| Alabama Department of Transportation | Materials and Tests Bureau |
| Alaska Department of Transportation & Public Facility | Manuals |
| California Department of Transportation | Manuals |
| The Colorado Non Point Source Council | Mountain Driveways BMP Manual |
| Connecticut Department of Transportation | Geotechnical Engineering Manual |
| Florida Department of Transportation | Soils and Foundations Handbook |
| Georgia Department of Transportation | Guidelines for Geotechnical Studies |
| Idaho Transportation Department | Materials Manual |
| Illinois Department of Transportation | Geotechnical Documents, Manuals and Procedures |
| Indiana Department of Transportation | Geotechnical Manuals and Guidelines |

| State | Geotechnical Guidance |
|---|---|
| Kentucky Transportation Cabinet | Geotechnical Manual |
| Maryland Department of Transportation | Standard Specifications for Subsurface Explorations |
| Michigan Department of Transportation | Geotechnical Investigation and Analysis Requirements for Structures |
| Montana Department of Transportation | Manuals, Guidelines and Catalogs |
| Nebraska Department of Roads | Geotechnical Policy and Procedures Manual |
| New Mexico State Highway and Transportation Department | Consultant Procedures Manual and Handbook |
| New York State Department of Transportation | Geotechnical Manuals |
| Ohio Department of Transportation | Geotechnical Specifications, Manuals and Reports |
| North Carolina Department of Transportation | Geotechnical Guidelines and Procedures Manual |
| Texas Department of Transportation | Geotechnical Manual |
| Virginia Department of Transportation | Geotechnical Engineering |
| Washington State Department of Transportation | Geotechnical Design Manual |
| West Virginia Department of Transportation | Materials Procedures |

# SECTION 3 GEOTECHNICAL INVESTIGATIONS

This chapter presents FLH guidance on site and subsurface investigation. The guidance has evolved from FLH experience in these areas, and with the Primary and Secondary sources cited.

The primary purpose of a geotechnical investigation is to provide design engineers with knowledge of the subsurface conditions at a specific project site. The investigation should also provide the construction project engineers and contractors with information concerning the materials and conditions that may be encountered in the field. Due to the varying complexity of projects and subsurface conditions, it is not appropriate to establish a rigid format to be followed in conducting geotechnical investigations. However, there is fundamental information that should be obtained and basic steps that should be followed for any project investigation. The collected field data and assessments are the basis for all subsequent engineering decisions and, as such, are of paramount importance to the design and success of a project. Outlining and describing these procedures and steps helps to establish a scope of geotechnical investigation that fulfills the project needs.

The following fundamental information should be obtained during a geotechnical investigation:

- Identification and delineation of existing soil and rock strata.
- Condition and performance of existing transportation structures.
- Qualitative and quantitative information on the character and engineering properties of the soil and rock strata.
- Groundwater levels.
- Slope stability, faults and other geologic hazards or constraints.
- Environmental concerns.

The high cost component of geotechnical site investigations is drilling; therefore, FLH philosophy on site investigations is to maximize the use of geologic interpretations and test pitting (i.e. progress from simpler to more complex) in order to minimize the amount and cost of drilling required - yet still achieve a level of knowledge commensurate with good engineering practice for similar locations/applications. Geotechnical investigations should not be attempted until certain project-specific information has been obtained, as set forth in the following section.

## 3.1 PLANNING AND MANAGEMENT

This section presents guidelines to plan the scope of a geotechnical investigation, including a subsurface exploration and testing program. However, as requirements and conditions will vary with each project, engineering judgment is essential in tailoring the investigation to the specific project. Primary reference is NHI 132031, which provides extensive information on planning and conducting a geotechnical investigation. AASHTO MSI-1 and GEC-5 are important secondary sources. Also refer to NHI 132012. FHWA-ED-88-053 provides valuable guidance for planning using checklist prompts.

### 3.1.1 PROJECT REQUIREMENTS

Coordinate project work with FLH project managers and their delegates to perform services that are cost-effective, timely, and compatible with the size, complexity and goals of the project. The geotechnical professional should facilitate a flow of communication and information with planning and project development. The geotechnical professional should have periodic discussions with the field inspector while the investigation program is ongoing and should notify the project manager or the design engineer of any unusual conditions or difficulties encountered, and any changes made in the investigation program or schedule. The frequency of these communications depends on the critical nature of the project, and on the nature and seriousness of the problems encountered. Refer to NHI 132031.

The first step in performing a geotechnical investigation is a thorough review of the project requirements. Base the reconnaissance on a clear understanding of project goals, objectives, constraints, values, and criteria. The geotechnical professional should facilitate a flow of communication and information with planning and project development. The geotechnical professional should review the following project details and limitations before planning and performing the geotechnical reconnaissance:

- Project location and size.
- Project type (realignment, reconstruction, improvement, bridge, embankment or rehabilitation).
- Project criteria (alignments, potential structure locations, approximate structure loads, probable bridge span lengths and pier locations, and cut and fill area locations).
- Project constraints (context-sensitive design issues as defined by the partner agency, right-of-way, environmental and biological assessments, and permitting).
- Project design and construction schedules and budgets.

Geotechnical investigations should not be attempted until certain project-specific information has been obtained. Exhibit 3.1–A identifies typical project requirements and suggests where the necessary information on specific subjects may be obtained.

The scope and cost of a geotechnical investigation should be adjusted to the size and complexity of the proposed project, the variability of subsurface conditions, and the constraints of project funding and schedule. Geotechnical judgment is essential in developing a subsurface exploration plan that satisfies the requirements of a specific project and allows the geotechnical professional to make reasonable design assumptions. Obtain adequate geotechnical data to provide designers, contractors and construction personnel information on anticipated materials, conditions and potential problems, while minimizing impacts to federal lands. The potential for catastrophic failure and/or failure consequences should be evaluated when establishing the scope of the investigation.

A comprehensive investigation program should start with a series of preliminary office studies, including a study of project objectives and preliminary plans, and review of existing information. Perform a desk review of available geotechnical information as the first step in planning an efficient geotechnical investigation. Exhibit 3.1–A identifies typical project requirements and suggests where the necessary information may be obtained. Determine the location and size of structures, embankments and cuts. Identify potential geotechnical design issues, and formulate a preliminary exploration and testing plan, as well as a list of anticipated analyses and office studies. The exploration program would typically include detailed reconnaissance, test pits and/or conventional borings and other specialized investigative or in situ testing methods. The planning of subsurface explorations also includes identification of appropriate laboratory testing and engineering analyses to support geotechnical design needs for the specific project. Following the preliminary office studies, a field reconnaissance should be performed and modifications made, if necessary, to the exploration and testing plan to provide applicable information in a feasible and cost-effective manner. Early identification of landforms and geologic conditions is used to optimize the subsurface exploration program. Consider the amount of risk that unknown subsurface conditions could bring to the project when planning the number and types of exploration methods. When subsurface conditions appear uniform and competent, it may be possible to reduce the amount of explorations.

**Exhibit 3.1–A POSSIBLE SOURCES FOR SITE SPECIFIC INFORMATION**

| Project Specifics | Information Sources |
|---|---|
| Type of proposed project.<br><br>Proposed project termini.<br><br>Funds available.<br><br>Schedule requirements.<br><br>Items requiring investigation.<br><br>Local authorities to contact.<br><br>Location and type of utilities present. | Planning and Coordination or<br>Project Development Unit |
| Scope of investigation desired.<br><br>Availability of equipment. | Geotechnical Engineer |
| Location of structures. | Project Development and Bridge Unit |
| Site maps and field reference systems. | Project Development and Survey Unit |
| Specific site restriction such as water quality, environmental considerations, or client agency considerations. | Project Development and Environmental Unit |
| Right-of-entry (access) restrictions. | Applicable property owners. |

The amount and type of data obtained during a geotechnical investigation are often constrained by limitations of time, manpower, equipment, access, environmental constraints, or funds. However, an important goal of the investigation program should be to provide sufficient data for the Geotechnical Discipline to recommend the most appropriate and efficient design. Otherwise, more conservative designs with higher factors of safety would be required, which may cost considerably more than a properly conceived exploration program. The investigation program should be of sufficient extent to minimize the possibility of "Changed Conditions" claims and modifications during construction.

Subsurface exploration programs should be conducted using a phased approach. This allows the results from critical design areas, or with the most uncertainty, to be evaluated early in the project. If subsurface information shows materials significantly different from those assumed in the planning stages, modifications could be made to the scope of the investigation. Modification to the scope may include boring depths, number of samples, type of samples, and number and location of explorations.

Coordinate investigation activities with pavement and bridge design engineers to minimize redundancy in explorations. The overall investigation costs can be reduced significantly if, for example, the information for a structure and the roadway centerline can be obtained from a single exploration hole.

Planning geotechnical investigations requires determining the appropriate number, depth, spacing, and type of exploration holes, as well as sampling and testing. Determine the amount of site investigations consistent with FLH standards (PDDM Exhibit 6.3-C and PDDM Exhibit 6.3-D). The FHWA publication FHWA-ED-88-053 that presents geotechnical checklists includes "Table 2" that presents recommended guidelines for planning in terms of exploration spacing and depths. This table is good guidance, is the source of FLH standards, and is reproduced here as Exhibit 3.1–B. The key differences between the FLH standards and the guidance in this table are with respect to reaching locations off the road, such as the face of a retaining wall or sites above and below landslides.

The low volume use of most FLH roads and the environmental impacts of some investigations are the justification for the FLH standards being somewhat different from FHWA-ED-88-053. The risk and cost associated with mischaracterizing the subsurface conditions and having to make contract adjustments while under construction is often more tolerable in these settings. Sometimes even the FLH standard investigation may be inappropriately invasive or expensive and it may be desired to go to bid and construction with even less subsurface in formation. In concept, this would be close to a design-build approach, where it would be expected of the contractor to collect the necessary subsurface information during the construction phase and to include the costs for doing this and the risks of unknowns in their bid.

There are also FLH projects where the level of investigation desired during preliminary engineering exceeds the standards in the PDDM and FHWA-ED-88-053. These are typically settings where the risk of having unanticipated conditions exposed during construction is unusually high. This could be because a certain potential condition could be a fatal flaw to the project, could cause considerable construction delays, perhaps carrying over into additional years because of limited construction windows, etc. In these conditions, the Geotechnical Professional should explain to the Project Manager the value of additional exploration and seek budget and schedule approval.

**Exhibit 3.1–B GUIDELINE "MINIMUM BORING" CRITERIA**

| Geotechnical Feature | Minimum Number of Borings | Minimum Depth of Borings |
|---|---|---|
| Structure Foundation | 1 per substructure unit under 30 m (100 ft) in width<br><br>2 per substructure unit over 30 m (100 ft) in width<br><br>Additional borings in areas of erratic subsurface conditions | Spread footings: 2B where L<2B; 4B where L>2B and interpolate for L between 2B and 4B.<br><br>Deep foundations: 6m (20 ft) below tip elevation or two times maximum pile group dimension, whichever is greater.<br><br>If bedrock is encountered: for piles core 3 m (10 ft) below tip elevation; for shafts core 3D or 2 times maximum shaft group dimension below tip elevation, whichever is greater. |
| Retaining Structures | Borings spaced every 30 to 60 m (100 to 200 ft). Some borings should be at the front of and some in back of the wall face. | Extend borings to depth of 0.75 to 1.5 times wall height. When stratum indicates potential deep stability or settlement problem, extend borings to hard stratum |
| Bridge Approach Embankments over Soft Ground | When approach embankments are to be placed over soft ground, at least one boring should be made at each embankment to determine the problems associated with stability and settlement of the embankment. Typically, test borings taken for the approach embankments are located at the proposed abutment locations to serve a dual function. | Extend borings into competent material and to a depth where added stresses due to embankment load is less than 10% of existing effective overburden stress or 3 m (10 ft) into bedrock if encountered at a shallower depth.<br><br>Additional shallow explorations (hand auger holes) taken at approach embankment locations to determine depth and extent of unsuitable surface soils or topsoil. |
| Centerline Cuts and Embankments | Borings typically spaced every 60 m (200 ft) (erratic conditions) to 120 m (400 ft) (uniform conditions) with at least one boring taken in each separate landform.<br><br>For high cuts and fills, should have a minimum of 3 borings along a line perpendicular to centerline or planned slope face to establish geologic cross-section for analysis. | <u>Cuts</u>: (1) in stable materials extend borings minimum 5 m (15 ft) below depth of cut at the ditch line and, (2) in weak soils extend borings below grade to firm materials or to twice the depth of cut whichever occurs first.<br><br><u>Embankments</u>: Extend borings to a hard stratum or to a depth of twice the embankment height. |

| Geotechnical Feature | Minimum Number of Borings | Minimum Depth of Borings |
| --- | --- | --- |
| Landslides | Minimum 3 borings along a line perpendicular to centerline or planned slope face to establish geologic cross-section for analysis. Number of sections depends on extent of stability problem. For active slide, place at least on boring each above and below sliding area | Extend borings to an elevation below active or potential failure surface and into hard stratum, or to a depth for which failure is unlikely because of geometry of cross-section.<br><br>Slope inclinometers used to locate the depth of an active slide must extend below base of slide. |
| Ground Improvement | Varies widely depending in the ground improvement technique(s) being employed. For more information see FHWA-SA-98-086R. | |
| Material Sites (Borrow sources, Quarries) | Borings spaced every 30 to 60 m (100 to 200 ft). | Extend exploration to base of deposit or to depth required to provide needed quantity. |

Usually the extent of sampling and testing is established as the site investigation progresses in the field. The criteria in Exhibit 3.1–C, which is a reproduction of "Table 2 Continued" in FHWA-ED-88-053, are considered reasonable "guidelines" to follow to produce the "minimum" subsurface data needed to allow cost-effective geotechnical design and construction and to minimize claim problems. FLH standards follow these guidelines with the exceptions that (a) the vane shear device is not often used and (b) FLH standard practice is to photograph all rock core in the core barrel or box, and before it is transported.

The guidelines focus on borings, but this technique may need to be supplemented or substituted by other subsurface exploration methods, such as test pits and hand-auger holes. Optimize the use of field reconnaissance, geologic mapping, geophysical surveys, and simple test pits/test holes (where permissible) to minimize the amount of high-cost site explorations required (such as drilled borings and in situ tests). In addition, borings can be augmented using in situ or geophysical techniques to improve the interpretation of the stratigraphy and subsurface characterization for design. Foreknowledge of geologic conditions will provide more accurate estimates of the number and depth of boring needs. In some cases, the presence of unsuitable materials could necessitate additional explorations.

These guidelines represent the general exploration/testing and analysis requirements for common highway projects and may need to be adapted to the specific requirements of each individual project. The required investigative effort is dependent on the type, significance and complexity of the geotechnical design, as well as the significance of the road and the proposed construction. The frequency of subsurface explorations for low-volume roads could be less than that indicated in Exhibit 3.1–C.

Determine whether specialized in situ testing and/or instrumentation are required to provide parameters for analyses. Sufficient lead time should be allowed since specialized testing and instrument purchases might necessitate outsourcing

In establishing the type and number of laboratory tests to be conducted, the geotechnical professional considers many factors, including: 1) project scope, 2) potential problem soils and variability within project limits, 3) proposed foundation types and magnitude of loads, 4) seismicity, 5) settlement constraints, both total and differential, and 6) height and slope angle of proposed cuts and fills.

**Exhibit 3.1–C GUIDELINE SAMPLINE AND TESTING CRITERIA**

| |
|---|
| Sand or Gravel Soils<br><br>SPT (split-spoon) samples should be taken at 1.5 m (5 ft) intervals or at significant changes in soil strata. Continuous SPT samples are recommended in the top 4.5 m (15 ft) of borings made at locations where spread footings may be placed in natural soils. SPT jar or bag samples should be sent to lab for classification testing and verification of field visual soil identification. |
| Silt or Clay Soils<br><br>SPT and "undisturbed" thin wall tube samples should be taken at 1.5 m (5 ft) intervals or at significant changes in strata. Take alternate SPT and tube samples in same boring or take tube samples in separate undisturbed boring. Tube samples should be sent to lab to allow consolidation testing (for settlement analysis) and strength testing (for slope stability and foundation bearing capacity Analysis). Field vane shear testing is also recommended to obtain in-place shear strength of soft clays, silts and well-rotted peat. |
| Rock<br><br>Continuous cores should be obtained in rock or shales using double or triple tube core barrels. In structural foundation investigations, core a minimum of 3 m (10 ft) into rock to insure it is bedrock and not a boulder. Core samples should be sent to the lab for possible strength testing (unconfined compression) if for foundation investigation. Percent core recovery and RQD value should be determined in field or lab for each core run and recorded on boring log. |
| Groundwater<br><br>Water level encountered during drilling, at completion of boring, and at 24 hours after completion of boring should be recorded on boring log. In low permeability soils such as silts and clays, a false indication of the water level may be obtained when water is used for drilling fluid and adequate time is not permitted after boring completion for the water level to stabilize (more than one week may be required). In such soils a plastic pipe water observation well should be installed to allow monitoring of the water level over a period of time. Seasonal fluctuations of water table should be determined where fluctuation will have significant impact on design or construction (e.g., borrow source, footing excavation, excavation at toe of landslide, etc.). Artesian pressure and seepage zones, if encountered, should also be noted on the boring log. In landslide investigations, slope inclinometer casings can also serve as water observations wells by using "leaky" couplings (either normal aluminum couplings or PVC couplings with small holes drilled through them) and pea gravel backfill. The top 0.3 m (1 ft) or so of the annular space between water observation well pipes and borehole wall should be backfilled with grout, bentonite, or sand-cement mixture to prevent surface water inflow which can cause erroneous groundwater level readings. |

| |
|---|
| Soil Borrow Sources<br><br>Exploration equipment that will allow direct observation and sampling of the subsurface soil layers is most desirable for material site investigations. Such equipment that can consist of backhoes, dozers, or large diameter augers, is preferred for exploration above the water table. Below the water table, SPT borings can be used. SPT samples should be taken at 1.5 m (5 ft) intervals or at significant changes in strata. Samples should be sent to lab for classification testing to verify field visual identification. Groundwater level should be recorded. Observations wells should be installed to monitor water levels where significant seasonal fluctuation is anticipated. |
| Quarry Sites<br><br>Rock coring should be used to explore new quarry sites. Use of double or triple tube core barrels is recommended to maximize core recovery. For riprap source, spacing of fractures should be carefully measured to allow assessment of rock sizes that can be produced by blasting. For aggregate source, the amount and type of joint infilling should be carefully noted. If assessment is made on the basis of an existing quarry site face, it may be necessary to core or use geophysical techniques to verify that nature of rock does not change behind the face or at depth. Core samples should be sent to lab for quality tests to determine suitability for riprap or aggregate. |

Refer to the PDDM Section 6.3.1.1 for standards on investigation tasks.

The primary source supporting investigation standards and guidance is NHI 132031. Secondary sources are AASHTO MSI-1 and GEC-5

Refer to Appendix B.1, Geotechnical Project Planning.

## 3.1.2 TYPICAL PROJECT PRACTICE

### 3.1.2.1 Roadway Alignment Investigations (General Earthwork)

The vast majority of Federal Lands Highway projects require a roadway investigation. The guidelines presented may be applied to all lengths of roadway projects but the frequency of testing and sampling should be adjusted based upon site specific problems and practical engineering judgment. The following describe the basic approach for typical projects.

Soil survey explorations are made along the proposed roadway alignment for the purpose of defining the geotechnical properties of materials. This information is used to:

- Design cut and fill slopes.
- Assess material suitability for embankment construction.
- Define the limits of potential borrow materials.
- Assess the suitability of foundation materials.
- Evaluate settlement or slope stability problems.
- Quantify the depths of topsoil and volumes to be removed.
- Design remedial measures in areas of poor materials.
- Aid the designer of the pavement section.

Explorations could be located along centerline or staggered left and right of the centerline, depending on the locations of maximum cuts and fills as well as the interpreted geology. Exploration holes may be spaced further apart if the project does not have significant earthwork or structures and available information indicates the presence of uniform subsurface conditions. Samples should be of adequate size to permit classification, moisture content testing, gradation testing, and Atterberg limits tests. Undisturbed samples should be obtained for any anticipated strength, consolidation, or other specialized testing needs. Large culverts may require additional exploration. Surface reconnaissance, trenches, or hand auger borings may suffice for smaller structures. Corrosion testing should be performed for each material type for culvert evaluations.

In areas of highly variable soil conditions and in areas of significant cut or fill where stability analysis is anticipated, additional borings may need to be included in the transverse direction to determine the three-dimensional variability of subsurface materials. In situ testing and instrumentation may be necessary to determine shear strength and groundwater levels over time. Where slope stability is a concern, inclinometer instrumentation could be installed during the design phase and used later as a baseline for monitoring slope stability during construction.

Suitable excavation materials should be evaluated for shrink/swell. Unsuitable materials should be identified. Disposal sites for unsuitable or excess material may need to be identified.

Some road alignments may require cut slopes in rock. A geologic reconnaissance is essential to evaluate geologic conditions and rock structure. Consider whether it is necessary to map rock outcrop and geologic structure, obtain oriented core or utilize in-borehole photography to determine the alignment of rock structure. Intact rock strength/hardness can be estimated by point load tests, or determined by performing unconfined compression tests. Design phase tasks could include stereonet analysis, stability analysis, and design of mitigation measures.

Review applicable FHWA checklists (FHWA-ED-88-053) for site investigations (Exhibit 5.1–A), centerline cuts and embankments (Exhibit 5.1–B), embankments over soft ground (Exhibit 5.1–C), and ground improvement (Exhibit 5.1–D).

#### 3.1.2.2 Material Source Investigations

Evaluate the quality and quantity of materials available at existing and prospective sources within the vicinity of a project. These materials could include gravel base, crushed surfacing materials, pavement and concrete aggregates, riprap, wall backfill, borrow excavation, and select backfill materials. In general, existing government-owned or commercial material sources should be used when suitable sites are available within a reasonable haul distance to the project. Projects utilizing government-owned sources that require minimal or no royalty fees may realize substantial savings in material costs. Commercial or contractor provided sources should be used when haul costs or the cost of investigating an existing government source or new source outweigh these savings. The risks associated with development of a new source should also be considered in the economic analysis. These risks include changed condition claims, unanticipated crushing problems, and source development and reclamation issues.

#### 3.1.2.2.1 Evaluation of Existing Government-Owned Material Source

Determine quantities and material properties of existing government-owned sites by obtaining field measurements, collecting grab samples for rock quality testing, and reviewing historic data. Review previous subsurface investigations to verify that sufficient quantity exists for the proposed construction project. The following minimum information should be determined for proposed material sources:

- Expected quality of processed materials and procedures necessary to obtain that quality.
- Boundary limits of proven materials and limits of previously used areas.
- Specific areas and elevation of unsuitable materials.
- Previous uses of material from the source.
- Recommendations on uses and limitations for processed materials.
- Listing of potential development, processing and handling problems that may occur during construction.
- Legal description of the location of the site.
- Regulatory permits.
- Reclamation requirements (if different than existing).

A geotechnical investigation of existing government-owned or leased material sources may be required in order to determine the quality and quantity of materials available for construction projects. Any proposed material source expansion area should be investigated for rock quantity and quality.

#### 3.1.2.2.2 Evaluation of Existing Commercial Material Source

Quantities and material properties at commercial sites can be identified by obtaining statements and test data from the operators (the ultimate verification would be compliance testing during construction). However, advance testing may be prudent if there are limited commercial sources and the impact to the project is great if quantities and material quality do not meet specifications. Commercial sources do not normally require a separate subsurface investigation.

The following minimum information should be determined for proposed material sources:

- Expected quality of processed materials and procedures necessary to obtain that quality.
- Previous uses of material from the source.
- Recommendations on uses and limitations for processed materials.
- Listing of potential processing and handling problems during construction.

#### 3.1.2.2.3 Expansion of Existing Sources and Development of New Material Sources

Any source expansion or new material source site needs to be large enough to meet the quantity and quality requirements of the construction project, with adequate work and storage areas. Sources may be located at large rock cuts within the proposed project or off-site. Review local permit requirements regarding the establishment and operation of material source sites. When developing materials source sites, evaluate reclamation requirements and aesthetic considerations to preserve or enhance the visual quality of the highway and local surroundings. This is especially important along scenic highways and adjacent to residential developments. Exposed sites that cannot be visually reclaimed might not be suitable for development as a material source (refer to project criteria). The development of potential material sources often presents unusual and site-specific problems that require coordination with environmental planning sections as well as the project designer

Determine the stratigraphy for the material source from site geology, borings and test pits. Develop geologic cross-sections to demonstrate the distribution and quality of material available at the site. Identify overburden and waste material encountered in the borings, test results and groundwater levels on the geologic cross-sections. Additional capacity for future projects and maintenance needs should also be considered. Reference NHI 132031 provides guidance that is applicable for the investigation of material sources.

The materials source investigation typically consists of the following elements:

- Determine preliminary rock quality by collecting and testing representative grab samples of the source material.
- Review site geology maps and publications, aerial photographs and contour maps.
- Review FHWA checklist for material source investigation in Exhibit 5.1–E (Section J of FHWA-ED-88-053).
- A reconnaissance level review of the material source site is made to begin the process of developing an understanding of the specific geology at the site. The reconnaissance phase review includes mapping existing outcrops and developing the exploration drilling and sampling program.
- Obtain permits to allow exploration.
- Exploration equipment that allows direct observation and sampling of the subsurface layers is preferred. The equipment can consist of backhoes, bulldozers, large diameter augers, or core drilling methods. For riprap sources, fracture mapping includes careful measurement of the spacing of fractures to assess rock block sizes that can be produced by blasting. Also, identification of the type and amount of joint infilling is noted. If assessment is made on the basis of an existing quarry site face, it may be necessary to core or use geophysical techniques to verify that the nature of the rock does not change behind the face, or at depth. The extent of the exploration will depend on the size of the source area, the amount and type of material needed, and the amount of sampling required to characterize the site and collect representative samples for testing.

- Groundwater levels should be recorded during the site investigation. Where significant seasonal groundwater fluctuation is anticipated, observation wells should be installed to monitor water levels.
- Samples should be obtained to permit classification, moisture, compaction, permeability, and/or corrosion testing of each material type, as applicable.
- Submit representative samples for laboratory testing. Testing will include soil identification and index tests for gravel deposits. The number of tests submitted will vary with the variability of material throughout the site.

### 3.1.2.3 Structure Foundation Investigations

#### 3.1.2.3.1 Bridge Structure Investigations

For widening of existing structures, the total number of exploration holes may be reduced, depending on the quality of information available for the existing structure. Additionally, for projects that include nonredundant piers or deep foundations under water, at least one exploration hole should be performed at each pier location.

All structure borings should include Standard Penetration Testing (SPT) at regular intervals unless other sampling methods and/or testing are being performed. In relatively cohesive soils, undisturbed samples are often obtained to determine shear strengths and material properties (such as moisture contents, unit weight, Atterberg limits, gradation). The borings can sometimes be supplemented with in situ tests such as the pressuremeter if field-developed p-y curves are needed for lateral pile analysis. The groundwater level should be determined.

In situ vane shear tests are recommended where soft clay, peat or other soft or highly organic materials are encountered. Representative undisturbed samples should be obtained in these materials for index testing and possible laboratory shear strength and consolidation testing.

Corrosion tests are required on all new bridge projects. As a minimum, one test should be performed on each soil and rock type and also a separate test on a water sample. In the case of a water crossing, samples of streambed materials and each underlying stratum should be obtained for determination of the median particle diameter, D50, for scour analysis.

Review FHWA checklists for structure foundations in Exhibit 5.1–F, Exhibit 5.1–G, and Exhibit 5.1–H (Sections F, G, and H of FHWA-ED-88-053).

#### 3.1.2.3.2 Retaining Structure Investigations

The following types of retaining wall structures will likely have investigation requirements beyond those stated herein: gravity and semi-gravity walls, cantilevered walls, MSE walls, and soil nailed and anchored walls. The Geotechnical Discipline should establish additional requirements prior to development of the investigation program. In the case of soil nailed and anchored walls, samples should be collected for testing throughout the zone that anchors/nails will be constructed. Provide allowable bearing pressure (Exhibit 4.3–B) and pile capacity (Exhibit 4.3–C), as applicable.

Review FHWA checklist exhibits (based on FHWA-ED-88-053) for retaining structures (Exhibit 5.1–I), as well as the appropriate structure foundation checklist (spread footings (Exhibit 5.1–F), driven piles (Exhibit 5.1–G) or drilled shafts (Exhibit 5.1–H).

#### 3.1.2.3.3 Buildings

In general, one boring should be made at each corner and one in the center. This may be reduced for small buildings. For extremely large buildings or highly variable site conditions, one boring should be taken at each support location. Refer to building foundation manuals and text books for additional guidance in planning geotechnical investigations (details are not extensive in this manual since buildings are not commonly designed by Federal Lands Highway). Refer to chapter 17 of WSDOT WA-M-46-03.

#### 3.1.2.3.4 Tunnels

Due to the extreme variability of conditions under which tunnels are constructed, investigation criteria for tunnels should be established for each project on an individual basis. Refer to tunneling texts for detailed guidance, or consult with an expert in tunneling, including the FHWA tunneling expert at the headquarters' office. Refer to FHWA-IF-05-023.

#### 3.1.2.3.5 Mast Arm Supports (Signals, Message Signs)

Standard foundations for sign bridges, cantilever signs, cantilever signals and strain pole standards are based on allowable lateral bearing pressure and angle of internal friction of the foundation soils. The determination of these values can be estimated by Standard Penetration Test (SPT). One boring should be made at each designated location. Borings should extend 15 m (50 ft) into suitable soil or 1.5 m (5 ft) into competent rock. Deeper borings may be required for posts with higher torsional loads or if large boulders are anticipated. Other criteria are the same as for bridges. Refer to AASHTO HB-17 and chapter 17 of WSDOT WA-M-46-03.

### 3.1.2.4 Landslides and Mitigation Measures

There are unique tasks required for landslide investigation and design of mitigations. To design a landslide remediation, the size and depth of the slide should be determined. Inclinometers and piezometers should be installed to accurately define the depth of movement and existing piezometric levels acting on the shear zone. When monitored over several months or years this instrumentation can be very valuable in determining the behavior of the landslide and the relationship between periods of active slide movement and seasonal groundwater levels.

As a minimum, two instrumented borings should be drilled along the cross-section (axis of movement) of the slide. Larger slides will usually require four or more borings to adequately define the failure shear zone. Borings should extend through the full depth of landslide material, terminating at least 5 m (15 ft) into underlying stable material. Generally, the boring depths for at least one or two borings should be made even deeper to verify an accurate interpretation of the depth of the failure and to identify any underlying zones of weakness that could affect the mitigation design. Shallow slides (less than 6 m (20 ft) deep) can sometimes be effectively evaluated using test pits or trenches, which can expose and allow positive identification of the failure shear zone, its shape and inclination.

It is essential that survey information extend beyond the landslide limits to provide sufficient data for stability analysis. Monitoring slide movement can be augmented with a line of survey hubs, referred to as a tagline. The results can help define the type of slide, rate of movement, changes in the slide limits and areas of greatest activity. The vector sums can be plotted and used to interpret the possible shape of the failure surface.

Piezometer instrumentation should be installed at appropriate depths to accurately record specific groundwater heads that act on the failure shear zone. Placement of piezometers at specific target areas demands an understanding of the slide geometry, which may require a second mobilization once the inclinometers have shown the actual depth of movement. Simply increasing the depth range covered by the slotted portion of an observation well will generally not provide good results and the water level readings will tend to be ambiguous and unusable. This can be avoided if the slotted zone is more targeted and controlled by appropriate seals or if vibrating wire piezometers are used.

Laboratory work could include the following tests: standard index, unit weights, Atterberg limits, shear strength (triaxial undrained peak shear tests on overburden materials, and ring shear or repeated direct shear to determine the residual strength along the slide failure zone).

Review Exhibit 5.1–J that presents the FHWA checklist for landslide correction (Table 4 and Section D of FHWA-ED-88-053).

### 3.1.2.5 Pavement Subgrade Investigations

FLH guidance to be drafted.

Refer to PDDM Section 6.3.1.2.5 for standards on investigation tasks.

The primary source supporting investigation standards and guidance is NHI 132031. Secondary sources are AASHTO MSI-1 and GEC-5.

Refer to Appendix B.1, Geotechnical Project Planning.

## 3.1.3 SAFETY

The nature of the equipment used and climatic conditions often present potential hazards that should be evaluated on a site-specific basis. It is the responsibility of the geotechnical professional, as well as field crew members to adjust the investigation program and/or provide equipment, training, and other means to provide safe working conditions. It may be advisable to prepare a unique Safety Plan for each project to provide guidance for field staff, which could include unique safety practices, emergency contact information and considerations for first aid in the event of an injury.

Safety guidance for drillers is described in NDA and BOR Drillers Safety. Additional guidance is described in NHI 132031 and USACE EM 1110-1-1804. Field crews should evaluate if traffic safety control is necessary. Refer to MUTCD requirements. The U.S. Department of Labor's Construction Safety and Health Regulations, OSHA Section 29, as well as regulations of any other governing agency should be reviewed and followed prior to excavation of exploration test pits, particularly in regard to shoring requirements.

All field personnel, including geologists, engineers, technicians and drill crews should be familiar with the general health and safety procedures as well as any additional safety requirements of the project or governing agency.

Minimum protective gear for all field personnel should include hard hat, safety boots, eye protection, and gloves.

The Underground Services Alert (1-800-227-2600) (USADIG) or local utility locate services should be called a minimum of two working days (preferably four days) prior to conducting subsurface explorations. Review the proposed boring locations following utility locations to determine if any borings need to be relocated to avoid buried utilities. The presence of utilities may need to be rechecked for the adjusted boring locations.

Unknown or unexpected environmental problems may be encountered during a site investigation. For example, discolored soils or rock fragments from prior spills, or contaminated groundwater may be detected. The geotechnical professional and the field supervisor should attempt to identify possible contamination sources prior to initiating fieldwork. Based on this evaluation, a decision should be made whether a site safety plan should be prepared. Environmental problems can adversely affect investigation schedules and cost, and may require obtaining permits from State or Federal agencies prior to drilling or sampling.

Refer to PDDM Section 6.3.1.3 for standards on safety.

The primary sources supporting safety standards and guidance are NDA for drilling and MUTCD for traffic. Secondary sources are BOR Drillers Safety, USACE EM 1110-1-1804, and FHWA-CFL-04-002.

Refer to Appendix B.1, Geotechnical Project Planning.

## 3.2 METHODS AND PRACTICE

The following sections describe the methods used to complete the planned investigation tasks.

### 3.2.1 PRELIMINARY STUDY AND RECONNAISSANCE

For the project definition phase, the geotechnical recommendations provided will be at the conceptual/feasibility level. Perform the preliminary study and reconnaissance to the extent necessary to disclose the probable materials and conditions to be encountered, as well as potential hazards, risks and uncertainties. The investigation for this phase usually consists of a field reconnaissance by the geotechnical professional and a review of existing records, aerial photographs and geologic maps. For projects that lack significant geotechnical information or are complex, some soil borings may be drilled at critical locations for development of the project definition. A key role of the geotechnical professional in this stage of a project is to identify potential fatal flaws with the project, potential constructability issues, and geotechnical hazards such as earthquake sources and faults, liquefaction, landslides, rockfall, and soft ground. Additional information includes drainage issues, unsuitable soils, material sources, and preliminary structure types and foundation options.

In the early stages of a project the geotechnical professional may be requested to perform an evaluation of several possible roadway alignments or structure locations. The purpose of this effort is to identify geologic conditions or constraints that could affect the selection decision. This project phase generally does not require subsurface explorations. It is normally limited to preliminary office studies and field reconnaissance.

Route selection typically depends on information from office studies and the field reconnaissance findings, as well as historic information. Where time is available, some geotechnical issues may benefit from an extended period of instrumentation and monitoring to measure critical geotechnical parameters, such as fluctuations of groundwater levels and, in the case of slope stability, the location and shape of the landslide failure surface. A properly conducted study of alignment options can potentially result in significant cost savings, especially if there is flexibility given to the designers to locate the new roadway and structures in the most geotechnically favorable locations.

The geotechnical professional should provide conceptual hazard avoidance or mitigation plans to address all the identified geotechnical issues. An assessment of potential geotechnical impacts on construction staging and overall constructability should be performed. An initial scope of future geotechnical services is determined at this time.

Refer to PDDM Section 6.3.2.1 for standards on preliminary study and reconnaissance.

The primary supporting sources are NHI 132031 for office and field work, and FHWA-ED-88-053 for reporting. Secondary sources are AASHTO MSI-1 and USACE EM 1110-1-1804

Refer to Appendix B.2, Site Investigation.

### 3.2.1.1 Office Review of Available Data

After gaining a thorough understanding of the project requirements, all relevant available information on the project site should be collected and reviewed. Available data may consist of reports, maps, journal articles, aerial photographs, previous as-built plans and communication with individuals with local knowledge. Review of this information can provide a basis for understanding the geology, topography, and geomorphology of the area. An initial understanding of the engineering properties of subsurface materials and groundwater characteristics can often be obtained from available data. The wide range of geographical areas where projects may be located requires access to geotechnical information from a variety of sources. Exhibit 3.2–A provides an initial listing of potential sources of regional geotechnical information and a brief description of information available. Each geotechnical unit should supplement the sources listed in the exhibit by establishing and maintaining a file of commonly used regional information. Refer to NHI 132031 for further guidance.

**Exhibit 3.2–A SOURCES OF REGIONAL GEOTECHNICAL INFORMATION**

| Source | Type of Information | Description |
|---|---|---|
| USGS<br>*Index of Publications*<br>Superintendent of Documents<br>US Government Printing Office<br>Washington, DC 20402 | • geologic maps<br>• water supply papers<br>• bulletins<br>• professional papers<br>• circulars<br>• annual reports<br>• monographs | General physical geology emphasizing all aspects of earth sciences, including mineral and petroleum resources, hydrology and seismicity, and groundwater resources. |
| USDOI<br>Geological Survey<br>National Center<br>12201 Sunrise Valley Drive<br>Reston, VA 22092 | • index maps<br>• quadrangle maps<br>• topographic maps | Maps of each State showing coverage and sources of published geological maps.<br>Maps that support the older geological folios including aerial and bedrock maps.<br>Contour maps for all States. |
| Geological Society of America<br>P. O. Box 9140<br>3300 Penrose Place<br>Boulder, CO 80302 | • monthly bulletins<br>• special papers<br>• memoirs<br>• geological maps | Specialized geological subjects and intensive investigations of local geology.<br>Maps of glacial deposits and Pleistocene Aeolian deposits. |
| USDA Soil Conservation Service<br>*List of Published Soil Surveys*<br>Washington, DC | • Soil Maps and Reports | Surveys of surface soils described in agriculture terms. Physical geology is summarized. |
| State Geological Surveys/ Geologist Offices | • Geological Maps and Reports | Maps/reports covering specific areas or features in the publications of the State geologist. |

### 3.2.1.2 Field Reconnaissance

The geotechnical professional should perform a field reconnaissance to evaluate the geographic, topographic, geologic and geotechnical issues and hazards along the subject route(s). As part of the reconnaissance, key site locations and conditions should be photographed and documented. Assess existing cut and fill slopes for indications of stable slope angles and potential problems. Identify geologic conditions that may tend to adversely affect project development plans, such as landslides, faults, springs, rockfall, and erosion. A site plan, large-scale topographic map, or Quadrangle map of the project area is necessary for field mapping. Note any features that may assist in the concepts evaluation. For rock slopes, performance of slopes and rockfall history are important indicators of how a new slope in the same material will perform. Inspect structures to determine foundation performance and whether damage has occurred. More detailed guidance on performing reconnaissance and surface investigations is described in NHI 132031. Secondary references include AASHTO MSI-1 and USACE EM 1110-1-1804.

A reconnaissance should be performed only after an understanding of the project requirements has been reached, a review of the existing data has been completed, and applicable right-of-entry permit(s) have been obtained. The final objective is to brief the project team on the key issues that will influence project design.

Obtain pertinent project information (project development documents) and other conceptual information from the project designer before performing the site visit. As part of the reconnaissance, key site locations and conditions, and exploration equipment access routes should be photographed and documented.

The following factors should be defined by the field reconnaissance:

- Stratigraphy – Compare stratigraphy to information obtained from available data.
- Key Outcrops – Identify outcrops or exposures that warrant further investigation.
- Existing Slopes – Assess the stability of major slope-forming geologic units. Natural slopes and any existing soil or rock slope failures should be evaluated and documented. Cut slope angles and orientations should be measured and their relative performance evaluated.
- Groundwater and Surficial Water – Estimate the general nature of surface water and groundwater regimes at the project site from surface evidence, such as drainage channels and springs. Develop concepts for future investigations.
- Geologic Constraints – Identify geologic conditions that may tend to adversely affect project development plans (landslides, faults, flooding, erosion, etc.). Devise methods of investigating the degree of potential impact.
- Environmental Considerations – Identify potential impacts the project may have on subsurface materials, landforms, and the surrounding area. Determine if project areas are governed by special regulations or have protected status.

Prior to performing any fieldwork, the geotechnical professional may need to obtain Entry Permits through the Right-of-Way office.

It is necessary for the geotechnical professional to perform a field reconnaissance to develop an appreciation of the topographic, geologic and geotechnical concerns at the project site and become knowledgeable of access and working conditions. A reconnaissance should be performed only after an understanding of the project requirements has been reached, a review of the existing data has been completed, and applicable right-of-entry permit(s) have been obtained. The final objective is to brief the project team on the key issues that will influence project design.

Obtain pertinent project information (project development documents) and other conceptual information from the project designer before performing the site visit. As part of the reconnaissance, key site locations and conditions, and exploration equipment access routes should be photographed and documented. Guidelines are provided in NHI 132031. Exhibit 3.2–B provides a form that could be used to gather and document this preliminary information.

The following factors should be defined by the field reconnaissance, in addition to those listed in Section 3:

- Key Outcrops – Delineate outcrops or exposures that warrant structural mapping.
- Explorations – Determine the type(s) of exploration and the kinds of samples that would best accomplish the project needs.
- Drilling Logistics – Define the type, approximate locations and depths of geotechnical borings. Determine approximate routes of access to each drilling location. Make note of any feature that may affect the boring program, such as accessibility, structures, overhead utilities, evidence of buried utilities, or property restrictions. Evaluate potential water sources for use during drilling operations. Evaluate potential concerns that may need to be addressed while planning an exploration program (permits, overhead utilities, equipment security, private property, etc.). If possible, exploration locations should be located with a field supervisor. If this is not possible, a field supervisor should be consulted regarding borehole location feasibility.
- Environmental Considerations – Identify potential impacts the exploration program may have on subsurface materials, landforms, and the surrounding area. Determine the need for low-impact methods such as track-mounted drilling equipment, and methods that reduce access disturbance (cranes and helicopters).
- Permits – Determine the various types of permits that may be required. Consider all applicable jurisdictions, which could include partner agencies, state DOT's, regulatory agencies, and local government agencies. Permits could include right-of-entry, well permits, special use permits, utility clearances, etc.

### 3.2.1.3 Preliminary Study and Reconnaissance Reporting

The findings of the reconnaissance are further evaluated in the office and discussed with Planning and Roadway Design staff. Evaluate geotechnical issues that were identified during the reconnaissance and identify conceptual mitigation options. The studies should include an assessment of risk and uncertainty associated with each of the feasible options. Prepare a memorandum to summarize the initial data and reconnaissance findings and to present preliminary options and recommendations for project development. Also included in the memorandum should be an initial workscope for geotechnical investigations to support project development, particularly the potential design options. Guidelines on preparing reports and memoranda are described in Section 5. Checklists for evaluating the report contents and completeness of investigations are also presented.

## Exhibit 3.2–B PRELIMINARY GEOTECHNICAL INVESTIGATION FORM

### Preliminary Information for Geotechnical Investigations

Project: ______________________ Date: ______________________

Account Number: ______________________ Estimated quantity needed: ______________________

Funding: ______________________ Information needed by: ______________________

Type of Investigation? Structure Foundation Roadway Slope Analysis Materials Source Landslide Other ________

Report Type? Preliminary Final Informal Formal

**Site Specific Information**

Location: ______________________

Termini: ______________________ To: ______________________

Field Reference Available (stakes, MP, etc.): ______________________

Terrain/Access? Easy Moderate Difficult Very Difficult

Utilities? Water Electric Telephone Unknown

Local Contacts:

Agency Name: ______________________ Property Owner: ______________________

Address: ______________________ Address: ______________________

Telephone: ______________________ Telephone: ______________________

**Additional Information Needed By Geotechnical**

Mapping? Not Available Availability Date: ______________________

*Structure Foundation Projects*

Structure Type: ______________________ Bridge Spans (No. & Length) ______________________

Max. Wall Height: ______________ Max. Loads Expected: ______________________

Availability of Preliminary Plans: ______________________

Restrictions: ______________________

Comments: ______________________

*Roadway Projects*

Type? Overlay Widening Reconst. New Alignment Other

Pavement Surface Type: ______________________

Traffic Data Availability: Where? ______________ When? ______________

Restrictions: ______________________

Comments: ______________________

*Material Source Projects*

Use of Material? A.C. Pavement Base Borrow Other __________

Amount Needed: ______ ______ ______ ______
($m^3$) ($ft^3$)

Suggested Source: ______________________

Previous Use: ______________________

*Slope/Landslide Projects:*

History/Maint. Problems: ______________________

Estimated Max. Movement Per Year: ______________________

Previous Correction Attempts: ______________________

Initial Correction Concepts: ______________________

*Estimated Number of Holes:*

Depth: ______________________ Backhoe or Dozer work required? ______________________

Is Water Available? ______________________ How Far? ______________________

Estimated Conditions: ______________________

## 3.2.2 SURFACE EXPLORATION METHODS

There are three main surface exploration methods used by the FLH Geotechnical Discipline: 1) field reconnaissance, 2) geologic mapping, and 3) field developed cross-sections. Guidance is available in NHI 132031. Secondary references include NHI 132035 and AASHTO MSI-1.

> Refer to PDDM Section 6.3.2.2 for standards on surface exploration methods.
>
> The primary source supporting the standards and guidance is NHI 132031. Secondary sources are AASHTO MSI-1 and NHI 132035.
>
> Refer to Appendix B.2, Site Investigation.

### 3.2.2.1 Geologic Field Mapping and Measurements of Rock Discontinuities

Field mapping should begin by observing road cuts, drainage courses, and bank exposures. A site plan or large-scale topographic map of the project area is essential for field mapping. The main objective of these observations is to confirm the general types of soil and rock present and topographic and slope features. Note any features that may assist in the engineering analysis, such as the angle and performance of existing slopes, or the stability of open excavations or trenches. Note the type and condition of vegetation, which may give an indication of ground and surface water regimes, as well as an indication of landslide or slope stability concerns.

Inspect structures to ascertain their foundation performance and their susceptibility to damage from construction-related ground vibrations or settlement due to embankment placement.

For rock slopes, performance of slopes and the rockfall history are important indicators of how a new slope in the same material will perform. More detailed rock structural mapping entails observing and measuring lithologic contacts and the engineering characteristics and orientation of rock discontinuities that make up the rock mass. A more detailed discussion on rock structural mapping is given in NHI 132035. AASHTO MSI-1 describes the procedures for engineering geological mapping. It also provides suggestions for preparing geologic maps for different applications, such as project area geologic maps, R-O-W geologic maps, file geologic maps, site geologic maps, and other special mapping. Exhibit 3.2–C presents a FLH form for documenting observed rock structure data. Exhibit 3.2–D presents a stereoplot form for analysis of rock structure. This data is often used in computer programs that can plot them on a stereonet and in some cases perform kinematic analyses.

### 3.2.2.2 Field-Developed Cross-Sections

Field-developed cross-sections are applicable to nearly all types of site-specific geotechnical investigations. Their use can be applied to excavation and placement of materials; foundations and slopes; specific development of borrow and aggregate resources; and for the graphic portrayal and analysis of significant features related to slope stability, seismicity, drainage, or other characteristics.

## Exhibit 3.2–C FIELD MAPPING – ROCK STRUCTURES *

### Field Mapping – Rock Structures*

1. Location, orientation, and number of planes are numeric. Data units are alphabetic and/or numeric. All other information is alphabetic.
2. Surface type, line type and rock type are three letter codes. Infilling water, form, roughness, and termination are one letter codes.
3. Record all codes and their full proper descriptions on a reference chart.
4. Record position within the data unit or traverse under location. Each data unit should include data from within one structural unit only.
5. Thickness, spacing, and length are entered according to the size notation given below.

| Traverse (Data Unit) Information | | | | | | |
|---|---|---|---|---|---|---|
| | Data Unit | Northing | Easting | Elevation | Inclination | |
| | Bearing | Length | No. Points | Structural Unit | Formation | Declination | Observer |

Remarks: ______________________________________________

______________________________________________

**Size Notations** (A through I mm, J through S m)

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| A | <0.26 | E | 12.5–25 | J | 0.3–0.6 | O | 9.0–18.0 |
| B | 0.25 | F | 25–50 | K | 0.6–1.2 | P | 18.0–30.0 |
| C | 0.25–6.5 | G | 50–100 | L | 1.2–2.4 | Q | 30.0–60.0 |
| D | 6.5–12.5 | H | 100–200 | M | 2.4–4.5 | R | 60.0–120.0 |
| | | I | 200–300 | N | 4.5–9.0 | S | >120.0 |

| Surface type | Infilling | Water | Form | Roughness | Term |
|---|---|---|---|---|---|
| C – Contact | A – Clay | W – Wet | P – Planar | V – Very rough(JRC=25) | 0 – neither end visible |
| F – Fault | F – Iron Metals | D – Dry | C – Curved | R – Rough(JRC=15) | 1 – one end visible |
| S – Shear | W – Calcite | M – Moist | U – Undulating | S – Smooth(JRC=5) | 2 – both ends visible |
| J – Joint | K – Chlorite | | S – Stepped | P – Polished(JRC=0) | |
| B – Bedding | Q – Quartz | | I – Irregular | | |
| L – Schistosity or Foliation | P – Pyrite | | | | |
| V – Vein | | | | | |

| Data Unit | | | | Infillings | | | Thick | Water | Form | Rough | Spec | No. of Planes | Length | Term | Job number | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Location | Surface | Orientation | | | | | | | | | | | | | Line | Orientation | | Rock Type |
| | Type | Dip | Direction | 1 | 2 | 3 | | | | | | | | | Type | Dip | Direction | |
| | | | | | | | | | | | | | | | | | | |
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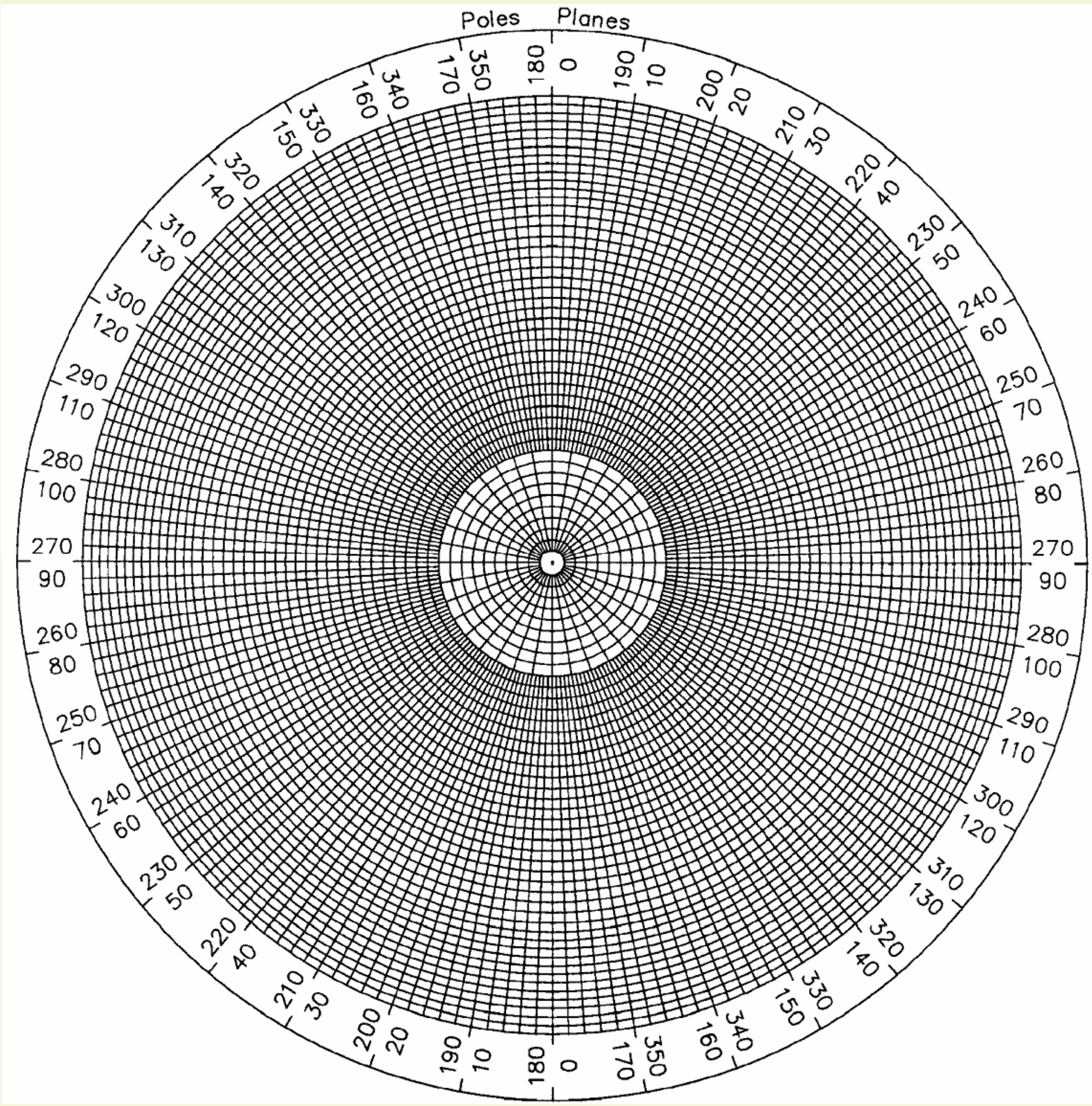
*Detailed instruction in Rock Slopes Design, Excavation, & Stabilization (FHWA-TS-89-045)

**Exhibit 3.2–D FIELD MAPPING – ROCK STRUCTURES STEREOPLOT**

Project ______________________________________________________________

Location _____________________________________________________________

Plotted by: ______________________________________ Date: ________________




Note: plot for data collected from field mapping of the rock structure.

Measurements should include all slope breaks and other identifiable, geological features such as landslide cracks and groundwater features. Describe the significance of each feature in the field notebook. Since slope breaks can result from characteristics of the subsurface material changes, the slope breaks could represent contacts between different soil and / or rock units. Measurements of the contact orientation (strike, dip and surface trace) should be denoted where appropriate in the field notebook.

Include the interpretations of the surface and subsurface materials and relationships on the section along with relevant estimates of engineering parameters. The section should show the distribution of soil and rock units, estimated location/elevation(s) of surface and subsurface water, and original ground lines prior to any previous excavation, filling or slope movements. As these interpretations are developed, plan any explorations that may be needed to confirm the subsurface model that will be used in the analysis and design phase.

For further guidance on field-developed cross sections, refer to USFS EM 7170-13 (slope investigation manual).

### 3.2.3 SUBSURFACE EXPLORATION METHODS

Once right-of-entry and utility clearances have been obtained, field explorations can begin. Many methods of field explorations exist. The subsections below address equipment and safety and contain brief descriptions of the most common methods. Perform explorations, materials sampling, testing and instrumentation consistent with applicable standards. The primary recommended reference is NHI 132031. Secondary and general references include GEC-5, FHWA-ED-88-053, NHI 132035; USACE EM 1110-1-1804; and AASHTO MSI-1.

#### 3.2.3.1 Geotechnical Equipment

Sometimes to perform geotechnical investigations, specialized subsurface investigation equipment is required. In cases of sporadic use or when highly trained technicians must be assigned exclusively to operate equipment, the geotechnical unit may prefer to use consultants and/or contractors to provide such services in lieu of actually purchasing and maintaining such equipment. Below are typical sources for technical assistance to obtain equipment or expertise:

- Other FLH Offices.
- FHWA Research and Implementation units.
- Local government agencies.
- Other Federal and State Government agencies.
- Universities.
- Private consultant.
- Equipment manufacturers.

Exploration equipment includes various types of drill rigs (hollow stem auger, and large and small rotary core drills), geophysical (including seismic and refraction), in situ test devices, instrumentation and backhoe test pitting. Exhibit 3.2–E provides guidelines for the type of equipment and the frequency of use that is typical for different types of FLH geotechnical investigations. The standard approach and considerations used for selecting exploration methods is in PDDM Exhibit 6.3-A (borings) and PDDM Exhibit 6.3-B (probes, test pits, trenches and shafts).

**Exhibit 3.2–E GENERAL INVESTIGATION EQUIPMENT REQUIREMENTS**

| Type of Investigation | Use by Equipment Type | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Hollow Stem Auger Drill | Large Rotary Core Drill | Small Rotary Core Drill | Seismic | Resistivity | In situ Strength Devices | In situ Monitors | Back-hoe |
| **Roadway Soils** | 1 | 3 | 4 | 2 | 3 | 3 | 4 | 1 |
| **Foundations** | 1 | 1 | 2 | 2 | 3 | 2 | 2 | 3 |
| **Material Sources** | 2 | 1 | 2 | 2 | 3 | 4 | 3 | 1 |
| **Landslides** | 1 | 1 | 2 | 3 | 3 | 2 | 1 | 2 |
| **Cut slopes** | 1 | 1 | 2 | 2 | 3 | 3 | 2 | 1 |

Use Code:
1 = Frequently 2 = Occasional 3 = Seldom 4 = Usually Inappropriate

### 3.2.3.2 Geophysical Methods

Geophysical methods are used to gather information on the geological subsurface features. Generally, geophysical methods are used as a reconnaissance investigation to cover large areas and/or to supplement information between bore holes. These exploration techniques are most useful for extending the interpretation of subsurface conditions beyond what is determined from small diameter borings. The methods given in Exhibit 3.2–F are some of the most common. This exhibit and FHWA-Geophysical should be considered to determine when geophysical testing may provide an economical means of obtaining information.

Many benefits may be obtained by use of geophysical tests, but specific procedures and limitations of the testing methods should always be considered. A limitation of these techniques is that no samples are recovered. It must be emphasized that geophysical methods might not be successful in all situations and should be carefully evaluated to determine whether any methods are appropriate for the specific project requirements and site conditions. For detailed guidance, refer to FHWA-Geophysical. Additional references include NHI 132031 and USACE EM 1110-1-1802.

The reliability of geophysical results can be limited by several factors, including the presence of groundwater, nonhomogeneity of soil stratum thickness, gradation or density, and the range of wave velocities within a particular stratum. Subsurface strata that have similar physical properties can be difficult to distinguish with geophysical methods. Because of these limitations, for most design applications, geophysics should be considered a secondary exploration method to drilling, and should generally be accompanied by conventional borings. An experienced professional should interpret the field data.)

**Exhibit 3.2–F GUIDELINES FOR USING GEOPHYSICAL METHODS**

| Method | Basic Properties and Measurements | Applications | Frequency of Use |
|---|---|---|---|
| Electrical Resistivity<br><br>ASTM D6432 | Electrical conductivity of subsurface materials as measured by apparent resistance. | Identify layers of less competent material lying below more competent layers. Interpolate surface condition between bore holes | Rare |
| Seismic Refraction<br><br>ASTM D5777 | Density and elasticity of subsurface material as measured by velocities of compression waves. | Estimate depth of more competent materials underlying less competent material. Interpolate subsurface condition between bore holes. Estimate Rippability. | Common |
| Ground Penetrating Radar (GPR)<br><br>ASTM D6432 | Analyzes reflection of radar signals transmitted into ground by low frequency antenna. | Provide profile and subsurface material interfaces and location of subsurface objects. Depth limited in finer grained soils. | Occasional |

Refer to PDDM Section 6.3.2.3.2 for standards on geophysical methods.

The primary source supporting the guidance is FHWA-Geophysical. Secondary sources are NHI 132031 and USACE EM 1110-1-1802.

Refer to Appendix B.2, Site Investigation.

### 3.2.3.3 Drilling and Soil Sampling

Exploration, materials sampling, test, and instrumentation procedures and documentation of results shall comply with the following applicable standards.

- American Association of State Highway and Transportation Officials (AASHTO).
- American Society for Testing and Materials (ASTM).
- Other recognized methods as approved by the Federal Lands Highway Division.

Methods for exploratory borings should be in accordance with AASHTO and ASTM whenever these standards satisfy FLH policies as described in the TGM and in Chapter 6 of the PDDM. When deviations are required they should be as minimal as possible to satisfy FLH policies and they should be documented. Sometimes state DOTs and/or local partners have practices that can be followed so that the data collected are consistent with data collected on nearby projects. Detailed information on drilling and sampling methods is given in NHI 132031 which lists applicable AASHTO and ASTM drilling and sampling specifications and test methods. Additional references include AASHTO MSI-1, GEC-5, FHWA-ED-88-053, NHI 132012, NHI 132035, USACE EM 1110-1-1804, USACE EM 1110-1-1906, and FHWA-FL-91-002.

Current copies of these standards and manuals should be maintained in the Division for ready reference. The geotechnical professional and field inspector should be thoroughly familiar with the contents of these documents, and should consult them whenever unusual subsurface situations arise during the field investigation.

Borings can be advanced using a number of methods. The purpose of a drilling and sampling program is to log subsurface conditions and obtain samples that reasonably represent subsurface conditions over the entire project site. Guidance for selection of the standard boring types used by FLH is in PDDM Exhibit 6.3-A.

Boring and sampling type and frequency is dependent upon both the type of material encountered and the purpose of the investigation. Guidelines on boring and sampling frequency are provided in Exhibit 3.1–B and Exhibit 3.1–C (Table 2 of FHWA-ED-88-053).

Soil samples obtained for engineering testing and analysis, in general, are of two main categories: 1) Disturbed (but representative), and 2) Undisturbed. Refer to GEC-5.

#### 3.2.3.3.1 Disturbed Samples

Disturbed samples are those obtained using equipment that destroy the in situ structure of the soil but do not alter its mineralogical composition. Specimens from these samples can be used for determining the general lithology of soil deposits, for identification of soil components and general classification purposes, for determining grain size, Atterberg limits, and compaction characteristics of soils. The most commonly used in-situ test for surface investigations is the Standard Penetration Test (SPT), AASHTO T 206.

Some common problems or procedural errors that can provide misleading SPT results are given in Exhibit 3.2–G. Refer to GEC-5 for additional guidance on potential errors with SPT testing. The use of automatic hammers for SPT is recommended if standard drop height and hammer weight can be maintained. SPT values obtained with automatic hammers should be calibrated by field comparisons with standard drop hammer methods. All SPT values should be corrected for overburden pressure. The Bazaraa method as given in NHI 132012 is often used for FLH projects.

#### 3.2.3.3.2 Undisturbed Samples

Undisturbed samples are obtained in fine-grained soil strata for use in laboratory testing to determine the engineering properties of those soils. It should be noted that the term “undisturbed” soil sample refers to the relative degree of disturbance to the soil’s in-situ properties. Undisturbed samples are obtained with specialized equipment designed to minimize the disturbance to the in-situ structure and moisture content of the soils. Specimens obtained by undisturbed sampling methods are used to determine the strength, stratification, permeability, density, consolidation, dynamic properties, and other engineering characteristics of soils.

Disturbed and undisturbed samples can be obtained with a number of different sampling devices as summarized in Table 3-4 of NHI 132031. Detailed information on sampling methods and equipment including appropriate application and limitations of each, are given in the NHI manual.

Refer to PDDM Section 6.3.2.3.3 for standards on drilling and sampling.

The primary source supporting the standards and guidance is NHI 132031. Secondary sources are AASHTO MSI-1 and GEC-5.

Refer to Appendix B.2, Site Investigation.

### 3.2.3.4 Rock Coring

Rock samples can be obtained from outcrops, test pits, or rock cores through drilling operations. Samples obtained from outcrops or test pits are termed "grab samples". Typically, the sample sizes should be small enough to carry, but large enough to be tested in a point load device or utilized as hand specimens. These samples should be labeled, and the location where they were obtained should be identified on a site map. Refer to GEC-5.

Rock cores are obtained using core barrels equipped with diamond or tungsten carbide tipped bits. There are three basic types of core barrels: 1) single tube, 2) double tube, and 3) triple tube. Because single tube core barrels generally provide poor recovery rates, their use is not recommended. Double and triple tube core barrel systems are preferred. To protect the integrity of the core from damage (minimize extraneous core breaks), a hydraulic ram should be used to expel the core from the core barrel. Refer to AASHTO T 225 and ASTM D 2113.

Rock cores should be photographed in color as soon as possible after being taken from the bore hole and before laboratory testing.

#### 3.2.3.4.1 Oriented Core

In some rock slope applications, it is important to understand the precise orientation of rock discontinuities for the design. Orienting recovered rock core so it can be properly mapped and evaluated, as though it were still in place, requires special core barrels. In the past, inclined borings were used. with core barrels weighted on one side allowing the core to be properly oriented when removed. Other techniques, such as using clay to make an impression of core run ends, have also been used. Currently, specialized core barrels that scribe a reference mark (line) on the side of the core as it is drilled are more routinely used. Special recording devices within the core barrel relate the known azimuth orientations to the reference mark so that when the core is subsequently removed from the core barrel, the core can be oriented to its exact in situ position. These specialized core barrels are relatively expensive and require additional training to use properly and interpret results. Refer to NHI 132031.

#### 3.2.3.4.2 Borehole Television Camera

In special cases, boreholes can be photographed/imaged to visually inspect the condition of the sidewalls and distinguish gross changes in lithology by using specialized television cameras. These down-hole cameras can also be used to identify fracture zones, shear zones, and joint patterns in rock core holes. This technique can also be used to identify/interpret the orientation of rock core. Refer to AASHTO MSI-1, Section 6.1.2.

Refer to PDDM Section 6.3.2.3.4 for standards on rock coring.

The primary source supporting the standards and guidance is NHI 132031. Secondary sources are AASHTO MSI-1 and GEC-5.

Refer to Appendix B.2, Site Investigation.

### 3.2.3.5 Test Pits, Trenches, and Surface Exposures

These are the simplest methods of observing subsurface soils. They consist of excavations performed by hand, backhoe, or dozer. Exploration pits and trenches permit detailed examination of the soil and rock conditions at shallow depths and relatively low cost. Exploration pits can be an important part of geotechnical explorations where significant variations in soil conditions occur (vertically and horizontally), large soil and/or non-soil materials exist (boulders, cobbles, debris) that cannot be sampled with conventional methods, or buried features must be identified and/or measured. Hand excavations are often performed with posthole diggers or shovels. They offer the advantages of speed and ready access for sampling. They are severely hampered by limitations of depth; and they cannot be used in hard/dense soils, large boulders or cohesionless soils below the water table. Another potential drawback is that test pits can cause a relatively large area of disturbance and visual impact, which might not be permissible at some partner agency sites/locations. Standard use of trenching and test pits is summarized in PDDM Exhibit 6.3-B.

A recommended primary reference is NHI 132031. Additional guidance is contained in AASHTO MSI-1 and CalTrans 2001.

Upon completion, the excavated test pit should be backfilled with the excavated material or other suitable soil material. Any test pit or excavated area located near planned structure footings or pavement must be surveyed to determine the precise location of the excavation. This information should be presented in the Contract Plans and Special Provisions to ensure the area will be re-excavated and properly compacted to the extent required. In the case of test pits excavated through existing pavements, the pavement should be properly patched. Where pits are located in agricultural areas or other areas used to support plant growth, the backhoe operator should be instructed to keep the topsoil or at least the finer upper-layer of the profile, and overburden separate from any gravel encountered in the pit. Upon completion of the pit, the operator should backfill in a sequence, generally with the coarsest material in the bottom of the pit, such that the backfilled pit area is reestablished to support vegetation. Reseeding may be necessary to comply with partner agency requirements.

Refer to PDDM Section 6.3.2.3.5 for standards on various explorations and sampling.

The primary source supporting the standards and guidance is NHI 132031. Secondary sources are AASHTO MSI-1 and CalTrans 2001.

Refer to Appendix B.2, Site Investigation.

### 3.2.3.6 Boring and Test Pit Closure

All borings should be properly closed at the completion of the field exploration. The primary reference is NHI 132031. This is typically required for safety considerations and to prevent cross contamination of soil strata and groundwater. Boring closure is particularly important for tunnel projects since an open borehole exposed during tunneling may lead to uncontrolled inflow of water or escape of compressed air. In many parts of the country, methods to be used for the closure of boreholes are regulated by state agencies. NCHRP RR 378 contains extensive information on sealing and grouting. The regulations generally require that any time groundwater or contamination is encountered, the borehole be grouted using a mixture of powdered bentonite, Portland cement and potable water. Some state agencies require grouting of all boreholes exceeding a certain depth. The geotechnical professional and the field supervisor should be knowledgeable about local requirements prior to commencing the borings. Also refer to AASHTO R 22-97.

Refer to PDDM Section 6.3.2.3.6 for standards on closing exploration sites.

The primary source supporting the standards and guidance is NHI 132031. Secondary sources are NCHRP RR 378 and AASHTO R 22-97.

Refer to Appendix B.2, Site Investigation.

### 3.2.3.7 Care and Retention of Samples

Soil samples and rock cores obtained represent a considerable investment of time and money. The samples should be properly labeled, transported, and stored. A detailed treatment of procedures for handling and storing samples is provided in NHI 132031 and AASHTO MSI-1. Refer to ASTM D 4220 and ASTM D 5079 for practices of preserving and transporting soil and rock core samples (ASTM Stds). However, any method that satisfactorily protects a sample from shock, large temperature changes, and moisture loss may be used. GEC-5 also provides guidance. All containers used for storage should be identified with the following:

- Project name and number
- Box number of total set
- Bore hole number
- Sample number
- Applicable depth information
- Date of sampling
- Name of person logging and sampling

The identification markings should be on the exterior as well as the interior of the storage container. All samples not used in laboratory testing will be retained as described in the next section.

In general, there are three types of samples obtained by the FLH Geotechnical Discipline and geotechnical consultants: disturbed soil samples (includes sack samples from test pits), undisturbed soil samples, and rock cores. Undisturbed samples typically degrade significantly and are not useful for testing purposes after about 3 to 6 months.

FLH standard is to retain rock core samples after laboratory testing until construction is awarded. In some situations and if space is available, core may be held until construction is complete and it is clear that claims related to the rock are not forthcoming. Final boring logs and photographs taken at the time of drilling are the permanent record of rock core and, therefore, they must be accurate and clear. Rock core obtained by consultants should be delivered to the designated storage area as part of the deliverables associated with the geotechnical task. All samples of soil or rock that are obtained on behalf of the agency by consultants and transported to the Laboratory will become the property of Federal Lands Highway.

Sample retention requirements unique to each Division, if different from those presented here, are available at EFL Link, CFL Link, and WFL Link.

Refer to PDDM Section 6.3.2.3.7 for standards on care and retention of samples.

The primary source supporting the standards and guidance is NHI 132031. Secondary sources are AASHTO MSI-1 and GEC-5.

Refer to Appendix B.2, Site Investigation.

### 3.2.3.8 Exploration Issues and Difficulties

As discussed in the AASHTO MSI-1 (Section 7.8), limitations and difficulties may be encountered during explorations, which are common to all exploratory techniques. They are usually a result of site-specific geologic conditions and/or a function of the improper equipment or technique being utilized. Several of these limitations and difficulties are described below.

Geotechnical professionals and field supervisors need to be aware of potential drilling problems and to avoid them in order to properly obtain field information and samples. See AASHTO MSI-1 (Section 7.12) and NHI 132031 (Section 3.5).

Some common problems or procedural errors with performing the Standard Penetration Test (SPT) that can provide misleading results are given in Exhibit 3.2–G. Refer to GEC-5 for additional guidance on potential errors with SPT testing.

Occasionally, sampling is attempted and little or no material is recovered. It is appropriate to make a second attempt to recover the material immediately following the first failed attempt. In such instances, the sampling device may be modified. For undisturbed samples, it may be appropriate to change the sampling method and/or the sampling equipment. Refer to NHI 132031 (Section 3.1.5).

The termination of an exploration above the required design depth due to boulders, fill material, excessively dense materials, and other obstructions may occur during any investigation. Specialized tools or coring techniques are available for use in difficult ground conditions when operating conventional drilling equipment. In some cases when obstacles are anticipated, a solution is to redrill the boring a meter (few feet) away.

**Exhibit 3.2–G COMMON PROCEDURAL ERRORS USING STANDARD PENETRATION TEST**

| Circumstance/Cause | Problem |
|---|---|
| Inadequate cleaning and/or seating of sampler. | Sludge and debris trapped in sampler or in bottom of hole. |
| Failure to maintain adequate hydrostatic pressure and/or over washing ahead of casing. | Fill-up inside casing.<br>Disturbance of in-situ material.<br>Too large pump. |
| Use of damaged and/or inadequate equipment. | Tip of sampler damaged by heavy driving.<br>Drive weight nonstandard or does not strike drive cap evenly. |
| Hammer does not free-fall and/or correct height of fall is not maintained. | More than 1.5 turns around cat head or wire line will restrict fall.<br>Proper height is not maintained by operator. |
| Operator and/or inspector errors. | Incorrect blow count.<br>Incorrect location and/or depth.<br>Sampler overdriven. |

### 3.2.4 SOIL AND ROCK CLASSIFICATION

Subsurface materials should be described/classified using a consistent approach so that users of the information can properly understand and interpret the subsurface conditions. Reference should be made to the soil and rock classification standards in PDDM Exhibit 6.3-E for soil and PDDM Exhibit 6.3-F for rock, and to guidance and tables in Chapter 4 of NHI 132031 and to Exhibit 3.2–H for guidance on soil classifications. Material classifications are important for design and could hold significant importance during claims disputes. It is therefore necessary for the method of reporting this data to be standardized. Records of subsurface explorations should follow the format presented in this section and the cited references. Additional references include: AASHTO MSI-1, NHI 132035, GEC-5, NHI 132012, and NAVFAC DM-7.1.

Material "descriptions" are based on the visual-manual method, which employs visual observations and simple manual index tests to estimate the physical and behavioral properties of the material (ASTM D 2488). Material "classifications" are based on laboratory index tests (ASTM D 2487). The classification of soil and rock includes consideration of the physical and engineering properties of the material. The detail of the classification should not be dictated by the complexity or objectives of the project. Classification should always be as complete as possible and based on factual information. Determine the USCS (Unified Soil Classification System) designation by following the procedures specified in ASTM D 2487. The USCS designation, as reported on exploration logs, will be an approximation based on the visual-manual soil description (ASTM D

2488). Where classifications are based on grain-size and Atterberg limits tests, the USCS designation will be more precisely defined.

Typical order of describing soil classifications is as follows (see Exhibit 3.2–H):

- Apparent Consistency or Density adjective
- Color
- Secondary soil constituent
- Primary soil constituent
- Additional soil constituents (minor) and inclusions
- Moisture content adjective
- Geologic name or formation

Rock descriptions for engineering purposes consist of two basic assessments: intact and in situ characters of the rock mass (NHI 132012). Both characteristics are the basis for rock slope design and excavation. Typical order of describing rock classifications is as follows:

- Rock type
- Color
- Grain size and shape
- Stratification/foliation
- Mineral composition
- Weathering and alteration
- Strength
- Hardness
- Discontinuities
- Rock Quality Designation (RQD)
- Formation name

Refer to PDDM Section 6.3.2.4 for standards on soil and rock classification.

The primary source supporting the standards and guidance is NHI 132031 and the secondary source is GEC-5.

Refer to Appendix B.3, Soil and Rock Classification.

**Exhibit 3.2–H BORING LOG TERMINOLOGY (Soil Description)**

| Soil Description[1] | Terms |
|---|---|
| Soil Density, Consistency or Hardness | See PDDM Exhibit 6.3-E for standards |
| Color | See below |
| Major Grain Size | The grain size that is > 50% of sample |
| Modifying Term | "and" – 40% to 50% of minor grain size<br>"some" – 10% to 40% of minor grain size<br>"trace" – Less than 10% of minor grain size |
| Minor Grain Size | The next visible grain size. |
| Moisture Content | D – dry<br>M – moist<br>W - wet |
| Laboratory Classification | (i.e., USCS, AASHTO) |

| Soil Color | Color Code | Soil Color | Color Code |
|---|---|---|---|
| Brown | BR | Grey-Red | GR RD |
| Lt. Brown | LT BR | Brown-Red | BR RD |
| Dk. Brown | DK BR | Yellow | YELL |
| Grey | GREY | Purple | PURP |
| Lt. Grey | LT GR | Green | GREEN |
| Dk. Grey | DK GR | White | WHITE |
| Red | RED | Grey-Green | GR GN |
| Black | BLK | Mottled | MOTT |
| Blue | BLUE | Orange | ORAN |
| Grey-Brown | GR BR | | |

<u>Note:</u>

1. *The following provides an example of soil description:*
   *Medium dense, reddish brown, SILT, some fine sand, trace of Clay (moist), ML.*

## 3.2.5 EXPLORATION LOGS

FLH standard is to produce a log of results for every subsurface exploration boring, hand-auger hole, probe hole and test pit performed. The log can also be used to describe inspected cut slopes. Refer to NHI 132031. An inspector, as well as the driller performing the work, should prepare a field log for each investigation. The geotechnical professional prepares a final edited log based on the field log, visual classification of the soil samples and the results of laboratory testing.

The drill inspector should maintain regular contact with the geotechnical professional, especially when unanticipated conditions or difficulties are encountered, significant schedule delays are anticipated, and prior to terminating the exploration and installing instrumentation. The driller should complete a daily drill report at the end of each workday. This is also required of any contract driller working for Federal Lands Highway. At the completion of each workweek these reports will be put in the project file.

As a minimum, groundwater levels should be measured and recorded prior to the daily commencement of drilling activities and upon completion of piezometer installation. Subsequent monitoring is at the discretion of the geotechnical professional.

Prior to de-mobilizing, the drill inspector should ensure location information (e.g., station, offset, elevation and/or state plane coordinates) of all the explorations are recorded on the field logs. This information should be provided with the field logs to the geotechnical professional. Required documentation for test pits should include a scale drawing of the excavation and photographs of the excavated faces and spoils pile. Drilling and sampling methods and in situ measurement devices that were used should also be documented.

The logs should contain basic reference information at the top, including project name, purpose, specific location and elevation, exploration hole, number, date, drilling equipment, procedures, drilling fluid, etc. Describe each sample fully. Care should be taken when referencing a previous sample since the samples might have slight distinguishing features that should be noted. Record the depth of each stratum contact, discontinuity, and lens. The reason for terminating an exploration hole and a list/description of instrumentation installed should be written at the end (bottom) of each exploration log.

### 3.2.5.1 Field Log

The field log is a record containing all the information obtained from an exploratory hole, whether or not it may seem important at the time of exploration. All soil and rock samples are to be fully described immediately on recovery. Depths of samples, top and bottom of each stratum/layer, discontinuities, field tests, and groundwater level(s) should be measured to the nearest 25 mm (0.1-foot). The depth(s) of drilling stoppage and date/time (i.e., end of shift) should be recorded. The material that is not recovered is frequently significant in the design of foundations, excavations, performance of fills, and other geotechnical applications. Therefore, any comments with regard to the character of drilling and difficulties encountered while advancing the boring should be included on the exploration log. Refer to ASTM D 5434.

#### 3.2.5.2 Final Boring Log

The final log is prepared from the field log after completing routine laboratory tests. Information provided on the logs should be typed. The final log includes factual descriptions of all materials, conditions, drilling remarks, and results of field tests and any instrumentation. Where groundwater observation wells or piezometers are installed, several measurements are usually necessary following drilling to verify that measured groundwater levels or pressures have achieved equilibrium. An explanation key should always accompany exploration logs whenever they are presented. Final boring logs should contain the information shown in NHI 132031. AASHTO MSI-1 is another good guidance document for boring logs.

GEC-5 provides guidance on the ISRM rock and rock mass classification system. This is the preferred classification for rock, though others are sometimes used (NHI 132031).

Refer to PDDM Section 6.3.2.5 for standards on exploration logging.

The primary source supporting the standards and guidance is NHI 132031. Secondary sources are AASHTO MSI-1 and GEC-5.

Refer to Appendix B.3, Soil and Rock Classification.

### 3.2.6 IN SITU SOIL TESTING

FLH standard is that sufficient field testing be performed to represent in situ conditions. In situ testing can be very beneficial on projects where obtaining representative samples suitable for laboratory testing is difficult, such as those involving very soft or loose silts and sands, cohesionless soils below the water table, and structured soils and material with large inclusions (gravel, cobbles, etc.). Additional benefits of in situ testing include avoidance of soil disturbance (and stress changes) and large scale testing when size requirements exceed common sample dimensions. Some in situ tests are performed in conventional drilled boreholes, whereas other more specialized tests require a separate borehole or different insertion equipment. The discussion for each test includes a brief description of the test method, the equipment and the uses of the data. The primary reference is NHI 132031. Diagrams, photographs, and example test results are contained in the following references: FHWA-SA-91-043, FHWA-SA-91-044, AASHTO MSI-1, NHI 132031, GEC-5; and NHI 132012. Field in situ borehole tests can be grouped into three categories: 1) correlation tests, 2) strength and deformation tests, and 3) permeability tests.

#### 3.2.6.1 Correlation Tests

Data obtained through these tests may be correlated to a number of different design parameters, including relative density, angle of internal friction, and shear strength. Further information and guidance is contained in NHI 132031 and AASHTO MSI-1.

#### 3.2.6.1.1 Standard Penetration Test (SPT)

SPT tests are often performed for geotechnical applications, being the most widely used field test in the United States. It has the advantages of simplicity, the availability of a wide variety of correlations for its data, and that a sample is typically obtained with each test. Although this in situ technique is extensively used in subsurface exploration, depending on the application, the test results should be augmented by other field and laboratory tests, particularly when dealing with clays. Standard Penetration Tests are performed in accordance with ASTM D 1596 and AASHTO T 206.

The SPT values should not be used indiscriminately. They are sensitive to the materials encountered and fluctuations in individual drilling practices and equipment used, such as the type of hammer (hammer efficiency), diameter and length of drill rods, presence of a liner in the sampler, and diameter of the drill hole. Noting the type of hammer used during the investigation is required on the boring logs, since this affects the actual input driving energy (hammer efficiency correction) transferred to the sampler. Because the rope and cathead method is not as consistent, the automatic drop system is preferred on FLH projects. SPT values obtained with automatic hammers should be calibrated by field comparisons with standard drop hammer methods. Some common problems or procedural errors that can provide misleading results are given in Exhibit 3.2–G. All SPT values should be corrected for overburden pressure. The Bazaraa Method as given in the FHWA publication *Soils and Foundation Workshop Manual* (NHI 132012) should be used for FLH projects. Exhibit 3.2–I provides empirical soil parameters from corrected SPT values for granular soils.

#### 3.2.6.1.2 Dynamic Cone Penetrometer Test (DCP)

This test consists of driving a cone shaped probe and the blow count results provide an indication of the uniformity or consistency of soils. Since no samples are recovered, dynamic cone penetrometer tests should only be used as a supplement to profile interpretations determined from standard borehole sampling techniques. Experience has shown that the DCP can be used effectively up to depths of 4.5 to 6 m (15 to 20 ft). Dynamic Cone Penetrometer Tests are performed in accordance with ASTM D 3441. Reference: FHWA-SA-91-043.

### 3.2.6.2 Strength and Deformation Tests

Various in situ tests are available for measuring strength and deformation properties.

#### 3.2.6.2.1 Cone Penetrometer Test (CPT) and Piezocone Penetrometer Test (PCPT)

Cone penetrometer tests are specialized quasi-static penetration profiling tests performed independently of drilled borings. The penetrometer can be used in sands or clays, but not in rock, dense sands, or soils containing appreciable amounts of gravel. This type of test is useful for subsurface materials, such as fibrous peat or muck that are very sensitive to sampling techniques. A disadvantage is that no samples are obtained so there is no positive identification of soil types. This method should only be used to supplement sampled borings, not to replace them. Piezocone penetrometers are electric penetrometers that are capable of measuring pore-water pressures during penetration. Cones can also be equipped with time-domain sensors that allow the cone to measure shear wave velocity. Tests are conducted in accordance with ASTM D 3441 (mechanical cones) and ASTM D 5778 (piezocones). Reference: FHWA-SA-91-043.

There are published correlations relating CPT data to soil type and several engineering properties. Many correlations of the cone test results to other soil parameters have been made, and design methods are available for spread footings and piles.

**Exhibit 3.2–I EMPIRICAL VALUES, RELATIVE DESNISTY AND MASS DENSITY OF GRANULAR SOILS** (Metric)

| Description | Very Loose | Loose | Medium | Dense | Very Dense | |
|---|---|---|---|---|---|---|
| Relative Density, $D_r$ | 0 | 0.15 | 0.35 | 0.65 | 0.85 | 1.00 |
| Corrected Standard Penetration N′ blows | | 4 | 10 | 30 | 50 | |
| Approximate Angle of Internal Friction $\phi$ (note 2) | 25-30º | 27-32º | 30-35º | 35-40º | 38-43º | |
| Approximate Range of Moist Mass Density (p) kg/m$^3$ | 1100-1600 | 1400-1800 | 1700-2100 | 1700-2200 | 2100-2400 | |

Notes:

1. *The table provides empirical values for ($\phi$), Relative density ($D_R$) and unit mass (y) of granular soils based on corrected N′ (Correlations may be unreliable in silts containing gravel.)*
2. *For $\phi$, use larger values for granular material with 5% or less fine sand and silt.*

### 3.2.6.2.2 Pressuremeter Test (PMT)

The pressuremeter measures stress/strain properties of soils by inflating a probe placed at a desired depth in a borehole. The PMT provides much more direct measurements of soil compressibility and lateral stresses than do the SPT or CPT. Results are interpreted based on semi-empirical correlations from past tests and observation. In situ horizontal stresses, shear strength, bearing capacities, and settlement can be estimated using these correlations. The pressuremeter test is a delicate tool, and is very sensitive to borehole disturbance. This test requires a high level of technical expertise to perform and is time consuming. Tests are completed in accordance with ASTM D 4719. Reference: FHWA-IP-89-008.

#### 3.2.6.2.3 Dilatometer Test (DMT)

The flat-plate dilatometer test uses pressure readings from an inserted plate at the base of a borehole to determine stratigraphy and obtain estimates of at-rest lateral stresses, elastic modulus, and shear strength of loose to medium dense sands (and to a lesser degree, silts and clays). The dilatometer test is not widely used, and the analysis and design methods based on DMT results are not yet as thoroughly developed as other techniques. Calibration is needed to correlate to local geologic environments. Through developed correlations, information can be deduced concerning material type, pore water pressure, in situ horizontal and vertical stresses, void ratio or relative density, modulus, shear strength parameters, and consolidation parameters. Tests are completed in accordance with ASTM D 6635. Reference: FHWA-SA-91-044.

#### 3.2.6.2.4 Field Vane Shear Test (VST)

This test consists of pushing a four-bladed vane at the base of a borehole into very soft to medium stiff cohesive soil or organic deposits to the desired depth and applying a torque at a constant rate until the material fails in shear. The torque measured at failure provides the undrained shear strength. A second test run immediately after remolding at the same depth provides the remolded strength of the soil and soil sensitivity. Tests are completed in accordance with ASTM D 2573 and AASHTO T 223.

#### 3.2.6.2.5 Borehole Shear Tests (BST)

Borehole shear strength tests are performed in an uncased borehole, where the apparatus is positioned and then expanded to apply horizontal pressure against the sides of the hole. The shear strength is determined by measuring the resistance while pulling up on the shear device. The test is repeated at increasing horizontal pressures to develop a plot of maximum shear stress to normal stress. This test is dependent on achieving “drained” conditions, and is more reliable on sand and silt soils.

### 3.2.6.3 Permeability Tests

Several in situ hydraulic conductivity tests exist with the most commonly used methods being the pumping test and the slug test. The selection of the appropriate aquifer test method for determining hydraulic properties by well techniques is described in ASTM D 4043. In general, refer to NHI 132031, BOR Geology Manual, and NAVFAC DM-7.1.

#### 3.2.6.3.1 Pumping Test

The pumping test requires not only a test well to pump from, but also one to four adjacent observation wells to monitor the changes in water levels as the pumping test is performed. Pumping tests are typically used in large-scale investigations to more accurately measure the permeability of an area for the design of dewatering systems. Refer to ASTM D 4050.

### 3.2.6.3.2 Slug Test

The slug test is quicker to perform and much less expensive because observation wells are not required. It consists of affecting a rapid change in the water level within a well by quickly injecting or removing a known volume of water or solid object known as a slug. The natural flow of groundwater out of or into the well is then observed until equilibrium in the water level is obtained. Refer to ASTM D 4044.

### 3.2.6.3.3 Packer Tests

These tests are performed in a borehole by placing packers above and below the soil/rock zone to be tested. One method is to remove water from the material being tested (Rising Water Level Method). Another method is to add water to the borehole (Falling Water Level Method and Constant Water Level Method). A third method utilizes water under pressure rather than gravity flow. The coefficient of permeability that is calculated provides a gross indication of the overall mass permeability. Refer to FHWA-TS-89-045 and NHI 132031.

### 3.2.6.3.4 Open Borehole Seepage Tests

Methods include “Falling Water Level,” “Rising Water Level,” and “Constant Water Level,” and are selected based on the relative permeability of the subsurface soils and groundwater conditions. Further detail is provided in Chapter 6 of NHI 132031.

### 3.2.6.3.5 Infiltration Tests

Two types of infiltrometer systems are available: sprinkler type and flooding type. Sprinkler types attempt to simulate rainfall, while the flooding type is applicable for simulating runoff conditions. Applications for these tests include the design of subdrainage and dry well systems. The most common application is the falling head test, performed by filling (flooding) a test pit hole and monitoring the rate the water level drops. Refer to ASTM D 4043.

Refer to PDDM Section 6.3.2.6 for standards on applying the SPT and other in-situ testing.

The primary source supporting the standards and guidance is NHI 132031. Secondary sources are FHWA-SA-91-043 and FHWA-SA-91-044.

Refer to Appendix B.4, In-Situ Testing.

## 3.2.7 LABORATORY TESTING

FLH standard is that sufficient laboratory testing be performed to represent in situ conditions. After collecting soil and rock samples, laboratory tests are routinely performed to quantify material properties and verify design assumptions. The type and number of tests required are primarily a function of the variability of the site, the purpose of the investigation, and the amount of risk and potential consequences of failure. Perform sufficient testing so that the geotechnical professional is satisfied that the test results are representative of in-situ conditions. Requesting and transmitting samples for laboratory testing and evaluation is handled differently in each FLH Division.

Exhibit 3.2–J provides a guideline for estimating laboratory test requirements for the different types of analysis. This exhibit is representative of past experience with FLH projects and is not intended to limit either the type of laboratory test or the frequency of testing but to provide a starting point for evaluation. Chapters 7 through 10 of NHI 132031, GEC-5, and Chapters 2 and 3 of NHI 132012 provide overviews of testing and correlations, as well as criteria to consider when planning the scope of the testing program. Additional references include: AASHTO MSI-1, NHI 132012, NHI 132035, USACE EM 1110-2-1906, and FHWA-FL-91-002.

**Exhibit 3.2–J GUIDELINES FOR SELECTION OF LABORATORY TESTS**

| Laboratory Tests Selection Frequency | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Test | A | B | C | D | E | F | G | H | I |
| Analysis Type | | | | | | | | | |
| Roadway Soil | F | F | F | L | F | L | M | R | L |
| Structural Foundation | F | M | F | L | L | F | M | L | M |
| Retaining Wall | F | M | F | M | L | F-M | F-M | L | M |
| Material Source | F | F | F | R | F | R | M-L | R | R |
| Landslides | F | F | F | M | L | F-M | F-M | L | R |

*Test Description:*

A — Gradation (Classification) AASHTO T88, T89, T90, T100
B — Fine Grain Analysis AASHTO T88
C — Atterberg Limits AASHTO T89, T90
D — Permeability Tests AASHTO T215
E — Remolded Density AASHTO T180 or T99
F — Unconfined Compression AASHTO T 208
G — Direct Shear AASHTO T 236
H — Triaxial AASHTO T296, T297
I — Consolidation AASHTO T216

*Selection Legend:*

F — Frequent/Routine Use
M — Moderate Use
L — Limited Use
R — Rarely Used

For each project, complete a testing request form to plan the laboratory program and to convey the plan to the geotechnical laboratory. The complexity of testing required for a particular project may range from simple moisture content determinations to specialized strength testing. Engineering judgment should be exercised in setting up a testing program that will produce the information required to resolve the technical issues for each specific project. It is important for the geotechnical professional to develop a prioritized and cost-effective testing program. The ideal laboratory program should provide the geotechnical professional with sufficient data to complete an economical design, yet not incur superfluous testing costs. When planning the laboratory testing program, the geotechnical professional should first examine all samples and evaluate the accuracy of descriptions in the field logs by performing visual classification tests.

Perform soil tests to determine specific soil properties and how the soil will respond to imposed conditions. Types of behavior depend on the strength, compressibility, permeability, corrosivity, and index properties. There are a number of tests that can be used to determine the desired properties, depending on the soil type and application. Refer to the Chapter 7 of NHI 132031. Detailed procedures for soil testing are presented in AASHTO Stds HM-25-M and USACE EM 1110-2-1906.

Laboratory testing on rock samples is to determine certain properties such as strength, elasticity, and degradation potential. The results are applied for the design of rock slopes, foundations, and material applications. Typically, the properties of in situ rock are determined by the presence of joints, bedding planes, etc., that cannot be modeled in the laboratory, since tests are performed on samples of limited size. A summary of rock tests and standards, grouped by rock behavior category, is listed in NHI 132031. Exhibit 3.2–K (soil) and Exhibit 3.2–L (rock) present a summary of laboratory tests, along with ASTM and AASHTO standard designations.

**Exhibit 3.2–K SUMMARY OF LABORATORY SOIL TESTS**

| Test Category | Soil Tests | ASTM | AASHTO |
|---|---|---|---|
| Visual Identification | Classification of Soils for Engineering Purposes (Unified Classification System | D 2487 | |
| | Description and Identification of Soils (Visual-Manual Procedure) | D 2488 | |
| Index Properties | Specific Gravity of Soils | D 854 | T 100 |
| | Materials Finer than No. 200 Sieve in Mineral Aggregates by Washing | C 117 | T 11 |
| | Sieve Analysis of Fine and Coarse Aggregate | C 136 | T 27 |
| | Particle-Size Analysis of Soils | D 421<br>D 422 | T 87<br>T 88 |
| | Materials Finer than No. 200 Sieve in Mineral Aggregates by Washing | D 1140 | |
| | Shrinkage Factors of Soils by the Mercury Method | D 427 | T 92 |
| | Liquid Limit, Plastic Limit, and Plasticity Index of Soils | D 4318 | T 89<br>T 90 |

| Test Category | Soil Tests | ASTM | AASHTO |
|---|---|---|---|
| Index Properties (continued) | Compaction Characteristics of Soil Using Standard Effort | D 698 | T 99 |
| | Compaction Characteristics of Soil Using Modified Effort | D 1557 | T 180 |
| | Harvard Miniature Compaction Device | | T 272 |
| | Water (Moisture) Content of Soil and Rock | D 2216 | T 265 |
| | Maximum Index Density and Unit Weight of Soils Using A Vibratory Table | D 4253 | |
| | Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density | D 4254 | |
| | Crumb Test to Identify Dispersive Clays | D 6572 | |
| Chemical Tests | pH of Peat Materials | D 2976 | |
| | pH of Soils | D 4972 | |
| | pH of Soil for Use in Corrosion Testing | G 51 | T 289 |
| | Sulfate Content | D 4230 | T 290 |
| | Resistivity | D 1125 & G 57 | T 288 |
| | Chloride Content | D 512 | T 291 |
| | Moisture, Ash, and Organic Matter of Peat and Other Organic Soils | D 2974 | T 267 |
| Strength Testing | Unconfined Compressive Strength of Cohesive Soil | D 2166 | T 208 |
| | Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression | D 2850 | T 296 |
| | Consolidated Undrained Triaxial Compression Test for Cohesive Soils | D 4767 | T 297 |
| | Consolidated Drained Triaxial Compression Test | | |
| | Direct Shear Test of Soils Under Consolidated Drained Conditions | D 3080 | T 236 |

| Test Category | Soil Tests | ASTM | AASHTO |
|---|---|---|---|
| Strength Testing (continued) | Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil | D 4648 | |
| | Consolidated Undrained Direct Simple Shear | D6528 | |
| Dynamic Properties | Modulus and Damping of Soils by the Resonant-Column Test | D 4015 | |
| | Modulus and Damping of Soils by the Cyclic Triaxial Apparatus | D 3999 | |
| | Load Controlled Cyclic Triaxial Strength of Soil | D 5311 | |
| | Consolidated Undrained Direct Simple Shear | D 6528 | |
| Permeability | Permeability of Granular Soils (Constant Head) | D 2434 | T215 |
| | Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter | D 5084 | |
| | Hydraulic Conductivity of Porous Materials Using a Rigid-Wall, Compaction-Mold Permeameter | D 5856 | |
| Compression Properties | One-Dimensional Consolidation Properties of Soils | D2435 | T 216 |
| | One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading | D 4186 | |
| | One-Dimensional Swell or Settlement Potential of Cohesive Soils | D 4546 | T 258 |
| | Measurement of Collapse Potential of Soils | D 5333 | |

**Exhibit 3.2–L SUMMARY OF LABORATORY ROCK TESTS**

| Test Category | Rock Tests | ASTM | AASHTO |
|---|---|---|---|
| Index Properties | Absorption and Bulk Specific Gravity of Dimension Stone | C 97 | |
| | Specific Gravity and Absorption of Coarse Aggregate | C 127 | T 85 |
| | Preserving and Transporting Rock Core Samples | D 5079 | |
| Point Load Strength | Point Load Strength Index of Rock | D 5731 | |

| Test Category | Rock Tests | ASTM | AASHTO |
|---|---|---|---|
| Compressive Strength | Triaxial Compressive Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements | D 2664 | T 226 |
| | Unconfined Compressive Strength of Intact Rock Core Specimens | D 2938 | |
| Direct Shear | Laboratory Direct Shear Strength Tests – Rock Specimens Under Constant Normal Stress | D 5607 | |
| Tensile Strength | Splitting Tensile Strength of Intact Rock Core Specimens | D 3967 | |
| Permeability | Permeability of Rocks by Flowing Air | D 4525 | |
| Durability | Slake Durability of Shales and Similar Weak Rock | D 4644 | |
| | Rock Slab Testing – Riprap Soundness, By Use of Sodium/Magnesium Sulfate | D 5240 | |

Refer to PDDM Section 6.3.2.7 for standards on laboratory testing.

The primary source supporting the standards and guidance is NHI 132031. Secondary sources are AASHTO MSI-1 and AASHTO Stds HM-25-M.

Refer to Appendix B.5, Laboratory Testing

### 3.2.8 INSTRUMENTATION AND MONITORING

Geotechnical instruments are used to characterize site conditions, verify design assumptions, monitor the effects of construction, enforce the quality of workmanship, and provide early warning of impending failures. In these regards, they are used to augment standard investigation practices and visual observations where conditions would otherwise be difficult to evaluate or quantify due to their location, magnitude or rate of change. Ralph Peck's paper on the "Observational Method" (Peck 1969), describes how instrumentation should be utilized during critical parts of construction to supplement the geotechnical professional's design and analyses.

During the investigation and design phase, instrumentation can be used to determine in situ conditions. Instrumentation monitoring can be extended over several months, as needed, to measure seasonal effects. During construction, instruments can be used to monitor in situ conditions to verify design assumptions and to warn of possible changed conditions or impending hazards. In addition, instrumentation may be used to monitor performance of embankments, slopes and foundation soils in response to construction (such as stability and the magnitude and rate of settlement of new embankments).

In all cases, instruments should be used to answer specific questions and provide engineering insight to a problem. There are a multitude of instruments available for design and construction. The geotechnical professional should become familiar with the different types of instrumentation available in order to understand their uses; how they are installed and operated; instrument accuracy, precision, and sensitivity; monitoring requirements; potential errors; environmental limitations and the effects of nearby activities. Consultation with equipment suppliers and instrumentation consultants is advisable if more complex types of instruments are required. General guidance is provided in NHI 132031. Key references that provide significant detail and useful schematics regarding various instrumentation methods include:

- NHI 13241
- AASHTO MSI-1, Appendix G – Instrumentation
- Dunnicliff 1993
- NCHRP Synthesis 89
- TRB SR 247

When ordinary inspection, investigation, and testing are insufficient to verify the intended performance, there may be a need for instrumentation. A successful instrumentation program involves creating a plan that matches appropriate instruments to the project needs and the resources available for implementation, monitoring and data reduction.

The planning task should consider several factors, which include the following:

- Objectives for instrumentation (What is the performance, property, or behavior that needs to be known?).

- Identification of instruments (What instrument functional applications are needed? Which instruments provide measurement of the desired objectives? What accuracy and reliability are needed? How often and how long will the instruments need to be monitored? Is the complexity of the instrument type appropriate to the skill-level of personnel available for monitoring? What are the simplest instruments that will meet the objectives and get the job done?).

- Site Plan (Showing instrument locations, preferably in safe places. Identify installation depths. Determine if monument covers and other warning/protection devices are needed to protect instruments).

- Acquisition of instruments (Will the Department acquire the instruments directly or will the construction contractor or a Consultant be asked to provide instruments? How will this be done? Specifications may be necessary to get equipment or results desired. Cost considerations could affect the types and extent of instrumentation selected).

- Calibration (Is this necessary for the selected types of instruments?).

- Installation (Identify who will perform and supervise the installations. Are there installation procedures that are unique for this project? Check that the installation approach will not compromise the quality of the expected data. Are protective measures needed?).

- Training personnel (Will the new instrumentation require training by the geotechnical professional or manufacturer's representative? Are the available Department technicians familiar with the selected instruments?).

- Monitoring (Determine the monitoring requirements and frequency, which could be variable depending on construction progress and ground behavior. Determine the means of collecting and storing data. Should monitoring be done manually each time, or should continuous data collection systems be used?).

- Data analysis (Determine how the data will be reduced, evaluated, and plotted. Identify the types of plots that are relevant to meet the objectives of the engineering evaluation).

- Documentation (Determine how the data and interpretations, including graphical plots, should be prepared and displayed. Determine the scheduling and recipients of reports/memorandums and recommendations).

- Follow-Up (Plan to check construction practices in response to instrumentation results, and be prepared to deal with potential concerns).

Instruments are often exposed to the environment and are susceptible to damage due to accidental impacts or vandalism. Environmental factors can also affect instrument performance. Therefore, consider the installation location and whether the instrument may be affected by conditions such as water, melting snow, heat, or subfreezing temperatures. Surface casings or monument covers may be installed for protecting instruments that are exposed near the ground surface. In addition, placing barricades, posts and warning flags around the instrument location can achieve further protection.

Maintenance of instruments and readout devices should be performed at recommended intervals in order to maintain accuracy and dependability. Deteriorated or damaged components should be immediately repaired. Periodic calibrations are required for some instruments and readout devices.

Most instruments require initial readings to be repeated (duplicate set) to crosscheck that the reference data set is accurate and dependable. The monitoring frequency should be determined by the geotechnical professional to fit the anticipated construction schedule and ground behavior.

Data analysis should usually be performed immediately, according to the most recent guidance provided by FHWA, AASHTO, geotechnical instrumentation publications, and the manufacturer. The reasonableness of the data and the analysis results should be verified by comparing them to previous data sets and known site conditions, as well as performing error checks. The instrumentation results should be documented promptly. The timing of the reports and memorandums should be established to fit the construction schedule in order to be of value. Sometimes, the reports may need to be made the same day as the readings, or possibly the next morning. Further guidance is provided in the following commentary.

Compare the results to previously projected scenarios and determine if there are any concerns. Apply correction factors, if necessary, to make the data relevant and usable. The data results are most useful when plotted in a form that is easily understood and relates to the construction and instrumentation program objectives. Ordinarily, graphs compare the specific measured results against time; however, the results can be used in several ways. For example, for settlement monitoring, graphs can relate and compare such things as fill placement height versus settlement, static groundwater level or pore pressure head versus fill placement height, or settlement versus time. Trends in the data should be evaluated to determine if there is an unusual condition or a common theme demonstrated by the results. A determination should be made if there is a performance hypothesis that is consistent with the instrumentation results. This will help evaluate the validity of data and interpretations. The results should be reviewed by experienced personnel.

For monitoring of unstable slopes and landslides, refer to TRB SR 247, WSDOT Unstable Slope Management System (WSDOT USMS Guidelines) and WSDOT PowerPoint presentation for Real Time Monitoring (WSDOT Monitoring).

Refer to PDDM Section 6.3.2.8 for standards on instrumentation and monitoring.

The primary source supporting the standards and guidance is NHI 132031. Secondary sources are AASHTO MSI-1 and NHI 132012.

Refer to Appendix B.6, Instrumentation

# SECTION 4 ANALYSIS AND DESIGN

Geotechnical design is based on the evaluation of site investigation data and performing studies and analyses to assess probable performance of constructed facilities and mitigation measures. An engineering analysis combines the information obtained from the geotechnical field investigation and the laboratory test results to determine the engineering properties and drainage characteristics of the subsurface materials. In addition, the analysis should alert designers, contractors and construction personnel of potential problems and provide economical solutions with consideration given to alternatives. Finally, the analysis should provide an assessment of risk associated with each of the possible solutions.

The quality of the analysis depends on several factors. Knowledge of engineering principles and practical experience in application of these principles is of course very important; but a thorough analysis cannot be accomplished without a clear understanding of the proposal details. This understanding requires a flow of communication and information between Project Development, Bridge Design, and the Geotechnical Discipline. To provide an acceptable analysis of geotechnical information that is practical, economical, and of sufficient detail, final alignment and grade are necessary. The project development process should provide this information and incorporate sufficient time to allow proper investigation and analysis.

FHWA geotechnical design manuals have been used to address areas not specifically covered by AASHTO, and are considered secondary relative to the AASHTO manuals for establishing geotechnical design direction. Alternative procedures could be used where justified by research, local experience, or described in nationally recognized design manuals. In such applications, the justification for deviating from AASHTO and FHWA standards should be documented. FLH standard is that the Geotechnical Discipline will determine the appropriate geotechnical standards of practice that are consistent with FLH and partner agency practices. FLH standard is that studies include an assessment of risk and uncertainty associated with each of the possible options. It is FLH standard that the scope of geotechnical analyses be commensurate with the scope of the project and consistent with FLH practices.

This section does not give detailed textbook solutions to engineering problems but will provide general guidelines and specific references to assist the engineer in performing a detailed analysis.

- Evaluation of Data. Gather the geotechnical data necessary to allow future completion of the PS&E level design work including final geometric data, test hole data, etc. Develop representative soil and rock properties for design based on the data collected, field observations and other results of the investigations. Prepare subsurface profiles for representative sections and critical locations where significant earthwork or structures are planned. The subsurface profiles should reasonably depict subsurface conditions and include design-level soil and rock properties relevant to the engineering evaluation to be performed. GEC-5 provides a rational approach for selecting soil and rock properties for engineering design. Additional references include NHI 132031 and EPRI EL-6800. Exhibit 3.2–I provides empirical soil parameters from corrected SPT values for granular soils.

- Review of Project Requirements and Preliminary Plans. The geotechnical professional should facilitate a flow of communication and information with planning and project development. It is in this phase that the Division office, or civil consultant, refines and defines the project's alignment, sets profiles and grade, and identifies specific project elements to be addressed by specialty groups within Federal Lands Highway or delegated to consultants. Once the preliminary project elements and alignments for the project are established, the geotechnical professional should assess feasible cut and fill slopes to enable roadway and bridge designers to establish the right-of-way needs for the project. The type, size and length of structures should also be obtained. Where walls may be needed, using approximate wall locations and heights identified by the design section, an assessment of feasible wall types is performed by the geotechnical professional, primarily to establish right-of-way and easement needs, and to provide preliminary cost estimates. Reference can be made to NHI 132031 and FHWA-ED-88-053.

- Planning the Scope of Analyses and Design Studies. FLH standard is that studies include an assessment of risk and uncertainty associated with each of the possible options. Many factors should be considered during the analysis and design phase of projects. Table 3 in FHWA-ED-88-053 provides guidelines for types of analyses that should be considered, based on design categories and soil types. Another valuable reference is GEC-5. Design categories in the table include "Embankment and Cut Slopes," "Structure Foundations" and "Retaining Structures." Additional issues of design could arise that are not defined in the exhibit and the geotechnical professional would need to consult the references cited in the PDDM that provide suggested methods of analysis and design. The geotechnical professional should keep abreast of the state-of-the-art practice for appropriate and economical designs. The scope of geotechnical analyses should be commensurate with the scope of the project and consistent with FLH practices.

## 4.1 EVALUATION OF DATA, PROJECT REQUIREMENTS, AND DESIGN PARAMETERS

*The first phase of the analysis and recommendations stage of project work is to evaluate the data present and the needs of the project. Evaluate if the data are suitable, the project needs are understood, and the appropriate scope of analysis is included in the budget. Evaluate if the data are suitable to support the analyses necessary to identify feasible design options, including assessments of cost, risk and uncertainty associated with each. Standard practices for data evaluation are as follows:*

- *Confirm understanding of project requirements and design criteria. Review preliminary plans and provide guidance and recommendations on geotechnical issues involving roadway alignment selection and the type, size, and location of roadway structures.*

- *Evaluate the accuracy and relevance of the available geotechnical data and whether they were collected according to standard or documented procedures. Section 6.3.3 provides standard site investigation methods and practices.*

- *Confirm suitability of data. Recommend supplemental explorations when additional geotechnical information is needed.*
- *Organize, tabulate, and format the field and laboratory data in order to extract suitable soil and rock properties and design parameters, and representative subsurface profiles and cross-sections supportive of required roadway and structure analyses.*
- *Document design parameters and design assumptions provided by others.*
- *Select values for geotechnical properties and design parameters with an understanding of uncertainty and variability. Refer to Section 2.2 for geotechnical discussion of risk management.*

FLH Standards for this task are in PDDM Section 6.4.1.

The primary source supporting the standards and guidance is GEC-5. Secondary sources are NHI 132031 and EPRI EL-6800.

Refer to Appendix B.7, Soil and Rock Properties.

## 4.2 SCOPE OF ANALYSIS

*Perform analyses to address specific project requirements. FLH standard practice is to use simple, inexpensive methods when they suffice, such as simply inspecting and comparing with precedent on the project or in the vicinity. These methods usually suffice when there is abundant precedent and the consequence of failure is low. An example is new cut slopes of less than 15 feet height on a route that contains many such stable slopes already.*

*Use more rigorous methods where there is not ample precedent and where the consequence of failure is more significant. Most structures and some earthwork features (embankments and cuts) fall into this category. For unique conditions and uncertainties, project features, or project risk tolerance, use multiple methods to evaluate the same design criteria. For example, combine limit equilibrium and finite element analysis of slope stability, or use alternate methods of drilled shaft capacity or settlement.*

*Conduct analyses and provide recommendations to accommodate evolving roadway and structure options and locations by providing recommendations that can be used for a variety of configurations where possible (e.g. plots of bearing capacity versus depth and diameter for drilled shafts).*

*Regardless of how simple or rigorous the analyses are, maintain analyses and calculations, including problem statements, given input, assumptions, reasoning, solution, and conclusions in a file. Follow the established QA/QC for the FLH Division.*

FLH standards for this task are in PDDM Section 6.4.2.

The primary source supporting the standards and guidance is FHWA-ED-88-053.

Refer to Appendix B.8, Computer Programs.

## 4.3 STRUCTURE FOUNDATIONS

This section covers the geotechnical design of bridge and wall foundations. Both shallow (e.g., spread footings) and deep (piles, shafts, micropiles, etc.) foundations are addressed. In general, the AASHTO service load design approach (SLD) will be used unless the project specific design requirements specify use of the newer load and resistance factor design approach (LRFD). The Bridge Engineer typically determines the design method. When using SLD, the geotechnical professional uses actual or unfactored loads for the design provided by the Bridge Engineer. Recommended safety factors and load combinations are outlined in the AASHTO bridge design code references.

FLH standard is to base selection of foundation type on an assessment of the magnitude and direction of loading, depth to suitable bearing materials, potential for liquefaction, undermining or scour, swelling potential, frost depth and ease and cost of construction. 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. Open communication and a close working relationship with the structure design engineer is necessary to provide efficient, cost-effective analysis of foundations. Foundation design is generally an iterative process between the geotechnical professional and Bridge Engineer. These iterations mean that the geotechnical professional 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. General guidance on foundations is provided in NHI 132012 and AASHTO HB-17, as well as NHI 132031 and FHWA-ED-88-053. Also, refer to USACE EM 1110-1-1904 and USACE EM 1110-1-1905.

FLH standards for this task are in PDDM Section 6.4.3.

The primary source supporting the standards and guidance is NHI 132012. Secondary sources are FHWA-ED-88-053 , AASHTO HB-17, USACE EM 1110-1-1905 and USACE EM 1110-1-1904.

Refer to Appendix B.9, Foundations.

### OFFICE STUDIES

Office studies could include evaluation of shallow and deep foundation options and specific analyses and design procedures for the selected foundation type. To systematically evaluate types of foundations, the following steps should be considered:

- Review as-constructed plans for existing structures at or near the proposed project site.
- Review any geotechnical reports and subsurface information for structures in the vicinity.
- Obtain a bridge layout sheet from the Bridge Unit.
- Discuss structure type and foundation requirements with the bridge designer.
- Identify design and constructability requirements (e.g. provide grade separation, transfer loads from bridge superstructure, provide for dry excavation) and their effect on the geotechnical information needed.
- Identify performance criteria (e.g. limiting settlements, right of way restrictions, proximity of adjacent structures) and schedule constraints.
- Summarize soil profile information. Identify areas of concern on site and potential variability of local geology
- Determine applicable soil and rock properties
- Subjectively assess the applicability of each type of foundation for their capability of carrying the required loads and estimate the amount of bearing capacity and settlement. Typically, shallow foundations should be considered first because of inherent cost savings if feasible (see Exhibit 4.3–A).
- Eliminate obviously unsuitable foundation types and prepare detailed studies and/or tentative designs for new foundations.
- Recommend the foundation type that meets structure requirements and is best suited and most economical for site subsurface conditions.
- Develop likely sequence and phases of construction and their effect on foundations.
- Identify engineering analyses to be performed and the engineering properties and parameters required for these analyses.
- Recommended bottom of footing or pile tip elevations, along with ultimate bearing capacity and appropriate factors of safety (see Exhibit 4.3–B and

  Exhibit 4.3–C). Address limitations and/or potential problems with the recommended foundation type.
- Identify potential construction problems and recommended construction control measures.

Review FHWA checklists on spread footings (Exhibit 5.1–F) driven piles (Exhibit 5.1–G), and drilled shafts (Exhibit 5.1–H). Refer to design requirements in AASHTO HB-17.

**Exhibit 4.3–A PRELIMINARY FOUNDATION TYPE SELECTION**

| Foundation Type | Use | Applicable Soil Conditions |
|---|---|---|
| Spread Footing | Individual columns, walls, bridge piers. | Any conditions where bearing capacity is adequate for applied load.<br>May use on single stratum; firm layer over soft layer or soft layer over firm layer.<br>Check immediate, differential and consolidation settlements. |
| Mat Foundation | Same as spread and wall footings.<br>Very heavy column loads.<br>Usually reduces differential settlements and total settlements. | Generally soil bearing value is less than for spread footings; over one-half area of building covered by individual footings.<br>Check settlements. |
| Friction Piles | In groups to carry heavy column, wall loads.<br>Requires pile cap. | Low strength surface and near surface soils. Soils of high bearing capacity 18 m to 45 m (60 ft to 150 ft) below ground surface, but by disturbing load along pile shaft solid strength is adequate.<br>Corrosive soils may require use of timber or concrete pile material. |
| End Bearing Piles | In groups of at least 2 to carry heavy column, wall loads.<br>Requires pile cap. | Low strength surface and near surface soils. End of pile located on soils 7.5 m to 30 m (25 ft to 100 ft) below ground surface. |
| Drilled Shafts (End bearing) | Larger column loads than for piles but eliminates pile cap by using caissons as column extension. | Low strength surface and near surface soils. End of shaft located on soils 7.5 m to 30 m (25 ft to 100 ft) below ground surface. |
| Sheetpile | Temporary retaining structures for excavations, alloy waterfront structures, cofferdams. | Any soil.<br>Waterfront structures may require special or corrosion protection.<br>Cofferdams require control of fill material. |

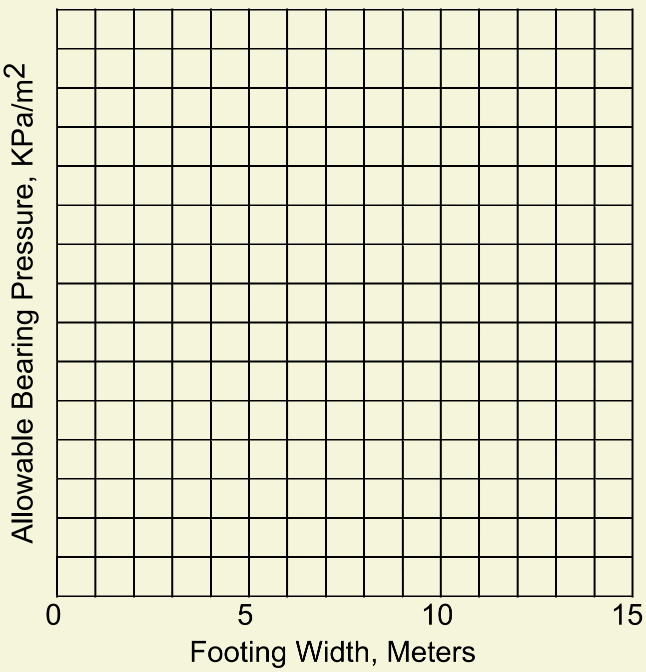
**Exhibit 4.3–B ALLOWABLE BEARING PRESSURE FOR SPREAD FOOTINGS (Metric)**

**Allowable Bearing Pressure for Spread Footings**

Project: ____________________________________________

Footing Location: ____________________________________________

Designer: ______________________________ Date: ____________




Design Criteria:

- Soil Type: ______________________________
- Factor of Safety: ______________________________
- Minimum Soil Above Footing Elevation: ______________________________
- Minimum Depth to Water Table: ______________________________
- Settlement at Bearing Pressure: ______________________________
- Maximum Total Settlement: ______________________________

**Exhibit 4.3–C ALLOWABLE PILE CAPACITY CURVE (Metric)**

Allowable Pile Capacity, kN

Depth of Penetration, Meters

Pile Tip Elevation, Meters

Project: ______________________________

Location: ______________________________

Substructure Unit: ______________ Factor of Safety: ________

## FOUNDATION DESIGN PHASES

There are several stages in developing foundation designs for transportation structures. A similar design process should be used if a consultant or design-builder is performing one or both design functions.

*Conceptual Bridge Foundation Design*

Initial information is provided to the Bridge Unit, consisting of a brief description of the anticipated site conditions, an estimate of the maximum slope feasible for the bridge approach fills for the purpose of determining bridge length, conceptual foundation types, and evaluation of potential geotechnical hazards such as existing slope instability, liquefaction potential or presence of large boulders. The purpose of these concept-level recommendations is to provide enough geotechnical information to allow the bridge preliminary plan (TS&L: Type, Size, and Location) to be produced. This type of conceptual evaluation could also be applied to other types of structures, such as tunnels.

*Preliminary Foundation Design*

Geotechnical data and recommendations are developed to assist the Bridge Unit in their structural analysis and modeling for the conditions considered for the selected structure. The Bridge Unit typically provides basic structure information to the Geotechnical Discipline to adequately develop the preliminary foundation design (refer to Commentary). The information provided by the Bridge Unit typically includes:

- Anticipated structure type and magnitudes of settlement (both total and differential) the structure can tolerate.
- At abutments, the approximate maximum elevation feasible for the top of the foundation in consideration of the foundation depth.
- For interior piers, the anticipated number of columns and whether single or multiple foundation elements would be used for each column.
- At stream crossings, the depth of scour anticipated, if known. Typically, the Geotechnical Discipline will pursue this issue with the Hydraulics Unit.
- Any known constraints that would affect the foundations in terms of type, location, or size, or any known constraints, which would affect design assumptions (e.g., utilities that must remain, construction staging, and constructability).

The preliminary geotechnical data is not suitable for publication or for potential bidders. The foundation recommendations are subject to change depending on the results of the structural analysis and modeling and the effect that modeling and analysis has on foundation types, locations, sizes, and depths, as well as any design assumptions made by the geotechnical designer. Preliminary foundation recommendations may also be subject to change depending on the construction staging needs and other constructability issues that are discovered during this design phase.

Geotechnical work conducted during this stage typically includes development of feasible foundation types and capacities (ultimate and allowable), foundation depths or pile tip elevations, p-y curve data and dynamic properties for seismic modeling, seismic site characterization and estimated ground acceleration, recommendations to address known constructability issues, and limitations and/or potential problems with the recommended foundation types. Provide a description of subsurface conditions and a preliminary subsurface profile at this stage, but detailed boring logs and laboratory test data are usually not provided yet. The Bridge Unit provides feedback to allow the Geotechnical Discipline to check the appropriateness of the preliminary foundation design recommendations.

*Final Foundation Design*

Checks the preliminary foundation design recommendations in consideration of the structural foundation design results determined by the Bridge Unit, and makes modifications to the preliminary foundation design as needed to accommodate the structural design provided by the Bridge Unit. A formal geotechnical report is produced that provides final geotechnical recommendations for the design and construction of the foundations for the subject structure. It is possible that much of what was included in the preliminary foundation design memorandum may be copied into the final geotechnical report, if no design changes are needed.

## SCOUR CONSIDERATIONS

Typically, the Hydraulics Engineer determines the depth of scour with input from the geotechnical professional. 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. 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. In some cases, it is more economical to design deep foundations to mitigate for scour potential. Refer to PDDM Chapter 7 for guidance on hydraulics procedures.

## SELECTION OF FOUNDATION TYPES

Foundations are classified as shallow, deep, or hybrid. Shallow foundations consist of spread footings or mats. Deep foundations include driven piles, drilled shafts and micropiles. Hybrid foundations are a combination of shallow and deep foundations. The most economical foundation type depends on subsurface soils and groundwater conditions, potential obstructions, site constraints, design loads, design scour elevations, potential for liquefaction, performance and serviceability requirements, construction sequence, surface water impacts, slope stability, and constructability. Shallow foundations are typically very cost effective, given the right set of conditions. Depth to suitable bearing material is the first factor to consider when choosing between shallow or deep foundations.

Foundation cost is always a consideration. While shallow foundations are typically less expensive, the choice between shallow or deep foundation systems usually depends in most cases on whether the site is either suitable for shallow foundations. Cost comparison becomes more of a factor when comparing different types of deep foundations or when comparing ground improvement costs with shallow foundations.

The geotechnical professional should consider local practice for founding structures when choosing a foundation system. Sometimes, local practice precludes a foundation type for reasons that might not be readily evident, which would require further evaluation.

Generally only one type of foundation is recommended for a structure, however, in some cases, multiple types of foundations may be specified. The evaluation of the proposed foundation type should be included in the structure geotechnical report as well as the other foundation types considered. Describe the reasons and/or analysis for the selection or exclusion of a particular foundation type.

The type of foundation for all supports of a structure need 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 professional should seek the input of the Bridge Engineer.

## 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. Measures to mitigate liquefaction are described in the following section. The seismic evaluation and design of soil-pile interaction is an area of active research and updates on design practices should be consulted GEC-3; NHI 132039A and WSDOT WA-M-46-03 provide detailed seismic design procedures. Also, refer to Section 4.11.

*Liquefaction Potential and Mitigation*

Generally, if liquefiable soils are present, some means of mitigation is required to protect structure foundations. In keeping with the no collapse philosophy, bridge approach embankments should be designed to remain stable during the design seismic event because of the potential to damage or initiate collapse of the structure should they fail. The aerial extent of approach embankment seismic design and mitigation (if necessary) should be such that the structure is protected against instability or loading conditions that could result in collapse. The typical distance of evaluation and mitigation is within 100 feet of the abutment. Instability or other seismic hazards such as liquefaction, lateral spread, downdrag, and settlement may require mitigation near the abutment to ensure that the structure is not compromised during a design seismic event. The geotechnical professional 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. GEC-3; AASHTO HB-17; and WSDOT WA-M-46-03 detail liquefaction analysis procedures. Also, refer to Section 4.11.

## FOUNDATION DESIGN RECOMMENDATIONS

Foundations are designed to provide adequate structural capacities within tolerable settlements, acceptable stability, and to meet minimum requirements for static and seismic conditions. NHI 132012 provides general guidance on foundation design. Refer to the Site Investigation Information Checklist (Exhibit 5.1–A) to ensure adequate scope of analyses. Review checklist items for spread footings (Exhibit 5.1–F), driven piles (Exhibit 5.1–G), and drilled shafts (Exhibit 5.1–H) as applicable.

Foundation Settlement Criteria should be established for specific subsurface conditions and structural requirements. The typical settlement criterion is a maximum 40 to 50 mm (1.5 to 2 inches) settlement that corresponds to 20 to 25 mm (0.8 to 1 inch) differential settlement between substructure units at allowable structural loadings.

FLH standard recommended minimum and typical ranges for factor of safety for the geotechnical soil-substructure interaction under static conditions are as follows in Exhibit 4.3–D. The selection of appropriate FS values within the ranges indicated depends on the uncertainty of geotechnical parameters and subsurface conditions as well as the level of importance of the facility and the risk and possible consequences.

**Exhibit 4.3–D AASHTO FOUNDATION CRITERIA (Factors of Safety)**

| Analysis Condition | Minimum Factor of Safety, FS |
|---|---|
| **Shallow Foundations:** | |
| • Bearing Capacity | 3.0 |
| • Sliding Along Base | 1.5 |
| • Overturning (Rotational Failure) | 2.0 |
| **Deep Foundations:** | |
| • Driven Piles (Static Method) | 2.0 to 3.0 |
| • Drilled Shafts | 2.0 to 2.5 |
| **Slope Stability at Structure Foundation Locations:** | |
| • Global Stability | 1.3 to 1.5 |

Note: Auger-cast piles are not included in this manual because quality assurance of auger-cast pile integrity and capacity needs further development, and most US highway agencies do not currently allow their use. For example, WSDOT policy does not allow auger-cast piles for bridge foundations.

Foundation selection criteria include:

- FLH standard is to base selection of foundation type on an assessment of the magnitude and direction of loading, depth to suitable bearing materials, potential for liquefaction, undermining or scour, swelling potential, frost depth and ease and cost of construction.

- The ability of the foundation type to meet performance requirements (e.g., deformation, bearing resistance, uplift resistance, lateral resistance/deformation), given the soil or rock conditions encountered. Refer to Exhibit 4.3–A for a summary of applicable soil conditions for different foundation types.

- The constructability of the foundation type.
- The impact of the foundation installation (in terms of time and space required) on traffic and right-of-way.
- The environmental impact of the foundation construction.
- The constraints that may impact the foundation installation (e.g., overhead clearance, access, and utilities).
- The impact of proposed foundation system on the performance of adjacent foundations, structures or utilities, considering both the design of the adjacent foundations, structures, or utilities, and the impact that installation of the new foundation will have on these adjacent facilities.
- The cost of the foundation, considering all of the issues listed above.

The following sections describe attributes and factors for each foundation type that influence the selection process:

Refer to PDDM Section 6.4.3 for general standards on structure foundations.

The primary source supporting the standards and guidance is NHI 132012. Secondary sources are FHWA-ED-88-053 , AASHTO HB-17, USACE EM 1110-1-1905 and USACE EM 1110-1-1904.

Refer to Appendix B.9, Foundations.

### 4.3.1 SHALLOW FOUNDATIONS

Spread footings work best in hard or dense soils that have adequate bearing resistance and exhibit tolerable settlement under load. If suitable material is at a reasonable depth, the geotechnical professional 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 affects bearing capacity and constructability. Groundwater flow into footing excavation can loosen potential bearing material and make forming and pouring a footing difficult. Construction sequence could impact the bearing capacity and settlements of shallow foundations. Footings can get rather large in medium dense or medium stiff soils to keep bearing stresses in an allowable range, or for structures with tall columns or which otherwise are loaded in a manner that results in large eccentricities at the footing level, or which result in the footing being subjected to uplift loads. Footings are not effective where soil liquefaction can occur at or below the footing level, unless the liquefiable soil is confined, not very thick, and well below the footing level. However, footings may be cost effective if inexpensive soil improvement techniques such as over-excavation, deep dynamic compaction, and stone columns, etc. are feasible. Other factors that affect the desirability of spread footings include the need for a cofferdam and seals when placed below the water table, the need for significant over-excavation of unsuitable soil, the need to place footings deep due to scour and possibly frost action, the need for significant shoring to protect adjacent existing facilities, and inadequate overall stability when placed on slopes that have marginally adequate stability. Footings may not be feasible where expansive or collapsible soils are present near the bearing elevation. The design scour depth could make excavating to construct shallow foundations unfeasible.

Since deformation (service) often controls the feasibility of spread footings, footings may still be feasible and cost effective if the superstructure can be designed to tolerate the settlement (e.g., flat slab bridges, bridges with jackable abutments, etc.).

The design of shallow foundations on soils is generally controlled by allowable settlement criteria, not by shear failure. The geotechnical professional should focus efforts on settlement evaluation, rather than bearing capacity for most sites. To design a spread footing, first determine the maximum allowable bearing capacity and check that a reasonable sized footing can support the design loads (refer to Exhibit 5.1–F). Estimate the magnitude of settlement (total and differential) and its time rate of consolidation to determine whether the predicted settlement performance is within acceptable limits. Refer to GEC-6, AASHTO HB-17, FHWA-RD-86-185, NHI 132012 and NHI 132037A-1 for the current practice for the design of spread footings. See the FHWA checklist for shallow foundation investigations and design (Exhibit 5.1–F).

FLH standards for shallow foundations are in PDDM Section 6.4.3.

The primary source supporting the standards and guidance is GEC-6. Secondary sources are AASHTO HB-17, NHI 132012, and FHWA-RD-86-185 .

Refer to Appendix B.9, Foundations.

### 4.3.2 DRIVEN PILE FOUNDATIONS

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.

Deep foundations are the best choice when spread footings cannot be founded on competent soils or rock at a reasonable cost. At locations where soil conditions would normally permit the use of spread footings but the potential exists for scour, liquefaction or lateral spreading, deep foundations bearing on suitable materials below such susceptible soils should be used as a protection against these problems. Deep foundations should also be used where an unacceptable amount of spread footing settlement may occur. Deep foundations should be used where right-of- way, space limitations, or other constraints as discussed above would not allow the use of spread footings.

The availability of raw materials, cost of labor, regional geology and proprietary systems 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.

- Driven Piles - Driven piles may be more cost effective than drilled shafts where pile cap construction is relatively easy, where the depth to the foundation layer is large (e.g., more than 30 m (100 feet)), or where the pier loads are such that multiple shafts per column, requiring a shaft cap, are needed. The tendency of the upper loose soils to flow, requiring permanent shaft casing, may also be a consideration that could make pile foundations more cost effective.

The design of a pile foundation involves an evaluation of a number of different design and constructability issues. Some of the items to be considered include:

- Design loads (axial and lateral).
- Predicted scour depth.
- Pile types (displacement or non-displacement? friction or end-bearing? precast concrete or steel? splicing?).
- Anticipated driving depths (consistent or variable? is splicing anticipated?).
- Pile drivability and acceptance criteria (hammer types, dynamic pile driving formulas and software – WEAP, CAPWAP).
- The presence of obstructions/boulders.
- Corrosion protection (steel piles).
- Site constraints (horizontal and vertical clearance, access, etc.).
- Impact to adjacent structures (settlement, vibration damage, noise).

Primary guidance on driven pile foundations is presented in NHI 132021 and NHI 132021. Secondary references include NHI 132038A; NHI 132012, WSDOT WA-M-46-03, and AASHTO HB-17 for the current design practice of driven pile foundations.

## AXIAL CAPACITY

Both compression and uplift axial capacities should be calculated for deep foundations. Evaluate subsurface conditions and construction sequence to determine 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. Provide pile or shaft capacity (Exhibit 5.1–G) and Exhibit 5.1–H). References NHI 132038A and AASHTO LRFD-3 outline design procedures for driven piles. FHWA-IF-99-025 describes the design procedures for drilled shafts. FHWA-SA-97-070 describes the practice for micropiles.

## LATERAL CAPACITY

In general, provide soil parameters to the Bridge Engineer so that LPILE program or other approved programs (including COM624) can be used for the lateral analysis of the foundation. Evaluate construction methods and sequence when developing LPILE parameters. Assumed conditions with the parameters should be described for the Bridge Engineer. References NHI 132038A; WSDOT WA-M-46-03; NHI 132021 and FHWA-IP-84-011 are helpful references for lateral load design procedures.

## DESIGN VERIFICATION FOR FOUNDATIONS

The Geotechnical Discipline is responsible for ensuring that pile foundations can be installed to design requirement without damage. In situations where concrete piles, high loads or difficult installation is anticipated, dynamic pile analyses are often performed. The wave equation computer program is often used to establish installation equipment requirements and pile stress during construction. The geotechnical professional 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. Detailed information on these procedures are provided in NHI 132012; NHI 132022, and NHI 132069.

Load testing is sometimes performed during design for high-capacity foundations. 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, particularly for high-capacity drilled shafts. 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 professional to use lower factors of safety for design, which results in lower construction costs. There are several methods for load testing piles and drilled shafts. These include static, dynamic, Statnamic, and Osterberg load cell tests. In general, static load tests are not feasible for drilled shafts due to the need of a high reaction force frame system. FHWA-IF-99-025 describes load test procedures for drilled shafts. NHI 132022 and NHI 132069 describe load test procedures for driven piles.

Refer to PDDM Section 6.4.3.2 for standards on pile foundation analysis and design.

The primary source supporting the standards and guidance, including a step by step procedure, is NHI 132021. Secondary sources are AASHTO HB-17, NHI 132012 and WSDOT WA-M-46-03.

Refer to Appendix B.9, Foundations.

### 4.3.3 DRILLED SHAFT FOUNDATIONS

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.

Deep foundations are the best choice when spread footings cannot be founded on competent soils or rock at a reasonable cost. At locations where soil conditions would normally permit the use of spread footings but the potential exists for scour, liquefaction or lateral spreading, deep foundations bearing on suitable materials below such susceptible soils should be used as a protection against these problems. Deep foundations should also be used where an unacceptable amount of spread footing settlement may occur. Deep foundations should be used where right-of- way, space limitations, or other constraints as discussed above would not allow the use of spread footings.

The availability of raw materials, cost of labor, regional geology and proprietary systems 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.

- <u>Drilled Shafts</u> **-** Shaft foundations are most advantageous where dense intermediate strata must be penetrated to obtain the desired bearing, uplift, or lateral resistance, or where obstructions such as boulders or logs must be penetrated. Drilled shafts may also become cost effective where a single shaft per column can be used in lieu of a pile group with a pile cap, especially when a cofferdam or shoring is required to construct the pile cap. However, drilled shafts may not be desirable where contaminated soils are present, since contaminated soil would be removed, requiring special handling and disposal. Drilled shafts should be used in lieu of piles where deep foundations are needed and pile driving vibrations could cause damage to existing adjacent facilities. Artesian pressure in the bearing layer could preclude the use of drilled shafts due to the difficulty in keeping enough head inside the shaft during excavation to prevent heave or caving under slurry. The primary advantage drilled shafts have over driven piles, aside from constructability concerns, is their large size and resultant large capacity.

Drilled shafts are generally used where design loads (axial and/or lateral) are very large, where the use of drilled shafts eliminates the need for a cap, or where the use of driven piles is not viable. Design of drilled shafts varies from driven piles because the construction processes are different. For example, 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. Due to the possible influence on design assumptions, consider the various construction methods that could be used for drilled shafts. Construction techniques that negatively influence the design assumptions should be restricted from use and specified in the contract documents. Based on load test data, the design approach outlined in current AASHTO codes could be over-conservative and does not adequately utilize the combined end-bearing/side shear axial load capacity of rock socketed drilled shafts. The design approach is to use the end-bearing and side shear components in combination.

To quantify these components and economize the overall design, load test(s) could be performed. A method shaft could be included on significant structures to insure that production shafts will be constructed properly. Drilled shafts could be constructed with steel access tubes to allow for cross-hole sonic logging. The percentage of shafts that are tested will vary among projects, however, the contract documents typically include a minimum quantity of about 30% of the total number of shafts.

References FHWA-IF-99-025 and AASHTO HB-17 outline design procedures for drilled shafts. FHWA provides more complete narrative and construction considerations, but design methodologies are similar. Additional references include Exhibit 5.1–H, NHI 132038A, FHWA-RD-95-172, FHWA-IP-84-011, NHI 132012, and WSDOT WA-M-46-03. When very large diameter drilled shafts are being considered, especially those with rock-sockets, industry groups (ADSC) should be contacted to determine the constructability of the proposed foundation.

## AXIAL CAPACITY

Both compression and uplift axial capacities should be calculated for deep foundations. Evaluate subsurface conditions and construction sequence to determine 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. Provide pile or shaft capacity (Exhibit 5.1–G) and Exhibit 5.1–H). References NHI 132038A and AASHTO LRFD-3 outline design procedures for driven piles. Reference FHWA-IF-99-025 describes the design procedures for drilled shafts. FHWA-SA-97-070 describes the practice for micropiles.

## LATERAL CAPACITY

In general, provide soil parameters to the Bridge Engineer so that LPILE program or other approved programs (including COM624) can be used for the lateral analysis of the foundation. Evaluate construction methods and sequence when developing LPILE parameters. Assumed conditions with the parameters should be described for the Bridge Engineer. References NHI 132038A, WSDOT WA-M-46-03, NHI 132021 and FHWA-IP-84-011 are helpful references for lateral load design procedures.

## DESIGN VERIFICATION FOR FOUNDATIONS

The Geotechnical Discipline is responsible for ensuring that pile foundations can be installed to design requirement without damage. In situations where concrete piles, high loads or difficult installation is anticipated, dynamic pile analyses are often performed. The wave equation computer program is often used to establish installation equipment requirements and pile stress during construction. The geotechnical professional 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. Detailed information on these procedures are provided in NHI 132012; NHI 132022, and NHI 132069.

Load testing is sometimes performed during design for high-capacity foundations. 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, particularly for high-capacity drilled shafts. 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 professional to use lower factors of safety for design, which results in lower construction costs. There are several methods for load testing piles and drilled shafts. These include static, dynamic, Statnamic, and Osterberg load cell tests. In general, static load tests are not feasible for drilled shafts due to the need of a high reaction force frame system. FHWA-IF-99-025 describes load test procedures for drilled shafts. NHI 132022 and NHI 132069 describes load test procedures for driven piles.

Refer to PDDM Section 6.4.3.3 for standards on drilled shaft foundation analysis and design.

The primary source supporting the standards and guidance, including a step by step procedure, is FHWA-IF-99-025. Secondary sources are AASHTO HB-17, FHWA-RD-95-172, NHI 132012 and WSDOT WA-M-46-03.

Refer to Appendix B.9, Foundations.

### 4.3.4 MICROPILE FOUNDATIONS

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.

Deep foundations are the best choice when spread footings cannot be founded on competent soils or rock at a reasonable cost. At locations where soil conditions would normally permit the use of spread footings but the potential exists for scour, liquefaction or lateral spreading, deep foundations bearing on suitable materials below such susceptible soils should be used as a protection against these problems. Deep foundations should also be used where an unacceptable amount of spread footing settlement may occur. Deep foundations should be used where right-of- way, space limitations, or other constraints as discussed above would not allow the use of spread footings.

The availability of raw materials, cost of labor, regional geology and proprietary systems 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.

- Micropiles - Micropiles are small diameter drilled piles. For situations where existing structures must be retrofitted or underpinned to improve foundation resistance or where limited headroom is available or in close proximity to settlement-sensitive existing structures or difficult geology (e.g. large boulders), micro-piles may be the best alternative and should be considered. Micropiles do not provide much lateral support, and therefore other methods may be required if lateral resistance is a project-specific structural criteria.

Micropiles are small diameter drilled piles. Micropiles are typically used where site restrictions prohibit the use of larger foundation construction equipment. Micropiles are installed by with specialized drills typically used to install tiebacks. Due to their size, micropiles have lower total capacity than larger deep foundation elements, and production rates are typically lower for micropile rigs than for other types of deep foundation construction. These factors contribute to the relatively high of cost of micropiles. The geotechnical professional typically performs a preliminary design of the micropile element to estimate capacities and minimum size requirements. On the contract drawings, provide the micropile specialty contractor with the footing layout and design loads, and the contractor will be responsible for the final design of the pile element. Micropiles do not provide significant lateral support, and therefore other methods may be required if lateral resistance is a project-specific structural criteria. Lateral load demand on a micropile foundation is resisted by the horizontal component of batter piles or by tieback anchors. The lateral resistance provided by the soil-pile interaction is small and generally not included in the design. The AASHTO design codes do not currently include provisions for micropiles. Primary reference is FHWA-NHI-05-039. Secondary reference FHWA-SA-97-070 should be referred to for the current design practice. An additional reference is NHI 132038A.

## AXIAL CAPACITY

Both compression and uplift axial capacities should be calculated for deep foundations. Evaluate subsurface conditions and construction sequence to determine 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. Provide pile or shaft capacity (Exhibit 5.1–G) and Exhibit 5.1–H). References NHI 132038A and AASHTO LRFD-3 outline design procedures for driven piles. Reference FHWA-IF-99-025 describes the design procedures for drilled shafts. FHWA-SA-97-070 describes the practice for micropiles.

## LATERAL CAPACITY

In general, provide soil parameters to the Bridge Engineer so that LPILE program or other approved programs (including COM624) can be used for the lateral analysis of the foundation. Evaluate construction methods and sequence when developing LPILE parameters. Assumed conditions with the parameters should be described for the Bridge Engineer. References NHI 132038A; WSDOT WA-M-46-03; NHI 132021 and FHWA-IP-84-011 are helpful references for lateral load design procedures.

## DESIGN VERIFICATION FOR FOUNDATIONS

The Geotechnical Discipline is responsible for ensuring that pile foundations can be installed to design requirement without damage. In situations where concrete piles, high loads or difficult installation is anticipated, dynamic pile analyses are often performed. The wave equation computer program is often used to establish installation equipment requirements and pile stress during construction. The geotechnical professional 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. Detailed information on these procedures are provided in NHI 132012; NHI 132022, and NHI 132069.

Load testing is sometimes performed during design for high-capacity foundations. 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, particularly for high-capacity drilled shafts. 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 professional to use lower factors of safety for design, which results in lower construction costs. There are several methods for load testing piles and drilled shafts. These include static, dynamic, Statnamic, and Osterberg load cell tests. In general, static load tests are not feasible for drilled shafts due to the need of a high reaction force frame system. FHWA-IF-99-025 describes load test procedures for drilled shafts. NHI 132022 and NHI 132069 describe load test procedures for driven piles.

Refer to PDDM Section 6.4.3.4 for standards on micropile foundation analysis and design.

The primary source supporting the standards and guidance, including a step by step procedure, is FHWA-NHI-05-039. The secondary source is FHWA-SA-97-070.

Refer to Appendix B.9, Foundations.

## 4.4 EARTH RETENTION SYSTEMS

Retaining walls and reinforced slopes are typically included in projects to minimize construction in wetlands, to widen existing facilities, and to minimize the amount of right-of-way needed in urban environments. Projects modifying existing facilities often need to retrofit or replace existing retaining walls or widen abutments for bridges. All abutments, walls, and reinforced slopes within right-of-way should be designed and constructed in accordance with AASHTO requirements and this manual.

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 Discipline. Geotechnical professionals should be able to develop their own designs as well as evaluate and review standardized and proprietary wall designs.

There are a number of factors that control wall type selection.

- Magnitude and direction of loading.
- Depth to suitable foundation support.
- Potential for earthquake loading.
- Presence of deleterious environmental conditions.
- Proximity of physical constraints.
- Cross sectional geometry.
- Tolerable total and differential settlement.
- Facing durability and aesthetics.

- Ease and cost of construction.

Retaining walls are typically semi-gravity cantilever or gravity type retaining walls. In certain cut applications, nongravity cantilever, anchored walls or soil nail walls may be considered as an option. The geotechnical professional should have an understanding of the applications of each wall type, exploration and design requirements, construction methods, and relative costs. The GEC-2 provides an overview of wall types with general information pertaining to selection criteria, and design and analysis procedures. Guidance is provided in AASHTO HB-17, FHWA-ED-88-053, and NHI 132012. Refer to FHWA checklist for investigations and designs of retaining walls, Exhibit 5.1–I. The US Forest Service publication FHWA-FLP-94-006 is a valuable reference for retaining walls on low-volume road and forest lands projects. WSDOT WA-M-46-03 provides detailed technical guidance.

AASHTO recommended factors of safety should be used for wall designs, as shown in Exhibit 4.4–A. 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.

**Exhibit 4.4–A AASHTO RETAINING STRUCTURE CRITERIA (Factors of Safety)**

| Analysis Condition | Minimum Factor of Safety (FS) (from AASHTO HB-17) |
|---|---|
| • Sliding (static)<br>• Sliding (seismic) | 1.5<br>1.125 |
| • Overturning (static)<br>• Overturning (seismic) | 2.0 for footings on soil<br>1.5 for footings on rock<br>1.5 for footings on soil<br>1.125 for footings on rock |
| • Bearing Capacity (static)<br>• Bearing Capacity (seismic) | 3.0 (shallow foundations)<br>1.5 (shallow foundations) |

Retaining structures should be designed to structurally withstand the effects of total and differential settlement estimated for the project site, both longitudinally and in cross-section, as prescribed in the AASHTO Specifications. The WSDOT WA-M-46-03 (Chapter 15) describes recommended settlement criteria for retaining systems.

FLH standards follow AASHTO codes that require retaining structures be designed to withstand lateral earth and water pressures, surcharges, and earthquake loads. All retaining walls should be designed with adequate soil resistance against bearing, sliding, overturning, and overall stability as specified by the governing design code.

FLH standard is to design retaining structures for a minimum service life of 75 years and provide adequate structural capacities within tolerable movements, acceptable stability, and meet minimum requirements for static and seismic conditions. The geotechnical professional should work closely with the Bridge Engineer to determine the appropriate earth pressure loadings so that the design is performed in accordance with current FLH procedures.

Abutments for bridges have components of both foundation design and wall design. Abutment walls, wingwalls, and curtain walls that are backfilled prior to constructing the superstructure should be designed using active earth pressures, allowing for wall rotation/deflection. Active earth pressures should be used for abutments that are backfilled after construction of the superstructure, if the abutment can move sufficiently to develop active pressures. If the abutment is restrained, at-rest earth pressure should be used. Abutments that are "U" shaped or that have curtain/wing walls should utilize at-rest pressures in the corners, as the walls are constrained.

The geotechnical design responsibility for a proprietary wall involves the determination of the maximum allowable bearing pressure, minimum lateral pressure (or maximum $\varphi$ to be used for determination), minimum foundation depths, and overall stability.

Other than the pre-approval process for proprietary wall systems, there is usually no need for specialized field testing of common wall elements or systems. However, high-capacity ground anchors commonly require pre-production load testing.

*Alternate and Proprietary Retaining Structures*

Proprietary wall designs require review and comparisons of specific wall design parameters. When proprietary wall designs are suitable for specific site conditions, alternative bid procedures are recommended. To ensure that these alternatives are equal, a review by the Geotechnical Discipline prior to advertisement of the construction contract is required. Designs submitted for approval should contain all calculations and assumptions made by the proprietary wall design. Included with the submittal should be copies of any computer programs used, in a format compatible with FLH software. Refer to Section 257 of FP-XX.

*Wall Drainage*

Drainage should be provided for all retaining walls and structures. Section 5.5.3 of GEC-2 provides an overview of drainage considerations for walls. Also refer to FHWA-FLP-94-006. Geotextiles are often used in wall drainage applications. The type of geosynthetics to be used depends on the application and site conditions. Generic material property and construction method specifications are included in the FP-XX. Criteria for geosynthetics are application-dependent and are described in FHWA-HI-95-038. Additional geosynthetic design guidance is described in WSDOT WA-M-46-03 and Koerner 1994. Recent experiences are documented in ASCE GSP 103. Evaluate the permeability and gradation of the in situ soil. Determine geotextile strength requirements based on the drain rock size, height that rock could be dropped onto the geotextile, and other constructability issues. The permittivity and aperture size are important properties of the geotextile to select to be compatible with onsite soils and water flow expectations.

For instances where wall drainage cannot be provided, include the hydrostatic pressure from the water in the design of the wall. In general, wall drainage should be in accordance with the Standard Plans, General Special Provisions, and the PDDM. The following are exceptions:

- Gabion walls and rock walls are generally considered permeable and do not typically require wall drains.

- Soil nail walls should include composite drainage material centered between each column of nails. The drainage material is connected to weep holes using a drain gate or wrapped around an underdrain.

Cantilever and Anchored wall systems using lagging should include composite drainage material attached to the lagging face prior to casting the permanent facing. Walls without facing or walls using precast panels are not required to use composite drainage material provided the water can pass through the lagging unhindered.

> Refer to PDDM Section 6.4.4 for standards on wall selection and analysis tasks.
>
> The primary source supporting the standards and guidance is GEC-2. Secondary sources are AASHTO HB-17, FHWA-FLP-94-006 and USACE EM 1110-2-2502
>
> Refer to Appendix B.10, Retaining Walls and Earth Retaining Structures.

### 4.4.1 CONCRETE 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. AASHTO HB-17 provides guidelines and design charts for analysis of static and seismic conditions. Seismic design issues are discussed further in Section 4.11.

The FLH 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. Provide all soil parameters needed to design walls that are not covered in the FLH Standard Plans, as well as determine the suitability of use of the walls covered in the FLH Standard Plans for the project.

*The Structures Discipline designs concrete walls, usually and preferably according to FLH Standard Plans. The Structures Discipline will use geotechnical recommendations to confirm the applicability of the standard plans. Provide soil, rock and groundwater design parameters for concrete gravity and cantilever walls. Include recommendations for the foundation and the retained soil, requirements for backfill, and the suitability of onsite material.*

### 4.4.2 MSE WALLS

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. FLH has specific procedures and requirements for the design of MSE walls. A recent, comprehensive reference on MSE walls is FHWA-NHI-00-043 (NHI 132042). AASHTO HB-17 describes the state of the practice design procedure for MSE walls. Additional references include: FHWA-FLP-94-006, GEC-2, FHWA-NHI-00-044, and WSDOT WA-M-46-03. MSE walls are commonly used for medium to large wall and grade separation projects because they are often less expensive than concrete cantilever walls.

It is FLH standard practice for the Geotechnical Discipline to perform the external stability analysis in the design phase of the project, and provide the required reinforcement lengths to the Bridge Office to be included in the Construction Plans.

Most MSE wall applications use proprietary systems where the internal design is performed by the wall vendor. 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 professional reviews the submitted calculations and shop drawings for approval. The following commentary provides information regarding FLH pre-approvals of proprietary wall systems.

A number of proprietary wall systems have been extensively reviewed by the Bridge Office. This review has resulted in pre-approving some proprietary wall systems. This allows the manufacturers to competitively bid a particular project without having a detailed wall design provided in the contract plans. Note that proprietary wall manufacturers may produce several retaining wall options, and not all options from a given manufacturer have been preapproved. The Bridge Office should be contacted to obtain the current listing of preapproved proprietary systems prior to including such systems in projects. Incorporation of non-approved systems requires the wall supplier to completely design the wall prior to advertisement for construction. All of the manufacturer's plans and details would need to be incorporated into the contract documents. Several manufacturers may need to be contacted to maintain competitive bidding.

*Modular Gravity Walls*

Modular gravity walls use interlocking soil or rock-filled concrete, masonry, 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 and systems. Determine earth pressures for modular gravity walls using the same procedures as for standard cantilever walls. Because many of these wall types are proprietary, it is recommended that geotechnical professionals use manufacturers' literature for design, and check them with generic methods. Seismic design issues are discussed further in Section 4.11. 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, and therefore the tolerable settlements of upslope structures should be considered.

Refer to PDDM Section 6.4.4.2 for standards on MSE wall analysis and design.

The primary source supporting the standards and guidance is FHWA-NHI-00-043. Secondary sources are WSDOT WA-M-46-03 and FHWA-NHI-00-044.

Refer to Appendix B.10, Retaining Walls and Earth Retaining Structures.

### 4.4.3 SOIL NAIL WALLS

Soil nails are closely spaced, passive steel bar reinforcements used to strengthen existing ground. They are constructed in a top down manner and are used to support an excavation face with temporary or permanent shotcrete facing. Soil nail walls are an economical alternative to soldier pile and ground anchored walls when installed in the appropriate soil conditions. The AASHTO design codes do not include provisions for the design of soil nail walls. Primary reference is GEC-7. FHWA-SA-96-069R is also recommended for design of soil nail walls. Analysis programs such as GoldNail (available from FHWA) and Snailz (download available) could 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. FHWA-SA-93-068 describes construction procedures. An additional reference is WSDOT WA-M-46-03. There are special issues that should be evaluated, as described in the following commentary.

The following are some of the items that should be taken into account when considering use of soil nail walls:

- Temporary standup of the excavation face requires some cohesion or cementation of the subject soils.
- Drilling into cohesionless materials requires the use of temporary casing during drilling, which 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.
- Soil nail walls are not recommended where the ground could deform, such as landslides.

Refer to PDDM Section 6.4.4.3 for standards on soil nail wall analysis and design.

The primary source supporting the standards and guidance is GEC-7. Secondary sources are FHWA-SA-96-069R and FHWA-SA-93-068.

Refer to Appendix B.10, Retaining Walls and Earth Retaining Structures.

### 4.4.4 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 low-height walls in cut sections, 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 4.5 m

(15 ft) 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. Bridge engineers determine the appropriate sizes of the structural elements based on the applied loads. Primary reference is GEC-2. Secondary references include AASHTO HB-17 and NAVFAC DM 7.2.

Refer to PDDM Section 6.4.4.4 for standards on pile wall analysis and design.

The primary source supporting the guidance is GEC-2. Secondary sources are AASHTO HB-17 and NAVFAC DM 7.2.

Refer to Appendix B.10, Retaining Walls and Earth Retaining Structures.

### 4.4.5 GROUND ANCHOR 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 for uncommon construction equipment and skills. However, they are well suited where deformations of adjacent structures are of concern and where high-capacity anchorage is required. Items that should be evaluated when considering use of ground-anchored walls are described in the following commentary.

Items that should be evaluated when considering use of ground-anchored walls include:

- Underground easements should be obtained to protect the anchors throughout their functional life.
- 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 pre-production tests and long-term monitoring may also be required.
- Anchors bonded in clays might have capacity and long-term creep limitations.

The contract documents should require the contractor to determine the anchor bond length necessary to resist the applied anchor force. Refer to GEC-4, GEC-2, FHWA-DP-68-1R and PTI 2004. In addition to static and seismic conditions of the completed wall configuration, evaluate the stability for each stage in the construction sequence of a ground anchored wall. The controlling condition is often a construction case.

Refer to PDDM Section 6.4.4.5 for standards on ground anchor systems and wall design.

The primary source supporting the standards and guidance is GEC-4. Secondary sources are PTI 2004 and FHWA-DP-68-1R.

Refer to Appendix B.10, Retaining Walls and Earth Retaining Structures.

### 4.4.6 ROCKERIES

Design rockeries for global and internal stability. Rockeries are slender "structures" primarily used for surficial erosion protection and do not provide significant lateral support compared to retaining walls. Therefore, rockeries should only be used where the retained material is at least minimally stable without the rockery (a minimum slope stability factor of safety of 1.2). Alternative structures that satisfy lateral earth pressure requirements include rock buttresses, designed using limit equilibrium slope stability methods (refer to Landslides section). Rockeries should have a batter of 6V:1H or flatter. The rocks should increase in size from the top of the wall to the bottom at a uniform rate. The recommended minimum rock sizes are shown in Chapter 15 of WSDOT WA-M-46-03, which is based on past experience. Rockeries should not exceed 12 feet height. Rockeries that are used to retain fill (and associated lateral earth pressures) should not exceed 6 feet total height if the rocks are placed concurrent with backfilling. Design of rockeries is further described in ARC 2000 and in papers by Gifford & Kirkland 1978, FHWA-FLP-94-006, and Hephill.

Refer to PDDM Section 6.4.4.6 for standards on rockery analysis and design.

The primary source supporting the standards and guidance is CFLHD Rockery. Secondary sources are ARC 2000 and WSDOT WA-M-46-03.

Refer to Appendix B.10, Retaining Walls and Earth Retaining Structures.

### 4.4.7 TEMPORARY CUTS AND SHORING

Temporary shoring and cut slopes are frequently used during construction to provide lateral support and as part of dewatering operations. Since the contractor has control of the construction operations, the contractor should be made responsible for the stability of temporary cut slopes, as well as the safety of the excavations. Because excavations are recognized as one of the most hazardous construction operations, temporary cut slopes should be designed to meet Federal regulations (OSHA Section 29).

The contractor is responsible for internal and external stability of temporary shoring, as well as global slope stability, soil bearing capacity, and settlement of temporary shoring walls. FLH design might include shoring requirements in unusual materials, loading situations and adjacent to critical structures. Shoring within railroad right-of-way typically requires railroad review and therefore FLH may need to design the shoring to obtain the railroad's concurrence prior to advertisement of the contract.

A wide range of temporary shoring systems are available for cut applications. Each temporary shoring system has advantages and disadvantages, conditions where the system is suitable or not suitable, and specific design considerations. Refer to FHWA-RD-75-128 and CalTrans 2001. The textbook by Ratay 1996 as well as WSDOT WA-M-46-03 (Chapter 15) provide overviews of shoring methods. Soil nail wall shoring applications are described in GEC-7.

Trench boxes are routinely used to protect workers during installation of utilities and other construction operations requiring access to excavations deeper than 4 feet. Trench boxes could be suitable for trenches where the depth is greater than the width of the excavation. Trench boxes are not appropriate for excavations that are deeper than the trench boxes.

Braced cuts are used to provide support to the excavated slope by extending structural supports to temporary foundations and reactions within the excavation area. Braced cuts need to be designed to account for lateral earth pressures and global stability.

Sheet piling is a common temporary shoring system in cut applications and is particularly beneficial as the sheet piles can act as a diaphragm wall to reduce groundwater seepage into the excavation. There are two general types of sheet pile walls: cantilever and anchored/braced. The ability for sheet piling to be anchored by means of ground anchors or deadman anchors (or braced internally) allows sheet piling to be used where deeper excavations are planned. One disadvantage of sheet piling is that it is installed by vibrating or driving; thus, in areas where vibration sensitive structures or soils are present, sheet piling may not be appropriate.

Soldier pile walls are frequently used as temporary shoring in cut applications. Ground anchors, internal bracing, rakers, or deadman anchors can be incorporated in soldier pile walls where the wall height is higher than about 12 feet, or where backslopes or surcharge loading are present. Soldier piles placed in predrilled holes are particularly effective adjacent to existing improvements that are sensitive to settlement, vibration, or lateral movement. The sheet piles are best used in fine-grained soils for ease of driving, whereas boulders create difficulties.

Modular blocks are used to form a gravity structure, which relies on the soil having adequate standup time so that the excavation can be made and the blocks placed without excessive caving. One disadvantage to using modular blocks is that voids in the backfill zone could cause movement to occur. Modular blocks need to be designed the same as gravity structures.

Soil nail walls reinforce the soil behind the excavation face. For this method to be feasible, the soils should be capable of adequate standup time to allow placement of the steel wire mesh and/or reinforcing bars to be installed and the shotcrete to be placed. Design of soil nail walls requires a detailed geotechnical investigation to characterize the reinforced soils and the soil located below the base of excavation.

While most temporary retaining systems are used in cut applications, some temporary retaining systems are also used in fill applications. Typical examples include the use of MSE walls to support preload fills that might otherwise encroach into a wetland or other sensitive area, the use of modular block walls or wrapped face geosynthetic walls to support temporary access road embankments or ramps, and the use of temporary wrapped face geosynthetic walls to support fills during intermediate construction stages.

The following shoring systems require special analysis and design:

- Diaphragm/slurry walls.
- Secant pile walls.
- Sheet pile cellular cofferdams.
- Frozen soil walls.
- Deep soil mixing (DSM).
- Jet grouting.

The factors that influence the choice of temporary shoring include: cost, subsurface constraints, site constraints, and local practice. The following are examples of such factors.

- Cut/Fill Height. Some retaining systems are more suitable for supporting deep excavations/fill thicknesses than others. Temporary modular block walls are typically suitable only for relatively short fill embankments (less than 4.5 m (15 ft)), while MSE walls can be designed to retain significant fill heights. In cut applications, the common cantilever retaining systems (sheetpiling and soldier piles) are typically most cost effective for retained soil heights of 4 to 5 m (12 to 15 ft) or less, whereas taller shoring could require reinforcement, either external (struts, rakers, etc.) or internal (ground anchors or dead-man anchors).

- Soil Conditions. Factors include: 1) dense soils, boulders and obstructions, 2) caving conditions, 3) permeability, 4) bottom heave and piping, and 5) compressibility and deformability.

- Groundwater. The groundwater level with respect to the proposed excavation depth could have a substantial influence on the temporary shoring system selected. Excavations that extend below the groundwater table and that are underlain by relatively permeable soils would also likely require dewatering. Large dewatering efforts might cause settlement of nearby structures. Considerations for barrier systems include the depth to an aquitard to seal off groundwater flow.

- Space Limitations. Space limitations include external constraints, such as right-of-way issues and adjacent structures, and internal constraints such as the amount of working space required. Permanent easements may be required if the shoring systems include support from ground anchors or dead-man anchors that remain after construction is complete.

- Adjacent Infrastructure. The location and sensitivity of infrastructure adjacent to the shoring application could influence the selection of the temporary shoring method. Existing underground utilities that cannot be relocated may have an impact on the choice of temporary shoring system.

For open temporary cuts, evaluate the following:

- Limiting traffic, construction equipment, stockpiles or building supplies at the top of the cut slopes to a setback distance from the top of the cut.

- Whether raveling of the slope face could occur and whether the exposed cut should be covered with waterproof tarps, plastic sheeting or shotcrete.

- Designating a maximum time period that the temporary cut is left open.
- Erosion control measures.
- Surface water impacts and means to divert the water away from the temporary cut slope.
- Whether the temporary slopes should be monitored to confirm adequate stability.

The primary difference between temporary shoring and permanent retaining structures is design life. Because of the shorter design life, temporary shoring is typically not designed for seismic loading and corrosion protection is generally not necessary. Additionally, more options for temporary shoring are available due to limited requirements for aesthetics.

The temporary shoring design should consider both internal and external stability. Internal stability includes assessing the components that comprise the shoring system, such as the reinforcing layers for MSE walls, the bars or tendons for ground anchors and soil nails, and the structural steel members for sheet pile walls and soldier piles. External stability includes an assessment of overturning, sliding, bearing resistance, settlement and global stability. Consider the actual construction-related loads that could be imposed on the shoring system. References include: FHWA-RD-75-128, NHI 132012, GEC-4, and GEC-7.

Refer to PDDM Section 6.4.4.7 for standards on temporary cuts and shoring.

The primary source supporting the standards and guidance is OSHA Section 29. Secondary sources are Ratay 1996 and CalTrans 2001.

Refer to Appendix B.10, Retaining Walls and Earth Retaining Structures.

## 4.5 OTHER STRUCTURES

### 4.5.1 CULVERTS AND PIPES

FLH guidance to be drafted.

Refer to PDDM Section 6.4.5.1 for standards on geotechnical recommendations for culverts and pipes.

The primary source supporting the standards and guidance is USACE EM 1110-2-2902. Secondary sources are Spangler & Handy 1982 and FHWA-RD-98-191.

Refer to Appendix B.10, Retaining Walls and Earth Retaining Structures.

### 4.5.2 BUILDING FOUNDATIONS

FLH guidance to be drafted.

Refer to PDDM Section 6.4.5.2 for standards on building foundations.

The primary source supporting the standards and guidance is NAVFAC DM 7.2 and the secondary source is NAVFAC DM-7.1.

Refer to Appendix B.10, Retaining Walls and Earth Retaining Structures.

### 4.5.3 MICROTUNNELS AND TRENCHLESS CONSTRUCTION

FLH guidance to be drafted.

Standards for microtunneling and trenchless construction is in PDDM Section 6.4.5.3.

The primary source supporting the standards and guidance is FHWA-IF-02-064 and the secondary source is CI/ASCE 36-01.

Refer to Appendix B.10, Retaining Walls and Earth Retaining Structures.

## 4.6 EARTHWORK

The vast majority of FLH projects require a roadway investigation. The guidelines presented for investigations may be applied to all lengths of roadway projects but the frequency of testing and sampling should be adjusted based upon site specific problems and practical engineering judgment in order to adequately characterize subsurface conditions.

Classify the subsurface materials encountered during soil explorations and interpret the stratigraphy. Soils should be grouped by stratum or with similar engineering properties. 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. Geotechnical interpretation may be used in grouping and stratifying soil types. Obtain sufficient subsurface information to define characteristics that may affect the design.

Roadway soils analysis factors are identified in Exhibit 4.6–A. Define and address the following issues (refer to Summary of Soil Survey form, Exhibit 4.6–B:

- Design cut and fill slope ratios.
- Suitable materials for embankments.
- Shrink/swell factors for excavation and embankment.
- Areas requiring mitigation.
- Poor and unsuitable soils
- Wet areas (seepage, excessive water) – Refer to Exhibit 4.6–C
- Potential areas of instability.

Where the presence of problematic soils or unfavorable bedrock structure is known early in a project's development, inform the designer of possible constraints and potential solutions for evaluation of mitigation measures and alternative alignments.

Adverse geotechnical conditions can often be mitigated by several different options. Evaluate the conceptual options for feasibility, constructability, risk-management, ability to satisfy project constraints and environmental criteria, future consequences and preliminary estimates of cost.

Design criteria for cut and fill slopes are based on an assessment of slope stability and available right-of-way. Generally accepted slopes are 1V:1.5H for cuts and 1V:2H for fills (common material). Granular fills where interlocking of coarse fragments will occur can be designed with 1V:1.5H slopes. Weaker materials and groundwater can affect slope stability and gentler slope angles or mitigation measures should be considered. Most small cuts and fills are designed using engineering judgment and precedents in the vicinity.

Cut and fill slopes are generally not designed for seismic conditions unless slope failure could impact adjacent structures. For seismic analysis, if applicable, a safety factor of 1.1 is used for slopes adjacent to walls and structures. For other significant cut and fill slopes, a minimum safety factor of 1.05 is recommended. Review of liquefaction potential should also be performed.

NHI 132012 and the WSDOT WA-M-46-03 are recommended sources of technical information for earthwork design and analysis. Chapter 9 in FHWA-DF-88-003 should be consulted regarding mass diagrams. Refer to geotechnical checklists Exhibit 5.1–B and Exhibit 5.1–C (based on FHWA-ED-88-053). TRB SAR 8 and BOR Earth Manual provide guidance regarding earthwork construction and testing practices.

**Exhibit 4.6–A ROADWAY SOILS ANAYSIS FACTORS**

| Identifying Characteristic | Potential Problem/Condition | | | | | |
|---|---|---|---|---|---|---|
| | Soil/Rock Interface | Variability of Pavement | Settle-ment | Frost Heave | Poor Drainage | Slope Instability |
| **In-Situ Properties** | | | | | | |
| Soil Classification | X | X | X | X | X | X |
| Plasticity | | | X | X | X | X |
| Natural Moisture | X | | X | | X | X |
| Subgrade Strength | | X | | X | | |
| **Existing Conditions** | | | | | | |
| Standing/Seeping Water | X | | X | X | X | X |
| Subgrade Support | | X | | | | |
| Pavement Thickness | | X | | X | X | |
| Slope Ratio | X | | | | | X |
| Pavement Distress | | X | X | X | X | X |

**Exhibit 4.6–B SAMPLE OF SUMMARY SOIL SURVEY**

**Summary of Soil Survey**

Project: ______________________________ Date Performed: ____________

Beginning Reference Location: _______________________ Performed by: ___________

| Station to Station | Description of Soil or Rock | Recommended Slope Ratios | Shrink/Swell Factor | Water Problem Area (Yes/No) | Remarks |
|---|---|---|---|---|---|

**Exhibit 4.6–C SUMMARY OF WATER PROBLEM AREAS**

**Summary of Water Problem Areas**

Project: ______________________________

Beginning Reference Location: ______________________________

Performed By: ____________________ Date Performed: ____________________

| From Station to Station | Description of Problem | Recommended Solution |
|---|---|---|

Standards for this general earthwork standard and for non-standardized tasks, is found in PDDM Section 6.4.6.

The primary source supporting the standards and guidance is NHI 132012. Secondary sources are WSDOT WA-M-46-03, TRB SAR 8 and BOR Earth Manual.

Refer to Appendix B.11, Earthwork.

### 4.6.1 RIPPABILITY

The degree of rippability is estimated based on experience and correlations with seismic wave velocities from geophysical surveys. It is usually preferred to avoid blasting if the rock materials are rippable. The degree of difficulty that can be experienced when ripping can affect construction time and cost. Refer to FHWA-Geophysical and Exhibit 4.6–D. Also refer to NHI 132035.

Standards for applying these standards and for completing non-standardized tasks, is found in PDDM Section 6.4.6.1

The primary source supporting the standards and guidance is FHWA-Geophysical, for seismic velocity, and the secondary source is NHI 132035.

Refer to Appendix B.11, Earthwork.

### 4.6.2 SHRINK/SWELL FACTORS

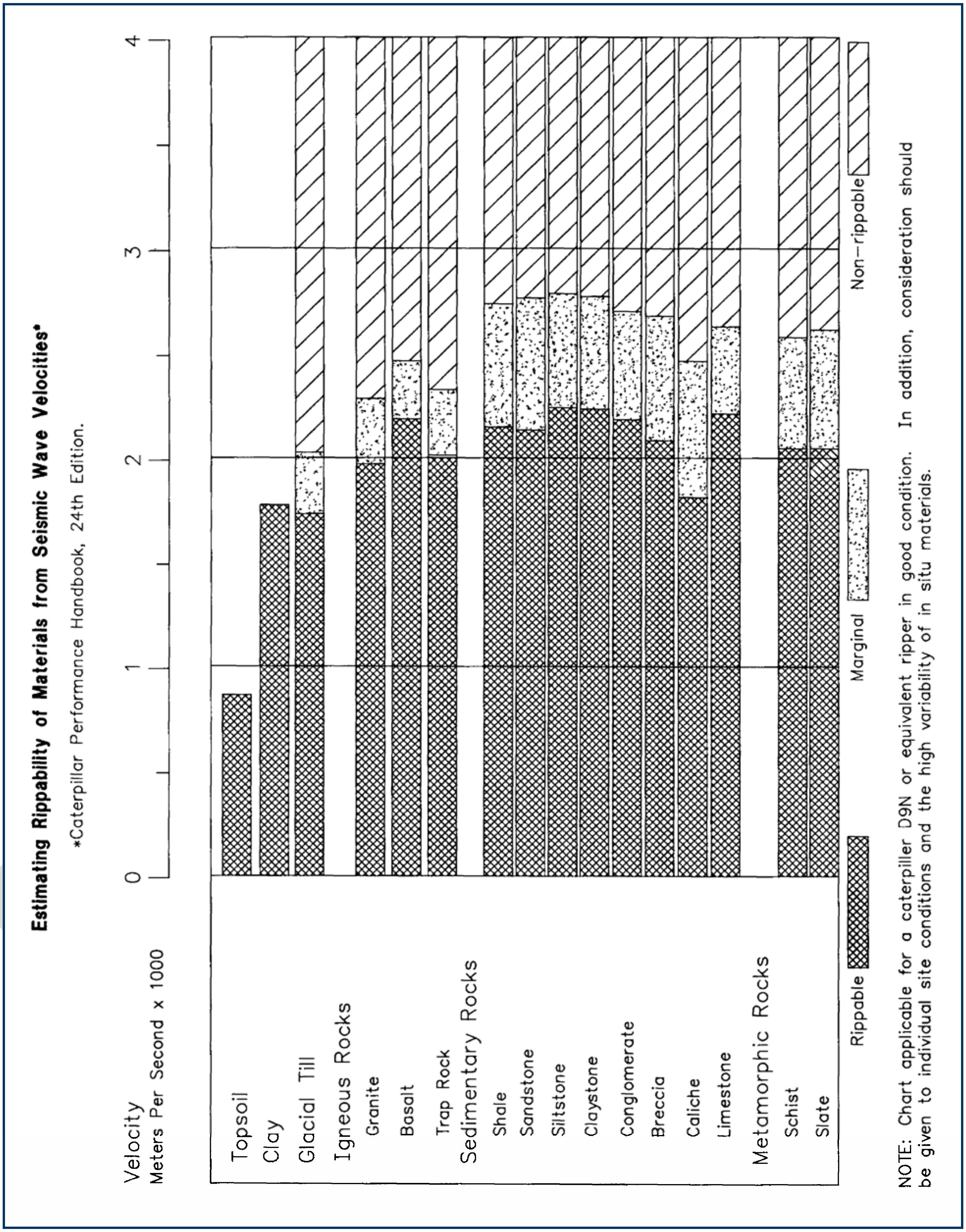
Shrink and swell factors are used in analyses to balance earthwork quantities on a project. Determine the shrinkage or swell factors for the predominant material types to be encountered. Earthwork factors (shrink/swell) used in estimating cut and fill quantities are typically based on local experience and correlations. The values of shrink/swell factors vary considerably depending on the method of fill construction or disposal. In addition, apparent shrink/swell on a project can come from numerous sources, such as poor survey control, poor excavation limits control, improper section construction, poor cut/fill slope control, improper compaction, missed quantities in the design package, haulage losses. These issues can cause uncertainties in the estimates of quantities for earthwork planning. Typical values of shrink/swell are described in (metric) and Exhibit 4.6–F (US Customary units). Additional sources of construction advice on estimating shrink/swell for mass-haul planning and developing mass diagrams are available, such as Burch 2006 and Church 1981.

Standards for applying these standards and for completing non-standardized tasks is found in PDDM Section 6.4.6.2

The primary source supporting the standards and guidance is Burch 2006 and the secondary source is Church 1981.

Refer to Appendix B.11, Earthwork.

**Exhibit 4.6–D ESTIMATING RIPPABILITY OF MATERIALS FROM SEISMIC WAVE VELOCITIES**



## Exhibit 4.6–E SHRINK/SWELL FACTORS FOR COMMON MATERIALS (Metric)

| Material | Measured | | | | |
|---|---|---|---|---|---|
| | In-Situ | Loose | | Embankment | |
| | Mass Density[1] kg/m[3] | Mass Density[2] kg/m[3] | % Swell[3] | Mass Density[2] kg/m[3] | % Swell or Shrink[3] |
| Andesite | 2930 | 1760 | 67 | 2050 | 43 |
| Basalt | 2935 | 1790 | 64 | 2160 | 36 |
| Bentonite | 1600 | 1185 | 35 | — | — |
| Breccia | 2400 | 1800 | 33 | 1890 | 27 |
| Calcite-Calcium | 2670 | 1600 | 67 | | |
| Caliche | 1440 | 1245 | 16 | 1900 | -25 |
| Chalk | 2410 | 1285 | 50 | 1810 | 33 |
| Charcoal | — | 610 | — | — | — |
| Cinders | 760 | 570 | 33 | 840 | -10 |
| Clay | | | | | |
| Dry | 1910 | 1275 | 50 | 2120 | -10 |
| Damp | 1985 | 1180 | 67 | 2205 | -10 |
| Conglomerate | 2205 | 1660 | 33 | | |
| Decomposed rock | | | | | |
| 75% R. 25% E. | 2445 | 1865 | 31 | 2185 | 12 |
| 50% R. 50% E. | 2225 | 1610 | 38 | 2375 | -6 |
| 25% R. 75% E. | 2005 | 1405 | 43 | 2205 | -9 |
| Diorite | 3095 | 1855 | 67 | 2165 | 43 |
| Diatomaceous earth | 870 | 540 | 62 | | |
| Dolomite | 2890 | 1725 | 67 | 2015 | 43 |
| Earth, loam | | | | | |
| Dry | 1795 | 1230 | 50 | 2090 | -12 |
| Damp | 2000 | 1400 | 43 | 2090 | -4 |
| Wet, mud | 1745 | 1745 | 0 | 2090 | -20 |
| Feldspar | 2615 | 1565 | 67 | 1825 | 43 |
| Gabbro | 3095 | 1855 | 67 | 2165 | 43 |
| Gneiss | 2700 | 1615 | 67 | 1885 | 43 |
| Gravel (Dry) | | | | | |
| Uniformly Graded | 1770 | 1600 | 10 | 1870 | -5 |
| Average Gradation | 1945 | 1620 | 20 | 2120 | -8 |
| Well Graded | 2180 | 1645 | 33 | 2450 | -11 |
| Gravel (Wet) | | | | | |
| Uniformly Graded | 1965 | 1870 | 5 | 1870 | -5 |
| Average Gradation | 2160 | 1950 | 10 | 2120 | -2 |
| Well Graded | 2425 | 2090 | 16 | 2450 | -1 |
| Granite | 2695 | 1565 | 72 | 1880 | 43 |
| Gumbo | | | | | |
| Dry | 1915 | 1275 | 50 | 2120 | -10 |
| Wet | 1985 | 1200 | 67 | 2205 | -10 |
| Gypsum | 2420 | 1410 | 72 | | |

| Material | Measured | | | | |
|---|---|---|---|---|---|
| | In-Situ | Loose | | Embankment | |
| | Mass Density[1] kg/m$^3$ | Mass Density[2] kg/m$^3$ | % Swell[3] | Mass Density[2] kg/m$^3$ | % Swell or Shrink[3] |
| Igneous rocks | 2795 | 1675 | 67 | 1960 | 43 |
| Kaolinite | | | | | |
| Dry | 1915 | 1275 | 50 | | |
| Wet | 1985 | 1190 | 67 | | |
| Limestone | 2600 | 1595 | 63 | 1910 | 36 |
| Loess | | | | | |
| Dry | 1910 | 1275 | 50 | 2120 | -10 |
| Wet | 1985 | 1190 | 67 | 2205 | -10 |
| Marble | 2680 | 1600 | 67 | 1875 | 43 |
| Marl | 2220 | 1330 | 67 | 1555 | 43 |
| Masonry, rubble | 2325 | 1395 | 67 | 1630 | 43 |
| Mica | 2885 | 1725 | 67 | | |
| Pavement | | | | | |
| Asphalt | 1920 | 1150 | 50 | 1920 | 0 |
| Brick | 2400 | 1440 | 67 | 1685 | 43 |
| Concrete | 2350 | 1405 | 67 | 1645 | 43 |
| Macadam | 1685 | 1010 | 67 | 1685 | 0 |
| Peat | 700 | 530 | 33 | | |
| Pumice | 640 | 385 | 67 | | |
| Quartz | 2585 | 1550 | 67 | 1780 | 43 |
| Quartzite | 2680 | 1610 | 67 | 1875 | 43 |
| Rhyolite | 2400 | 1435 | 67 | 1700 | 43 |
| Riprap rock | 2670 | 1550 | 72 | 1870 | 43 |
| Sand | | | | | |
| Dry | 1710 | 1535 | 11 | 1920 | -11 |
| Wet | 1835 | 1915 | 5 | 2050 | -11 |
| Sandstone | 2415 | 1495 | 61 | 1795 | 34 |
| Schist | 2685 | 1610 | 67 | 1880 | 43 |
| Shale | 2640 | 1470 | 79 | 1775 | 49 |
| Shale | 1920 | 1410 | 36 | 2310 | -17 |
| Siltstone | 2415 | 1495 | 61 | 2705 | -11 |
| Slate | 2670 | 1540 | 77 | 1870 | 43 |
| Talc | 2750 | 1650 | 67 | 1930 | 43 |
| Topsoil | 1440 | 960 | 56 | 1945 | -26 |
| Tuff | 2400 | 1600 | 50 | 1810 | 33 |

*Notes:*

1. *Subject to average ±5% variation.*
2. *Mass densities are subject to adjustments in accordance with modified swell and shrinkage factors.*
3. *Based on average in-situ densities. A negative number represents shrinkage. Factors subject to ±33% variation.*

**Exhibit 4.6–F SHRINK/SWELL FACTORS FOR COMMON MATERIALS (U.S. Customary)**

| Material | Measured | | | | |
|---|---|---|---|---|---|
| | In-Situ | Loose | | Embankment | |
| | Mass Density[1] lb/yd[3] | Mass Density[2] lb/yd[3] | % Swell[3] | Mass Density[2] lb/yd[3] | % Swell or Shrink[3] |
| Andesite | 4,950 | 2,970 | 67 | 3,460 | 43 |
| Basalt | 4,950 | 3,020 | 64 | 3,640 | 36 |
| Bentonite | 2,700 | 2,000 | 35 | — | — |
| Breccia | 4,050 | 3,040 | 33 | 3,190 | 27 |
| Calcite-Calcium | 4,500 | 2,700 | 67 | | |
| Caliche | 2,430 | 2,100 | 16 | 3,200 | -25 |
| Chalk | 4,060 | 2,170 | 50 | 3,050 | 33 |
| Charcoal | — | 1,030 | — | — | — |
| Cinders | 1,280 | 960 | 33 | 1,420 | -10 |
| Clay | | | | | |
| Dry | 3,220 | 2,150 | 50 | 3,570 | -10 |
| Damp | 3,350 | 2,010 | 67 | 3,720 | -10 |
| Conglomerate | 3,720 | 2,800 | 33 | | |
| Decomposed rock | | | | | |
| 75% R. 25% E. | 4,120 | 3,140 | 31 | 3,680 | 12 |
| 50% R. 50% E. | 3,750 | 2,710 | 38 | 4,000 | -6 |
| 25% R. 75% E. | 3,380 | 2,370 | 43 | 3,720 | -9 |
| Diorite | 5,220 | 3,130 | 67 | 67 | 43 |
| Diatomaceous earth | 1,470 | 910 | 62 | | |
| Dolomite | 4,870 | 2,910 | 67 | 3,400 | 43 |
| Earth, loam | | | | | |
| Dry | 3,030 | 2,070 | 50 | 3,520 | -12 |
| Damp | 3,370 | 2,360 | 43 | 3,520 | -4 |
| Wet, mud | 2,940 | 2,940 | 0 | 3,520 | -20 |
| Feldspar | 4,410 | 2,640 | 67 | 3,080 | 43 |
| Gabbro | 5,220 | 3,130 | 67 | 3,650 | 43 |
| Gneiss | 4,550 | 2,720 | 67 | 3,180 | 43 |
| Gravel (Dry) | | | | | |
| Uniformly Graded | 2,980 | 2,700 | 10 | 3,150 | -5 |
| Average Gradation | 3,280 | 2,730 | 20 | 3,570 | -8 |
| Well Graded | 3,680 | 2,770 | 33 | 4,130 | -11 |
| Gravel (Wet) | | | | | |
| Uniformly Graded | 3,310 | 3,150 | 5 | 3,150 | -5 |
| Average Gradation | 3,640 | 3,290 | 10 | 3,570 | -2 |
| Well Graded | 4,090 | 3,520 | 16 | 4,130 | -1 |
| Granite | 4,540 | 2,640 | 72 | 3,170 | 43 |
| Gumbo | | | | | |
| Dry | 3,230 | 2,150 | 50 | 3,570 | -10 |
| Wet | 3,350 | 2,020 | 67 | 3,720 | -10 |
| Gypsum | 4,080 | 2,380 | 72 | | |

| Material | Measured | | | | |
|---|---|---|---|---|---|
| | In-Situ | Loose | | Embankment | |
| | Mass Density[1] lb/yd$^3$ | Mass Density[2] lb/yd$^3$ | % Swell[3] | Mass Density[2] lb/yd$^3$ | % Swell or Shrink[3] |
| Igneous rocks | 4,170 | 2,820 | 67 | 3,300 | 43 |
| Kaolinite | | | | | |
| Dry | 3,230 | 2,150 | 50 | | |
| Wet | 3,350 | 2,020 | 67 | | |
| Limestone | 4,380 | 2,690 | 63 | 3,220 | 36 |
| Loess | | | | | |
| Dry | 3,220 | 2,150 | 50 | 3,570 | -10 |
| Wet | 3,350 | 2,010 | 67 | 3,720 | -10 |
| Marble | 4,520 | 2,700 | 67 | 3,160 | 43 |
| Marl | 3,740 | 2,240 | 67 | 2,620 | 43 |
| Masonry, rubble | 3,920 | 2,350 | 67 | 2,750 | 43 |
| Mica | 4,860 | 2,910 | 67 | | |
| Pavement | | | | | |
| Asphalt | 3,240 | 1,940 | 50 | 3,240 | 0 |
| Brick | 4,050 | 2,430 | 67 | 2,840 | 43 |
| Concrete | 3,960 | 2,370 | 67 | 2,770 | 43 |
| Macadam | 2,840 | 1,700 | 67 | 2,840 | 0 |
| Peat | 1,180 | 890 | 33 | | |
| Pumice | 1,080 | 650 | 67 | | |
| Quartz | 4,360 | 2,610 | 67 | 3,000 | 43 |
| Quartzite | 4,520 | 2,710 | 67 | 3,160 | 43 |
| Rhyolite | 4,050 | 2,420 | 67 | 2,870 | 43 |
| Riprap rock | 4,500 | 2,610 | 72 | 3,150 | 43 |
| Sand | | | | | |
| Dry | 2,880 | 2,590 | 11 | 3,240 | -11 |
| Wet | 3,090 | 3,230 | 5 | 3,460 | -11 |
| Sandstone | 4,070 | 2,520 | 61 | 3,030 | 34 |
| Schist | 4,530 | 2,710 | 67 | 3,170 | 43 |
| Shale | 4,450 | 2,480 | 79 | 2,990 | 49 |
| Shale | 3,240 | 2,380 | 36 | 3,890 | -17 |
| Siltstone | 4,070 | 2,520 | 61 | 4,560 | -11 |
| Slate | 4,500 | 2,600 | 77 | 3,150 | 43 |
| Talc | 4,640 | 2,780 | 67 | 3,250 | 43 |
| Topsoil | 2,430 | 1,620 | 56 | 3,280 | -26 |
| Tuff | 4,050 | 2,700 | 50 | 3,050 | 33 |

Notes:

1. *Subject to average ±5% variation.*
2. *Mass densities are subject to adjustments in accordance with modified swell and shrinkage factors.*
3. *Based on average in-situ densities. A negative number represents shrinkage. Factors subject to ±33% variation.*

### 4.6.3 MATERIAL SOURCES AND EXCAVATION

FLH guidance to be drafted.

Standards for analysis of materials are in PDDM Section 6.4.6.3.

The primary source supporting the standards and guidance is FHWA-ED-88-053 and the secondary source is WSDOT WA-M-46-03.

Refer to Appendix B.12, Material Sources.

#### 4.6.3.1 Use of Excavated Materials

Determine the extent of each soil unit, the preferred uses for each unit (such as common fill, structural fill, drain rock, and riprap), and any measures to improve soil units to meet specifications. Soil excavated from within the roadway prism intended for use as embankment fill should generally meet Standard Specifications for common borrow. If all weather use is desired, the material should meet the specifications for select borrow per Standard Specifications. Identify any soil units considered unsuitable for reuse, such as moisture-sensitive silts, highly plastic soil, peat, and muck, for possible avoidance or disposal.

Where there is a potential for a significant volume of boulders, determine the estimated percentage of the excavation quantity. The excavated materials that will not be suitable for unrestricted use in embankments and fills should also be determined. Evaluate if these materials may be used in restricted locations and/or with special construction considerations.

Consideration should be given to the location and time of year that construction will likely take place when using moisture-sensitive soils. Techniques to treat wet soils, such as adding Portland cement, can lower the moisture content of soil a few percent and provide some strength. However, concerns regarding the pH of runoff water from the project site may limit the use of this technique on some sites. FHWA-SA-93-004/5 provides additional information on soil amendments.

#### 4.6.3.2 Material Sources

For aggregate sources, determine the rock quality characteristics through the suite of aggregate quality tests including Los Angeles Abrasion (AASHTO T 96), Sodium Sulfate Soundness (AASHTO T 104), and Fine and Coarse Durability (AASHTO T 210), For paving aggregate sources, preliminary asphalt concrete mix design may be evaluated with Preliminary Immersion Compression tests (AASHTO T 165 and 167). Identify potential difficulties with development of the material sources. Evaluate slope stability for proposed excavation slopes, which is usually based precedent and professional judgment. In some cases, depending on complexity, materials and depths involved, slope stability analyses might be necessary. Provide recommendations for temporary excavation and final slope ratios, bench heights, and bench widths. A primary reference is FHWA-ED-88-053, and additional guidance is provided in WSDOT WA-M-46-03.

Determine the estimated waste and appropriate shrink/swell factors to convert the needed cubic yards to yards in place (bank yards) at the proposed source. This does not address or account for losses or wastage on construction. Include a safety factor when estimating the quantity of available material present at the site. The total quantity of materials available from all material sources provided for a specific project should be 10 to 50 percent in excess of the project needs. Extrapolation beneath the depth of explorations should not be made.

#### 4.6.3.3 Mining and Reclamation Plans

The source development plan consists of a map and cross sections that indicate how the resource will be developed and demonstrates the logic for excavation and material extraction of the site. Identify any special problems associated with the material present at the site, such as oversize material or excessive overburden. Identify the stockpile and waste areas for overburden and crusher reject material. Designate locations of haul roads, gates, fences, and source limits. Provide cross sections of slope angles and the proposed elevation of the source floor.

Designate recommended slope angles for final reclamation. For quarry sites, slopes should be based on the rock parameters mapped and identified specifically at the quarry. Show the locations of roads, stockpile storage, waste, overburden and proposed finished topography reclamation plan map.

### 4.6.4 SUBGRADE STABILIZATION

Determine the limits of all subgrade soils considered unsuitable for embankment or roadway construction. The strength and compressibility of these deposits and impacts on construction should be evaluated. The geotechnical professional should provide a recommendation based on an evaluation that considers potential mitigation measures and any long term maintenance considerations. Primary references are FHWA-SA-93-004/5. Additional references include FHWA-IP-80-002, FHWA-HI-95-038 and FHWA-TS-80-236. For soil stabilization design utilizing geotextiles, evaluate subgrade shear strength data for reinforcement design. Geotextile strength properties are the primary parameter to design (see FP-XX and ASCE GPS 76). For geotextile separation design applications, evaluate soil classifications and gradations. Determine if other geotextile functions should be included, such as drainage and filtration. Often, the construction process imparts significant stresses on the geotextile and should be accounted for in the selection of geotextile strength properties.

> Geotechnical standards on soil amendments are provided in PDDM Section 6.4.6.4.
>
> The primary source supporting the standards and guidance is FHWA-SA-93-004/5. Secondary sources are FHWA-HI-95-038 and FHWA-TS-80-236.
>
> Refer to Appendix B.12, Material Sources.

### 4.6.5 EMBANKMENTS

The suitability of in situ materials for use as roadway Embankment Borrow is determined by analysis of site investigations and testing. Embankment materials and construction requirements should follow FLH Standard Specifications, unless special provision specifications are needed (such as special compaction requirements when using degradable rock). General earthwork guidance is provided in NHI 132033, FHWA-ED-88-053, NHI 132012, FHWA-NHI-00-043 and TRB SAR 8. References for use of degradable rock materials include: FHWA-TS-80-219 and ORDOT EFR OR 83-02.

Determine if materials within the proposed project excavations meet required specifications of Borrow Embankment materials. Estimate the available quantities of each identified material type. Identify the location of material types in order to provide information for project designers and contractors regarding potential earthwork and staging 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.

Standards for embankment analysis and design are in PDDM Section 6.4.5.

The primary source supporting the standards and guidance is NHI 132033. Secondary sources are WSDOT WA-M-46-03, and USACE EM 1110-1-1904 for settlement, and USACE EM 1110-2-1902 and FHWA-SA-94-005 for stability.

Refer to Appendix B.12, Material Sources.

#### 4.6.5.1 Embankment Settlement

The total amount and time rate of settlement can significantly influence a project's design, duration and cost. Primary consolidation and secondary compression can continue to occur long after the embankment is constructed. Post construction settlement can damage structures and utilities located within the embankment. Embankment settlement near an abutment could create an unsafe dip in the roadway surface, or down drag and lateral forces on the foundations. If the primary consolidation is allowed to occur prior to placing utilities or building structures that would otherwise be impacted by the settlement, the impact is essentially mitigated. However, it can take weeks to years for primary settlement to occur, and significant secondary compression of organic soils can continue for decades. Many construction projects cannot absorb the scheduling impacts associated with waiting for primary consolidation and/or secondary compression to occur. Therefore, estimating the time rate of settlement is often as important as estimating the magnitude of settlement.

Coordination with the designer is necessary to determine what time is available for construction waiting periods to allow for settlement to occur. When large settlements are anticipated, embankment stability may also likely be a problem. The majority of the solutions to slope instability also apply to embankment settlement. The geotechnical professional should evaluate both issues at the same time to determine the most feasible solution.

NHI 132012, NHI 132033, and Chapter 9 of WSDOT WA-M-46-03 provide technical guidance for estimating settlement. The geotechnical professional may use FoSSA or EMBANK software distributed by FHWA to estimate settlements. Secondary references include FHWA-SA-92-045, FHWA-SA-98-086R, and USACE EM 1110-1-1904.

Evaluate mitigation measures if design analyses indicate excessive settlement magnitude or duration. Mitigation measures could include: 1) surcharging/preloading, 2) removal of settlement-prone material, 3) installation of vertical drainage systems, 4) ground improvement, 5) decreasing the height of embankments, 5) lightweight fill, and 6) spanning the compressible area with a pile-supported structure. Refer to Section 5.3.7.3 and Section 5.7 for discussion of ground improvement options and NCHRP Synthesis 147.

### 4.6.5.2 Embankment Slopes

Small fill slopes are generally designed based on local precedence and engineering judgment. In general, design by precedence is used when new fills will be constructed less than 3 m (10 ft) at the same angle of inclination and there is no evidence of instability. Fill slopes greater than 3 m (10 ft) in height usually require geotechnical studies. For slope stability analysis of significant fill slopes, the desired minimum safety factor (FS) is typically in the order of 1.3 to 1.5, depending on the levels of importance and risk, and the uncertainty in the slope analysis input parameters. Unstable slopes that can be reliably back-analyzed to determine representative shear strengths may be redesigned with a lower factor of safety of 1.2 to 1.3. Factors of safety for short-term conditions (i.e., during construction) can be less than the long-term factors of safety, typically about 1.1 to 1.2. To mitigate the problem of embankment slope instability, the geotechnical professional should evaluate the feasibility of flatter slopes, alternate horizontal or vertical alignments, soil reinforcement with geosynthetics, partial/total removal of weak foundation soils, controlled filling, counterweight berms, shear keys, lightweight fill, and installation of subsurface drainage. Section 5.5.1 provides guidance for performing slope stability analyses and Section 5.5.2 provides guidance for evaluating stability of landslides and possible mitigation measures. Design guidance is described in NHI 132012 and FHWA-SA-94-005. Additional guidance is described in USFS EM 7170-13, NHI 132033, Chapter 10 of WSDOT WA-M-46-03, and FHWA-ED-88-053.

### 4.6.5.3 Embankment Mitigation Methods

#### 4.6.5.3.1 Surcharging (Preloading)

The primary purpose of a preload/surcharge is to speed up the consolidation process. Based on previous experience, the preload fill needs to be at least one-third the design height of the embankment to provide any significant time savings. Using a preload or surcharge typically will not completely eliminate secondary compression, but it has been successfully used to reduce the magnitude of secondary settlement. However, for highly organic soils or peat where secondary compression is expected to be high, the success of a surcharge to reduce secondary compression may be quite limited. Other more positive means may be needed to address the secondary compression in this case, such as removal. Design guidance is described in Chapter 10 of WSDOT WA-M-46-03. Two significant design and construction considerations for using surcharges include embankment stability and re-use of the additional fill materials. New embankments constructed over soft soils can result in stability problems. Adding additional surcharge fill would only exacerbate the stability problem. After the settlement objectives have been met, the surcharge would need to be removed and preferably incorporated for necessary embankment use elsewhere on the project. When surcharge soils must be handled multiple times, it may be advantageous to use sand and/or gravel borrow to reduce workability issues.

#### 4.6.5.3.2 Vertical Drainage Systems (Wick Drains)

Vertical drainage systems can be used to speed the rate of consolidation. Select a drain spacing that results in a consolidation that meets the project construction time requirements. The more common type of vertical drainage is a wick drain, which consist of a plastic drainage core wrapped in a nonwoven geotextile, installed by a rig that drives the wicks into place. 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 embankment is then constructed on top of the sand blanket. Refer to FHWA-RD-86-168 and FHWA-SA-98-086R.

#### 4.6.5.3.3 Controlled Filling

This mitigation measure consists of constructing the embankment in stages to not exceed the available strength of the foundation soil. Use limit equilibrium and primary consolidation analyses to develop guideline heights and staging durations to maintain stability during construction. Ladd 1991.

#### 4.6.5.3.4 Overexcavation/Replacement

This mitigation measure consists of excavating the soft compressible soils from below the embankment footprint and replacing these materials with higher quality, less compressible soil. Because of the high costs associated with excavating and disposing of unsuitable soils as well as the difficulties associated with excavating below the water table, this method has limited application. The relevant considerations include:

- The area requiring overexcavation is relatively shallow;
- Temporary shoring and dewatering are not required;
- The unsuitable soils can be wasted on site or readily disposed;
- Suitable excess or borrow materials (usually granular) are readily available to replace the over-excavated unsuitable soils.

#### 4.6.5.3.5 Light-Weight Fill

Light fill materials could be considered when embankment slope stability using conventional materials is unacceptable. Lightweight fill can consist of a variety of materials including polystyrene blocks (geofoam, EPS), light weight aggregates (rhyolite, expanded shale, blast furnace slag, fly ash), wood fiber and fresh sawdust, and shredded rubber tires. Lightweight fills are generally used for two conditions: the reduction of the driving forces contributing to instability and reduction of potential settlement resulting from consolidation of compressible foundation soils. Situations where lightweight fill may be appropriate include conditions where the construction schedule does not allow the use of staged construction, where existing utilities or adjacent structures are present that cannot tolerate the magnitude of settlement induced by placement of typical fill, and at locations where post-construction settlements may be excessive under conventional fills. Refer to FHWA-SA-98-086R, and NCHRP RR 529.

#### 4.6.5.3.6 Counterberms

This type of mitigation improve the stability of an embankment by placing an adjoining smaller embankment at the toe, which increases the resistance along the potential failure surface. Other terms for this mitigation measure include "toe berms" and "counterbalances." Counterberms could be considered when embankment slope stability for conventional materials is inadequate (due to low shear strengths in foundation soils) and there is ample right-of-way. Counterbalances are sized using stability analyses, and should be checked for both short-term and long-term cases. Design guidance is described in FHWA-SA-94-005, and chapter 10 of WSDOT WA-M-46-03.

#### 4.6.5.3.7 Subdrainage

Methods to minimize the harmful affects of groundwater may be necessary in embankments and to mitigate slope stability problems. Refer to section 5.6.1 for subdrainage guidelines and FHWA-TS-80-224. For geotextile separation/filter design applications, evaluate soil classifications and gradations. Determine if other geotextile functions should be included. Often, the construction process imparts significant stresses on the geotextile and should be accounted for in the selection of geotextile strength properties.

#### 4.6.5.3.8 Embankment Reinforcement

This mitigation measure improves embankment stability, especially where right-of-way is constrained. One method is to place horizontal layers of reinforcement. Common reinforcement includes geogrids and high-strength woven geotextiles. Another method is to construct a rock-filled shear key. Both methods require geotechnical stability analysis to determine size requirements.

#### 4.6.5.3.9 Embankment Zoning

There may be reasons to use different materials in embankment construction, such as aggregate subdrains, foundation subexcavation, and shear keys. When materials of different gradations are placed, the contact zone might require a filter blanket or geotextile to prevent piping and subsidence.

### 4.6.6 SLOPE REINFORCEMENT

Steepened slopes (steeper than 1V:1.5H) should be evaluated for internal and external stability including all failure possibilities; sliding, deep-seated overall instability, bearing capacity failure, and excessive settlement. Design slope face treatments to minimize erosion. FHWA-NHI-00-043 and GEC-2 provide design guidance.

> Standards for slope reinforcement are in PDDM Section 6.4.6.6
>
> The primary source supporting the standards and guidance is FHWA-NHI-00-043 and the secondary source is GEC-1.
>
> Refer to Appendix B.12, Material Sources.

## 4.7 SLOPE STABILITY

Types of slope stability analyses can be performed for rotational, sliding block, irregular and infinite slope failure surfaces. Detailed assessment of soil and rock stratigraphy is critical to the proper assessment of slope stability. It is important to define any thin weak layers and slickensides. Long-term or short-term stability considerations affect the selection of soil and rock shear strength parameters. Detailed assessment should be made of the groundwater regime within and beneath the slope and potential seepage at the face of the slope.

Limit equilibrium methodologies are generally used to assess slope stability of cuts and fills. The Modified Bishop, simplified Janbu, Spencer, or other widely accepted slope stability analysis methods should be used to evaluate various failure mechanisms.

For very simplified cases, design charts to assess slope stability are available. Examples of simplified design charts are provided in NAVFAC DM-7.1, USFS EM 7170-13 and FHWA-SA-94-005. Simplified design charts can be used for preliminary evaluations, as well as final design of non-critical slopes that are less than 6 m (20 ft) high and that are consistent with the associated simplified assumptions.

For slope stability analysis of significant cuts and fills, the desired minimum safety factor (FS) is typically in the order of 1.3 to 1.5, depending on the levels of importance and risk, and the uncertainty in the slope analysis input parameters. Unstable slopes that can be reliably back-analyzed to determine representative shear strengths may be redesigned with a lower factor of safety of 1.2 to 1.3. Factors of safety for short-term conditions (i.e., during construction) can be less than the long-term factors of safety, typically about 1.1 to 1.2. A primary reference for slope stability analysis is USACE EM 1110-2-1902. Refer also to FHWA-SA-94-005. Additional guidance is described in USFS EM 7170-13, NHI 132012, Duncan & Wright 2005, WSDOT WA-M-46-03, and FHWA-ED-88-053.

If the potential slope failure mechanism is anticipated to be relatively shallow and parallel to the slope face, with or without seepage affects, an infinite slope analysis may be applicable.

In evaluating the acceptability of the factor of safety of slope stability, the geotechnical professional should consider the method of analysis used, the reliability of the subsurface data, past experience with similar soils, uncertainties in analysis laboratory data and the consequences of slope failure. In addition, project constraints in scenic corridors might dictate the use of lower factors of safety to minimize impacts to environmental and cultural resources. The use of lower factors of safety could be justifiable considering the low traffic volumes and road user familiarity (AASHTO LV Roads; and FLH NPS Road Stds).

### 4.7.1 SOIL CUT SLOPES

Design of cut slopes considers the past performance of slopes in the project vicinity. Indirect relationships, such as subsurface drainage characteristics may be indicated by vegetative pattern. Assess whether tree roots may be providing anchoring of the soil and if there are any existing trees near the top of the proposed cut that may become a hazard after the cut is completed. Changes in ground surface slope angle may reflect differences in physical characteristics of soil and rock materials or the presence of water. Provide recommendations for maximum earth and/or rock slope ratios on a station-by-station basis.

Small cut slopes are generally designed based on local precedence and engineering judgment. In general, design by precedence is used when the following conditions apply: 1) new cuts will be made into an existing slope 3 m (10 ft) or less and at the same angle of inclination, 2) slope height does not increase significantly, 3) there is no evidence of instability, 4) material types at the excavation face appear consistent, and 5) there is no apparent seepage in the cut.

Cut slopes greater than 3 m (10 ft) in height usually require geotechnical studies. Situations that could require analysis include: 1) large cuts, 2) cuts with varying stratigraphy (especially if weak zones are present), 3) cuts where high groundwater or seepage forces are likely, 4) cuts involving weaker soils, or 5) cuts in old landslides or in formations known to be susceptible to landsliding.

For slope stability analysis of significant cuts, the desired minimum safety factor (FS) is typically in the order of 1.3 to 1.5, depending on the levels of importance and risk, and the uncertainty in the slope analysis input parameters. Unstable slopes that can be reliably back-analyzed to determine representative shear strengths may be redesigned with a lower factor of safety of 1.2 to 1.3. Factors of safety for short-term conditions (i.e., during construction) can be less than the long-term factors of safety, typically about 1.1 to 1.2. Section 5.5.1 provides guidance for performing slope stability analyses and Section 5.5.2 provides guidance for evaluating stability of landslides and possible mitigation measures. Design guidance is described in FHWA-SA-94-005. Additional guidance is described in USFS EM 7170-13, NHI 132012, Chapter 10 of WSDOT WA-M-46-03, and FHWA-ED-88-053.

Major causes of cut slope failures include undermining the toe of the slope and oversteepening the slope angle, or cutting into heavily overconsolidated clays. The base of the cut slope should be protected by not oversteepening the slope angle and by keeping drainage ditches near the toe a reasonable distance away. There are a number of options to increase the stability of a cut slope, including:

- Flattening slopes.
- Lowering the water table.
- Buttressing the slope with rockfill or rock inlay.

- Structural systems such as retaining walls or reinforced slopes (see Section 5.2).

Design should include establishing vegetation on the slope to prevent long-term erosion. It may be difficult to establish vegetation on slopes with inclinations greater than 1V:2H without the use of erosion mats or other stabilization methods.

Proper drainage provisions are very important when designing cut slopes. Surface drainage can be accomplished through the use of drainage ditches and berms located above the top of the cut, around the sides of the cut, and at the base of the cut. Cut slopes should be designed with adequate drainage and temporary and permanent erosion control facilities to limit erosion and piping.

Loessal soils require special design for cut slopes; refer to WSDOT WA-RD-69, and WSDOT WA-M-46-03.

Standards for cut slope stability are in PDDM Section 6.4.7.1

The primary source supporting the standards and guidance is USACE EM 1110-2-1902. Secondary sources are USFS EM 7170-13, Duncan & Wright 2005 and FHWA-SA-94-005.

Refer to Appendix B.13, Slope Stability and Landslides.

### 4.7.2 LANDSLIDES

Landslide analysis and remediation presents one of the more difficult geotechnical engineering challenges due to the variable size and complexity of landslides. Unique reconnaissance and exploration approaches apply. Landslides can be improperly diagnosed because of inadequate geologic reconnaissance or interpretation and poorly conceived exploration/instrumentation programs. Responding to and investigating a landslide is likened to a forensic investigation. The Geotechnical Engineer/Engineering Geologist 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 more extensive exploration and instrumentation programs, along with experienced engineering geology and geotechnical engineering expertise. In these cases, the causation mechanisms are probably complex and difficult to determine without such specialized, expert involvement.

For most Federal Lands Highway projects, the size of the landslide and costs of correction should be subjectively evaluated relative to the potential impacts of non-correction before detailed investigations are authorized and initiated. When remediation is authorized, use of lower-cost remediation methods (e.g., alignment shifts, grade changes, excavation, horizontal drains and/or drain trenches, rock buttresses, toe berms and shear keys) to remove driving forces or increase resisting forces should be routinely considered. More expensive structural alternatives should be considered (e.g. tieback or shear pile wall) when geometric, right-of-way or other constraints dictate, or when the risks and consequences of failure justify more expensive mitigation methods.

The information and procedure that are common for landslide analysis and design include the following:

- Physical limits and dimensions of the landslide. Extend survey information beyond the landslide limits to provide detailed analysis.
- Review the Landslide Correction Checklist in Exhibit 5.1–J to ensure all appropriate information is available.
- Determine the most probable causative factors for the landslide.
- Develop the best possible geologic model of the existing conditions. This would include at least one geologic cross section along the centerline of the landslide, developed from borings or other exploratory methods, that shows the most likely slide surface or shear zone, groundwater levels, and pore-water pressure on the sliding plane or zone.
- Magnitude and rate of existing movements. This should be determined from a combination of surface measurements and subsurface slope indicator measurements.
- Back-analysis of the existing slide to determine appropriate geotechnical parameters for evaluation of remedial options.
- Select desired remediation FS or percent increase in stability.
- Perform appropriate stability analysis (including back analysis, when appropriate).
- Evaluate conceptual options to improve stability. The reliability of a remedial option can influence the desired FS (e.g. a rockfill buttress usually provides a more reliable solution than drainage only. In this instance a lower FS may be acceptable for the buttress than the drainage only option).
- Recommendations and/or design alternatives for corrective actions and evaluation of future risk.

The geotechnical professional should evaluate several applicable options to determine which option is more cost-effective and best satisfies the project criteria and constraints. The options analysis includes an assessment of risk, uncertainty, possible consequences, constructability, material availability, environmental/cultural impacts, and costs for each option. Usually, the landslide remediation options also need to be constructable while providing access for traffic, which could include requiring the availability of one or more lanes and limiting time delays.

The factor of safety (FS) selected for landslide analysis and remediation should vary with the type of facility, potential damages, and amount and quality geotechnical data regarding size/depth of the landslide and groundwater levels. In addition, the reliability of site-specific rainfall and ground water level should be considered. Generally, for small landslides a remediation factor of safety in the 1.20 to 1.5 range is used, with 1.25 being most common. The use of a FS of 1.25 implies high confidence in the subsurface model, where the exploration provides good geometric and pore-water pressure data on the slip surface, allowing accurate "back-analysis" of soil shear strength at failure for FS equal 1.0. The back-analyzed shear strength is then used in the FS calculation for correction options.

Sometimes, lower remediation FS are used for larger landslides and when risks and consequences are acceptable, particularly when the cost of providing FS of 1.2 or greater is inordinately expensive. This is particularly relevant for rehabilitation or realignment of existing low-volume roads and where environmental and cultural impacts could be significant. An alternative to using factor of safety design criteria is to provide improvements in the level of stability based on restoring resistive forces that are slightly greater than that removed by construction (using an "Original Profile Analysis," Cornforth 2005). The following commentary provides further guidance on FS and stability improvement selection.

Cornforth 2005 provides excellent guidance on selection of appropriate FS or percent increase in stability. See section 1.4 titled Remediation of Landslides and Chapter 10 Stability Margin. Table 10.1 gives "Suggested Guidelines for Factor of Safety in Landslide Studies According to Level of Information and Landslide Size". Section 10.2 sets forth the "Principle of Original Profile Analysis" a concept developed by the author (Cornforth 1995) as a method of determining the amount of resistance required to reestablish equilibrium in a slope that has been destabilized by construction activities. If for example, the Original Profile Analysis indicates that the calculated factor of safety before construction was 1.14, the designer knows that a remedial factor of safety of 1.14 or higher is sufficient and does not need to provide values of 1.25 or some other arbitrary number. This Original Profile Analysis concept is particularly applicable to FLH low-volume road projects.)

The primary and secondary geotechnical references for landslide investigations and analyses for transportation projects are

- TRB SR 247
- FHWA-SA-94-005
- Cornforth 2005
- FHWA-RT-88-040
- GEC-4
- FHWA-ED-88-053

Mitigation methods that are considered "experimental" or have unusual structural requirements may warrant special field testing to verify the methods will perform as intended. For example, it is advisable to conduct pre-production load tests on high-capacity ground anchors to verify that anticipated capacities can be achieved. Pilot programs with horizontal drains, vertical relief wells, or test drainage shafts/tunnels can be valuable to ascertain the degree of drawdown that can actually be achieved in a specific geologic setting and to verify the adequacy of construction methods. Experimental methods could include stabilization with grout or chemical injection, for example.

For monitoring of unstable slopes and landslides, refer to TRB SR 247, WSDOT Unstable Slope Management System (WSDOT USMS Guidelines) and WSDOT PowerPoint presentation for Real Time Monitoring (WSDOT Monitoring).

Standards for landslides are in PDDM Section 6.4.7.2.

The primary source supporting the standards and guidance is TRB SR 247. Secondary sources are Cornforth 2005, FHWA-RT-88-040 and FHWA-ED-88-053.

Refer to Appendix B.13, Slope Stability and Landslides.

## 4.8 ROCK ENGINEERING

### 4.8.1 ROCK SLOPES

Rock slope designs primarily determine the suitable slope angle for stability and the ditch width necessary to control most rockfall. Evaluate 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 (reduces excavation quantities) that achieves acceptable cut slope performance (reduced rockfall and improved safety). 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. Primary guidance is presented in NHI 132035, with additional information in FHWA-TS-89-045, FHWA-OR-RD-01-04, and NHI 13211.

The subsurface evaluation includes characterization of the rock in the slope based on rock classification, degree of weathering and presence of discontinuities. Consider the degradability of rock (such as the weathering potential and erodibility) because these characteristics can adversely affect the long-term stability of the slope. Various forms of rock quality designations exist, including the RQD and RMR (GEC-5).

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 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. Determine the resistance to sliding by performing shear tests or develop an estimate by observing the inclinations of pre-existing failure surfaces.

Evaluate groundwater conditions 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 and can be measured with piezometers.

In some cases, right-of-way limitations or other factors, such as economics, may require the design slope to be steeper than desirable. In the event this results in potentially unstable conditions, mitigation measures might be necessary. Rock slope and rockfall issues can often be mitigated by several different options, including catch ditches, barriers, catch fences, draped mesh, rock bolting, dowels, shotcrete facing, cable lashing, buttresses, etc. Evaluate the conceptual options for feasibility, constructability, risk-management, ability to satisfy project constraints and environmental criteria, future consequences and preliminary estimates of cost.

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 (pre-shear) or trim (cushion) blasting, produces a cut slope with significantly less potential for rockfall.

Recommendations for maximum cut slope angles and fall-out ditch widths, along with recommended stabilization measures, should be presented on a station-to-station basis.

#### 4.8.1.1 Rock Slope Analysis

Design of rock cut slopes considers the past performance of slopes in the project vicinity. Small rock cuts are generally designed based on local precedence and engineering judgment. In general, design by precedence is used when the following conditions apply: 1) new cuts will be made into an existing slope 3 m (10 ft) or less and at the same angle of inclination, 2) slope height does not increase significantly, 3) there is no evidence of instability, and 4) rock types at the excavation face appear consistent. Taller rock cuts could be designed by evaluating the angle of primary sets of discontinuities and applying engineering judgment. However, slope stability analyses may be necessary when the rock is highly fractured or weathered and groundwater affects stability. Provide recommendations for maximum rock slope ratios on a station-by-station basis.

Kinematic analysis of the discontinuities may be performed to determine the most likely mode of potential slope failure. The kinematic analysis determines whether the orientations (dip and dip direction) of the various discontinuities could intersect 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 include: 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 kinematic analysis involves a comparison of the orientations of the dominant discontinuity sets with the orientation of the cut slope. Use a stereonet to display the discontinuity and slope data in this analysis. For detailed discussions of stereographic analysis, refer to NHI 132035, and FHWA-TS-89-045. Refer to checklists in Section A and B of FHWA-ED-88-053.

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. 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 too low or the rockfall potential onto the facility is unacceptably high. Although analyses should be performed, sound judgment should be applied because of uncertainties in conditions present within a rock mass. 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. Design references include: FHWA-CFL/TD-05-008, NHI 132035, FHWA-TS-89-045, FHWA-SA-93-057, and FHWA-OR-RD-01-04.

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 and type of discontinuities, shear strength and groundwater conditions.
- Size of slope and potentially unstable blocks.
- The criticality of the facility.
- Cost to provide additional stability.
- Whether the slope is temporary or permanent.
- The level of acceptable long-term risk.

For significant cuts in rock, 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. In addition, project constraints in scenic corridors might dictate the use of lower factors of safety to minimize impacts to environmental and cultural resources. The use of lower factors of safety could be justifiable considering the low traffic volumes and road user familiarity (AASHTO LV Roads; and FLH NPS Road Stds). Include rock slope stabilization and rockfall mitigation measures in the design 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.

#### 4.8.1.2 Rock Slope Construction Methods

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. Construction measures to enhance stability include installation of reinforcement, drainage, and erosion protection systems. Guidance is provided in NHI 13219, and additional references include FHWA-OR-RD-01-04, PTI 2004, USACE EM 1110-1-2907, FHWA-SA-93-057, and NHI 132035. Rockfall mitigation methods are described in Section 5.4.3. The following is a partial list of available techniques:

- <u>Controlled Blasting - </u>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 4.5 m (15 ft). Controlled blasting is routinely used for rock cuts that are 1V:0.75H or steeper. The limiting factor is the inability to maintain proper blast hole alignments on flatter slopes. Blasting specifications and guidance are described in FHWA-GA 7, and NHI 13211.

- Scaling - 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.

- 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. Refer to Section 10.6 of NHI 132035 (Not in chart to link) and Section 7.3.5 of NHI 13219.

- 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 that prevents build up of groundwater pressures. Refer to Section 10.6.6 of NHI 132035.

- 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. Refer to section 10.6.4 of NHI 132035 and section 7.3.4 of NHI 13219.

- 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.

Standards for rock slope analysis are in PDDM Section 6.4.8.1.

The primary source supporting the standards and guidance is NHI 132035. Secondary sources are FHWA-TS-89-045 and FHWA-HI-92-001.

Refer to Appendix B.14, Rock Slopes and Rockfall Mitigation.

### 4.8.2 ROCKFALL ANALYSIS

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. Evaluate rockfall situations to determine the likelihood of rocks reaching the road and recommend appropriate control or protection measures. Consider the consequences and probabilities of falling rocks reaching the road or facilities and weigh them against the cost of installing control measures. Refer to NHI 13219 and FHWA-OR-RD-01-04 for general discussions on rockfall control design. Refer also to section 10.8 of NHI 132035 and Chapter 12 of FHWA-TS-89-045. An additional reference is USACE EM 1110-1-2907. 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.).

The Rockfall Hazard Rating System (RHRS) is a technical method that provides a measure of rockfall risk (FHWA-SA-93-057). Criteria for evaluating rockfall mitigation measures are normally based on judgment and probabilistic analyses. Experience is the best predictor of the effectiveness of rockfall remedial design. Consult case histories in similar rock conditions to obtain additional guidance. Identify potential instabilities that could occur during construction as well as for permanent slope conditions.

Standards for rockfall hazard assessment are in PDDM Section 6.4.8.2.

The primary source supporting the standards and guidance is FHWA-SA-93-057. Secondary sources are FHWA-OR-RD-01-04 and NHI 132035.

Refer to Appendix B.14, Rock Slopes and Rockfall Mitigation.

### 4.8.3 ROCKFALL MITIGATION

Rockfall mitigation could consist of catch ditches, barriers, catch fences, draped mesh, rock bolting, dowels, shotcrete slope facing, cable lashing, buttresses, etc. Rockfall mitigation methods are described in NHI 13219 and FHWA-OR-RD-01-04, and are briefly summarized below.

- Fallout area and barrier design may be performed with the aid of the detailed design charts included in “Rockfall Catchment Area Design Guide” (FHWA-OR-RD-01-04). 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 (described in NHI 13219), is widely used for this purpose. The computer program RocFall available from Rocscience Inc. 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.

- 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. Scaling removes loose rock from the cut slope face and is routinely used to provide an immediate reduction in the rockfall potential. However, scaling is considered a temporary measure. Refer to Section 10.7 of ____ for descriptions on resloping and unloading, trimming, scaling, and rock removal operations.

- Reinforcement or external support methods including, shotcrete, dowels, rock bolts, rock anchors, cable lashing, or concrete buttresses, can provide longer-term protection. Rock bolting requirements and design guidelines are provided in NHI 132035, PTI 2004, and USACE EM 1110-1-2907.

- Screening 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. Draped mesh usually consists of gabion style mesh or higher strength wire meshes. A higher capacity system would include pinned-in-place mesh. Refer to Section 7.4 of NHI 13219, and WSDOT WA-RD-612.

- Catch Fence Systems can range gabion fences to proprietary systems, such as ring mesh and cable mesh systems. Refer to Section 7.4 of NHI 13219.

Standards for applying rockfall mitigation standards are in PDDM Section 6.4.8.3.

The primary source supporting the standards and guidance is FHWA-SA-93-085. Secondary sources are FHWA-CFL/TD-05-008, USACE EM 1110-1-2907, and NHI 132035.

Refer to Appendix B.14, Rock Slopes and Rockfall Mitigation.

### 4.8.4 FOUNDATIONS ON ROCK

Determine the influence that dominant joint sets in the rock have on foundation performance. Refer to GEC-6, NHI 132037A-1 and USACE EM 1110-1-2908 that provide a summary of methods to calculate bearing capacity of competent as well as jointed rock. More in-depth discussions are available in Wylie 1999 and Canadian Foundation.

Standards for foundations on rock are in PDDM 6.4.8.4.

The primary source supporting the standards and guidance is AASHTO HB-17. Secondary sources are USACE EM 1110-1-2908, Wyllie 1992 and Canadian Foundation.

Refer to Appendix B.14, Rock Slopes and Rockfall Mitigation.

### 4.8.5 TUNNELS

Due to the extreme variability of conditions under which tunnels are constructed and the uniqueness of each tunnel design, criteria for tunnels should be established for each project on an individual basis. Refer to tunneling texts for detailed guidance, or consult with an expert in tunneling, including the FHWA tunneling expert at the headquarters' office. Primary guidance I presented in FHWA-IF-05-023. Additional information is described in FHWA-Tunnel Inspection and FHWA-Tunnel Maintenance.

Standards for tunnel analysis and design are in PDDM 6.4.8.5.

The primary source supporting the standards and guidance is FHWA-IF-05-023

Refer to Appendix B.14, Rock Slopes and Rockfall Mitigation.

## 4.9 DRAINAGE, DEWATERING, AND EROSION CONTROL

Based on the results of the subsurface investigation, evaluate the need for surface and subsurface drainage within the roadway section, toe of slope cut, mid-slope, or other locations. Control of water starts with planning adequate surface drainage and routing of water away from sensitive areas and landslides. The presence of saturated soils, shallow ground water and seasonal variations in groundwater elevations can influence stability and road performance.

Sources of subsurface water include gravity flow of groundwater, capillary water that moves upward through soils, and artesian groundwater that is under pressure. In general, adverse effects of water and moisture could cause slope failures, landslides, piping erosion and subsidence, subgrade pumping, heave or blowouts in excavations, uplift of structures due to buoyancy, frost heave and unsatisfactory pavement performance.

In order to design a reliable, economic and adequate subsurface drain, evaluate the following information:

- Locations of all seepage areas in the vicinity of the roadway.
- Maximum rates of flow (measured or estimated).
- Locations of aggregate sources suitable for drain rock and filter material (or determine the suitability of using a filter geotextile).
- Climatic data and anticipated frost penetration depths.
- Laboratory tests indicating the potential for corrosion and frost susceptibility.

Surface drainage systems may include interceptor ditches, drainage channels, culverts, retention basins and dry wells. Subsurface drainage systems may include pavement underdrains and edge drains, deeper trench drains, shallow French drains, horizontal drains, vertical relief drains, granular drainage blankets and chimney drains, and interceptor drains. Exhibit 4.9–A through Exhibit 4.9–E present design details used by Federal Lands Highway for subsurface drainage.

The functions of subsurface drainage are to reduce adverse effects on roadways. These functions are more specifically stated in terms of the following requirements:

- To draw-down or lower groundwater levels in the area of a roadway, cut and fill slopes, embankment and structure foundations.

- To eliminate active springs or seeps beneath pavements and retaining structures by intercepting the seepage.
- To drain surface water infiltrating into the pavement structural section and retaining structures.
- To collect discharge from various drainage systems.

Do not overlook the importance of surface water management. It is generally easier to intercept and transport water before it runs on to an area of concern and before it infiltrates the ground. Surface water control is generally conducted through grading and may include the use of asphalt or concrete pavement, or shotcrete or geosynthetic clay liner (GCL) to provide an impermeable membrane to convey water with less infiltration. Drainage berms and ditches may need to be designed and constructed to accommodate movement when used on unstable slopes.

### 4.9.1 SURFACE DRAINAGE

Open channels and ditches help to divert surface water and shallow groundwater from flowing and percolating into critical locations, such as steep highway cut slopes or landslides. In addition, surface drains, collector pipes and culverts are used to carry discharge from subdrain systems.

A related design item is the discharge of collected surface water. Design would need to account for water quality requirements. Designs could include water detention areas and ditches (grassy swales), infiltration ponds, sumps and groundwater recharge systems. FLH standard practices should be considered in developing suitable discharge systems.

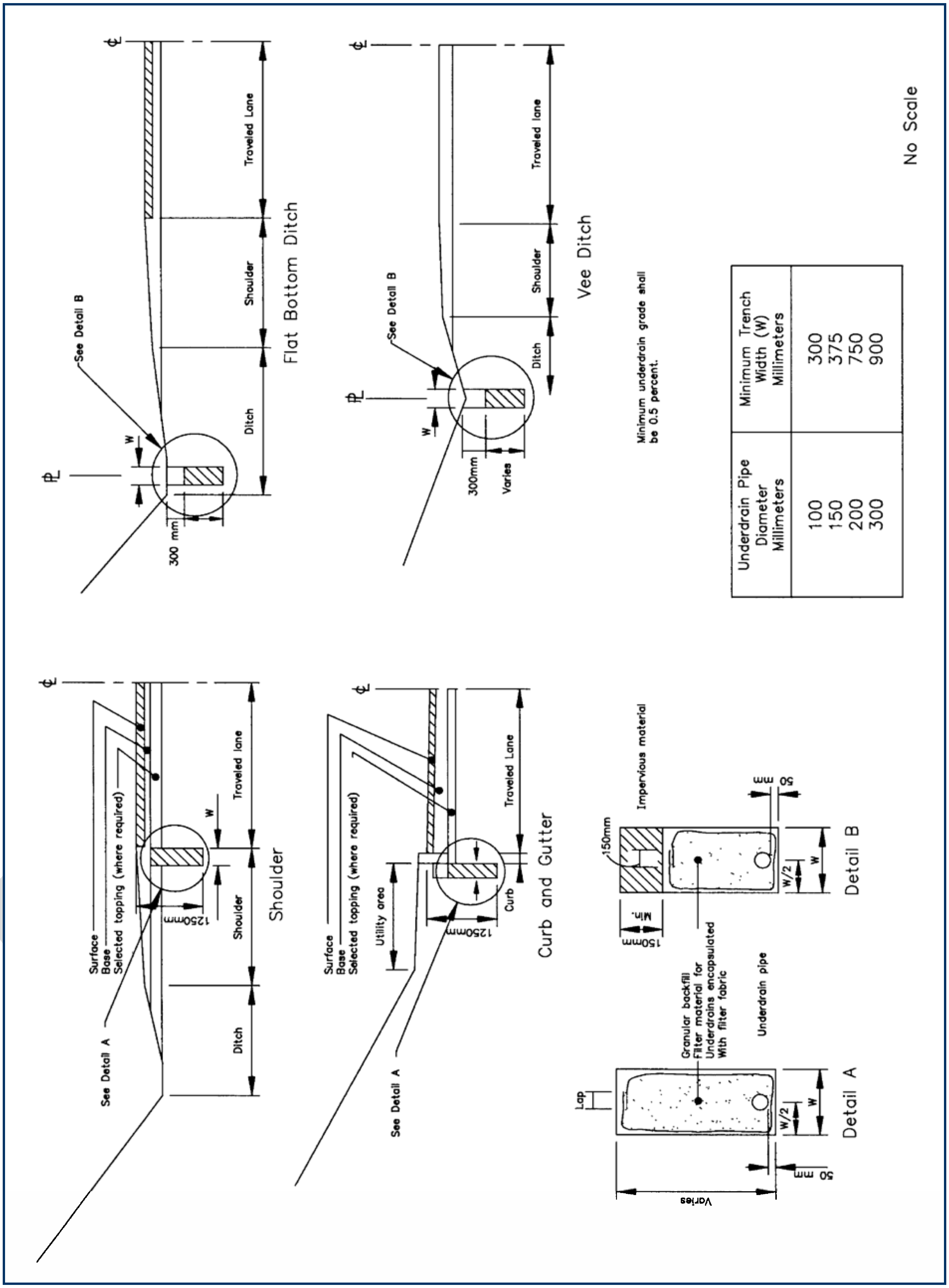
Guidance for surface drainage standards and guidelines can be found in FHWA manuals, such as FHWA-FLP-94-005, FHWA-RD-98-191, AASHTO Drainage, FHWA-RT-88-040, and FHWA-TS-80-218. A valuable secondary reference is Spangler & Handy 1982. A recent reference for infiltration pond design is WSDOT WA-RD-578.

Standards for surface drainage are in PDDM Section 6.4.9.1

The primary source supporting the standards and guidance is FHWA-FLP-94-005. Secondary sources are FHWA-TS-80-218 and FHWA-RT-88-040.

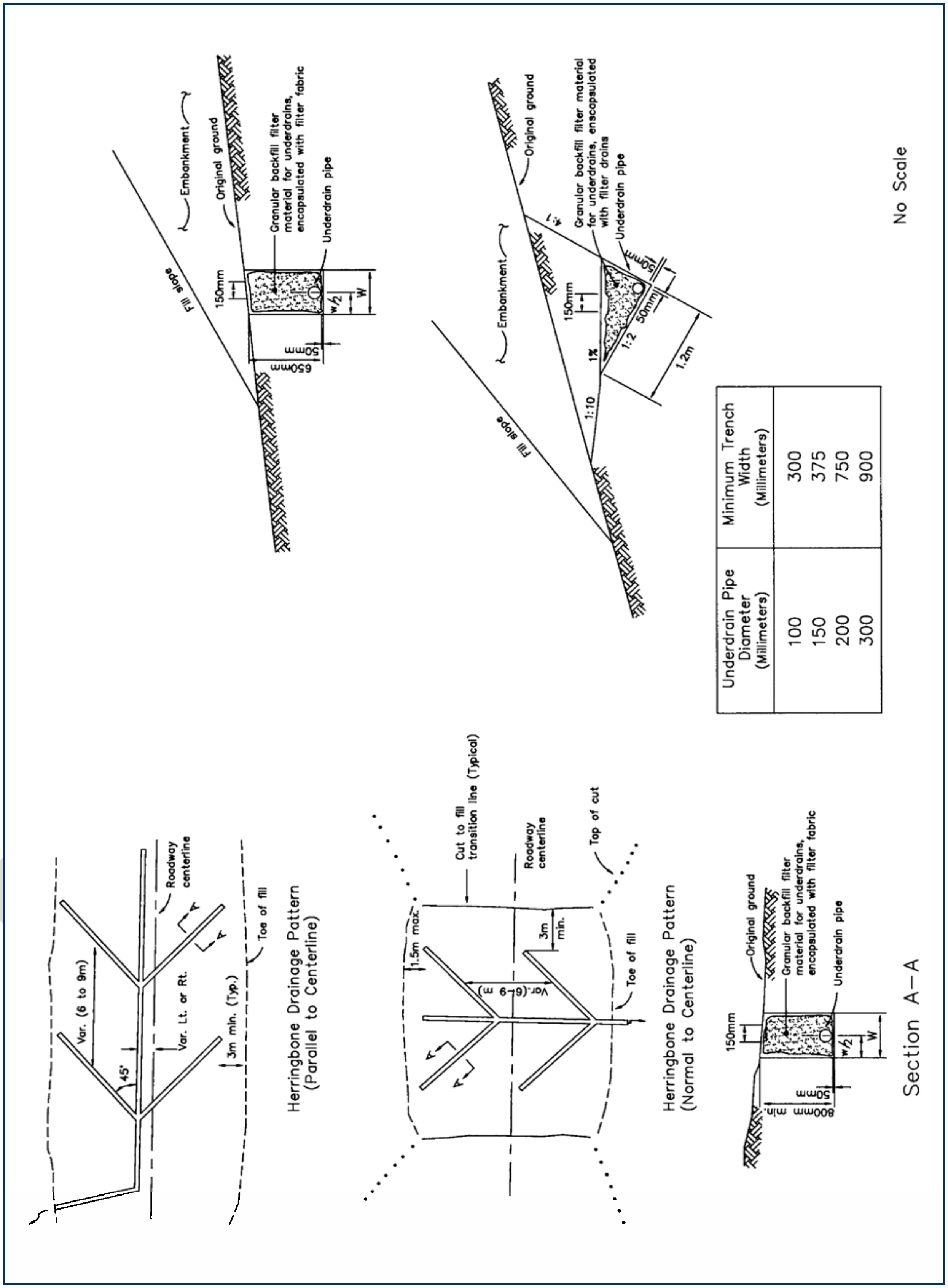
Refer to Appendix B.15, Drainage and Dewatering.

**Exhibit 4.9–A TYPICAL UNDERDRAIN INSTALLATION FOR ROADBEDS AND DITCHES (Metric)**




| Underdrain Pipe Diameter Millimeters | Minimum Trench Width (W) Millimeters |
|---|---|
| 100 | 300 |
| 150 | 375 |
| 200 | 750 |
| 300 | 900 |

**Exhibit 4.9–B TYPICAL UNDERDRAIN INSTALLATION IN EMBANKMENT AREAS (Metric)**




| Underdrain Pipe Diameter (Millimeters) | Minimum Trench Width (Millimeters) |
|---|---|
| 100 | 300 |
| 150 | 375 |
| 200 | 750 |
| 300 | 900 |

**Exhibit 4.9–C TYPICAL UNDERDRAIN INSTALLATION BENEATH THE ROADBED (Metric)**

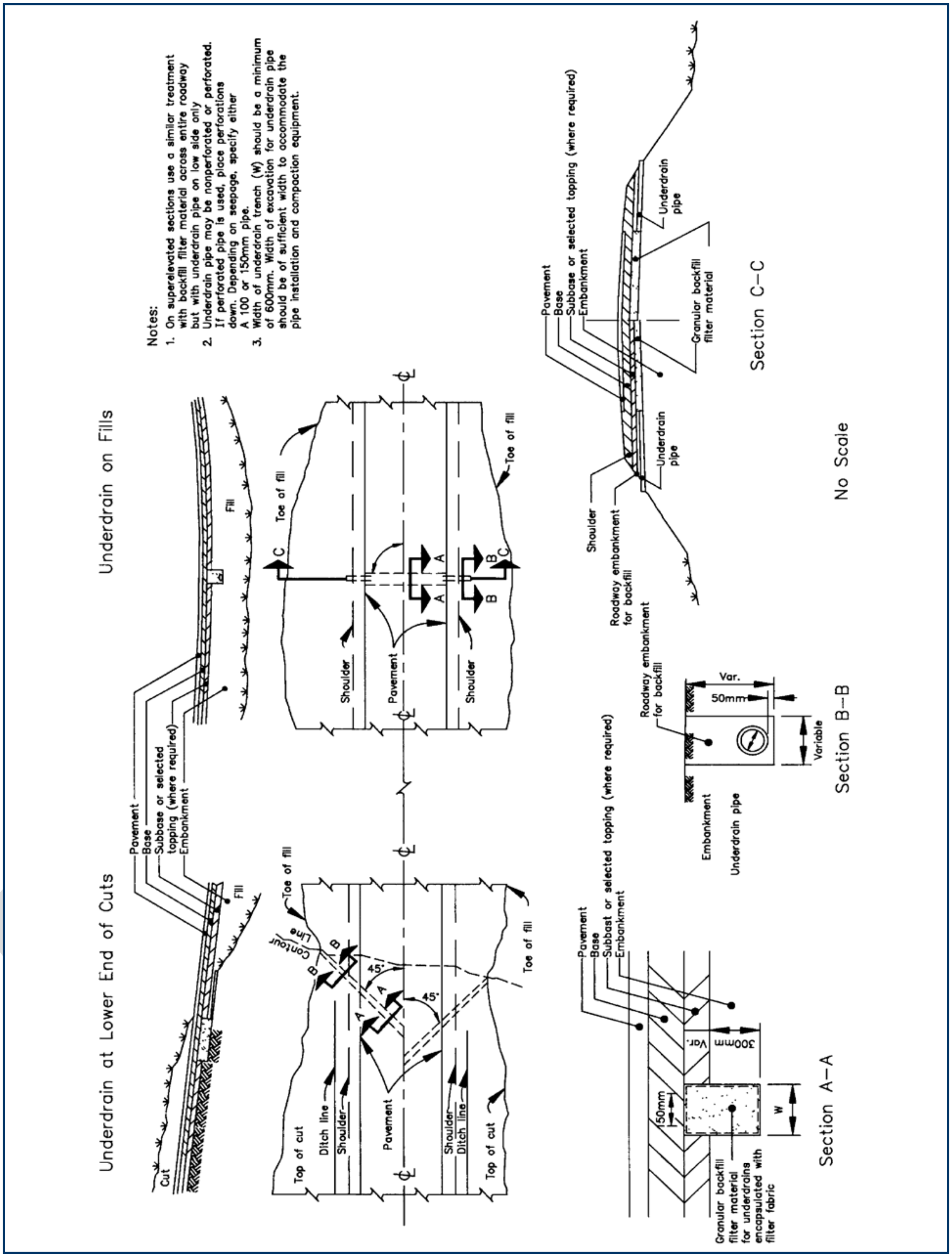


**Exhibit 4.9–D TYPICAL UNDERDRAIN INSTALLATION FOR SPRING AREAS (Metric)**

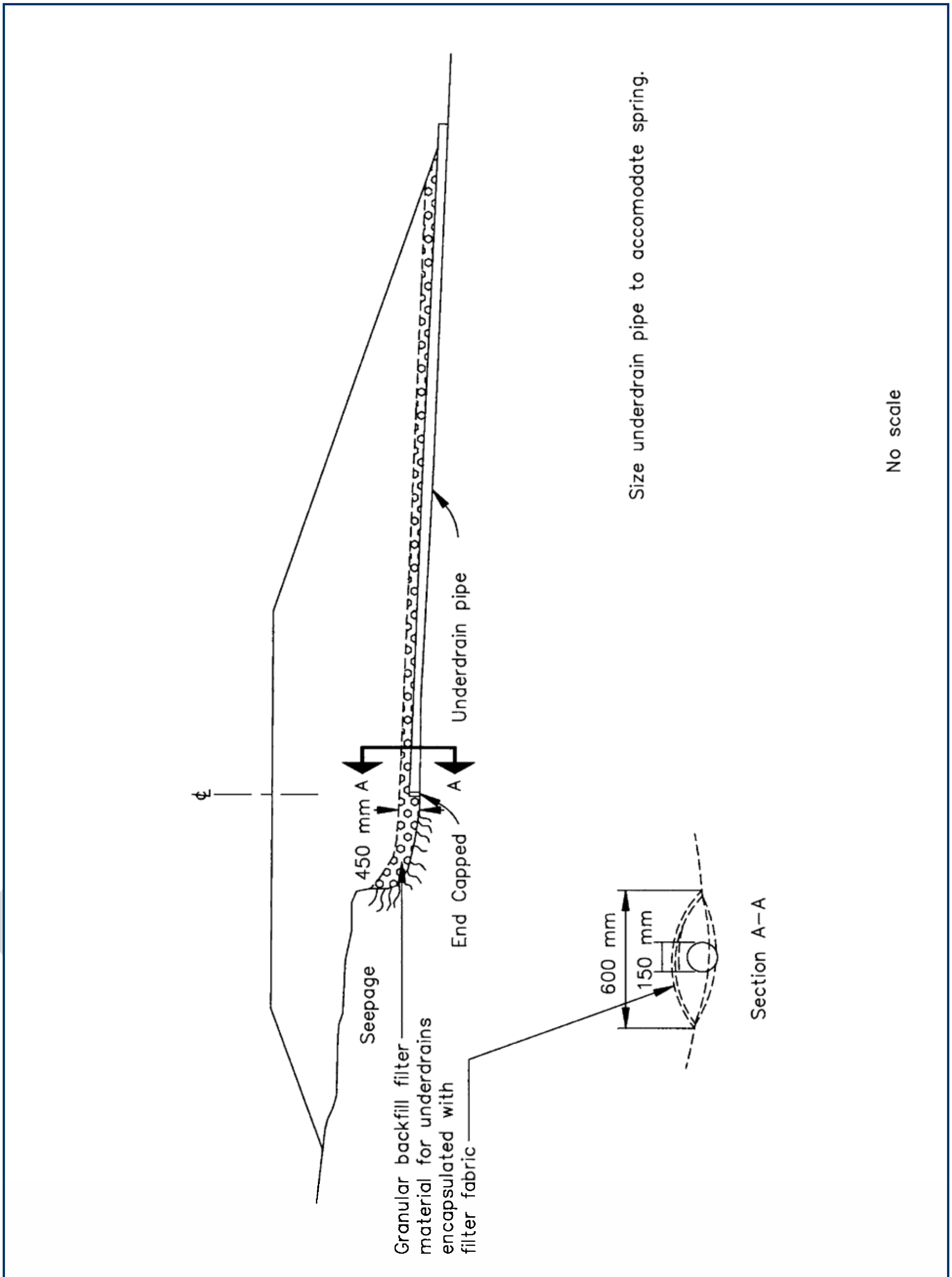
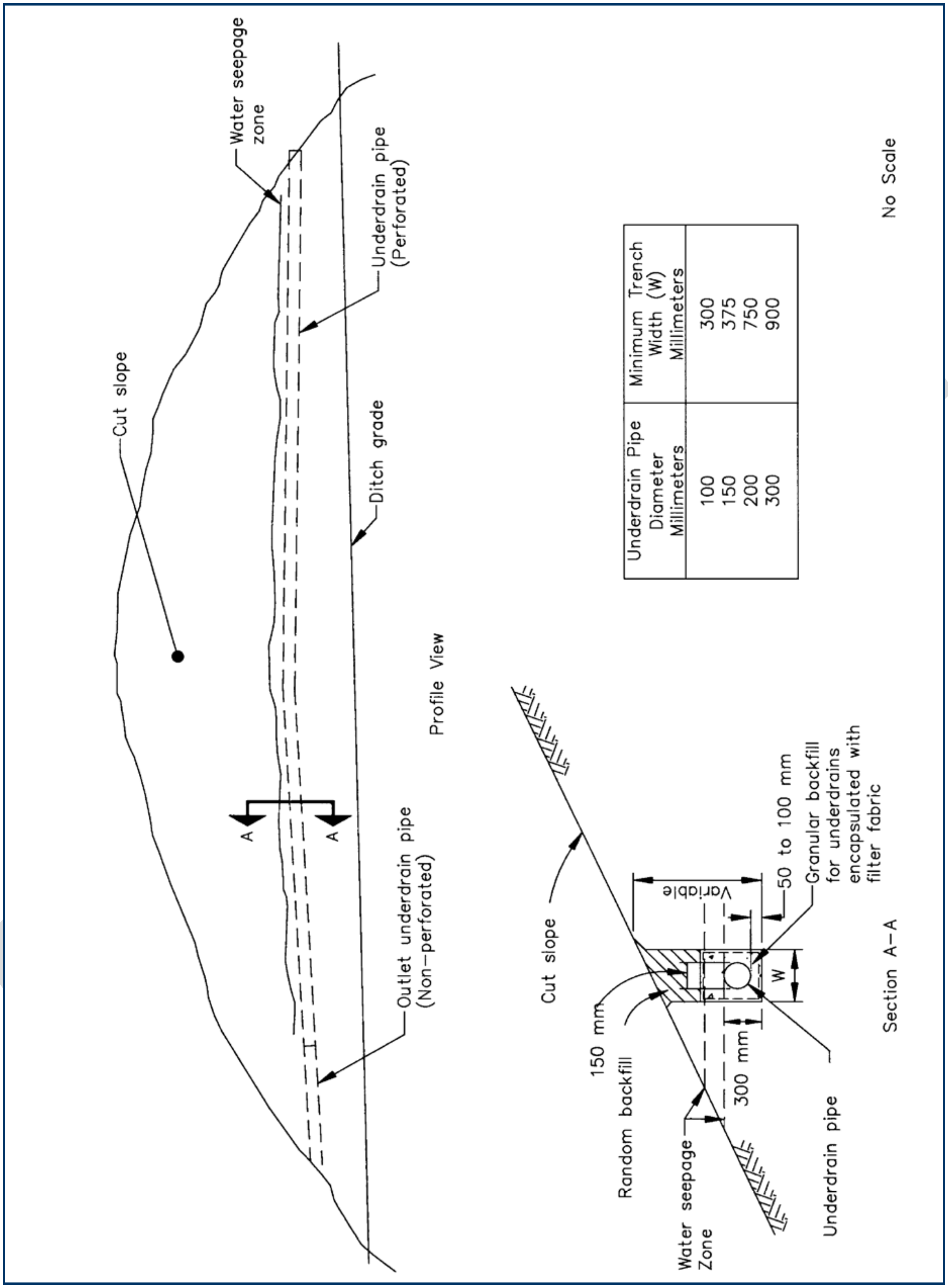


**Exhibit 4.9–E TYPICAL UNDERDRAIN INSTALLATION FOR BACKSLOPE (Metric)**




| Underdrain Pipe Diameter Millimeters | Minimum Trench Width (W) Millimeters |
|---|---|
| 100 | 300 |
| 150 | 375 |
| 200 | 750 |
| 300 | 900 |

### 4.9.2 SUBSURFACE DRAINAGE

Dewatering and subdrainage methods and systems are selected based on site conditions and desired objectives, which often limit the types that are applicable. Evaluate the options for feasibility, constructability, risk-management, ability to satisfy project constraints and environmental criteria, future consequences and preliminary estimates of cost.

Design criteria could include the slope stability Factor of Safety achieved by lowering groundwater levels, or a drawdown distance below anticipated excavation depths or pavement subgrade.

Typically, piezometers or observation wells are installed and periodic measurements made of groundwater levels. Shallow groundwater levels could indicate potential problems that should be mitigated before a failure occurs. Determine the groundwater levels that are critical to roadway performance and slope stability based on analyses. Monitoring can be used to verify the degree of groundwater drawdown achieved.

Subsurface drainage systems include underdrains, horizontal drains, drainage blankets, and relief wells, as described in the following paragraphs. Surface drains that could be incorporated with subdrainage systems are included. Primary reference is FHWA-TS-80-224. Additional references include FHWA-RD-86-171, FHWA-HI-95-038, FHWA-SA-93-004/5, and FHWA-CA-TL-80-16.

Geotextiles are often used in subdrain construction. For design, evaluate the permeability and gradation of the in situ soil. Determine geotextile strength requirements based on the drain rock size, height that rock could be dropped onto the geotextile, and other constructability issues. The permittivity and aperture size are important properties of the geotextile to select to be compatible with onsite soils and water flow expectations. Generic material property and construction method specifications are included in the FP-XX. Criteria for geosynthetics are application-dependent and are described in FHWA-HI-95-038. Additional geosynthetic design guidance is described in WSDOT WA-M-46-03 and Koerner 1994. Recent experiences are documented in ASCE GSP 103.

Standards for subsurface drainage are in PDDM Section 6.4.9.2

The primary source supporting the standards and guidance is FHWA-TS-80-224. Secondary sources are FHWA-RD-86-171, NHI 132013A, FHWA-SA-93-004/5 and FHWA-CA-TL-80-16.

Refer to Appendix B.15, Drainage and Dewatering.

#### 4.9.2.1 Underdrains

Underdrains (trench drains) are categorized as longitudinal drains if they are located parallel to the roadway centerline (both in the horizontal and vertical alignment) and as transverse drains if they run beneath the roadway either at right angles to the roadway centerline or skewed in the "herringbone" pattern. Refer to FHWA-TS-80-224. These drains are located not only at the edge of or under the pavement, but may also be constructed as aggregate trench drains in wet cut slopes. Typically, these drains involve a trench of substantial depth, a collector pipe, free draining aggregate, and a protective drainage/filter geotextile. In lieu of aggregate underdrains, prefabricated drainage systems (geocomposite drains) could be used. Refer to FHWA-RD-86-171, FHWA-SA-93-004/5, and FHWA-RD-72-30. Filter design is important to subsurface drain system designs. Guidance for geosynthetic filters and subdrains is provided in FHWA-HI-95-038. Depending on the source of subsurface water and the function of the drain, less sophisticated underdrains may be used. These may include French drains, consisting of a shallow trench filled with open graded aggregate, or a deep trench drain with filter fabric enveloping an open graded aggregate.

#### 4.9.2.2 Horizontal Drains

This drainage system consists of small-diameter pipes drilled horizontally into cut slopes or fill slopes to tap springs and lower groundwater levels. Construction considerations include the risk of damage to drain installations if the ground is moving. Determine the skew and inclination of horizontal pipes on a project-by-project basis and possibly adjust in the field as groundwater is encountered. In ordinary installation, the ends of the drain pipes are simply left projecting from the slope and the discharge is picked up in drainage ditches. Where greater protection is required (such as in freezing climates), a pipe collector system may be used to dispose of the water outside of the roadway limits or into a dry well. Design guidance is presented in FHWA-CA-TL-80-16. Horizontal drains require periodic maintenance as they tend to become clogged over time. Drain details should provide access for maintenance and cleaning, and where possible, monitoring. Horizontal drains often achieve a seasonal steady state discharge and if the monitored discharge drops from normal it may indicate drain problems before elevated water levels cause instability. If slope movement is ongoing or anticipated where horizontal drains are installed, the design should include provisions to accommodate movement, such as steeper drain installations and discharge ditches.

#### 4.9.2.3 Drainage Blankets

Drainage blankets are installed to provide a permeable layer, typically 0.3 to 1 m (1 to 3 ft) thick. Drainage blankets used in conjunction with a longitudinal underdrain can help to lower groundwater levels to improve the surface slope stability and reduce piping erosion of cut slopes. Horizontal drainage blankets can be used beneath or as an integral part of the embankment/pavement to remove ground water from both gravity and artesian sources. Refer to FHWA-SA-93-004/5 and FHWA-HI-95-038. Although relatively pervious granular materials are often used for base and subbase courses, these layers will not function as drainage blankets unless they are specifically designed and constructed for such purpose. This requires an adequate thickness of granular material with a very high coefficient of permeability, a positive outlet for the water, and an envelope of drainage geotextile. Drainage pipes are sometimes installed at the discharge ends of the blanket to ensure drainage in the event the aggregate were to become covered or blocked over time.

#### 4.9.2.4 Relief Wells

Vertical or inclined relief wells can be used to control the flow of ground water and relieve pore water pressures in potentially unstable highway slopes, including deep artesian water pressures. Wells are sometimes used in conjunction with other drainage systems to improve the ability to lower groundwater levels. Wells can be designed to be pumped and would require a collection and discharge system at the ground surface. Where permeable strata exist beneath the depth of the aquifer to be drained, the well would simply drain by gravity. Wells are generally not common in the construction of highway slopes. Refer to USACE EM 1110-2-1914.

### 4.9.3 DEWATERING

Dewatering methods include temporary ditches and trenches, open sumps, and pumped wells and wellpoints. Potential problems might occur with trenches and open sumps in liquefiable soils. Wells are often used when deep dewatering is necessary. Dewatering systems could be used alone to mitigate groundwater impacts when slope/trench stability is not an issue, or used in conjunction with shoring methods when both slope stability and drainage are important design considerations. Consider potential impacts to surrounding property and means to dispose of discharge water. Refer to Powers 1981, USACE EM 1110-2-1914, and ASCE 1985.

Standards for dewatering are in PDDM Section 6.4.9.3.

The primary source supporting the standards and guidance is Powers 1981. Secondary sources are USACE EM 1110-2-1914 and ASCE 1985.

Refer to Appendix B.15, Drainage and Dewatering..

### 4.9.4 EROSION CONTROL

Temporary erosion and sediment control measures are common on transportation construction projects. Evaluate erosion possibilities and identify best management practices to minimize erosion and to control sediments from affecting sensitive water bodies. Long-term mitigation measures may be necessary, such as riprap and rock blankets on erodible slopes. Primary reference is FHWA-FLP-94-005. Refer also to NHI 142054.

Geotextiles are used for some erosion and sediment control applications. Generic material property and construction method specifications are included in the FP-XX. Additional geosynthetic design guidance is described in Koerner 1994. For separation design applications, evaluate soil classifications and gradations. Determine if other geotextile functions should be included, such as drainage and filtration. Often, the construction process imparts significant stresses on the geotextile and should be accounted for in the selection of geotextile strength properties. Investigation for silt fences can generally be done by site inspection and “Best Management Practices” (BMP’s).

Standards for erosion control are in PDDM Section 6.4.9.4.

The primary source supporting the standards and guidance is FHWA-FLP-94-005 and the secondary source is NHI 142054.

Refer to Appendix B.16, Erosion and Sediment Control.

# 4.10 GROUND IMPROVEMENT

## 4.10.1 GENERAL

Ground improvement methods could be used when embankment foundations are weak and pose difficult construction issues or long-term performance and where potential liquefaction is a concern. Ground improvement can increase bearing capacity, shear strength, soil density, trench stability, and overall stability for structure and wall foundations and embankments. Types of ground improvement techniques include the following:

- Vibrocompaction techniques such as stone columns and vibroflotation.
- Deep dynamic compaction.
- Blast densification.
- Grout injection techniques and replacement of soil with grout such as compaction grouting, jet grouting, and deep soil mixing.
- Permeation grouting.
- Ground freezing (temporary application only).

Each of these methods has limitations regarding their applicability and the degree of improvement that is possible.

Evaluate the different conceptual ground improvement options for feasibility, constructability, risk-management, ability to satisfy project constraints and environmental criteria, future consequences and preliminary estimates of cost. Refer to NHI 132034, and NCHRP Synthesis 147. Refer to Exhibit 5.1–D which is based on the checklist in FHWA-ED-88-053.

Dynamic compaction and vibroflotation (including stone columns) increase the density of cohesionless soils. Silt and sand soils are best-suited for these types of mitigations techniques since they readily adjust to denser configurations resulting from dynamic motions.

Grout injection and mixing techniques (such as jet grouting) mix the grout with the soil in situ and can be successful for soils such as silt, sand and gravel. Clay soils may not break down sufficiently. The potential for obstructions such as boulders can limit the use of grouting methods. Permeation grouting is more limited in its application because the grout is introduced into the soil pore structure and voids, but not mixed. An environmental assessment of such techniques may also be needed, especially if there is potential to contaminate groundwater supplies. Concerns with grouting applications include potential heave of the ground surface, potential escape of grout, and blockage of natural groundwater seepage paths.

Ground freezing is a highly specialized technique that depends on soil characteristics and groundwater flow rates. Concerns include the time to freeze and thaw the ground, potential heave of the ground surface during freezing, settlement upon thawing, and temporary blockage of natural groundwater seepage paths.

Design criteria involve parameters that are required to achieve design objectives, which are determined by geotechnical analysis. The criteria include:

- Density gains to prevent liquefaction
- Shear strength gains to provide the required level of stability or bearing capacity

Design guidelines are provided in a variety of technical references, including: FHWA-SA-98-086R, NHI 132034, FHWA-SA-92-041, FHWA-RD-83-026, FHWA-RD-83-027, and GEC-1. Other references include WSDOT WA-RD-348, FHWA-AK-RD-01-6B, FHWA-RD-99-138, NAVFAC DM 7.3, and various ASCE publications that contain recent papers dealing with advances in ground improvement and grouting (see following list). Specialty contractors also have literature describing variations of the ground improvement methods. ASCE has many significant publications concerning ground improvement and grouting (refer to bibliography).

**Exhibit 4.10–A REFERENCES FOR GROUND IMPROVEMENT ANALYSIS AND DESIGN**

| | *Primary Source* | *Secondary Sources* |
|---|---|---|
| *General* | NHI 132034 | NCHRP Synthesis 147<br>FHWA-SA-98-086R<br>FHWA-SA-92-041<br>FHWA-ED-88-053 |

**Refer to Appendix B.17, Ground Improvement for tertiary sources.*

Good, general, readily available guidance is also available through USACE Publication Number ETL 110-1-185, Engineering and Design – Guidelines on Ground Improvement for Structures and Facilities, 1999.

## 4.10.2 GEOSYNTHETICS

Geotextiles are sometimes used in ground improvement methods. The type of geosynthetics to be used depends on the application and site conditions. Generic material property and construction method specifications are included in the FP-XX. Criteria for geosynthetics are application-dependent and are described in FHWA-HI-95-038. Reinforcement applications may require the analysis of a factor of safety for short-term and/or long-term conditions. Additional geosynthetic design guidance is described in WSDOT WA-M-46-03, FHWA-NHI-00-043 and Koerner 1994. Recent experiences are documented in *ASCE GSP 103*. Determine geotextile strength requirements based on the size and height that rock could be dropped onto the geotextile and other constructability issues. The permittivity and aperture size are important properties of the geotextile to select to be compatible with onsite soils and water flow expectations. For soil stabilization design utilizing geotextiles, evaluate subgrade shear strength data for reinforcement design in order to determine the geotextile strength properties. Refer to ASCE GPS 76.

Geosynthetics can be used in the following applications for highway earthwork designs:

- Underground drainage, including prefabricated drainage strips.

- Soil separation.
- Soil stabilization.
- Permanent erosion control (including riprap filters).
- Base reinforcement for embankments over soft ground.
- Reinforced soil slopes (RSS).
- Impermeable barriers.

The type of geosynthetics to be used depends on the application and site conditions. Generic material property and construction method specifications are included in the FP-XX. Criteria for geosynthetics are application-dependent and are described in FHWA-HI-95-038. Reinforcement applications may require the analysis of a factor of safety for short-term and/or long-term conditions. Additional geosynthetic design guidance is described in WSDOT WA-M-46-03 and Koerner 1994. Recent experiences are documented in ASCE GSP 103.

For underground drainage design, evaluate the permeability and gradation of the in situ soil. Determine geotextile strength requirements based on the drain rock size, height that rock could be dropped onto the geotextile, and other constructability issues. The permittivity and aperture size are important properties of the geotextile to select to be compatible with onsite soils and water flow expectations.

For soil stabilization design utilizing geotextiles, evaluate subgrade shear strength data for reinforcement design. Geotextile strength properties are the primary parameter to design. Refer to ASCE GPS 76.

Soil slopes reinforced with geotextiles can be designed by consulting the following references: FHWA-NHI-00-043 and GEC-2.

For separation design applications, evaluate soil classifications and gradations. Determine if other geotextile functions should be included, such as drainage and filtration. Often, the construction process imparts significant stresses on the geotextile and should be accounted for in the selection of geotextile strength properties.

For permanent erosion control design (such as filters for riprap), evaluate the gradation characteristics of the soil below the geotextile layer, as well as the granular fill or riprap gradation. Provide adequate geotextile drainage and filter properties. Usually, high strength requirements apply for the geotextile to survive the placement/drop of large rocks.

Investigation for silt fences can generally be done by site inspection and "Best Management Practices" (BMP's).

For geomembrane design, evaluate soil gradations and cover material specifications. Identify potential obstructions or protrusions that the geomembrane might need to accommodate.

**Exhibit 4.10–B REFERENCES FOR GEOSYNTHETICS**

| | *Primary Source* | *Secondary Sources* |
|---|---|---|
| *Geosynthetics* | NHI 132034 | Koerner 1994<br>WSDOT WA-M-46-03 |

**Refer to Appendix B.18, Geosythetics for tertiary sources.*

### 4.10.3 DEEP SOIL MIXING

FLH guidance to be drafted.

**Exhibit 4.10–C REFERENCES FOR DEEP SOIL MIXING**

| | *Primary Source* | *Secondary Sources* |
|---|---|---|
| *Deep Soil Mixing* | NHI 132034 | FHWA-RD-99-138 |

**Refer to Appendix B.17, Ground Improvement for tertiary sources.*

### 4.10.4 DYNAMIC COMPACTION

FLH guidance to be drafted.

**Exhibit 4.10–D REFERENCES FOR DYNAMIC COMPACTION**

| | *Primary Source* | *Secondary Sources* |
|---|---|---|
| *Dynamic Compaction* | NHI 132034 | GEC-1 |

**Refer to Appendix B.17, Ground Improvement for tertiary sources.*

### 4.10.5 BLAST DENSIFICATION

FLH guidance to be drafted.

**Exhibit 4.10-C REFERENCES FOR BLAST DENSIFICATION**

| | *Primary Source* | *Secondary Sources* |
|---|---|---|
| *Blast Densification* | NHI 132034 | WSDOT WA-M-46-03 |

**Refer to Appendix B.17, Ground Improvement for tertiary sources.*

### 4.10.6 SOIL STABILIZATION

FLH guidance to be drafted.

**Exhibit 4.10–E REFERENCES FOR SOIL STABILIZATION**

| | *Primary Source* | *Secondary Sources* |
|---|---|---|
| *Soil Stabilization* | NHI 132034 | FHWA-SA-93-004/5 |

**Refer to Appendix B.17, Ground Improvement for tertiary sources.*

### 4.10.7 STONE COLUMNS

FLH guidance to be drafted.

**Exhibit 4.10–F REFERENCES FOR STONE COLUMNS**

| | *Primary Source* | *Secondary Sources* |
|---|---|---|
| *Stone Columns* | NHI 132034 | FHWA-RD-83-026 |

**Refer to Appendix B.17, Ground Improvement for tertiary sources.*

## 4.11 GEOTECHNICAL EARTHQUAKE ENGINEERING

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 and seismic source 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-history 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 professionals 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.

FLH design procedures follow AASHTO guidelines for seismic design of transportation facilities. A primary reference is GEC-3. Article 3.21 of AASHTO HB-17 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 professional 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 professional is responsible for analyzing all three items. Additional guidance is described in WSDOT WA-M-46-03.

Providing geotechnical/seismic input parameters to the structural engineers for their use in structural design of the transportation infrastructure (e.g., bridges, retaining walls, ferry terminals, etc.). Specific elements to be addressed by the geotechnical designer include the design ground motion parameters, site response, and geologic/seismic hazards. Provide input for evaluation of soil-structure interaction (foundation response to seismic loading), earthquake induced earth pressures on retaining walls, and an assessment of the impacts of geologic hazards on the structures.

Geotechnical seismic design should be consistent with the philosophy for structure design that loss of life and serious injury due to structure collapse are minimized, to the extent possible and economically feasible. Bridges, regardless of their AASHTO classification, may suffer damage and may need to be replaced after a design seismic event, but they should be designed for noncollapse due to earthquake shaking and geologic hazards associated with a design seismic event. In keeping with the no collapse philosophy, bridge approach embankments and fills through which cut-and-cover tunnels are constructed should be designed to remain stable during the design seismic event because of the potential to damage or initiate collapse of the structure should they fail. The aerial extent of approach embankment seismic design and mitigation (if necessary) should be such that the structure is protected against instability or loading conditions that could result in collapse. The typical distance of evaluation and mitigation is within 100 feet of the abutment or tunnel wall. Instability or other seismic hazards such as liquefaction, lateral spread, downdrag, and settlement may require mitigation near the abutment or tunnel wall to ensure that the structure is not compromised during a design seismic event.

Evaluate the potential for differential settlement between mitigated and non-mitigated soils. Additional measures may be required to limit differential settlements to tolerable levels both for static and seismic conditions.

For the case where an existing bridge is to be widened and liquefiable soil is present, the foundations for the widened portion of the bridge and bridge approaches should be designed to remain stable during the design seismic event such that bridge collapse does not occur. In addition, if the existing bridge foundation is not stable, to the extent practical, measures should be taken to prevent collapse of the existing bridge during the design seismic event. Design the foundations for the widening in a way that the seismic response of the bridge widening can be made compatible with the seismic response of the existing bridge as stabilized in terms of foundation deformation and stiffness. If it is not feasible to stabilize the existing bridge such that it will not collapse during the design seismic event, consideration should be given to replacing the existing bridge rather than widening the existing bridge.

Evaluate all retaining walls and abutment walls for seismic stability internally and externally (i.e. sliding and overturning). Walls directly supporting the traveled way, or walls that are directly adjacent to the traveled way and are 3 m (10 ft) in height or more, should be designed to remain stable under seismic loading conditions and anticipated displacements associated with liquefaction. Mitigation to achieve overall stability may be required.

Walls where the face is more than 3 m (10 ft) from the traveled roadway, and walls that are less than 3 m (10 ft) in height, are not required to meet overall stability under seismic loading and/or liquefaction effects. These walls are considered to have relatively low risk to the traveling public. These walls may deform, translate, or rotate during a seismic event and overall stability may be compromised. Considering the excessive cost required to stabilize these walls for liquefaction effects, it is generally considered uneconomical to stabilize these lower risk walls.

Note that stabilizing retaining walls for overall stability due to design seismic events may not be practical for walls placed on or near large marginally stable landslide areas. In general, if the placement of a wall within a marginally stable landslide area (i.e., marginally stable for static conditions) has only a minor effect on the stability of the landslide, Federal Lands Highways will not design the wall to prevent global instability of the landslide during the design seismic event.

Standards for seismic design are in PDDM Section 6.4.11.

The primary source supporting the standards and guidance is GEC-3. Secondary sources are AASHTO HB-17, NHI 132039A, WSDOT WA-M-46-03 and Kramer 1996.

Refer to Appendix B.19, Seismic.

## 4.11.1 SEISMIC DESIGN

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 HB-17. 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.

For very large or critical structures, perform a site-specific seismic hazard evaluation. 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.

For the design of bridges according to Section 3.5.1 of Division 1A of AASHTO HB-17, the seismic response of the soils is expressed by the Site Coefficient (S), which is in turn determined by the Soil Profile Type. The Soil Profile Type is based on geotechnical subsurface explorations and classifications of the subsurface materials.

### 4.11.1.1 Slope Stability Issues

Earthquake shaking can result in failures of natural slopes and man-made embankments. The standard procedure for evaluating the stability of a non-liquefiable 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 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 1.0 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 1.0, but the actual deformation of the slope or embankment would be minimal and overall performance acceptable.

One method to estimate seismic deformations of non-liquefiable 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 GEC-3 and Kramer 1996 for more details on the deformation analyses.

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.

#### 4.11.1.2 Foundation and Retaining Structure Issues

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 HB-17 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 HB-17 states an alternate method that can be developed by a qualified professional. Develop the site-response spectrum and refer to GEC-3, which provides a summary of seismic design procedures, including selection of representative earthquake time histories.

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 AASHTO HB-17, Articles c6.4.3, c7.4.3, and c7.4.5 for the seismic requirements for abutments. Other references include: GEC-3, and Kramer 1996.

### 4.11.2 LIQUEFACTION

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. Measures to mitigate liquefaction are described in the following section. The seismic evaluation and design of soil-pile interaction is an area of active research and updates on design practices should be consulted GEC-3; NHI 132039A and WSDOT WA-M-46-03 provide detailed seismic design procedures.

Generally, if liquefiable soils are present, some means of mitigation is required to protect structure foundations. In keeping with the no collapse philosophy, bridge approach embankments should be designed to remain stable during the design seismic event because of the potential to damage or initiate collapse of the structure should they fail. The aerial extent of approach embankment seismic design and mitigation (if necessary) should be such that the structure is protected against instability or loading conditions that could result in collapse. The typical distance of evaluation and mitigation is within 100 feet of the abutment. Instability or other seismic hazards such as liquefaction, lateral spread, downdrag, and settlement may require mitigation near the abutment to ensure that the structure is not compromised during a design seismic event. The geotechnical professional 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. GEC-3; AASHTO HB-17; and WSDOT WA-M-46-03 detail liquefaction analysis procedures.

The selection of ground motion parameters for the lateral force design procedures discussed in Division 1A of the AASHTO HB-17 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 density will result in a liquefiable condition. Liquefaction has caused a number of bridge failures during past earthquakes. The recommended procedure to evaluate the liquefaction potential is based on Standard Penetration Test results. The liquefaction evaluation procedure is described in standard references on geotechnical earthquake engineering, including Chapter 8 of GEC-3. Additional guidance is described in NHI 132039A, WSDOT WA-M-46-03, and Kramer 1996.

Standards for liquefaction are in PDDM Section 6.4.11.

The primary source supporting the standards and guidance is GEC-3. Secondary sources are AASHTO HB-17 and WSDOT WA-M-46-03.

Refer to Appendix B.19, Seismic.

# SECTION 5 DOCUMENTATION AND SUPPORT

The purpose of geotechnical reports and memoranda is to present the design data in a clear systematic and concise manner, to draw conclusions from the data, and to make recommendations for the geotechnical aspects of the project. Pertinent information should consist of site specific physical, environmental, geological and subsurface exploration data; station by station field notes; geophysical field data; material properties and laboratory test results; discussion of analyses used; listing of all major assumptions and/or data used for analyses; and design and construction recommendations. Ensure that factual data is presented separately from interpretation and opinion, and that all interpretations are clearly identified as such. The results of the investigation phases are commonly documented with interim memoranda and subsequently culminate in a Final Geotechnical Report. In some cases, such as for Design-Build projects, geotechnical baseline reports could be required.

Written correspondence falls into the categories of formal and informal. Letters, memorandums, some emails, and reports are formal correspondence. Transmittal forms, some facsimiles and most emails are informal correspondence. Note that even informal means of correspondence are “public information” and readily shared with others. It is important to be careful and tactful in all forms of correspondence. Digital forms of correspondence must be placed in a computer file, backed up on an intranet server, and placed as a hardcopy in the Project file.

As a general rule, written correspondence should be as brief as possible, but also needs to deal comprehensively with the subject matter. This means beginning the writing by explaining the purpose of the correspondence. Not all correspondence requires a specific recommendation. Contributors should be given the opportunity to review reports citing their work. Finally, all correspondence should be processed in a timely manner.

Reports and memoranda are primarily intended for highway designers, but are also made available to project construction personnel and prospective bidders.

Standards for applying reporting and cooorespondence are in PDDM Section 6.5.

The primary source supporting the standards and guidance is NHI 132031. Secondary sources are FHWA-ED-88-053 for geotechnical reports and ASCE GBR for baseline reports.

Refer to Appendix B.20, Reports.

## 5.1 GEOTECHNICAL REPORTS AND DOCUMENTS

### 5.1.1 GENERAL

There are several different types of documentation used to report geotechnical data and findings, according to the work phase and the information to be communicated. All documentation utilize the principles described in the foregoing paragraphs. The following describes common types of documentation. When several of the following occur at the same time, it may be possible to combine them into a single document. It is advisable to use memorandum formats when the geotechnical work is preliminary, in draft form, and/or likely to be supplemented (rather than using “report” formats that can imply final documentation).

*Preliminary Study and Reconnaissance Memorandum*

During the early phase of a project, initial geotechnical evaluations are based on research of prior reports and geologic publications, review of project scope and criteria, and a preliminary reconnaissance of the site. The memorandum should provide preliminary and conceptual geotechnical guidance to assist the project team in advancing the scope of the project. Potential geotechnical problems and hazards should be identified, along with recommendations for avoidance or mitigation.

*Preliminary Geotechnical Investigation Memoranda*

Preliminary investigations and draft recommendations should be summarized in memoranda to communicate general direction to the design team. Where design recommendations are clearly identified as being preliminary or conceptual, all parties receiving those recommendations should understand that the recommendations are subject to change and should only be used for preliminary alternative and scope development purposes.

*Geotechnical Data Baseline Report*

A geotechnical baseline report may be required, particularly in conjunction with solicitations for Design-Build projects. The geotechnical baseline report outline is similar to the outline for a final geotechnical report, except that it is strictly a factual report, and no analyses or design recommendations are presented.

*Final Geotechnical and Foundation Report*

The final geotechnical report contains factual data, interpretations, engineering studies and analyses, and recommendations for design and construction. The format and contents of the geotechnical report are somewhat dependent on the type of project. The general outline for a geotechnical report should be consistent with the general guidelines presented in Section 6. A suggested outline is as follows:

- Title Page
- Table of Contents
- Executive Summary (optional)
- Introduction (scope, and other reports and investigations, project description)
- Procedures and Results
- Field Investigations
- Laboratory Analyses
- Summary of Engineering Analyses and Calculations
- Discussion
- Geologic Conditions and Seismicity (Local geology, faulting and seismicity)
- Anticipated Subsurface Conditions (soil, rock, groundwater)
- Geologic Hazards
- Specific Site Evaluations

- Design Recommendations
- Site Grading and Earthwork (use of materials, embankments, cut slopes, drainage)
- Rock Slopes (slope angles, stabilization, rockfall mitigation)
- Foundations (spread footings, deep foundations)
- Retaining Walls
- Construction Specifications
- Recommended Construction Observations, Testing and Instrumentation
- Maintenance Issues and Recommendations
- Figures
- Appendices

The geotechnical report should divulge all subsurface information used for design. The report writers and reviewers should be aware that the information contained in the report is typically used by contractors to prepare bids. The Geotechnical Report typically is also used by FLH as the basis for resolving contractor claims of changed conditions. Refer to FHWA-GT-15. Since some words and phrases can have double meanings, it is important to avoid the use of incomplete, ambiguous, and subjective statements. Independent reviews can help to eliminate ambiguity. Questionable words and phrases should be replaced with clearer terms. Geotechnical interpretations are needed to describe and justify the assumptions made in areas where conditions are unknown. Unnecessary interpretations and statements or overly optimistic statements should be avoided. Guidance for checking the completeness of Geotechnical Reports is provided for specific project features in Exhibit 5.1–B through Exhibit 5.1–J (based on FHWA-ED-88-053).

To maximize the benefits of the geotechnical investigation and report, the geotechnical professional should interact with the project design and construction engineers throughout the duration of the project. When the project approaches the final design stage, the geotechnical professional should determine if the geotechnical report should be revised to reflect modified assumptions and recommendations incorporated in the final design plans. It is preferable that only one Final Geotechnical Report be produced to avoid misapplication of previous versions and preliminary reports. A suggestion is to title all prior documentation as “Preliminary Memoranda.”

The geotechnical professional should provide recommendations for all earthwork, rock slopes, retaining walls, foundations and geotechnical problems. The excavated materials should be described in terms of their behavior and its suitability for use as Borrow material. Address how the materials satisfy FLH standards for Borrow materials. Unsuitable materials should be described and their locations identified. If groundwater or seepage could impact the project, describe any recommended drainage systems and their locations. Estimate earthwork shrink/swell factors to allow for computation of earthwork quantities.

Provide recommendations for embankment construction, including special methods to ensure slope stability and manage settlement. Estimate the magnitude and rate of settlement. Evaluate possible alternatives if magnitude or time required for settlement is unacceptable, and recommend treatment based on economic analysis, time and environmental constraints. When addressing stability, describe the factor of safety criteria and the level achieved with the recommended approach. Evaluate possible treatment alternatives if the factor of safety is too low. Landslide mitigation measures require detailed design recommendations. Provide recommendations for any ground improvement. Reinforced slopes, if to be used, should be detailed for design.

Provide rock slope recommendations including the design of slopes (appropriate cut slope angles and fallout area dimensions). The potential for rockfall should be described and any recommended mitigations should be detailed.

Provide foundation recommendations for all structures including bridges, soundwalls, earth retaining walls, channels, box culverts and poles. Address the use of both shallow and deep foundations and describe advantages and disadvantages for each. Provide detailed recommendations for preferred foundation types. For shallow foundations, provide the recommended elevations of bottom of footings and the allowable soil pressures based on settlements and bearing capacities. Describe suitable pile types and reasons for design selections and exclusions. Provide plots of soil resistance for selected pile size alternates. Depth of scour should be accounted for on each plot. Describe recommendations for piles that include: lateral capacity, vertical capacity, seismic criteria and design parameters, minimum pile length or tip elevation, minimum pile spacing, estimated pile settlement or pile group settlement, and maximum driving resistance to be encountered in reaching the estimated bearing elevation. In addition, recommend locations of test piles and pile installation criteria for dynamic monitoring and selection of load test types, locations and depths, where applicable.

Recommend the retaining wall types that are appropriate for the project. Provide detailed recommendations for design of the preferred wall type(s). Address global stability of walls. Include any requirements for tiebacks, geotextiles, reinforcing materials, etc. Include MSE reinforcement lengths and locations if lengths vary.

Describe any possible impacts of roadway construction (vibratory rollers, utility excavations, settlements, etc.) on surrounding structures. Provide recommendations to mitigate impacts.

The design recommendations should utilize FLH Standard Specifications wherever possible for simplicity and contractor familiarity. There is no need to repeat the Standard Specifications in the geotechnical report. Provide specifications and details where the Standard Specifications do not apply or do not address the planned construction operation for the project.

Provide recommendations for construction phase geotechnical testing, observations, and/or instrumentation, depending on the needs of the project and the relative complexity or criticality of the work to be performed. Describe the benefit of performing the testing and instrumentation, and the possible consequences if they are not performed or if the instruments are accidentally damaged. List the tests and instruments to be used and their planned locations.

Comments on construction issues are helpful to both the FLH and contractors. Provide information about known water, soil, and rock conditions that might affect construction operations, sequences, and methods. These conditions might include soft foundation soils, quick soils, extremely weathered or fractured rock, massive rock, high moisture contents, presence of subsurface boulders, buried drainage systems, springs that could interfere with construction. Identify design features that were specifically included to address geotechnical problems during construction. Discuss the design features and possible consequences if recommendations are not implemented. Identify restrictions, such as not being allowed to place fill or temporary stockpiles in sensitive or unstable areas, and provide information on temporary cut slopes.

Some construction measures and geotechnical mitigations require maintenance to extend their life and effectiveness. Lack of maintenance or identification of system problems could result in harm to roadway and structures. Maintenance practices should avoid methods that exacerbate unstable conditions, such as material removal at the toe of a marginally-stable slope or placing fill or waste materials onto outside road shoulders where slopes below could become oversteepened and overloaded. Rockfall mitigation measures such as catch fences, draped mesh and barriers can be susceptible to damage over their service life. The effectiveness of rockfall control could be reduced if no repair is done. In addition, rockfall is likely to accumulate in ditches and fallout areas, reducing catchment effectiveness and increasing the potential for rockfall to reach the roadway. The geotechnical professional should evaluate potential geotechnical problem conditions and provide guidance for inspection, maintenance and repair.

The geotechnical report might contain an appendix summarizing the analyses and calculations. An alternative if calculations are to be submitted is to package the calculations in a separate document. Refer to specific FLH Division requirements and guidelines.

*ADDENDUM REPORTS*

If the project design is altered as project development advances, the geotechnical recommendations may have to be modified from those presented in the original geotechnical memorandum or report. When the project approaches the final design stage, the geotechnical professional should determine if the geotechnical report should be revised to reflect modified assumptions and recommendations incorporated in the final design plans.

*Materials Source Report*

The geotechnical professional prepares a Materials Source Report that provides documentation for the investigation and subsequent development of a materials site. The report should review and discuss site geology, field data and testing information, slope stability, groundwater information, and provides the mining plan for development of the site. The following are considered in developing the report:

- <u>Introduction</u>**.** The legal description of the property location should be described (e.g., Township, Range, Section, ¼ sections). The description also includes the size of the material source in acres. Ownership is identified and any pertinent lease information. Also, any zoning restriction or other restrictions or constraints are identified. The general geomorphology and topography of the area are described, including drainage features. Vegetation and climate should also be discussed.

- <u>Procedures and Results</u>. The number and location of exploratory holes should be shown on a site map. Color photos of rock core samples are generally required. Representative samples are tested for quality and to verify field visual identification. The test results are used to interpret the distribution of the good and poor quality materials at the site. Existing stockpiles and waste piles are identified on the site plan map, along with estimated volumes.

- Analysis. A stratigraphy for the material source is developed from the site geology, borings and test pits. Geologic cross-sections are developed to demonstrate the distribution and quality of material available at the site. Overburden and unsuitable materials encountered in the borings, quality test results, and groundwater should be identified on the geologic cross-sections. Included in the discussion of the stratigraphy should be a description of rock quality as identified on each cross-section. Slope stability analyses may be necessary to evaluate the stability of the slopes during mining development, and for reclamation. The estimated quantities of useable materials present at the site is based on factual data from the explorations and geologic interpretations of the formations. The estimated quantities of materials should include a safety factor.

- Discussion. Summarize opinions made from the analyses and previous uses of materials from the same site. Compare alternatives. Discuss any special problems associated with materials present at the site, such as a description of large rock encountered or excessive overburden.

- Recommendations**.** Describe the recommendations for developing the materials source and limitations to avoid potential problem conditions. Identify the exploration locations, limits of proven reserves, stockpile and waste areas for overburden and reject material on the site map. The location of haul roads, gates, fences, and the elevation of the mining floor should also be included in the site map. Designate recommended slope angles, based on slope stability analyses, for interim and final reclamation. Reclamation plans are typically a cooperative effort with the designer, environmental specialist, and the client agencies.

### 5.1.2 REPORTING ORGANIZATION AND CONTENT

Organize geotechnical memoranda and reports to be consistent and to follow the same general format to allow for familiarity by even the occasional reader. Ensure that factual data is presented separately from interpretation and opinion, and that all interpretations are clearly identified as such. Describe potential problems disclosed by analyses and identify potential feasible solutions. Provide an assessment of cost, risk and uncertainty associated with each of the possible options. Include recommendations for both design and construction.

Standard report and memoranda organization is as follows. Each of these sections could be a sentence, a paragraph or a chapter depending on the scope of work and the purpose of the correspondence, and entire sections are omitted where they are not relevant.

STANDARD REPORTING ORGANIZATION AND CONTENT

EXECUTIVE SUMMARY: This may be included in larger reports with complicated scopes. Prepare executive summaries no more than a few pages long and direct the reader to the full report text because not all observations and recommendations can be included in this abbreviated summary.

A. INTRODUCTION: Only the report is introduced in this section. The introduction describes why this report was prepared (purpose/objectives), what’s included in it, how it relates to other reports prepared for this project, and how it is organized. The project is identified here, but that is typically all - the project introduction comes in the following section. List previous reports, authors and dates.

B. PROJECT DESCRIPTION: This is the section where the project is introduced and described. Include understanding of the project setting, project history, preliminary design, and design criteria. Include reference to a project location map and site map, and project features that could generally be identified prior to this investigation such as steep slopes or historic landslides. Include identification of previous projects on the route and any particular ties between this project and previous projects. Identify the extent of development of preliminary design including reference to the dated design drawings and stationing used for reference in the report. Identify special design criteria, such as preference for or against walls, specified culvert types, features that can't be impacted, non-standard factors of safety, etc.

C. GEOLOGY: Start the discussion of geology on a regional basis and end up with site-specific observations and hazards, including seismicity. This section may be very short or quite long if there are significant geologic hazards and or a site-specific seismic hazard assessment required.

D. SITE CONDITIONS: The conditions described in this section are based mostly on above ground observations including topography, drainage, vegetation, utilities, road, slope, and structure conditions, and mapping (including rock slopes, outcrops, etc.). These are observations made as part of the work being reported, enhancing the general descriptions presented in the Project Description section. Describe observations in a specific and quantitative way (e.g. 50-ft high, extremely weathered, 4V:1H rock slopes). This section may be short if there was no significant surface mapping or long, with many figures and tables if there was extensive geologic hazard or rock slope mapping. The data and observations presented in this section are factual – interpretations and recommendations come later.

E. SUBSURFACE CONDITIONS: The subsurface investigation procedures and results are presented in this section. Once again, the data and observations presented here are factual – interpretations and recommendations come later. This section includes procedures and results from geophysics, drilling, test pitting, laboratory testing and other subsurface investigations. Before presenting the results, answer the questions "who did what, when, and how". Tables and figures are often needed in this section to present the results in such a way that they can be readily understood and used. Appendices are typically used to present compiled data, such as boring logs and photographs. Study laboratory reports to see if the entire report should be included, or if a summary table would convey the relevant information. This section concludes the data presentation part of the report; the project geologists and engineers base the subsequent report sections on the interpretations of these data and from other studies and reports cited herein. Thus, the following sections present the results of professional judgments, which in many cases are made with the assistance of formal analyses.

F. ANALYSIS: Interpretation of data is the first phase of analysis and interpretations worth reporting should be reported here (e.g. connecting strata between borings to develop a subsurface profile or grouping soil samples by gradation or plasticity). Sometimes a good interpretation is all that is needed to develop the recommendations and identify the construction considerations presented later. Often, however, formal analyses are needed to confirm and quantify judgments. Such analyses should be presented in this section. In general, enough information should be given to allow others to duplicate the analysis as a form of checking the work, or possibly to start a parametric analysis to evaluate the significance of certain assumptions that may be called into question during further study or during construction. This typically requires identifying the analysis methods, assumptions, and input parameters used, and summarizing results. Some or all of this information can be included in the report appendices, which are to be cited from this section. The calculations themselves, whether done by hand or computer, are seldom included in the report text or appendices. Include calculations only where they cannot practically be separated from the assumptions, input or output.

Design alternatives often need to be evaluated from a geotechnical perspective. The evaluation of pros and cons, risks and costs, etc. is a form of analysis that also belongs in this section. Identify the preferred alternative from a geotechnical perspective first and then, if appropriate, a perspective that includes understanding of environmental, design, and aesthetic considerations, etc.

G. DESIGN RECOMMENDATIONS: The interpretation of analysis results is the first step in developing design recommendations. It is important that design recommendations for all project features and loading conditions be presented (or summarized if they were first presented in Section F) in this section. Base recommendations on the analysis results and other experience of the authors; the basis of all recommendations should be evident. For example, if a recommendation is not consistent with the results of analysis an explanation is required (i.e., disturbed sample, different boundary conditions, etc.). Provide concise recommendations directed toward the preferred alternative, and present on a station-by-station and/or feature-by-feature basis. Present recommendations in such a way to be readily applied to the design and construction specifications; for example, AASHTO, the FP-XX, and Standard SCRs. If, in addition to recommendations, actual designs for geotechnical features (such as soil nail walls) are to be presented, they are included in this section. It may be necessary to include sketches or drawings to convey recommendations or designs as Figures or, if many are necessary, in a designated Appendix.

If, through the process of investigation, analysis, and design, the need for further work is identified, it should be presented as a recommendation here. Explain the importance of the supplemental work.

H. CONSTRUCTION CONSIDERATIONS: Recommended construction specifications are identified here, and are based on the FP-XX and Standard and/or “Special” SCRs to the extent possible. This section also includes discussion of observations as they relate to construction of the recommended designs. Often this could include groundwater conditions, the presence of boulders, unstable slopes, and geotechnically based restrictions on the contractor’s activity, such as only opening up a small part of an excavation at a time. If a Geotechnical Advisory is recommended for inclusion in the construction contract it should be recommended here, and suggested text should be provided.

I. REFERENCES: List cited references only: including previous reports, communications and papers. Do not introduce new material or list reports not cited in previous sections – cite them earlier in the report if they are needed.

Reports and memoranda are prepared at all stages of projects and they are to be clearly identified as “preliminary”, “interim”, or “final” to refer to the stage of the project, not the correspondence. When correspondence at any stage is going through development or review it is identified as "draft". The content of reports should be in accordance with FHWA-ED-88-053. This document should also be used for the review of reports prepared by others.

Prepare geotechnical reports and memoranda using a formal technical report writing style. The reports serve as the permanent record of all geotechnical data known to be pertinent to the project and is referred to throughout the design, construction, and service life of the project. The reports and memoranda are read by various parties within FLH, by partner agencies, consultants, contractors, attorneys, etc. In litigation matters, the geotechnical reports could be read by opposing legal counsel looking for weaknesses, misstatements, errors, omissions, and evidence of substandard work or implied conditions (refer to FHWA-GT-15). Internal report reviews are critical to verify that reports meet FLH standards before the reports are distributed. Reports and report drafts are discoverable in legal proceedings, and can be used by opposing counsel in an attempt to cast doubt on the competency of the geotechnical professional. Guidance for geotechnical documentation and reports is described in FHWA-ED-88-053. Also, refer to NHI 132031, EFLH GeoData 1998, ASCE MREP 56, ASCE GBR, and ASFE 1978.

All geotechnical reports and memoranda should be consistent and organized to follow the same general structure to allow for familiarity by even the occasional reader.

The report should be formatted to present information using a standardized approach, so that users are able to locate information readily and consistently. The following is a typical expanded outline for geotechnical reports:

- Title Page
- Table of Contents (if report is large)
- Introduction
- Procedures and Results
- Analysis
- Discussion
- Recommendations
- Attachment Figures - Location Map, Drawings, etc
- Appendix A - Field Boring/Core Logs, Test Pit Logs, etc
- Appendix B - Laboratory Test Results
- Other appendices (as necessary) - Geophysical Test Results, Photographs, Specialized In situ Test Results, Instrumentation Data Results, etc.

The introduction section of the geotechnical report should contain information as to the specific location of the project site, the purpose of the report, authorization for the work and any limitations and restrictions that may apply. Include a review of the project and history of the site as background information when it is relevant to the investigation and/or proposed project.

The procedures and results section should contain factual information including field reconnaissance and exploration procedures and engineering properties determined from the tests. Include all test data, both field and laboratory, in the report and reference in the appropriate appendices. Include a summary of site conditions and pertinent geology. The subsurface conditions should be described along the route of the project, which might require splitting the discussion into sections along the alignment. Describe the engineering characteristics and anticipated behavior of each soil and rock unit. Identify potentially difficult or problematic conditions. Describe relevant historical information such as past slope performance or instabilities and ground settlement evidence. The groundwater regimes throughout the project should be described. Describe any potential geologic hazards, such as unstable slopes and rockfall hazards. Use data summaries, tables and charts whenever possible. Document any previous report and/or other specific references used to generalize conditions and estimate engineering parameters.

The analysis section of a geotechnical report should summarize the analyses that were performed. When appropriate, include the applicable analysis procedures, including limitations and pertinent assumptions.

The discussion section of the report should draw upon the gathered data and present the various possible alternative solutions that were considered for each specific feature or project. Include a general discussion that communicates the major advantages and disadvantages of each alternative and comparative cost estimates. Describe potential problems disclosed by analyses and identify potential feasible solutions. Provide an assessment of cost, risk and uncertainty associated with each of the possible options.

Recommendations in the report should be concise and directed to the preferred alternative. All detailed information necessary to design and construct the recommended alternative should be provided and all reference literature cited. Identify areas where special treatment may be required and make recommendations on the preferred type of mitigation or solution.

The appendices of a geotechnical report should contain all detailed laboratory test results, exploration logs and field test data used to generate the report. Specific calculations would not normally be included. Include standard terminology and reference charts.

## 5.1.3 REVIEW OF CALCULATIONS AND REPORTS

### 5.1.3.1 General

The geotechnical professional should arrange for a senior-level review of geotechnical reports to be conducted by a professional with the necessary geotechnical or engineering geology experience at the following key project junctures:

- Scope of work memorandum with estimated costs for geotechnical services
- Subsurface investigation plan
- Draft/Final Geotechnical Report

Formal review is generally not required for preliminary drafts of the subsurface investigation memorandum and EIS/planning phase memoranda.

Formal review should be conducted for design recommendations and those that are considered preliminary if significant design effort could be expended or if the recommendations could otherwise end up being treated as final recommendations.

In general, the individual who prepared the design and the first line reviewer should sign the report. If other individuals during the review process require technical design changes be made in the report, they should also sign the report.

Compile a complete set of the analysis computations to adequately document the basis of designs. Generally, the calculations should be saved in a separate file or report, but not usually included in the geotechnical report. In some cases, the geotechnical report might contain an appendix summarizing the analyses and calculations. Refer to specific FLH Division requirements and guidelines.

#### 5.1.3.2 FHWA Guidelines and Checklists

As a guide to ensure that all pertinent items are considered in geotechnical reports, geotechnical professionals should utilize checklists based on FHWA-ED-88-053, which are included in this manual for easy reference (see referenced exhibits below). All geotechnical staff preparing reports should be familiar with the contents, concepts and procedures presented in the checklists. The checklist located at the end of this section contains information that is generally common to all geotechnical reports. Checklists at the end of this section, based on Sections B through J of FHWA-ED-88-053, should be used for specific design categories that are addressed in the geotechnical report (see referenced exhibits below).

Checklists are provided for various types of geotechnical investigations and construction elements, in the following categories:

- Site investigation information (Exhibit 5.1–A)
- Centerline cuts and embankments (Exhibit 5.1–B)
- Embankments over soft ground (Exhibit 5.1–C)
- Landslide corrections (Exhibit 5.1–J)
- Retaining structures (Exhibit 5.1–I)
- Spread footings (Exhibit 5.1–F)
- Driven piles (Exhibit 5.1–G)
- Drilled shafts (Exhibit 5.1–H)
- Ground improvement techniques (Exhibit 5.1–D)
- Material sources (Exhibit 5.1–E)
- PS&E review checklists (Exhibit 5.2–A)

In addition, recent FHWA technical publications are available for different types of geotechnical construction (such as soil nailing and ground anchors). Analyses and computations should be checked by an independent geotechnical professional consistent with appropriate QA/QC procedures.

**Exhibit 5.1–A GTR REVIEW CHECKLIST FOR SITE INVESTIGATION**

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| Geotechnical Report Text (Introduction) | | | |
| 1. Is the general location of the investigation described and/or a vicinity map included? | | | |
| 2. Is scope and purpose of the investigation summarized? | | | |
| 3. Is concise description given of geologic setting and topography of area? | | | |
| 4. Are the field explorations and laboratory tests on which the report is based listed? | | | |
| 5. Is the general description of subsurface soil, rock, and groundwater conditions given? | | | |
| 6. Is the following information included with the geotechnical report (typically included in the report appendices):* | | | |
| a. Test hole logs? | | | |
| b. Field test data? | | | |
| c. Laboratory test data? | | | |
| d. Photographs (if pertinent)? | | | |
| Plan and Subsurface Profile | | | |
| 7. Is a plan and subsurface profile of the investigation site provided?* | | | |
| 8. Are the field explorations located on the plan view? | | | |
| 9. Does the conducted site investigation meet minimum criteria?* | | | |
| 10. Are the explorations plotted and correctly numbered on the profile at their true elecation and location? | | | |
| 11. Does the subsurface profile contain a word description and/or graphic depiction of soil and rock types? | | | |
| 12. Are groundwater levels and date measured shown on the subsurface profile? | | | |
| Subsurface Profile or Field Boring Log | | | |
| 13. Are sample types and depths recorded?* | | | |
| 14. Are SPT blow count, percent core recovery, and RQD values shown?* | | | |
| 15. If cone penetration tests were made, are plots of cone resistance and friction ratio shown with depth? | | | |
| Laboratory Test Data | | | |
| 16. Were lab soil classification tests such as natural moisture content, gradation, Atterberg limits, performed on selected representative samples to verify field visual soil identification?* | | | |
| 17. Are laboratory test results such as shear strength, consolidation, etc., incuded and/or summarized? | | | |

*Note: A response other than (yes) or (N/A) for any of the checklist questions with an * is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project. Since the most important step in the geotechnical design process is to conduct an adequate site investigation, presentation of the subsurface information in the geotechnical report and on the plans deserves careful attention.

**Exhibit 5.1–B GTR CHECKLIST FOR CENTERLINE AND EMBANKMENTS**

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| Are station-to-station descriptions included for: | | | |
| 1. Existing surface and subsurface drainage? | | | |
| 2. Evidence of springs and excessively wet areas? | | | |
| 3. Slides, slumps, and faults noted along the alignment? | | | |
| General Soil Cut or Fill: Are station-to station recommendations included for the following? | | | |
| 4. Specific surface/subsurface drainage recommendations? | | | |
| 5. Excavation limits of unsuitable materials? | | | |
| 6. Erosion protection measures for back slopes, side slopes, and ditches, including riprap recommendaions or special slope treatment.* | | | |
| Soil Cuts: Are station-to-station recommendations included for the following? | | | |
| 7. Recommended cut slope design?* | | | |
| 8. Are clay cut slopes designed for minimum F.S. = 1.50? | | | |
| 9. Special usage of excavated soils? | | | |
| 10. Estimated shrink-swell factors for excavated materials? | | | |
| 11. If answer to 3 is yes, are recommendations provided for design treatment | | | |
| Fills: Are station-to-station recommendations included for the following? | | | |
| 12. Recommended fill slope design? | | | |
| 13. Will fill slope design provide minimum F. S. = 1.25? | | | |
| Rock Slopes: Are station-to-station recommendations included for the following? | | | |
| 14. Are recommended slope designs and blasting specifications provided?* | | | |
| 15. Is the need for special rock slope stabiliazation measures, e.g., rockfall catch ditch, wire mesh slope protection, shotcrete, rock bolts, addressed?* | | | |
| 16. Has the use of “template” designs been avoided (such as designing all rock slopes on 0.25:1 rather than designing based on orientation of major rock jointing)? | | | |
| 17. Have effects of blast induced vibrations on adjacent structures been evaluated?* | | | |

*Note: A response other than (yes) or (N/A) for any of the checklist questions with an * is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project. In addition to the basic information listed in Section 4, is the following information provided in the project geotechnical report.

**Exhibit 5.1–C GTR CHECKLIST FOR EMBANKMENTS OVER SOFT GROUND**

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| Embankment Stability | | | |
| 1. Has the stability of the embankment been evaluated for minimum F.S. = 1.25 for side slope and 1.30 for end slope of bridge approach embankments?* | | | |
| 2. Has the shear strength of the foundation soil been determined from lab testing and/or field vane shear or cone penetrometer tests?* | | | |
| 3. If the proposed embankment does not provide minimum factors of safety given above, are recommendations given or feasible treatment alternatives, which will increase factor of safety to minimum acceptable (such as change alignment, lower grade, use stabilizing counterberms, excavate and replace weak subsoil, lightweight fill, geotextile fabric reinforcement, etc.)?* | | | |
| 4. Are cost caomparisions of treatment alternatives given and a specific alternate recommended?* | | | |
| Settlement of Subsoil | | | |
| 5. Have consolidation properties of fine-grained soils been determined from laboratory consolidation tests? | | | |
| 6. Have settlement amount and time been estimated?* | | | |
| 7. For bridge approach embankments, are recommendations made to get the settlement out before the bridge abutment is constructed (waiting period, surcharge, or wick drains)? | | | |
| 8. If geotechnical instrumentation is proposed to monitor fill stability and settlement, are detailed recommendations provided on the number, type, and specific locations of the proposed instruments? | | | |
| Construction Considerations | | | |
| 9. If excavation and replacement of unsuitable shallow surface deposits (peat, muck, topsoil) is recommended, are vertical and lateral limits of recommended excavation provided? | | | |
| 10. Where a surcharge treatment is recommended, are plan and cross-section of surcharge treatment provided in geotechnical report for benefit of the roadway designer? | | | |
| 11. Are instructions or specifications provided concerning instrumentation, fill placement rates and estimated delay times for the contractor? | | | |
| 12. Are recommendations provided for disposal of surcharge material after the settlement period is complete? | | | |

*Note: A response other than (yes) or (N/A) for any of the checklist questions with an * is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project. Where embankments must be built over soft ground (such as soft clays, organic silts, or peat), stability and settlement of the fill should be carefully evaluated. In addition to the basic information listed in Section 4 is the following information provided in the geotechnical report?

**Exhibit 5.1–D GTR CHECKLIST FOR GROUND IMPROVEMENT TECHNIQUES**

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| 1. For wick drains, do recommendations include the coefficient of consolidation for horizontal drainage, $c_h$, and the length and spacing of wick drains? | | | |
| 2. For lightweight fill, do recommendations include the material properties ($\phi$, c, $\gamma$), permeability, compressibility, and drainage requirements? | | | |
| 3. For vibro-compaction, do the recommendations include required degree of densification (e.g., relative density, SPT blow count, etc.), settlement limitations, and quality conrol? | | | |
| 4. For dynamic compation, do the recommendations include required degree of densifications (e.g., relative density, SPT blow count, etc.) settlement limitations, and quality control? | | | |
| 5. For stone columns, do the recommendations include spacing and dimensions of columns, bearing capacity, settlement characteristics, and permeability (seismic applications)? | | | |
| 6. For grouting, do the recommendations include the grouting method (permeation, compaction, etc.), material improvement criteria, settlement limitations, and quality control? | | | |

*Note: A response other than (yes) or (N/A) for any of the checklist questions with an * is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

In addition to the basic information listed in Section 4, if ground improvement techniques are recommended or given as an alternative, are conclusion/recommendations provided in the project foundation report for the following:

**Exhibit 5.1–E GTR REVIEW CHECKLIST FOR MATERIAL SOURCES**

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| 1. Material site location, including description of existing or proposed access routes and bridge load limits, if any? | | | |
| 2. Have soil samples representative of all materials encountered during pit investigation been submitted and tested?* | | | |
| 3. Are laboratory quality test results included in the report?* | | | |
| 4. For aggregate sources, do the laboratory quality test results (such as L.A. abrasion, sodium sulfate, degradation, absorption, reactive aggregate, etc.) indicate if specification materials can be obtained from the deposit using normal processing methods? | | | |
| 5. If the lab quality test results indicate that specification material cannot be obtained from the pit materials as they exist naturally, has the source been rejeceted or are detailed recommendations provided for processing or controlling production so as to ensure a satisfactory product? | | | |
| 6. For soil borrow sources, have possible difficulties been noted, such as above optimum moisture content for clay-silt soils, waste due to high PL, boulders, etc.?* | | | |
| 7. Where high moisture content clay-silt soils must be used, are recommendations provided on the need for aeration to allow the materials to dry out sufficiently to meet compaction requirements? | | | |
| 8. Are estimated shrink-swell factors provided? | | | |
| 9. Do the proven material site quantitities satisfy the estimated project quantity needs? | | | |
| 10. Have settlement amount and time been estimated?* | | | |
| 11. Have special permit requirements been covered? | | | |
| 12. Are pit reclamation requirements covered adequately? | | | |
| 13. Has a material site sketch (plan and profile) been provided for inclusion in the plans, which contains: | | | |
| a. Material site number? | | | |
| b. North arrow and legal subdivision? | | | |
| c. Test hole / test pit logs, locations, numbers & date? | | | |
| d. Water table elevation and date? | | | |
| e. Depth of unsuitable over burden to be stripped? | | | |
| f. Suggested overburden disposal area? | | | |
| g. Proposed mining area and previously mined areas? | | | |
| h. Existing or suggested access road? | | | |
| i. Bridge load limits? | | | |
| j. Reclamation details? | | | |
| 14. Are recommended special provisions provided? | | | |

*Note: A response other than (yes) or (N/A) for any of the checklist questions with an * is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project. Where embankments must be built over soft ground (such as soft clays, organic silts, or peat), stability and settlement of the fill should be carefully evaluated. In addition to the basic information listed in Section 4 is the following information provided in the geotechnical report?

**Exhibit 5.1–F GTR REVIEW CHECKLIST FOR SPREAD FOOTINGS**

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| Embankment Stability | | | |
| 1. Are spread footings recommended for foundation support? If not, are reasons for not using them discussed?* | | | |
| If spread footing supports are recommended, are conclusions and recommendations given for the following: | | | |
| 2. Is recommended bottom of footing elevation and reason for recommendation (e.g., based on frost depth, estimated scour depth, or depth to competent bearing material) given?* | | | |
| 3. Is recommended allowable soil or rock bearing pressure given?* | | | |
| 4. Is estimated footing settlement and time given?* | | | |
| 5. Where spread footings are recommended to support abutments placed in the bridge end fill, are special gradation and compaction requirements provided for select end fill and backwall drainage material?* | | | |
| Construction Considerations | | | |
| 6. Have the materials been adequately described on which the footing is to be placed so the project inspector can verify that material is as expected? | | | |
| 7. Have excavation requirements been included for safe slopes in open excavations, need for sheeting or shoring, etc.?* | | | |
| 8. Has fluctuation of the groundwater table been addressed? | | | |
| 9. If geotechnical instrumentation is proposed to monitor fill stability and settlement, are detailed recommendations provided on the number, type, and specific locations of the proposed instruments? | | | |

In addition to the basic information listed in Section 4, is the following information provided in the project foundation report?

*Note: A response other than (yes) or (N/A) for any of the checklist questions with an * is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

**Exhibit 5.1–G GTR REVIEW CHECKLIST FOR DRIVEN PILES**

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| Design Considerations | | | |
| 1. Is the recommended pile type given (displacement, non-displacement, steel pipe, concrete, H-pile, etc.) with valid reasons given for choice and/or exclusion?* | | | |
| 2. Do you consider the recommended pile type (s) to be the most suitable and economical? | | | |
| 3. Are estimated pile lengths and estimated tip elevations given for the recommended allowable pile design loads?* | | | |
| 4. Are recommended design loads considered reasonable? | | | |
| 5. Has pile group settlement been estimated (only of practical significance for friction pile groups ending in cohesive soil)? | | | |
| 6. Have the materials been adequately described on which the footing is to be placed so the project inspector can verify that material is as expected?* | | | |
| 7. Has design analysis (wave equation analysis) verified that the recommended pile section can be driven to the estimated or specified tip elevation without damage (especially applicable where dense gravel-cobble-boulder layers or other obstructions have to be penetrated)?* | | | |
| 8. Where scour piles are required, have pile design and driving criteria been established based on mobilizing the full pile design capacity below the scour zone? | | | |
| 9. Where lateral load capacity of large diameter piles is an important design consideration, are p-y curves (load vs. deflection) or soil parameters given in the geotechnical report to allow the structural engineer to evaluate lateral load capacity of all piles? | | | |
| 10. For pile supported bridge abutments over soft ground: | | | |
| a. Has abutment downdrag load been estimated and solutions such as bitumen coating been considered in design? Not generally required if surcharging of the fill is being performed. | | | |
| b. Is bridge approach slab recommended to moderate differential settlement between bridge ends and fill? | | | |
| c. If the majority of subsoil settlement will not be removed prior to abutment construction (by surcharging), has estimate been made of abutment rotation that can occur due to lateral squeeze of soil subsoil? | | | |
| d. Does the geotechnical report specifically alert the structural designer to the estimated horizontal abutment movement? | | | |
| 11. If bridge project is large, has pile load test program been recommended? | | | |

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| 12. For major structure in high seismic risk area, has assessment been made of liquefaction potential of foundation soil during design earthquake (only loose saturated sands and silts are susceptible to liquefaction)? (See GEC No. 3, FHWA-SA-97-076) | | | |
| Construction Considerations | | | |
| 13. Pile driving details such as: boulders or obstructions which may be encountered during driving; need for preaugering, jetting, spudding; need for pile tip reinforcement; driving shoes, etc. | | | |
| 14. Excavation requirements: safe slope for open excavation; need for sheeting or shoring; fluctuation of groundwater table? | | | |
| 15. Have effects of pile driving operation on adjacent structures been evaluated such as protection against damage caused by footing excavation or pile driving vibrations? | | | |
| 16. Is preconstruction condition survey to be made of adjacent structures to prevent unwarranted damage claims? | | | |
| 17. On large pile driving projects, have other methods of pile driving control been considered such as dynamic testing or wave equation analysis? | | | |

In addition to the basic information listed in Section 4, if pile support is recommended or given as an alternative, conclusions/recommendations should be provided in the project geotechnical report for the following:

*Note: A response other than (yes) or (N/A) for any of the checklist questions with an * is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

**Exhibit 5.1–H GTR REVIEW CHECKLIST FOR DRILLED SHAFTS**

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| Design Considerations | | | |
| 1. Are recommended shaft diameter(s) and length(s) for allowable design loads based on an analysis using soil parameters for side friction and end bearing?* | | | |
| 2. Settlement estimated for recommended design loads?* | | | |
| 3. Where lateral load capacity of shaft is an important design consideration, are p-y (load vs. deflection) curves or soils data provided in geotechnical report that will allow structural engineer to evaluate lateral load capacity of shaft?* | | | |
| 4. Is static load test (to plunging failure) recommended? | | | |
| Construction Considerations | | | |
| 5. Have construction methods been evaluated, i.e., can less expensive dry method or slurry method be used or will casing be required? | | | |
| 6. If casing will be required, can casing be pulled as shaft is concreted (this can result in significant cost shaving on very large diameter shafts)? | | | |
| 7. If artesian water was encountered in explorations, have design provisions been included to handle it (such as by requiring casing and a tremie seal)? | | | |
| 8. Will boulders be encountered? (If boulders will be encountered, then the use of shafts should be seriously questioned due to construction installation difficulties and resultant higher cost to boulders can cause) | | | |

In addition to the basic information listed in Section 4, if drilled shaft support is recommended or given as an alternative, are conclusion/recommendations provided in the project foundation report for the following:

*Note: A response other than (yes) or (N/A) for any of the checklist questions with an * is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

### Exhibit 5.1–I GTR REVIEW CHECKLIST FOR RETAINING STRUCTURES

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| Design Considerations | | | |
| 1. Recommneded soil strength parameters and groundwater elevations for use in computing wall design lateral earth pressures and factor of safety for overturning, sliding, and external slope stability.* | | | |
| 2. Is it proposed to bid alternate wall designs? | | | |
| 3. Are acceptable reasons given for the choice and/or exclusion of certain wall types?* | | | |
| 4. Is an analysis of the wall stability included with minimum acceptable factors of safety against overturning (F.S. = 2.0), sliding (F.S. = 1.5), and external slope stability (F.S. = 1.5)?* | | | |
| 5. If wall will be placed on compressible foundation soils, is estimated total, differential and time rate of settlement given? | | | |
| 6. Will wall types selected for compressible foundation soils allow differential movement without distress? | | | |
| 7. Are wall drainage details, including materials and compaction, provided? | | | |
| Construction Considerations | | | |
| 8. Are excavation requirements covered including safe slopes for open excavations or need for sheeting or shoring? | | | |
| 9. Fluctuation of groundwater table? | | | |
| Top-down Construction Type – See “Manual for Design & Construction Monitoring of Soil Nail Walls,” FHWA-SA-96-069R, and “Ground Anchors and Anchored Systems,” FHWA-IF-99-015. | | | |
| 10. For soil nail and anchor walls are the following included in the geotechnical report? | | | |
| a. Design soil parameters ($\phi$, c, $\gamma$) | | | |
| b. Minimum bore size (soil nails)? | | | |
| c. Design pullout resistance (soil nails)? | | | |
| d. Ultimate anchor capacity (anchors)? | | | |
| e. Corrosion protection requirements? | | | |

In addition to the basic information listed in Section 4, is the following information provided in the project geotechnical report? Refer to “Earth Retaining Structures” FHWA-NHI-99-025).

*Note: A response other than (yes) or (N/A) for any of the checklist questions with an * is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

## Exhibit 5.1–J GTR REVIEW CHECKLIST FOR LANDSLIDE CORRECTIONS

Each year hundreds of millions of dollars are spent to correct soil or rock-related instabilities on highways. The purpose of this technical note is to advise field engineers what technical support information is essential such that a complete evaluation can be performed. For the purpose of this technical note, soil and rock-related instabilities are defined as follows: "A condition that currently or threatens to affect the stability or performance the stability or performance of a highway facility and is the result of the inadequate performance of the soil or rock components." This includes major instabilities resulting form or associated with: landslides, rockfalls, sinkholes, and degrading shales. Technical support data needed are:

1. Site plan and typical cross-section(s) representing ground surface conditions prior to failure, along with subsurface configuration after failure. Photographs, including aerials, if available, would also be beneficial.
2. Cross-section(s) showing soil and/or rock conditions and water bearing strata as determined by drilling and possibly geophysical surveys.
3. Description of the latent state of the unstable mass, whether movement has stopped or is still occurring, and if so, at what rate.
4. Boring logs.
5. Shear strength test data and a description of the testing method utilized on the materials, through which failure is occurring. Where average shear strength is calculated using an assumed failure surface and a factor of safety of 1.0, the complete analysis should be provided and location of assumed water table(s) shown.
6. Proposed corrective schemes including: estimated costs, final safety factors, and design analysis for each alternative solution.
7. Narrative report containing instability history; record of maintenance costs and activity, and preventative measures taken, if any; reasons for inadequacy of the original design; description and results of subsurface investigation performed; summary and results of stability analysis performed; and recommendations for correction.

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| Design Considerations | | | |
| 1. Is a site plan and scaled cross-section provided showing ground surface conditions both before and after failure?* | | | |
| 2. Is the past history of the slide area summarized, including movement history, summary of maintenance work and costs, and previous corrective measures taken, if any?* | | | |
| 3. Is a summary given of results of site investigation, field and lab testing, and stability analysis, including cause(s) of the slide?* | | | |
| Plan | | | |
| 4. Are detailed slide features, including location of ground surface cracks, head scarp, and toe bulge, shown on the site plan?* | | | |

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| Cross-Section | | | |
| 5. Are the cross-sections used for stability analysis included with the soil profile, water table, soil unit weights, soil shear strengths, and failure plane shown as it exists? | | | |
| 6. Is slide failure plane location determined from slope indicators? | | | |
| 7. For an active slide, was soil strength along the sile failure plane back-calculated using a F.S. = 1.0 at the time of failure? | | | |
| Text | | | |
| 8. Is the following information presented for each proposed correction alternative (typical correction methods include buttress, shear key, rebuild slope, surface drainage, subsurface drainage-interceptor, drain trenches or horizontal drains, etc.) | | | |
| a. Cross-section of proposed alternative? | | | |
| b. Estimated safety factor? | | | |
| c. Estimated cost? | | | |
| d. Advantages and disadvantages? | | | |
| 9. Is recommended correction alternative (s) given that provide a minimum F.S. = 1.25 | | | |
| 10. If horizontal drains are proposed as part of slide correction, has subsurface investigation located definite water bearing strata that can be tapped with horizontal drains? | | | |
| 11. If a toe counterberm is proposed to stabilize an active slide has field investigation confirmed that the toe of the existing slide does not extend beyond the toe of the proposed counterberm? | | | |
| Construction Considerations | | | |
| 12. Where proposed correction will require excavation into the toe of an active slide (such as for buttress or shear key) has the "during construction backslope F.S." with open excavation been determined? | | | |
| 13. If open excavation F.S. is near 1.0, has excavation stage construction been proposed? | | | |
| 14. Has seasonal fluctuation of groundwater table been considered? | | | |
| 15. Is stability of excavation backslope to be monitored? | | | |
| 16. Are special construction features, techniques and materials described and specified? | | | |

In addition to the basic information listed in Section 4 is the following information provided in the landslide study geotechnical report?

*Note: A response other than (yes) or (N/A) for any of the checklist questions with an * is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

## 5.2 FINAL DESIGN AND REVIEW OF PLANS AND SPECIFICATIONS

To maximize the benefits of the geotechnical investigation and resultant report, the geotechnical professional should interact with the project design and construction engineers throughout the duration of the project. The geotechnical criteria and recommendations should be considered and incorporated into the project as the design is developed. Provide geotechnical support during design to ensure the Plans, specifications, and estimates of cost and/or quantity adequately reflect the geotechnical concerns and recommendations. The geotechnical professional evaluates the reasonableness and acceptability of risks and consequences of design options.

*DETAILS, DRAWINGS AND SPECIFICATIONS*

The geotechnical professional should prepare drafts of typical details or construction drawings to demonstrate unique geotechnical designs and provide them to the roadway or bridge designer. The FLH Standard Specifications (FP-XX), supplemental specifications, standard drawings and details (FLH Std Drawings) and special contract requirements (FLH SCR) should be used wherever possible. However, some situations such as use of new or experimental technologies may warrant unique specifications. Refer to guidance in Section 4 and referenced design manuals for example details and specifications.

*REVIEW OF PRELIMINARY PLANS AND SPECIFICATIONS*

Plans and specifications that are not consistent with geotechnical recommendations could result in contractor claims. In addition to writing the report, the geotechnical professional should carefully review the plans and specifications to verify that the geotechnical recommendations have been correctly incorporated. PS&E checklists providing guidance in checking the plans and specifications with geotechnical reports for consistency are available in FHWA-ED-88-053 (pages 31 through 38), and are reproduced at the end of this TGM section (Exhibit 5.2–A). Any discrepancies should be discussed with the roadway or bridge designer and resolved. Check that the correct geotechnical reports have been referenced in the contract documents. Provide any necessary “Geotechnical Advisory” statements and notations within the plans and specifications package.

*ASSISTANCE WITH ENGINEER’S COST ESTIMATE*

The geotechnical professional may be asked to provide unit cost estimates for specialized construction items. The cost estimating system used by Federal Lands Highway is FLH Engineer's Estimate Program. Guidance on unit costs is often described in various FHWA design manuals (see Section 4). Where cost data is insufficient or missing, contact industry representatives, the FHWA Resource Center, or State DOT geotechnical staff who have demonstrated expertise in the construction item being evaluated.

The geotechnical professional should arrange for a senior-level review of geotechnical reports to be conducted by a professional with the necessary geotechnical or engineering geology experience at the following key project junctures:

- Scope of work memorandum with estimated costs for geotechnical services
- Subsurface investigation plan

- Draft/Final Geotechnical Report

Formal review is generally not required for preliminary drafts of the subsurface investigation memorandum and EIS/planning phase memoranda.

Formal review should be conducted for design recommendations and those that are considered preliminary if significant design effort could be expended or if the recommendations could otherwise end up being treated as final recommendations.

In general, the individual who prepared the design and the first line reviewer should sign the report. If other individuals during the review process require technical design changes be made in the report, they should also sign the report.

Compile a complete set of the analysis computations to adequately document the basis of designs. Generally, the calculations should be saved in a separate file or report, but not usually included in the geotechnical report. In some cases, the geotechnical report might contain an appendix summarizing the analyses and calculations. Refer to specific FLH Division requirements and guidelines.

Standards for review of plans and specifications are in PDDM 6.5.2.

The primary source supporting the standards and guidance is FHWA-ED-88-053. The secondary sources are ASFE Guidelines, for reports, and Division QA/QC plans for calculations.

Refer to Appendix B.20, Reports.

**Exhibit 5.2–A PS&E REVIEW CHECKLISTS**

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| A. General | | | |
| 1. Has the appropriate geotechnical engineer reviewed the PS&E to ensure that the design and construction recommendations have been incorporated as intended and that the subsurface information has been presented correctly? This is absolutely necessary* | | | |
| 2. Are the finished profile exploration logs and locations included in the plans? | | | |
| 3. Have geotechnical designs prepared by Division or consultants been reviewed and approved by the geotechnical engineer?* | | | |
| 4. Do the contract documents contain the special provisions as provided in the project geotechnical report? | | | |
| 5. Have the following common pitfalls been avoided: | | | |
| a. Has an adequate site investigation been conducted (reasonably meeting or exceeding the minimum criteria)? | | | |

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| b. Has the use of "subjective" subsurface terminology (such as relatively soft rock or gravel with occasional boulders) been avoided? | | | |
| c. If alignment has been shifted, have additional explorations been conducted along the new alignment? | | | |
| d. Has a note been included in the contract indicating all subsurface information in available to bidders? | | | |
| e. Do you think the wording of the geotechnical special provisions are clear, specific and unambiguous? | | | |
| B. Centerline Cuts and Embankments | | | |
| 1. Where excavation is required, are excavation limits and description of unsuitable organic soils shown on the plans? | | | |
| 2. Are plan details and special provisions provided for special drainage blanket under sidehill fill, interceptor trench drains, etc.? | | | |
| 3. Are special provisions included for fill materials requiring special treatment, such as nondurable shales, lightweight fill, etc.? | | | |
| 4. Are special provisions provided for any special rock slope excavation and stabilization measures called for in plans, such as controlled blasting, wire mesh slope protection, rock bolts, shotcrete, etc? | | | |
| C. Embankments Over Soft Ground | | | |
| 1. Where subexcavation is required, are excavation limits and description of unsuitable soils clearly shown on the plans?* | | | |
| 2. Where settlement waiting period will be required, has estimated settlement time been stated in the special provisions to allow bidders to fairly bid the project?* | | | |
| 3. If instrumentation will be used to control the rate of fill placement, do special provisions clearly spell out how this will be done and how the readings will be used to control the contractor's operation?* | | | |
| 4. Do special provisions state that any instrumentation damage by contractor personnel will be repaired at the contractor's expense? | | | |
| D. Landslide Corrections | | | |
| 1. Are plan details and special provisions provided for special drainage details, such as lined surface ditches, drainage blankets, horizontal drains, etc.? | | | |
| 2. Where excavation is to be made into the toe of an active slide, such as for a buttress or shear key, and stage construction is required, do the special provisions clearly spell out the stage construction sequence to be followed?* | | | |

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| 3. Where a toe buttress is to be constructed, do the special provisions clearly state gradation and compaction requirements for the buttress material?* | | | |
| 4. If the geotechnical report recommends slide repair work not be allowed during the wet time of year, is the construction schedule in accord with this?* | | | |
| E. Retaining Structures | | | |
| 1. Are selected materials specified for wall backfill with gradation and compaction requirements covered in the specification?* | | | |
| 2. Are limits of required select backfill zones clearly detailed on the plans? | | | |
| 3. Are excavation requirements specified, e.g., safe slopes for excavations, need for sheeting, etc. | | | |
| 4. Where alternative wall types will be allowed, are fully detailed plans included for all alternatives?* | | | |
| 5. Were designs prepared by the wall supplier? | | | |
| 6. Were wall supplier's design calculations and specifications reviewed and approved by the structural and geotechnical engineers? | | | |
| 7. Where proprietary retaining walls are bid as alternates, does bid schedule require bidders to designate which alternate their bid is for, to prevent bid shopping after contract award? | | | |
| 8. Have FHWA guidelines for experimental designations for certain proprietary wall types been followed? | | | |
| 9. Is ROW limit or easements shown on plans and mentioned in specifications where anchors are to be installed? | | | |
| Top-down Construction Type Walls- See "Manual for Design & Construction Monitoring of Soil Nail Walls," FHWA-SA-96-069R, and "Ground Anchors and Anchored Systems," FHWA-IF-99-015. | | | |
| 10. For soil nail and anchor walls are the following included in the provisions? | | | |
| a. Construction tolerances? | | | |
| b. Minimum drill-hole size? | | | |
| c. Material requirements? | | | |
| d. Load testing procedures and acceptance criteria? | | | |
| e. Construction monitoring requirements? | | | |
| F. Spread Footings | | | |
| 1. Where spread footings are to be placed on natural soil, is the specific bearing strata in which the footing is to be founded clearly described, e.g., placed on Br. Sandy GRAVEL deposit, etc.?* | | | |

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| 2. Where spread footings are to be placed in the bridge end fill, are gradation and compaction requirements, for the select fill and backfill drainage material, covered in the special provisions, standard specifications, or standard structure sheets?* | | | |
| G. Driven Piles | | | |
| 1. Do plan details adequately cover pile splices, tip reinforcement, driving shoes, etc? | | | |
| 2. Where friction piles are to be driven in silty or clayey soils, significant setup or soil freeze affecting long-term capacity may occur. Do specifications require retapping the piles after 24 to 48 hour waiting period when required bearing is not obtained at estimated length at the end of initial driving?* | | | |
| 3. Where friction piles are to be load tested, has a reaction load of 4 times design load been specified to allow load testing the pile to plunging failure so that the ultimate soil capacity can be determined? | | | |
| 4. Where end bearing steel piles are to be load tested, has load test been designed to determine if higher than 62 MPa (9ksi) allowable steel stress can be used, e.g., 83 to 103 MPa (12-15ksi)? | | | |
| 5. Where cofferdam construction will be required, have soil gradation results been included in the plans or been made available to bidders to assist them in determining dewatering procedures?* | | | |
| 6. If a wave equation analysis will be used to approve the contractor's pile driving hammer, has a minimum hammer energy or estimated soil resistance in kN (tons) to be overcome to drive the piles to the estimated length, been given in the special provisions? | | | |
| 7. Has the appropriate safety factor, based on construction control method (static load test, dynamic load test, wave equation, etc.) been included? Have the specifications for the applicable construction control method been included? | | | |
| H. Drilled Shafts | | | |
| 1. Where drilled shafts are to be placed in soil, is the specific bearing stratum in which the drilled shaft is to be found clearly described, e.g., placed on Br. Sandy GRAVEL deposit, etc.?* | | | |
| 2. Where end bearing drilled shafts are to be founded on rock, has the rock elevation at the shaft pier locations been determined form borings at the pier locations? | | | |
| 3. Where drilled shafts are to be socketed some depth into rock, have rock cores been extracted at depth to 3m (10ft) below proposed socket at location within 3m (10ft) of the shaft? | | | |

| | Yes | No | Unknown or N/A |
|---|---|---|---|
| 4. Are shafts equipped with PVC access tubes to accommodate non-destructive testing (gamma/gamma logging, cross-hole sonic logging) of the shaft? Are provisions for the appropriate non-destructive testing methods included?* | | | |
| I. Ground Improvement Techniques | | | |
| 1. For wick drains, are contractors submittals required that include proposed equipment and materials, method (s) for addressing obstructions, and method(s) for splicing wick drains. | | | |
| 2. For lightweight fill, are min/max densities, gradation, lift thickness, and method of compaction specified? | | | |
| 3. For vibro-compaction, are contractor submittals required that include proposed equipment and materials? Are methods of measurement and acceptance criteria specified? | | | |
| 4. For dynamic compaction: | | | |
| a. If method specification is used, are the following specified: tamper mass and size; drop height; grid spacing; applied energy; number of phases or passes; site preparation requirements; subsequent surface compaction procedures? | | | |
| b. If performance specification is used, are the following specified: minimum soil property value to be achieved and method of measurement; maximum permissible settlement? | | | |
| 5. For stone columns, are the following specified: site preparation, backfill, minimum equipment requirements, acceptance criteria and quality assurance procedures? | | | |
| 6. For grouting, are contractor submittals required that include equipment and materials. Are methods of measurement and acceptance criteria specified? | | | |
| J. Material Sources | | | |
| 1. Is a material site sketch, containing basic information, included in the plans?* | | | |
| 2. Has the material site investigation established a proven quantity of material sufficient to satisfy the project estimated quantity needs?* | | | |
| 3. Where specification material cannot be obtained directly from the natural deposit, do the special provisions clearly state that processing will be required? | | | |
| 4. Are contractor special permit requirements convered in the special provisions? | | | |
| 5. Are pit reclamation requirements clearly spelled out on the plans and in the special provisions? | | | |

Plans and specifications (PS&E) reviews of projects with major or unusual geotechnical features should preferably be made by examining the plans, special provisions, and geotechnical report together. Certain checklist items are of vital importance to have been included in the PS&E. These checklist items have been marked with an asterisk (*). A negative response to any of these asterisked items is cause to contact the geotechnical engineer for clarification of this omission. The information covered in Section A, General will apply to all geotechnical features. The rest of the sections cover additional important PS&E review items that pertain to specific geotechnical features.
For purposes of this document, PS&E refers to a plan and specification review at any time during a project's development. Hence, the review may be at a preliminary or partial stage of plan development. When plan reviews are conducted at a partial stage the final geotechnical report may not be available.
*Note: A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

**Exhibit 5.2–B FINAL DESIGN AND REVIEW REFERENCES**

| Subject | Primary Source | Secondary Sources |
|---|---|---|
| Final Design | FHWA-ED-88-053 | |
| Plans and Specifications | FP-XX | FLH Std Drawings |
| Cost Estimates | FLH Engineer's Estimate Program | RS Means<br>USACE ER 1110-2-1302 |
| Instrumentation Monitoring | NHI 132041 | TRB SR 247 |
| Addendum Reports | FHWA-ED-88-053 | |
| Planning Geotechnical Services for Construction Phase | NHI 132012 | |

## 5.3 CONSTRUCTION SUPPORT

The geotechnical professional should assist the construction project manager and inspectors during construction. Unusual designs may require further explanation in terms of inspection, execution or acceptance. Troubleshooting may become necessary. Sometimes, specialized monitoring or testing is required. Refer to NHI 132012 for examples of construction support. The geotechnical professional should prepare a work scope for construction services with associated budget estimates.

Geotechnical professionals are involved during the construction phase of projects, which includes participating at Prebid and Preconstruction meetings (projects that have major or complex geotechnical issues and designs), reviewing contractor submittals, providing advice and responding to geotechnical issues encountered during construction. Geotechnical professionals may be asked to visit the site to observe problems firsthand.

Inform the construction project manager of any specialized geotechnical concerns or requirements and help provide related orientation or training for project inspectors. Projects with significant geotechnical construction items would warrant a memorandum specifically written for FLH construction staff, providing guidance for inspection, testing and monitoring. Provide guidance for identification of potential geotechnical problem conditions. For general guidance, refer to NHI 132031. Other references could include FHWA-RD-86-160, ADSC 1989, FHWA-SA-94-035, FHWA-SA-93-068, FLH Anchor Inspection, NHI 132012, TRB SAR 8, and FHWA-FL-91-002.

The geotechnical professional may be asked to comment on issues raised during the bid process to clarify designs and intent of specifications and required submittals.

The geotechnical professional should be prepared to participate in Prebid and Preconstruction Meetings for projects that have major or complex geotechnical issues and designs, at the request of the construction project manager.

During construction, in situ materials and construction methods may require inspection to assure compliance with the design assumptions and the project specifications. The inspection tasks may include subgrade and/or embankment compaction control, backfilling techniques around structural elements, footing foundations, drilled shafts, piles, soil nail or ground anchor installations, rock bolting, shotcreting and other specialized geotechnical items. The construction project manager and inspectors may need to be provided with geotechnical information and written guidelines to perform their tasks effectively.

Existing structures and potentially unstable slopes or landslides may need to be instrumented and monitored, including groundwater level changes, settlement, heave, and/or lateral displacement of the structures. Mitigating actions may be necessary to reduce the impact of construction-induced ground movements and other geotechnical issues.

### 5.3.1 REVIEW OF CONTRACTOR SUBMITTALS

Qualification and equipment/procedure submittals, shop drawings or calculations are typically provided for foundations, retaining systems, ground anchors, soil nails, underpinning, shoring and dewatering. Blasting plans are required for rock cuts. Submittals could also include contractor approaches to unique construction, such as landslide mitigation and staged embankment construction, value engineering and alternate wall proposals. Review turn-around time is specified in the contract documents: verify the time available for geotechnical review with the construction project manager.

For driven pile foundations, the geotechnical professional reviews the contractor's pile driving submittal and evaluates the proposed pile/hammer system to calculate driving stresses and the driving criteria that corresponds to the soil resistance loads. The geotechnical professional checks the contractor's dynamic pile driving analysis (WEAP) to verify the adequacy of the proposed equipment and methods and to verify driving criteria. FLH may run additional WEAP analyses if the contractor's analyses are deemed incorrect or marginally satisfactory, but generally it is the contractor's responsibility to prove the right equipment is being used to achieve tip elevations and capacities without damaging piles.

Typically, retaining wall vendors perform the internal design for MSE systems. Geotechnical professionals review calculations and shop drawings submitted by wall vendors for external and internal stability. When requested, geotechnical professionals assist inspectors to verify that engineering properties used by wall vendors match the actual field conditions, and that compaction techniques and efforts being employed are appropriate and adequate.

Temporary shoring, cofferdams, and dewatering are typically the responsibility of the contractor. Shoring requirements are specified in “Occupational Safety and Health Standards for the Construction Industry” (refer to OSHA Section 29) promulgated by the Occupational Safety and Health Administration, U.S. Department of Labor. The geotechnical professional may be asked to review shop drawings and calculations and assist the inspector when the shoring system is complex and critical facilities are located nearby. Dewatering may be necessary in excavation for bridge foundations and retaining walls to prevent base heave or subgrade softening, and to improve stability of side slopes. Instrumentation might be required in critical shoring and dewatering applications.

### 5.3.2 FIELD SUPPORT AND TROUBLESHOOTING

No matter how carefully projects are investigated and designed, the possibility exists for unforeseen problems to arise during construction. Geotechnical professionals should be prepared to investigate when such problems occur, and recommend design changes or changes in construction techniques to suit the conditions, while minimizing construction delays. Perform prompt investigations of claimed or apparent “changed conditions” to assist in the resolution of issues and design or construction changes. If it is determined that the cause of a problem has a geotechnical basis, the geotechnical professional should recommend remedial actions that will eliminate, or at least minimize, potential consequences. A quick evaluation and list of recommendations may be necessary to keep an emerging issue from becoming a major construction and safety problem. Refer to NHI 132012. Additional references include Peck 1969, ASCE MREP 56, NHI 132031, and OSHA Section 29. Examples of construction issues include the following:

- Excavation areas might encounter groundwater or springs. The geotechnical professional should evaluate the nature of groundwater flow and determine the appropriate drainage systems or other mitigation measures. Standard earthwork guidelines are described in TRB SAR 8.

- Projects involving ground improvement measures (such as surcharging, wick drains, stone columns, dynamic compaction, or grouting) typically require the geotechnical professional to provide support to the inspector. Once design assumptions are confirmed and the ground improvement contractor has established a routine, the geotechnical professional may decrease involvement to regular review of daily inspection reports and occasional site visits. The site visit is an opportunity to verify any ground variation from the anticipated subsurface conditions. If variations exist, evaluate the need to modify ground improvement operations to suit the different subsurface conditions. Refer to FHWA-RD-83-026 and FHWA-SA-98-086R.

- The geotechnical professional may be asked to review the contractor’s blasting submittals, and to observe and evaluate test blasts. Previously undisclosed rock slope problems could occur during construction. Typically, as construction proceeds, slope conditions, and the need for special measures such as rock bolts, can change due to blasting or scaling operations. Refer to NHI 13211.

- The geotechnical professional should be prepared to modify rockfall mitigation measures. Because rockfall mitigation measures are rarely applied, construction personnel generally have little experience with them. Provide on-site support for specialty work items such as slope scaling, slope screening, rock bolting, block underpinning, cable lashing, barrier system installation and shotcrete placement. Refer to NHI 13219.

- The geotechnical professional may be asked to evaluate problem conditions encountered in excavations for shallow foundations. Refer to GEC-6. Refer to OSHA Section 29 regulations that apply to temporary slopes and trenches.

- Excavation of material borrow sites may need to be reviewed if problematic variations in stratigraphy exist from those originally anticipated.

- On some projects, specifications will require that dynamic testing be performed to confirm design assumptions and foundation capacity, determine site-specific soil engineering properties, and to evaluate potential pile damage. Dynamic testing is performed using a Pile Driving Analyzer (PDA). Geotechnical professionals could be asked to review and evaluate the test data. Refer to NHI 132021 and NHI 13212. An inspector's guide is provided in FHWA-RD-86-160.

- Geotechnical professionals should be involved in load tests during construction to verify design assumptions and construction methods. Traditionally, load tests have only been required for complex or high capacity deep foundations. Designs may need to be modified as necessary based on the results of load tests. Refer to FHWA-RD-99-170 and NHI 132021, FHWA-SA-94-035, and ASTM D 2113 and ASTM D 5079 in the ASTM Stds

- The quality of a drilled shaft installation is dependent on the construction procedures. The geotechnical professional should evaluate the contractor's proposed construction methods before equipment is mobilized to the site. Geotechnical professionals should contact inspectors to discuss drilled shaft requirements. For drilled shafts socketed into rock, it is important to evaluate the quality of the rock socket. Refer to ADSC 1989 and NHI 132070 (inspector's manuals) and overall guidance in FHWA-IF-99-025.

- Various test methods are available to assess the quality of in-place deep foundation elements. These quality assurance tests need to be performed by qualified personnel, and the results analyzed and interpreted by experienced engineers in order to provide meaningful results. Pile Integrity Testing can be used to detect anomalies, such as necking or voids in some drilled shafts. Furthermore, since drilled shaft foundations carry high loads, it is common to perform high-resolution integrity testing on every shaft.

- Soil nail load tests are conducted by contractors and evaluated by Inspectors. Typically, the geotechnical professional is involved in the early stage of soil nail installation and when problem conditions are encountered. A comprehensive discussion on soil nail construction and inspection is included in the FHWA-SA-93-068, FWA-IF-99-026, and GEC-7.

- Ground anchor load tests are conducted by contractors and evaluated by Inspectors. Geotechnical professionals may be requested to assist with these evaluations. Inspectors should maintain well-documented written records of contractors' operations and installation details, especially in dealing with potential construction claims. The geotechnical professional assists the construction project manager/inspectors and reviews the test results for approval. A detailed discussion of load testing for post-tensioned ground anchors is available in GEC-4. Also, inspection guidelines are provided in FLH Anchor Inspection.

- Landslide mitigation for FLH projects are sometimes complex. Uncertainty is a factor in mitigation design. Assumptions regarding subsurface conditions are necessary because not all subsurface conditions can be identified in the investigations (particularly for complex slides), which could require geotechnical involvement during construction. It is important for the geotechnical professional to visit the construction of mitigation measures at complex slides to identify actual subsurface conditions and provide guidance on the management of unstable conditions. For example, in order to maintain slope stability when excavating and constructing a rockfill buttress at the toe of landslides, a staged construction approach is often used to limit the amount of excavation that is allowed at any one time (to minimize the loss of ground support). Geotechnical professionals should clearly communicate these requirements to Inspectors. Once construction begins, the geotechnical professional should evaluate potential differing site conditions that could require field adjustments. This situation is not uncommon, and therefore, a plan should be prepared in advance to address potential changed conditions scenarios in the event they occur. Refer to FHWA-SA-94-005, FHWA-RT-88-040, and TRB SR 247.

### 5.3.3 INSTRUMENTATION MONITORING DURING CONSTRUCTION

Field instrumentation could be used during construction to verify that actual field conditions are in agreement with the assumptions made for the design or to monitor performance of the facility and/or changes in the field. Instrumentation can provide early warning of potential problems, and should be monitored according to the schedule developed by the geotechnical professional. Immediate data reduction and evaluation is typically required. Problems identified by instrumentation often require immediate construction response and/or mitigation efforts. Refer to NHI 13241, NHI 132031, AASHTO MSI-1, TRB SR 247, and NCHRP Synthesis 89.

### 5.3.4 GEOTECHNICAL DOCUMENTATION

Valuable geotechnical information is gained from construction projects. Therefore, it is important to document this information for future applications. This data is often helpful during the design of other projects under similar conditions and often is valuable in addressing construction claims. Problems similar to those encountered during construction of completed projects can possibly be avoided in the future when the geotechnical professional has detailed records of the problems and solutions. Refer to NHI 132031.

Complete construction records of the geotechnical aspects should be kept. Note any specialized construction procedures or design changes. Special Provisions should be modified and improved based on experiences gained from past projects.

Geotechnical professionals should document observations made during each site visit. Documentation should include written descriptions of problem soil and rock conditions, as well as photographs.

If appropriate, the geotechnical professional should take cross-section measurements of problem areas. Cross-sections in conjunction with station and offset limits help quantify problem areas. Measurements should be linked to stationing or permanent benchmarks (identifiable on a site map). These measurements could be valuable when negotiating potential contractor claims.

**Exhibit 5.3–A CONSTRUCTION SUPPORT REFERENCES**

| Subject | Primary Source | Secondary Sources |
|---|---|---|
| Contractor Submittals | | |
| Footing Inspection | | |
| Pile Inspection | NHI 132022 | NHI 132021<br>NHI 132069 |
| Drilled Shaft Inspection | NHI 132070 | |
| Micropile Inspection | FHWA-NHI-05-039 | FHWA-SA-97-070 |
| MSE Wall Inspection | FHWA-NHI-00-043 | |
| Soil Nail Inspection | FHWA-SA-93-068 | |
| Anchor Inspection | GEC-4 | |
| Earthwork Inspection | TRB SAR 8 | |
| Ground Improvement Inspection | NHI 132034 | |
| Instrumentation Installation and Monitoring | NHI 132031 | AASHTO MSI-1<br>NHI 132012<br>NHI 132041<br>NCHRP Synthesis 89<br>TRB SR 247 |
| Geotechnical Documentation | NHI 132031 | |

## 5.4 POST-CONSTRUCTION MONITORING AND EMERGENCY RESPONSE

Instrumentation that was installed and monitored during the geotechnical investigation phase should continue to be monitored during Final Design since groundwater levels are usually seasonal, and ground movements could vary with time. Reconsider or modify designs if new subsurface conditions are identified during continued instrument monitoring that indicate significant deficiencies in the original design. In addition, longer records of instrumentation data can be very useful to contractors in planning excavations, such as knowing if groundwater could be encountered and whether special precautions or dewatering might be necessary. Refer to Section 4.10, NHI 13241 and TRB SR 247 for further guidance regarding instrumentation and monitoring.

Maintenance staff may contact the Geotechnical Discipline regarding specific problems, possibly requiring immediate assistance. Monitoring of problem conditions may include field evaluations, documentation, and instrumentation. Recommendations should be provided when problem conditions are significant or when Maintenance staff has plans to make repairs.

Typically, potential maintenance problems include roadway settlement/distortion, swelling ground, slope erosion, slope failures, rock slope degradation, rockfall hazards, and groundwater seepage. In addition, damage could occur to constructed items such as subdrains, horizontal drains, ground anchors, and wall systems. Earthquakes, heavy precipitation, fires and floods are hazards that can result in damage to facilities. Man-made hazards include vehicular damage to walls and foundations. Some existing constructed items may need occasional maintenance, such as flushing and surging horizontal drains and unplugging subdrain discharge pipes. When requested, geotechnical professionals should visit the site to observe and document the identified problems. By evaluating the problem areas, geotechnical professionals may decide to instrument and monitor, recommend interim mitigation measures (within available Maintenance budgets) and/or recommend permanent stabilization methods.

### 5.4.1 MONITORING GEOTECHNICAL PERFORMANCE

By regularly monitoring problem areas, geotechnical professionals can often reduce uncertainties in the design of mitigation measures and permanent stabilizations. Instrumentation could be as complicated as extensometers and slope inclinometers or as simple as survey points. Regardless of the complexity of the monitoring program, geotechnical professionals should place and secure instruments as needed to survive for the duration of the intended monitoring period. For example, placing PK nails in pavement, as survey points, would not be a good choice where snow is plowed or pavement repairs may occur during the monitoring period. Existing structures that are potentially sensitive to vibrations or movement should be monitored. It may also be desirable to monitor groundwater level changes, settlement, heave, and/or lateral displacement of the roadway, retaining systems and structures. Anchored wall systems might be instrumented with load cells, which should be monitored to verify continued acceptable performance. Primary references are FHWA-SA-93-057 and NHI 132031. Refer also to NHI 132041, AASHTO MSI-1, NHI 132012, TRB SR 247, and NCHRP Synthesis 89.

Rockfall potential is inherent along roadways in mountainous terrain. The Rockfall Hazard Rating System (RHRS) is a rock slope management tool for quantifying the potential hazard a rock slope poses to users. Details of the RHRS and the procedures involved are described in FHWA-SA-93-057. Refer to NHI 13219. Also, refer to WSDOT Unstable Slope Management System (WSDOT USMS Guidelines) and WSDOT PowerPoint presentation for Real Time Monitoring (WSDOT Monitoring).

### 5.4.2 REPAIR OF GEOTECHNICAL FEATURES

Some construction measures and geotechnical mitigations require maintenance to extend their life and effectiveness. Lack of maintenance or identification of system problems could result in harm to roadway and structures. Discharge ends of drain pipes and aggregate drainage blankets should be maintained free of obstructions to ensure the flow of water out of the systems. Leaks in pipes and lined ditches should be repaired to prevent infiltration and piping erosion. Horizontal drains should be regularly flushed.

Roadways that are experiencing subsidence or landslide cracks should be monitored and may require repair to eliminate hazardous conditions. Maintenance practices should avoid methods that exacerbate unstable conditions, such as material removal at the toe of a marginally-stable slope or placing fill or waste materials onto outside road shoulders where slopes below could become oversteepened and overloaded. The geotechnical professional should evaluate such problem conditions and provide guidance to the maintenance staff. Primary references are FHWA-RT-88-040 and NHI 13219. Also refer to TRB SR 247.

Rockfall mitigation measures such as catch fences, draped mesh and barriers can be susceptible to damage over their service life. The effectiveness of rockfall control could be reduced if no repair is done. In addition, rockfall is likely to accumulate in ditches and fallout areas, reducing catchment effectiveness and increasing the potential for rockfall to reach the roadway. The geotechnical professional should provide guidance for inspection and repair of rockfall mitigation systems. Refer to NHI 13219, and FHWA-OR-RD-01-04.

### 5.4.3 RESPONDING TO EMERGENCY REQUESTS

Generally, it is more cost effective to respond to maintenance requests than to emergencies. Working with Maintenance staff to investigate problem areas before they become emergencies helps to identify the cause of the problem and plan for an appropriate repair. Geologic conditions can present hazards at or near roadways. However, impending geologic hazards could be difficult to identify by Maintenance staff. Geotechnical professionals should evaluate the geologic conditions and potential hazards and provide recommendations regarding the relative risks that road users may face, as well as Maintenance staff who may be working in close proximity. In performing the site evaluation, Geotechnical professionals are cautioned to follow safety practices for their own protection as well as others involved. The Federal Lands Highway emergency program is described in ERFO. Refer to safety guidelines (OSHA Section 29 and MUTCD).

Geotechnical professionals should use available resources as time permits to identify the cause of problems. Geotechnical professionals should ask Maintenance staff for site history, consult geologic publications, examine air photos (stereo pairs), and research files for historic data, hazards, and prior project information. Review available relevant geotechnical reports and instrumentation memoranda for possible problematic conditions encountered during construction.

Information sources and suggestions are described in the referenced manuals for various types of hazards. When responding to a rockfall hazard, Geotechnical professionals should ask

Maintenance staff how often rockfall events occur, where it comes to rest, how much material is typical for a single event, and whether it is comprised of individual blocks or a volume of numerous pieces.

Occasionally geotechnical professionals are involved in evaluating existing structure foundations for new loading conditions. This typically occurs as part of a seismic or scour vulnerability assessment. An important part of the assessment is the type, depth, and condition of the structure foundation. Sources for this information include as-built drawings, construction records, and Plans for the structure.

For geotechnical site problems, geotechnical professionals are typically requested to assess potential hazards and risks. In situations that could imperil the public, warnings may need to be provided to road users and local property owners. In extremely hazardous situations, road closure may be required. In responding to such public safety issues, consult with the area Maintenance Foreman or Supervisor.

Depending on the maintenance cost of a problem and the availability of funds, geotechnical professionals may be asked to recommend interim solutions or "band-aid" mitigations rather than more permanent solutions. When responding to maintenance emergencies a rapid response is often necessary to ensure public safety and maintain the integrity of the roadway. As maintenance repairs proceed, on-site inspection and assistance should be provided to identify differing conditions and make field adjustments as required. Innovative and experimental mitigations may be appropriate, especially if they fit within budget constraints. Removal of slide debris from a roadway or ditch is often done to restore road service, but could cause additional slope distress and failures.

Short-term options to a variety of maintenance problems could include surface water control/diversion, draining of trapped water, slope modifications (flatter slopes or benched slopes), rock inlays, berms, horizontal drains, dewatering wells, fabric walls and gabion walls (MSE), soldier pile and sheetpile walls, pin piles, pavement patching, bio-remediation (seeding, willow wattles, etc.), interim buttresses, grouting, scaling, rock bolts/bars and beams to pin rock that is on the verge of toppling, and unloading a slope to slow slide movements until a permanent solution is constructed. Sometimes these solutions are implemented without complete engineering analyses in order to provide a rapid response; however, geotechnical professionals should explain the uncertainties and risks to the decision-makers and follow through with appropriate analyses to determine whether the implemented measures are adequate and whether additional measures are needed.

Sometimes the interim solutions are implemented to address immediate concerns, and are followed by permanent solutions if additional funding becomes available and Plans can be prepared. Frequently, interim repairs are relied upon much longer than initially intended. Therefore, before recommending an interim or experimental solution, geotechnical professionals should consider the implications in the event the measure becomes permanent. Geotechnical professionals should document and keep records of events regarding maintenance issues. With adequate risk/consequence evaluations, the Maintenance office will be in a position to consider the relative advantages of each option when making decisions on how to proceed.

Mitigation of slope failures and landslides that are not part of design and construction contracts are often managed differently due to smaller funding sources and emergency response timeframes. Feasible solutions might not fully stabilize the problem, but can reduce risk and maintenance. A source of guidance for maintenance-level slides is FHWA-RT-88-040 and TRB SR 247.

FLH standard practice is to monitor geotechnical instrumentation that is necessary to verify satisfactory performance of constructed facilities. Guidance on monitoring geotechnical performance is provided in Section 5.4.1. The TGM guidance is supported by Subsurface Investigations, NHI 132031 and Rockfall Hazard Rating System, FHWA-SA-93-057.

The Geotechnical Discipline provides emergency geotechnical support for evaluating geologic hazards and designing repairs to facilities harmed by natural disasters through the ERFO program. Guidance on repair is provided in Section 5.4.2. The TGM guidance is supported by FHWA-RT-88-040 for highway slopes in general and FHWA-SA-93-085 for rock slopes in particular. Exhibit 5.4–A provides links to these and other sources of guidance.

**Exhibit 5.4–A POST CONSTRUCTION MONITORING AND EMERGENCY RESPONSE REFERENCES**

| Subject | Primary Source | Secondary Sources |
|---|---|---|
| *Monitoring Geotechnical Performance* | NHI 132031<br>FHWA-SA-93-057 | AASHTO MSI-1<br>NHI 132012<br>NHI 132041<br>NCHRP Synthesis 89<br>TRB SR 247 |
| *Repair of Geotechnical Features* | FHWA-SA-93-085<br>FHWA-RT-88-040 | FHWA-OR-RD-01-04<br>TRB SR 247 |
| *Responding to Emergencies* | ERFO | OSHA Section 29<br>MUTCD |

# SECTION 6

# GEOTECHNICAL WORK BY CONSULTANTS

The responsibilities and procedures of the FLH Geotechnical Discipline and consultant geotechnical professionals are described to define agency expectations. It is not the intent of this section to describe the procedures of the consultant selection process or contracting with the agency.

Geotechnical services are occasionally needed which cannot be performed in-house due to time constraints or lack of in-house capabilities. Outsourcing may include all or part of the geotechnical services and may also include some or all other disciplines in the project delivery process. Information regarding the use of Consultants and FLH oversight of their work is covered in this chapter. Specifics on the outsourcing process are available at each Federal Lands Highway Division website:

EFLHD – http://www.fbo.gov/spg/DOT/FHWA/71/postdatePrevDays_1.html

CFLHD - http://www.cflhd.gov/procurement/ae/

WFLHD – http://www.wfl.fha.dot.gov/edi/

The Geotechnical Discipline is responsible for evaluating the geotechnical investigation and geotechnical design submitted by consultants for compliance with Federal Lands Highway Division and partner agency policies and standards. The extent of involvement by the Geotechnical Discipline will depend upon the nature of each project. The consultants are responsible for coordinating all activities related to accomplishing the geotechnical investigation, which may involve obtaining permits and/or having an approved traffic control plan. Geotechnical services provided by consultants for the design and construction of roadway projects that affect roadway right-of-way are subject to the same geotechnical requirements as for services performed by the Federal Lands Highway Division. Fieldwork, laboratory testing, analyses, and design recommendations should be in general accordance with the procedures and guidelines listed in this Manual.

Geotechnical consultants should provide services consistent with the guidance described within PDDM Chapter 6, the Geotechnical Guidance Manual (TGM), and the primary references cited.

## 6.1 RETAINING GEOTECHNICAL CONSULTANTS

Consultants might be retained where the agency does not have necessary equipment, expertise or available personnel. Most geotechnical work is conducted before and during the design phase of a roadway project, but services are also provided during the construction phase of the project. In rare instances, consultant geotechnical services could be retained for non-roadway items, such as buildings.

If a geotechnical consultant is needed, the first choice is to utilize a consultant working directly for the FLH Geotechnical Discipline. In general, geotechnical consultants working directly for the Geotechnical Discipline in any of the FLH Divisions will do so through an on-call IDIQ master agreement through which the consultant is assigned project specific tasks. A Task Order will be developed, with a list of services to be performed, milestone dates, and budget authorization. Through these task orders, the consultant is typically responsible to develop the detailed geotechnical investigation plan, perform the testing and design, and produce a geotechnical report. For these assignments, the consultant is viewed as an extension of the Geotechnical Discipline staff and is therefore subject to the same standards of design and review as in-house division staff. Frequent communication between the Geotechnical Discipline staff and the consultant is essential for a successful project.

When a geotechnical subconsultant is retained by a prime design consultant, the Geotechnical Discipline should assist in the development of the geotechnical scope and estimate for the project and in reviewing the geotechnical subconsultant work product.

## 6.2 INFORMATION PROVIDED BY FEDERAL LANDS HIGHWAY

The Geotechnical Discipline and project development staff should provide information upon request, which may be useful in the design of a project. The information could include items such as previous geotechnical reports, boring logs, laboratory test results, geologic mapping, and as-built plans and construction notes. As-built information should be sought on all rehabilitation projects to obtain knowledge of the existing construction prior to commencing fieldwork. For rehabilitation projects, pavement test data and related information may also be available. Project development information would also be provided, including available maps and cross sections. Provide guidance based on institutional experience with partner agency requirements at the subject roadway site.

## 6.3 SCOPE AND OVERSIGHT OF CONSULTANT GEOTECHNICAL SERVICES

FLH geotechnical staff will be responsible for evaluation of the quality of geotechnical work throughout a project. All geotechnical services provided for Federal Lands projects are reviewed by the Geotechnical Discipline for comments and approval. Therefore, the procedures and methods described above and in this manual are important in standardizing and expediting the information, reports, and techniques utilized by all involved in geotechnical services for FLH projects.

Consultants are expected to work independently but maintain communications as if they were Federal Lands Highway geotechnical staff. At all times, FLH geotechnical professionals should act in an advisory role as compared to a supervisory role.

The Geotechnical Discipline should be contacted prior to commencement of the consultant's geotechnical investigation. A FLH geotechnical professional is assigned as the primary contact. Communications between consultants and Geotechnical Discipline personnel are necessary to result in investigations that meet Federal Lands requirements. This will reduce any unnecessary delays during the subsequent report and design review process and avoid delaying scheduled construction bid dates. Geotechnical requirements of the project are to be agreed to by all parties prior to the start of the work.

The FLH Geotechnical Staff will participate in meetings, site visits, correspondence and review of products. For outsourced work, this document and other documents with geotechnical guidelines accessed through this document will be used to evaluate the quality of the geotechnical work elements and work products unless otherwise stated for specific work elements in the SOW.

The organization performing the investigation is responsible for the technical adequacy of their design and activities. For outsourced work, each delivered product is expected to meet an Acceptable Quality Level (AQL) with respect to the thoroughness and appropriateness of the data collection, analysis and recommendations, the clarity of presentation, and the accuracy of the completed products. Work conducted according to guidelines recommended here and documents referenced here, and satisfying any specific requirements of a Task Order SOW, will meet the thoroughness, appropriateness and clarity criteria of the AQL. The A/E may augment the guidance in this document with their own as long as the requirements here and in the SOW are satisfied.

## 6.4 CONSULTANT SITE INVESTIGATIONS

Federal Lands Highway procedures for site investigations are described in various sections in this manual and in NHI 132031 and FHWA-ED-88-053. On Federal Lands projects, FLH geotechnical professionals and/or geotechnical consultants should adhere to the work described in the Task Order and not exceed those requirements without authorization.

FLH geotechnical professionals are expected to respond to formal and informal requests for information submitted directly to the geotechnical professional or passed down from upper management on projects the geotechnical professional has been assigned. All requests should be responded to expeditiously.

Consultants should submit a quarterly status report on each Task Order.

The FLH geotechnical professional and consultant representatives are expected to attend status meetings, respond verbally to appropriate questions, and then follow up these conversations with written documentation on each of their projects. It is then the FLH geotechnical professional's responsibility to verify that consultants respond to those communications and take the appropriate action.

In-house correspondence between FLH geotechnical professionals and FLH management can be informal, but written records should be maintained. Conversations and meetings should also be documented.

Consultants are allowed to use their own standard forms in lieu of FLH versions, providing all required information is included.

Prior to disposal of soil and rock samples, Consultant should contact the Geotechnical Discipline so that they may take possession of the samples.

## 6.5 CHECKLISTS OF GEOTECHNICAL WORK

Consultants should prepare and submit a list of all services they are expected to perform prior to providing those services. These items are covered in the workscope in the Task Order. Reference should be made to Exhibit 5.1–A through Exhibit 5.1–J and Exhibit 5.2–A in this manual, which are based on FHWA-ED-88-053. While these are useful and convenient references, there may be other requirements and guidelines that should be identified and followed.

## 6.6 CONSULTANT GEOTECHNICAL REPORTS

A consultant should utilize their internal QA/QC procedures that are appropriate for the work performed. Analyses and computations should be checked by an independent geotechnical engineer working for the consultant. Original analysis computations should be documented and filed. It is the FLH geotechnical professional's responsibility to monitor and follow up to see that their recommendations are acknowledged, acted upon, and documented. The FLH geotechnical professional should evaluate the appropriate level of risk for project recommendations and designs. Review should also include use of the FHWA checklists in FHWA-ED-88-053.

All geotechnical documents such as calculations, reports, memos, and logs of borings prepared by the consultant should be provided to the agency in digital format in addition to copies on paper. Report text should be submitted in Microsoft Word, and logs of borings should preferably be in the gINT program format used by FLH. Guidance for geotechnical reports is described in Section 5.

A complete set of the analysis computations should be adequately documented and saved in a separate file or report. The FLH geotechnical professional may request a general review of consultant calculations for possible changes or additions. Consultants remain responsible for the accuracy and completeness of all analyses and deliverables.

When errors or omissions are identified, or when there are disagreements on analyses or conclusions/recommendations, the FLH geotechnical professional provides formal correspondence and maintains a permanent record for the files. The agency's final decision on the use of the consultant's recommendations should be documented (a memorandum to the FLH project manager).

The geotechnical report prepared by the consultant is to be signed, dated, and stamped by the consultant's Registered Professional Engineer in responsible charge of the project geotechnical work.

The consultant should review the preliminary plans and specifications to verify that recommendations have been adequately incorporated and that there are no inconsistencies with the geotechnical findings and recommendations and the Final geotechnical report.

## 6.7 CONSTRUCTION SUPPORT PHASE

Once the consultant's services have been completed and their recommendations have been accepted by the Geotechnical Discipline and incorporated into FLH documents, the FLH geotechnical professionals are expected to support the documents during construction, unless the consultant has been retained for geotechnical construction support.

Consultants could be asked to review contractor's submittals for general accuracy and completeness. Communications with contractors and suppliers follow formal processes routed through the FLH construction project manager. Shop drawings and calculations are typically submitted by contractors prior to commencing portions of the affected work. Occasionally, additional shop drawings and calculations may be required during the construction phase. Consultant review should be stamped by a Registered Professional Engineer. Prior to approving the shop drawings and calculations, the Geotechnical Discipline should coordinate with the Bridge design office on a joint review. If the shop drawings and/or calculations need to be returned for corrections, one memorandum should be written to the construction project manager by the Bridge Division incorporating all needed corrections.

The Geotechnical Discipline performs a cursory review after consultants have completed their review. For consultant-designed projects where the geotechnical consultant has not been retained for construction support, the FLH geotechnical professional should perform the submittal reviews.

Guidance for geotechnical services provided during construction is described in Section 5. Consultants might be requested to review the following:

- Change of site conditions
- Change order requests
- Value Engineering submittals
- Construction recommendations

## 6.8 CONSULTANT PERFORMANCE

While FLH geotechnical professionals have no supervisory role with consultants, they are the primary contact and have close relationships with these parties. FLH geotechnical professionals may be requested to provide comments about consultant's performance during projects. Geotechnical professionals providing material or comments for any evaluation process should use formal FLH correspondence methods and route the information to appropriate personnel.

# APPENDIX A LIST OF PRIMARY, SECONDARY AND OTHER CITED SOURCES (TERTIARY)


| | | | |
|---|---|---|---|
| 1. | AASHTO HB-17 | P-4.8.4<br>S-4.3, 4.3.1, 4.3.2, 4.3.3, 4.4, 4.4.4, 4.11 | *AASHTO, Standard Specifications for Highway Bridges, 17th ed., HB-17, 2002* |
| 2. | AASHTO MSI-1 | S-3.1.1, 3.2.2, 3.2.3, 3.2.4.2, 3.2.4.3, 3.2.4.5, 3.2.6, 3.2.8, 3.2.9, 5.3, 5.4 | *AASHTO, Manual on Subsurface Investigation, MSI-1, 1988.* |
| 3. | AASHTO R 22-97 | S-3.2.4.4 | *AASHTO, Standard Recommended Practice for Decommissioning Geotechnical Exploratory Boreholes, AASHTO R 22-97, Standard Specifications, 2005.* |
| 4. | AASHTO Stds HM-25-M | S-3.2.8 | *AASHTO, Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II: Tests, HM-25-M, 2005.* |
| 5. | ADSC 1989 | S-5.3 | *ADSC, Drilled Shaft Inspector's Manual, 1989.* |
| 6. | ARC 2000 | S-4.4.6 | *Associated Rockery Contractors, Rock Wall Construction Guidelines, Woodinville, WA, 2000 or current edition.* |
| 7. | ASCE 1985 | S-4.9.3 | *ASCE, Dewatering: Avoiding Its Unwanted Side Effects, ISBN 0-87262-459-5, 1985.* |
| 8. | ASCE GBR | S-5.1 | *ASCE, Geotechnical Baseline Reports for Underground Construction - Guidelines and Practices, 1997.* |
| 9. | ASCE MREP 56 | | *ASCE, Subsurface Investigation for Design and Construction of Buildings, Manual and Report on Engineering Practice No. 56, 1976.* |

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|---|---|---|---|
| 10. | BOR Drillers Safety | S-3.1.2 | *US Bureau of Reclamation, Driller's Safety Manual, US Department of the Interior, 1973.* |
| 11. | BOR Earth Manual | S-4.6 | *US Bureau of Reclamation, Earth Manual, Third Edition, US Department of the Interior, 1998.* |
| 12. | BOR Geology Manual | | *US Bureau of Reclamation, Geology Manual, US Department of the Interior, 2005.* |
| 13. | Burch 2006 | P-4.6.2 | *Burch, Deryl, Estimating Excavation, Craftsman Book Company, 2006.* |
| 14. | CalTrans 2001 | S-3.2.4.3, 4.4.7 | *CalTrans, Trenching and Shoring Manual, Revision 12, 2001* |
| 15. | Canadian Foundation | S-4.8.4 | *Canadian Geotechnical Society, Canadian Foundation Engineering Manual, 3rd ed., 1992.* |
| 16. | CFLHD Rockery | P-4.4.6 | *Rockery TD report (in preparation)* |
| 17. | Church 1981 | S-4.6.2 | *Church, H.K., Excavation Handbook, McGraw-Hill, 1981.* |
| 18. | CI/ASCE 36-01 | S-4.5.2 | *Standard Construction Guidelines for Microtunneling, 2001* |
| 19. | Cornforth 2005 | S-4.7.2 | *Cornforth, D.H., Landslides in Practice: Investigation, Analysis, and Remedial/Preventative Options in Soil, Wiley & Sons, 2005.* |
| 20. | Duncan & Wright 2005 | S-4.7.1 | *Duncan, J.M. & Wright, S. G., Soil Strength and Slope Stability, Wiley & Sons, 2005.* |
| 21. | EPRI EL-6800 | S-4.1 | *EPRI, Manual on Estimating Soil Properties for Foundation Design, Electrical Power Research Institute, Report No. EL-6800, 1990.* |
| 22. | ERFO | P-5.4 | *FHWA, Emergency Relief for Federally Owned Roads, Disaster Assistance Manual, FHWA-FLH-04-007, 2004.* |

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|---|---|---|---|
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# APPENDIX B TOPICAL BIBLIOGRAPHY

The following bibliography provides a general list of references. The references are grouped by topic and contain a broad listing of publications. The topic groups are sorted in alphabetical order and consist of the following topics:

- B.1 Geotechnical Project Planning (Section 3.1)
- B.2 Site Investigation (Section 3.2)
- B.3 Soil and Rock Classification (Section 3.2.4)
- B.4 In-Situ Testing (Section 3.2.6)
- B.5 Laboratory Testing (Section 3.2.7)
- B.6 Instrumentation (Section 3.2.8)
- B.7 Soil and Rock Properties (Section 4.1)
- B.8 Computer Programs (Section 4.2)
- B.9 Foundations (Section 4.3)
- B.10 Retaining Walls and Earth Retaining Structures (Section 4.4)
- B.11 Earthwork (Section 4.6)
- B.12 Material Sources (Section 4.6.3)
- B.13 Slope Stability and Landslides (Section 4.7)
- B.14 Rock Slopes and Rockfall Mitigation (Section 4.8.1)
- B.15 Drainage and Dewatering (Section 4.9)
- B.16 Erosion and Sediment Control (Section 4.9.4)
- B.17 Ground Improvement (Section 4.10)
- B.18 Geosythetics (Section 4.10.2)
- B.19 Seismic (Section 4.11)
- B.20 Reports (Section 5.1)

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# APPENDIX C PARTNER AGENCY ACTS, EXECUTIVE ORDERS AND GUIDANCE

## C.1 US FOREST SERVICE ACTS, EXECUTIVE ORDERS, AND GUIDANCE

USFS Forest Service Manual (FSM) Chapter 7170 (2000) is a partner agency policy that describes "Geotechnical and Materials Engineering" policies, responsibilities and definitions. Geotechnical and materials engineering services shall be used in planning, designing, maintaining and operating engineered facilities commensurate with the opportunities, values, hazards, and risks that are anticipated or associated with the facility.

The USFS (Washington) manual includes Title 2800 - Minerals and Geology (1990) that includes a summary of Federal directives and implications for geotechnical practice. Geologic resources and services activities are essential to Forest Service programs and are authorized by the following acts and executive orders:

1. Forest and Rangeland Renewable Resources Planning Act of August 17, 1974 (88 Stat. 476; 16 U.S.C. 1600-1614) as Amended by National Forest Management Act of October 22, 1976 (90 Stat. 2949; 16 U.S.C. 1609). (FSM 1920 and FSM 2550.) This act requires consideration of the geologic environment through the identification of hazardous conditions and the prevention of irreversible damages. The Secretary of Agriculture is required, in the development and maintenance of land management plans, to use a systematic interdisciplinary approach to achieve integrated consideration of physical, biological, economic, and other sciences.

2. Multiple Use Sustained Yield Act of June 12, 1960 (74 Stat. 215; 16 U.S.C. 528-531). (FSM 2501.1.) This act requires due consideration for the relative values of all resources and implies that the administration of nonrenewable resources must be considered.

3. Organic Administrative Act of June 4, 1897, as Amended (30 Stat. 34, as Supplemented and Amended; 16 U.S.C. 473-478, 482-482(a), 551. (FSM 2501.1.)

4. Resource Conservation and Recovery Act of 1976 (90 Stat. 2795; 42 U.S.C. 6901) as Amended by 92 Stat. 3081. This act, commonly referred to as the Solid Waste Disposal Act, requires protection of ground water quality and is integrated with the Safe Drinking Water Act of December 16, 1974, and Amendments of 1977 (42 U.S.C. 300(f)). (FSM 7420.1.)

5. Watershed Protection and Flood Prevention Act of August 4, 1954, as Amended (68 Stat. 666; 16 U.S.C. 1001). (FSM 1021.1, 2501.1.) This act authorizes the Secretary to share costs with other agencies in recreational development, ground-water recharge, and water-quality management, as well as the conservation and proper use of land.

6. Federal Water Pollution Control Act of July 9, 1956, as Amended (33 U.S.C. 1151) (FSM 2501.1); Federal Water Pollution Control Act Amendments of 1972 (86 Stat. 816) (FSM

2501.1), and Clean Water Act of 1977 (91 Stat. 1566; 33 U.S.C. 1251). (FSM 2501.1, 7440.1.) These acts are intended to enhance the quality and value of the water resource and to establish a national policy for the prevention, control, and abatement of water pollution. Groundwater information, including that concerning recharge areas, and information of geologic conditions that affect ground water quality are needed to carry out purposes of these acts.

7. Executive Order 12113, Independent Water Project Review, January 5, 1979. This Executive order requires an independent water project review by the Water Resources Council on preauthorization reports and preconstruction plans for Federal and federally assisted water and related land resource plans. The technical review will evaluate each plan for compliance with the Council's principles and standards, agency procedures, other Federal laws, and goals for public involvement.

8. National Environmental Policy Act of January 1, 1970 (83 Stat. 852 as Amended; 42 U.S.C. 4321, 4331-4335, 4341-4347). (FSM 1950.2.) This act directs all agencies of the Federal Government to utilize a systematic interdisciplinary approach which will ensure the integrated use of the natural and social sciences in planning and in decision-making that may have an impact on the environment. Geology is one of the applicable sciences.

9. Surface Mining Control and Reclamation Act of August 3, 1977 (30 U.S.C. 1201, 1202, 1211, 1221-43, 1251-79, 1281, 1291, 1309, 1311-16, 1321-28). This act enables agencies to take action to prevent water pollution from current mining activities, and also promote reclamation of mined areas left without adequate reclamation prior to this act.

10. Antiquities Act of 1906 (34 Stat. 225; 16 U.S.C. 432-433). (FSM 2361.01.)

11. Archeological and Historical Conservation Act of 1974 (88 Stat. 174; 16 U.S.C. 469). (FSM 2361.01.) This act requires all Federal agencies to notify the Secretary of the Interior when a construction project threatens to irreparably harm or destroy significant scientific, prehistoric, historic, or archeological data. The paleontologic resource may have significant scientific and historic value.

12. Executive Order 11593, Protection and Enhancement of Cultural Environment, May 13, 1971 (3 CFR 559, 1971-75 Compilation).

13. Wilderness Act of September 3, 1964 (78 Stat. 890; 16 U.S.C. 1131-1136). (FSM 2501.1.) This act describes a wilderness as an area which may also contain ecological, geological, or other features of scientific, educational, scenic, or historical value. These geological features are generally identified for wilderness classification purposes.

14. Wild and Scenic Rivers Act of October 2, 1968 (82 Stat. 906 as Amended; 16 U.S.C. 1271-1287). This act states that it is the policy of the United States that certain selected rivers of the Nation that, with their immediate environments, possess outstanding scenic, recreation, geologic, fish and wildlife, cultural, or other similar values, shall be preserved in free-flowing condition.

15. National Forest Roads and Trails Systems Act of October 13, 1964 (78 Stat. 1089; 16 U.S.C. 532-538). (FSM 7701.1.) This act provides for the construction and maintenance of an adequate system of roads and trails to meet the demands for timber, recreation, and other uses. It further provides that protection, development, and management of lands will be under the principles of multiple use and sustained yield of product and services (16 U.S.C. 532). Geologic conditions influence the final selection of route locations.

16. Mining and Minerals Policy Act of December 31, 1970 (84 Stat. 1876; 30 U.S.C.). This act provides for the study and development of methods for the disposal, control, and reclamation of mineral waste products and the reclamation of mined lands. This requires an evaluation of geology as it relates to ground water protection and geologic stability.

17. Disaster Relief Act of 1974 (88 Stat. 143). Section 202(b) states that the President shall direct appropriate Federal agencies to ensure timely and effective disaster warnings for such hazards as earthquakes, volcanic eruptions, landslides, and mudslides. The Federal Register, Vol. 42, No. 70 of April 12, 1977, "Warnings and Preparedness for Geologic Related Hazards," implies coordination with the U.S. Geological Survey in such warnings.

18. USFS Manuals and Guidance.

    a. Forest Service Manual FSM Chapter 7170 “Geotechnical and Materials Engineering,” 2000.

    b. Minerals and Geology (Title 2800), 1990.

    c. Guidelines for Geotech Drilling Exploration (D. Williamson), 1989.

    d. Transportation Structures Handbook, Chapter 3 Site Surveys, Section 3.1 Foundation Investigation (FSH 7709.56b), 1994.

    e. Bridge Foundation Investigation (Region 6, Portland OR), 1980.

19. Federal Lands Highway / US Forest Service Intergovernmental Agreement ?

## C.2 NATIONAL PARKS ACTS, EXECUTIVE ORDERS, AND GUIDANCE

Executive Order No. 11593 Protection and Enhancement of the Cultural Environment, 1971. The Federal Government shall provide leadership in preserving, restoring, and maintaining the historic and cultural environment. Federal agencies shall 1) administer cultural properties in a spirit of stewardship and trusteeship for future generations, 2) initiate measures to direct their policies, plans and programs so that federally owned sites, structures, and objects of historical, architectural or archaeological significance are preserved, restored and maintained for the inspiration and benefit of the people, and 3) institute procedures to ensure that Federal plans and programs contribute to preservation and enhancement of non-federally owned sites, structures and objects of historical, architectural or archaeological significance. Also, refer to National Historic Preservation Act of 1966 (80 Stat. 915, 16 U.S.C. 470 et seq.), Historic Sites Act of 1935 (49 Stat. 666, 16 U.S.C. 461 et seq.), and Antiquities Act of 1906 (34 Stat. 225, 16 U.S.C. 431 et seq.).

National Environmental Policy Act of 1969 (83 Stat. 852, 42 U.S.C. 4321 et seq.).

Endangered Species Act (16 U.S.C., 1531 et seq. 50 CFR Part 402, 450, 451, 452 & 453).

Federal Lands Highway / National Parks Intergovernmental Agreement ?

Geotechnical design manuals?

### C.2.1. Bureau of Indian Affairs Acts, Executive Orders, and Guidance

Indian Reservation Roads Program Final Rule (25 CFR Part 170). Transportation planning procedures and guidelines.

Federal Lands Highway / Bureau of Indian Affairs Intergovernmental Agreement ?

Geotechnical design manuals?

### C.2.2. Fish and Wildlife Service Acts, Executive Orders, and Guidance

Fish and Wildlife Coordination Act (16 U.S.C., 661 et seq.)

Federal Lands Highway / Fish and Wildlife Service Intergovernmental Agreement?

Geotechnical design manuals?

### C.2.3. Bureau of Land Management Acts, Executive Orders, and Guidance

Wild and Scenic River Act (43 CFR Part 8351). Reiterates that Wild and Scenic Rivers shall be managed to protect the natural, cultural, or historical features that make the river outstanding. Lands and water administered by the Bureau of Land Management may see closed or restricted uses if the Wild and Scenic Rivers Act is applicable. Mining regulations shall provide safeguards against pollution of System rivers or potential System additions and unnecessary impairment of the System's scenery.

BLM Land Use Planning Handbook, H-1601-1 (revised 2000).

Federal Lands Highway / Bureau of Land Management Intergovernmental Agreement?

Geotechnical design manuals?

### C.2.4. Bureau of Reclamation Acts, Executive Orders, and Guidance

Federal Lands Highway / Bureau of Reclamation Intergovernmental Agreement?

Policies. (not cited here)

Bureau of Reclamation Design Manuals. Includes geotechnical references and procedures, some of which are web based.

### C.2.5. Corps of Engineers Acts, Executive Orders, and Guidance

Marine Protection, Research, and Sanctuaries Act of 1972 (PL 92-532; 33 U.S.C. 1401-1445). Also known as the Ocean Dumping Act, it was enacted to regulate the dumping of all types of materials into ocean waters. The dumping permit program for dredged material authorizes the Secretary of the Army to issue permits for the transportation of dredged material for the purpose of disposal in the ocean where it is determined that the disposal will not unreasonably degrade human health, the environment, or economic potentialities.

Clean Water Act – Section 404 Wetlands (33 U.S.C., 1251-1387, 33 CFR Part 3300).

Federal Lands Highway / US Corps of Engineers Intergovernmental Agreement ?

Corps of Engineers Technical Manuals. Includes an extensive library of geotechnical manuals describing investigation guidelines and technical analysis and design procedures. Many manuals can be reviewed online.

## C.3 FEDERAL LAWS AND EXECUTIVE ORDERS

Clean Water Act – Section 404 Wetlands (33 U.S.C., 1251-1387, 33 CFR Part 3300).

Clean Water Act – Sections 313, 401 & 402 Water Quality (33 U.S.C., 1251-1387, 33 CFR).

Coastal Barrier Resources Act (16 U.S.C., 3501 et seq., 44 CFR 206 Subpart J, DOI CBRA Advisory Guidelines).

Coastal Zone Management Act (16 U.S.C., 1451 et seq., 15 CFR Part 930, Subpart D).

Disaster Relief Act of 1974 (88 Stat. 143). Section 202(b) states that the President shall direct appropriate Federal agencies to ensure timely and effective disaster warnings for such hazards as earthquakes, volcanic eruptions, landslides, and mudslides.

Endangered Species Act (16 U.S.C., 1531 et seq. 50 CFR Part 402, 450, 451, 452 & 453).

Estuary Protection Act (PL 90-454; 16 U.S.C. 1221-1226) is intended to strike a balance between the national need of conserving the beauty of the nation s estuaries and the need to develop these estuaries to further growth and development.

Executive Order No. 11593 Protection and Enhancement of the Cultural Environment, 1971. Federal agencies shall 1) administer the cultural properties under their control in a spirit of stewardship and trusteeship, 2) preserve federally owned sites, structures, and objects of historical, architectural or archaeological significance, and 3) contribute to the preservation and enhancement of non-federally owned sites, structures and objects of historical, architectural or archaeological significance.

Executive Order 11988, Floodplain Management, and Executive Order 11990, Wetland Protection (44 CFR Part 9). And "Floodplain Management Guidelines," Water Resources Council (WRC) describes a decision process from the determination that a proposed action is or is not located in the base floodplain through the implementation of agency actions. The Department of the Interior "520 DM 1 Floodplain Management and Wetlands Protection Procedures" generally adopts the WRC guidelines for floodplain management. Requires agencies to write compliance procedures and provides criteria for evaluation of procedures.

Farmland Protection Policy Act (7 U.S.C., 4201 et sea 7 CFR 658).

Federal Water Pollution Control Act of July 9, 1956, as Amended (33 U.S.C. 1151); Federal Water Pollution Control Act Amendments of 1972 (86 Stat. 816), and (Clean Water Act of 1977 (91 Stat. 1566; 33 U.S.C. 1251).

Fish and Wildlife Coordination Act (16 U.S.C., 661 et seq.)

Forest and Rangeland Renewable Resources Planning Act of August 17, 1974 (88 Stat. 476; 16 U.S.C. 1600-1614) as Amended by National Forest Management Act of October 22, 1976 (90 Stat. 2949; 16 U.S.C. 1609). This act requires consideration of the geologic environment through the identification of hazardous conditions and the prevention of irreversible damages.

Mining and Minerals Policy Act of December 31, 1970 (84 Stat. 1876; 30 U.S.C.).

Multiple Use Sustained Yield Act of June 12, 1960 (74 Stat. 215; 16 U.S.C. 528-531).

National Environmental Policy Act 1970 (83 Stat. 852, 42 U.S.C.A., 4321, 40 CFR Parts 1500-1508 (CEQ) 44 CFR Part 10 (FEMA)).

National Historic Preservation Act of 1966 (80 Stat. 915, 16 U.S.C. 470 et seq.). Historic Sites Act of 1935 (49 Stat. 666, 16 U.S.C. 461 et seq.). Antiquities Act of 1906 (34 Stat. 225, 16 U.S.C. 431 et seq.).

Organic Administrative Act of June 4, 1897, as Amended (30 Stat. 34, as Supplemented and Amended; 16 U.S.C. 473-478, 482-482(a), 551.

Resource Conservation and Recovery Act of 1976 (90 Stat. 2795; 42 U.S.C. 6901) as Amended by 92 Stat. 3081.

Surface Mining Control and Reclamation Act of August 3, 1977 (30 U.S.C. 1201, 1202, 1211, 1221-43, 1251-79, 1281, 1291, 1309, 1311-16, 1321-28).

Watershed Protection and Flood Prevention Act of August 4, 1954, as Amended (68 Stat. 666; 16 U.S.C. 1001).

Wild and Scenic Rivers Act 1968 (16 U.S.C., 1271-1287 et seq. 36 CFR Part 297, Subpart A).

Wilderness Act of September 3, 1964 (78 Stat. 890; 16 U.S.C. 1131-1136).